COMPACTION OF ASPHALT CONCRETE PAVEMENTS

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TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

LIST OF FIGURES (CONT'D)

LIST OF FIGURES (CONT'D)

LSIT OF FIGURES (CONT'D)

LIST OF FIGURES (CONT'D)

LIST OF FIGURES (CONT'D)

LIST OF FIGURES (CONT'D)

APPENDICES

Disclaimer

The opinions, findings, and conclusions expressed in this report are those of the authors and not necessarily those of the Bureau of Public Roads.

INTRODUCTION

The importance of proper compaction of asphalt pavements has been recognized for many years. Investigators have shown that pavement stability, durability, tensile strength, fatigue resistance, stiffness, and flexibility are controlled to a certain degree by the density of asphalt concrete.

To insure adequate compaction several agencies specify "in-place" density requirements. These in-place requirements are commonly expressed as a percent of a standard laboratory compaction density. Laboratory tests are intended to give the engineer needed information about the density of the surfacing material as it ultimately appears on the roadway. However, there is evidence that an increasing number of asphalt concrete pavements in Texas as well as other states are not stabilizing at a density equal to that obtained in the laboratory design of a companion paving mixture.

The reasons for this unpredictable behavior are probably many and complex. In an attempt to define more adequately the variables that may affect the long term density of a pavement, fifteen test sites were selected throughout the state of Texas, and compaction data were collected over a three-year span, covering a maximum life span of two years for any individual pavement. The results of this study are presented herein.

For the sake of simplicity in reporting the results of this study, the long term compaction has been separated into initial compaction or that which occurs during the construction of the pavement while the asphalt concrete is at an elevated temperature, and long term compaction. The latter compaction is considered to be due to the action of traffic and environment, and takes place after initial compaction has occurred. Furthermore, the data collected during the laboratory phase of the project have been separated from the data collected during the field phase of the project. Detailed results will be presented in each section of the report together with a brief literature review.

TEST PROGRAM

The test program can be conveniently separated into laboratory and field work. Laboratory compaction data were obtained on the paving mixtures obtained from fifteen full scale field test sites. For comparison purposes, similar measurements were made by the Texas Highway Department district laboratories.

The field work included site selection, preparation and placing of the test section, and regular sampling of the fifteen test sections.

As a result of the above mentioned laboratory and field data, comparisons have been made which suggest that the rate and amount of densification of a surface course of asphalt concrete is dependent upon a complex set of variables that cannot be easily separated.

Field Work

Test Section Layout: Fifteen test sites were selected in 6 highway districts. The test site selection was based on:

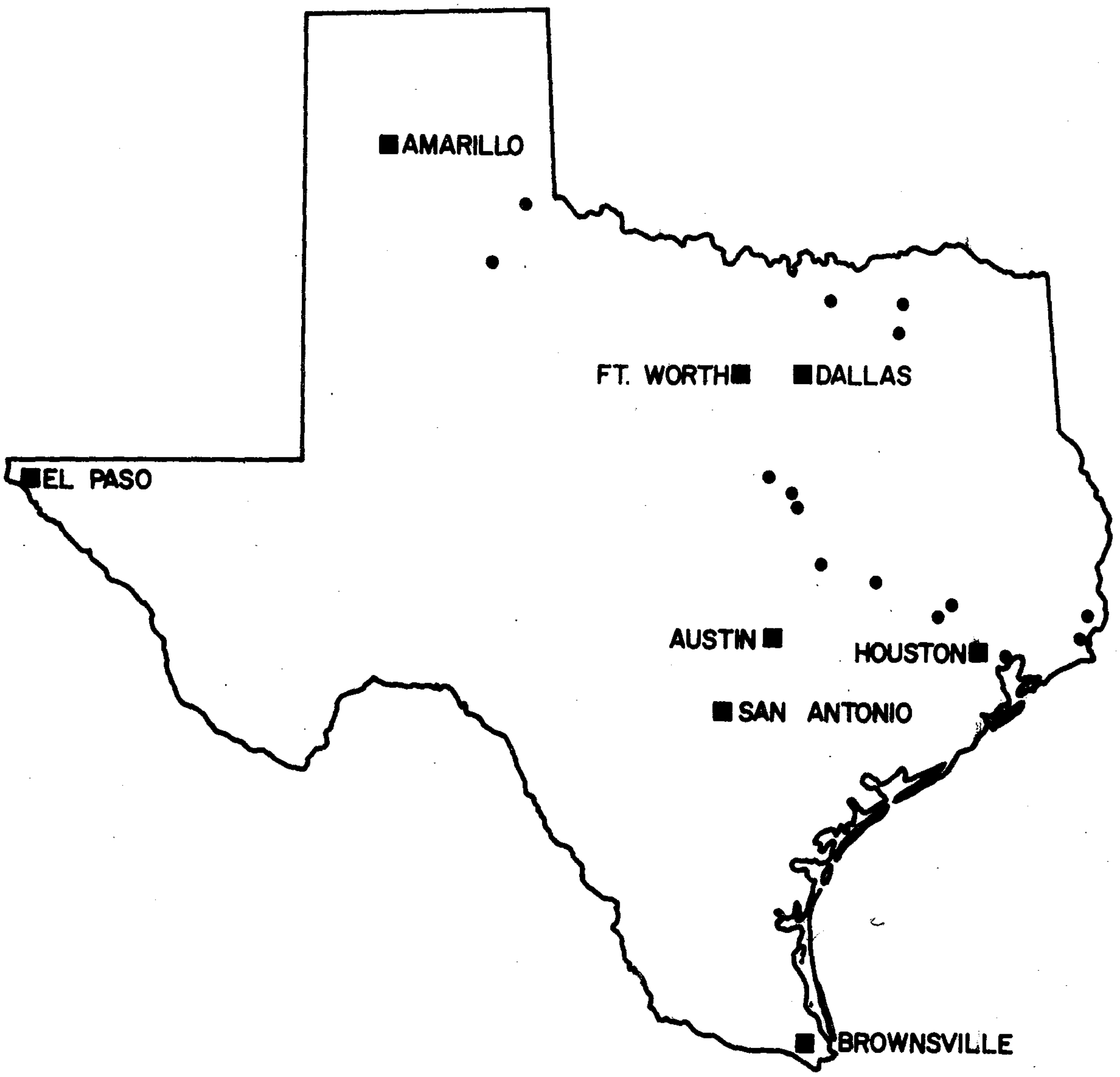
1. Contract work in progress
2. Traffic volume
3. Climatic conditions
4. Materials
5. Pavement type (flexible or rigid)
6. Construction type (new or overlay)

In addition the grade line was approximately level, there was no ingress or egress from the test sections, and all test sites were on tangents. The approximate location of each test site is shown in Figure 1. Details of the exact location are given in Table 1, together with the name of the project to be used in this report. Table 2, contains pertinent weather conditions on the day of construction.

Each test section was 600 feet in length and one traffic lane in width. The sections are further subdivided into three parts (A, B, and C) with each part or subdivision receiving a different amount of construction compaction.

A typical layout for a given test section is shown in Figure 2. The test cores were removed (as indicated in the figure) from the center portion of each subdivision. The space between sampling locations was provided so that the rollers would have sufficient maneuvering space, thus avoiding the effect of still another variable.

In order to obtain samples, 18-inch by 24-inch aluminum foil envelopes were placed on the existing surface or base. These envelopes consisted of a single sheet of aluminum foil folded to form an envelope as shown in Figure 3. The foil envelopes were prepared in the laboratory



TEST SITE LOCATIONS

FIGURE 1

before field construction, and were glued to the pavement with contact cement as shown in Figures 4 and 5. The pressure required to hold the foil in place was applied by an automobile tire. Figure 6 is a close-up view of an aluminum foil envelope in-place on an existing roadway. As shown in Figures 2 and 7, the aluminum foil envelopes are arranged in rows to correspond to the wheel paths of the vehicles with an additional row of envelopes between the wheel paths.

The hot mix asphaltic concrete was placed on the prepared roadway in the normal manner without damage to the foil envelopes. A single 4-inch diameter core was removed from each prepared location according to the following schedule:

1 day 18 cores @ 3 subdivisions = 54 samples
1 week 18 cores @ 3 subdivisions = 54 samples
1 month 9 cores @ 3 subdivisions = 27 samples
4 months 9 cores @ 3 subdivisions = 27 samples
1 year 9 cores @ 3 subdivisions = 27 samples
2 years 9 cores @ 3 subdivisions = 27 samples

Total number of samples per test site = 216

The sequence of coring proceeded against the traffic flow (Figure 2).

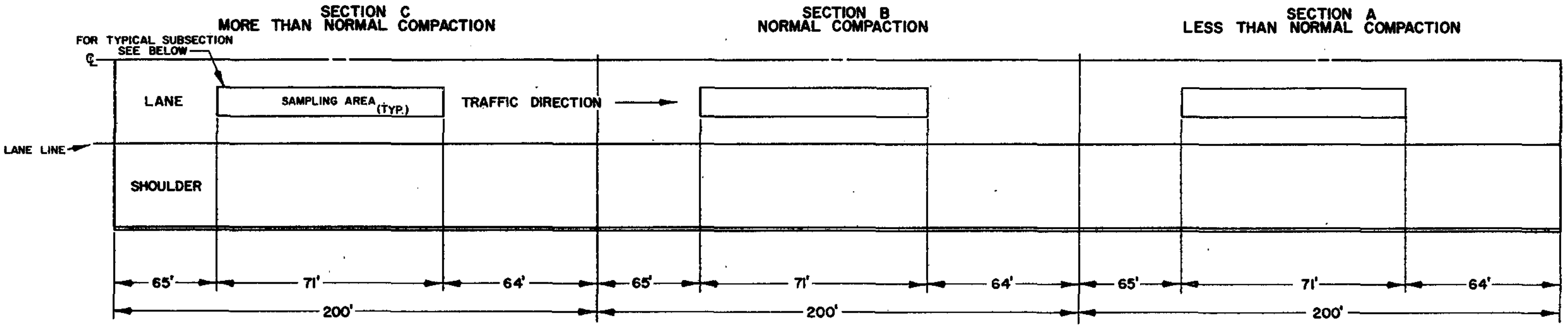
Test Results: The stiffness or supporting capacity of the "base" on which the asphalt concrete test section was placed, has been evaluated by the measurement of the pavement deflection. The pavement deflection was determined by the use of the Benkleman beam with an eighteen-kip axle load. These rebound deflection measurements were made initially, and in selected cases at regular intervals during the study. The initial measurements, (Table 3), were made at 30-foot intervals throughout the test section. Later measurements were made at the same locations during both summer and winter months to determine if the seasonal variations in

TABLE 2 CONSTRUCTION WEATHER CONDITIONS

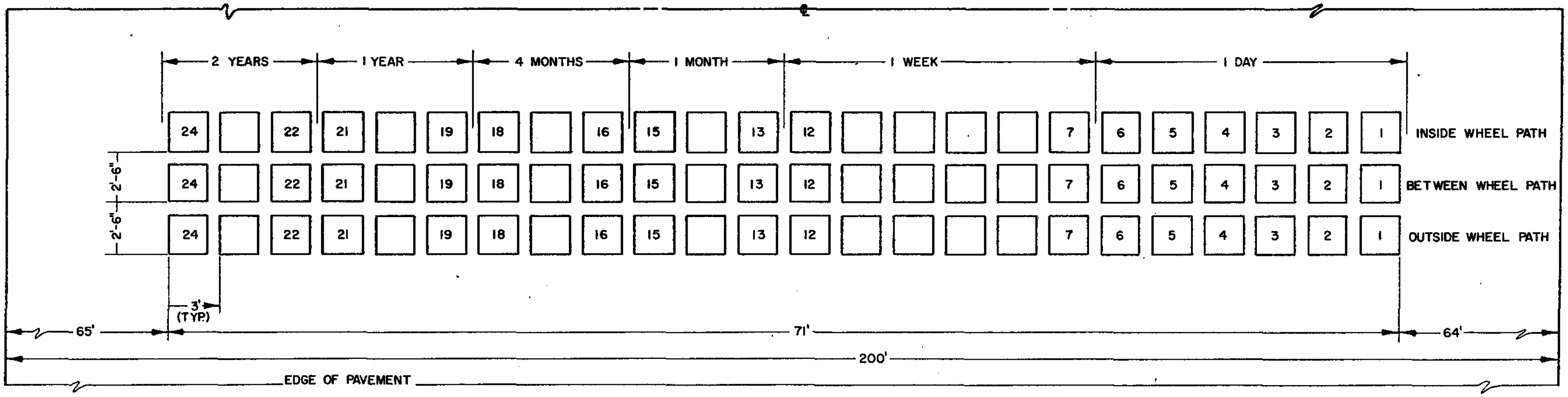
Test Section	Day of Construction	Weather Conditions	Maximum Temperature, °F	Minimum Temperature, °F
Childress US 287 25-42-9	5-3-66	Clear & Warm	79	54
Matador US 70 25-145-8	11-3-66	Clear	60	30
Sherman SH 5 1-47-3	10-10-66	Partly Cloudy	79	66
Cooper SH 24 1-136-3	9-3-67	Clear & Warm	85	60
Cumby IH 30 1-9-13	2-5-67	Clear & Cold	71	37
Clifton SH 6 9-258-7	7-25-66	Cloudy	93	86
Waco US 84 9-55-8	8-3-65	Clear	99	68
Robinson US 77 9-209-1	8-9-66	Partly Cloudy Hot	100	78
Milano SH 36 17-185-4	8-17-66	Partly Cloudy Hot	96	74
Bryan Spur 308 17-599-1	8-17-65	Partly Cloudy Hot	96	73
Tamina IH 45 12-110-4	7-6-66	Partly Cloudy Hot	92	71
Conroe FM 1485 12-1062-35	8-20-65	Partly Cloudy Hot	100	72
Baytown Spur 330 12-508-7	9-1-65	Partly Cloudy Hot	95	79
Orange SH 12 20-499-3	6-21-66	Partly Cloudy Hot	93	72
Bridge City IH 87 20-306-3	6-14-66	Partly Cloudy Hot	93	73

TABLE 1 TEST SITE DETAILS

Project Reference Number	Test Section Number	Reference Name	Highway	County
1	25-42-9	Childress	U.S. 287, 5.4 mi. NW of Red River Bridge	Hall
2	25-145-8	Matador	U.S. 70, 3 mi. W of Matador	Motley
3	1-47-3	Sherman	U.S. 75, 2.7 mi. N of Van Alstyne City Limit	Grayson
4	1-136-3	Cooper	SH 24, 3 mi. SW of Cooper City Limit	Delta
5	1-9-13	Cumby	IH 30, 2 mi. W of FM 275	Hopkins
6	9-258-7	Clifton	SH 6, .1 mi. W of SH 6 SH 6 and FM 217	Bosque
7	9-55-8	Waco	U.S. 84, .1 mi. W of SH 6 over pass	McLennan
8	9-209-1	Robinson	U.S. 77, 1.6 mi. N of Jct U.S. 77 and FM 2837	McLennan
9	17-185-4	Milano	SH 36, 3 mi. N of Jct. SH 36 and U.S. 79	Milam
10	17-599-1	Bryan	SH 308, .25 mi. N of Jct SH 308 and FM 60	Brazos
11	12-110-4	Tamina	IH 45, 1.5 mi. S of West Fork of San Jacinto River	Montgomery
12	12-1062-35	Conroe	FM 1485, 7 mi. NW of New Caney	Montgomery
13	12-508-7	Baytown	Spur 330, 3 mi. SE of IH 10	Harris
14	20-499-3	Orange	SH 12, 0.7 mi. E of Jct SH 12 and 62	Orange
15	20-306-3	Bridge City	SH 87, .2 mi. S of Railroad Bridge	Jefferson



OVERALL LAYOUT



ENLARGEMENT SHOWING FOIL PLACEMENT & SAMPLE SEQUENCE FOR ONE SUBSECTION

TYPICAL TEST SECTION LAYOUT

FIGURE 2

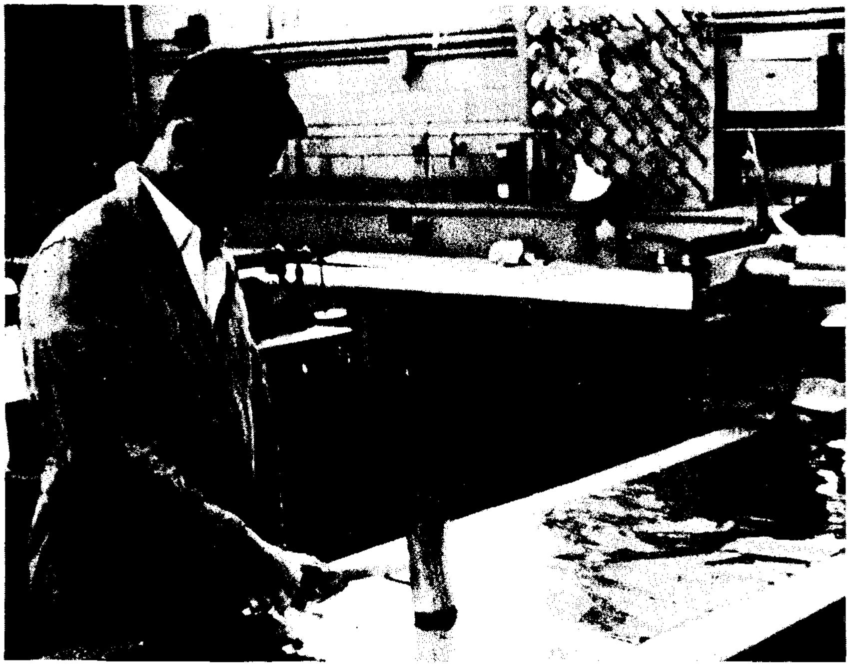


Figure 3. Preparation of aluminum foil envelopes.

Figure 4. Application of contact cement to the roadway.

Figure 5. Application of aluminum foil envelopes to the roadway.

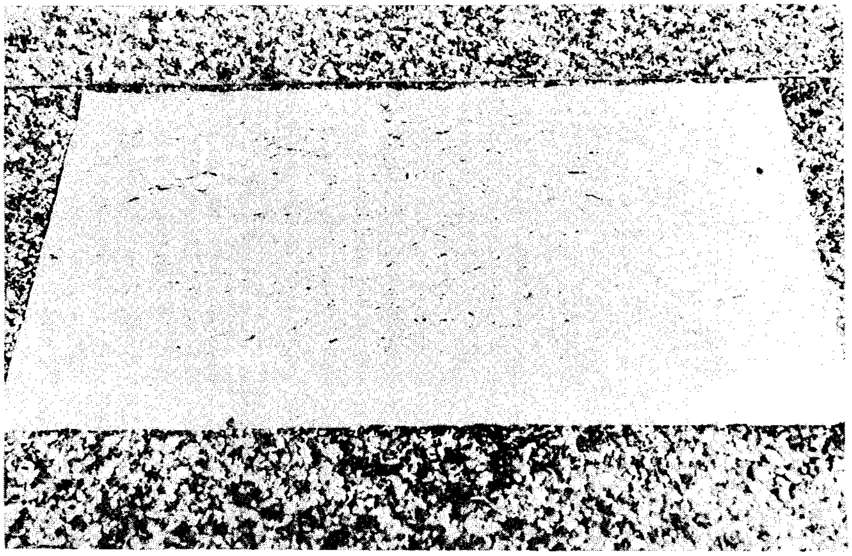


Figure 6. View of aluminum foil envelope in place.

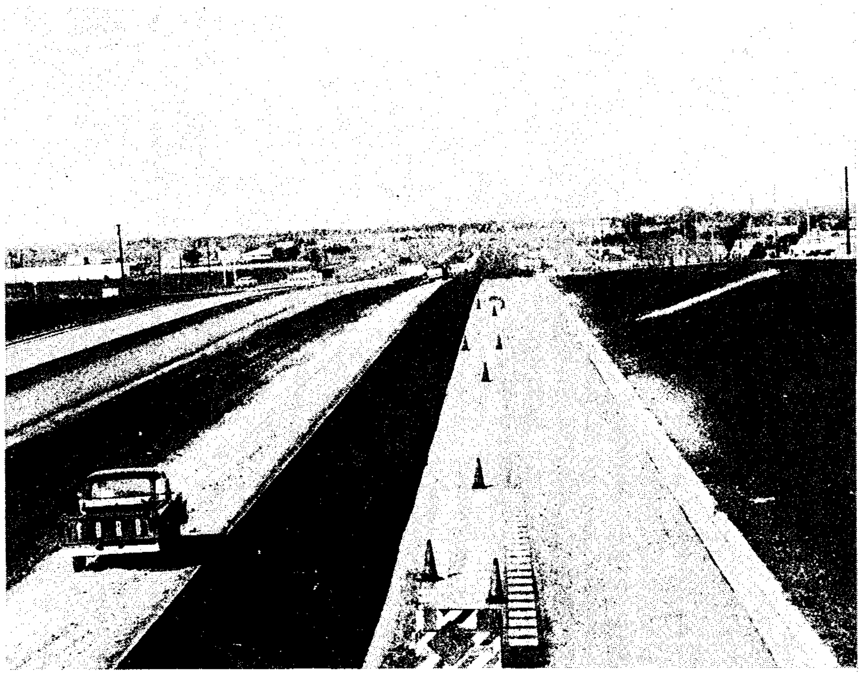


Figure 7. View of completed test section.

the pavement flexibility are a factor influencing the surface compaction (Table 3). The type of "base" material on which the test section was placed is given in Table 3. As shown both new and overlay construction on rigid and flexible pavement bases were used.

The amount of traffic on these test sections (Table 4, Part I) has been determined by the Texas Highway Department and presented in terms of equivalent 18,000-pound axle loads (Table 4, Part II). The equivalent 18,000-pound axle load considers not only the number of vehicles but also their directional distribution, the percentage of trucks, the weight of trucks and other factors.

The percent air voids for the pavements after initial compaction and at various times during a two-year span is given in Appendix A. As shown each test site has three subsections A, B, and C. Each subsection has been subjected to different amounts of compaction as follows:

subsection A - half as many roller passes as subsection B.

subsection B - normal rolling procedures for the given project.

subsection C - twice as many roller passes as subsection B.

It is believed that this range of roller passes would span the range encountered in practice. Compaction procedures for each project are given in Table 5. These data will be used subsequently.

Approximately twenty-five pounds of loose mixture was taken from the laydown machine at each section. These materials were used for the laboratory study explained in the next section.

TABLE 3 BENKLEMAN BEAM DEFLECTIONS

		Before Construction			After Construction			
Test Section	"Base" Material	Average Deflection, in.	Standard Deviation	Date of Test	Average Deflection, in.	Standard Deviation	Date of Test	Date of Construction
Childress US 287 25-42-9	New Black Base	0.02266	.00141	4-26-66				5-3-66
Matador US 70 25-145-8	HMAC	0.01789	.00281	4-26-66				11-2-66
Sherman SH 5 1-47-3	HMAC							10-10-66
Cooper SH 24 1-136-3	HMAC & Level Up							3-7-67
Cumby IH 30 1-9-13	New IH Spec.							2-5-67
Clifton SH 6 9-258-7	HMAC & Level Up	0.03521	.00044	7-66	0.009714	.00216	7-66	7-25-66
Waco US 84 9-55-8	New Flex Base	0.01562	.00441	7-23-65	0.01548	.00284	7-66	8-3-65
Robinson US 77 9-209-1	PC & HMAC	0.01957	.00353	7-66	0.016285	.00310	3-16-67	8-9-66
Milano SH 36 17-185-4	HMAC & Level Up							8-17-66
Bryan Spur 308 17-599-1	PC	0.0154	.00489	8-11-65	0.0144	.00443	2-4-66	8-17-65
Tamina IH 45 12-110-4	PC + 4 in. Iron Ore	0.003429	.00700	6-15-66				7-6-66
Conroe FM 1485 12-1062-35	HMAC	0.02351	.00206	8-11-65	0.02979	.00253	2-3-66	8-20-65
Baytown Spur 330 12-508-7	PC	0.0105	.00275	8-12-65	0.01147	.00780	2-3-66	9-1-65
Orange SH 12 20-499-3	PC & Level Up	0.00716	.00264	5-24-66				6-21-66
Bridge City IH 87 20-306-3	HMAC	0.0476	.00436	5-24-66				6-14-66

TABLE 4 TRAFFIC

Part I

Test Section	ADT		Increase In Traffic/Year		Trucks, Percent
	Beginning	Ending	Number	Percent	
Childress US 287 25-42-9	3190	3,461	136	4.25	18.90
Matador US 70 25-145-8	1170	1,264	47	4.02	24.40
Sherman SH 5 1-47-3	6410	7,754	672	10.48	12.30
Cooper SH 24 1-136-3	1860	1,975	57	3.09	9.60
Cumby IH 30 1-9-13	6210	7,502	646	10.40	15.40
Clifton SH 6 9-258-7	3060	3,310	125	4.08	14.90
Waco US 84 9-55-8	11270	12,343	536	4.76	6.00
Robinson US 77 9-209-1	2810	3,279	234	8.34	13.2
Milano SH 36 17-185-4	1720	1,866	73	4.24	22.20
Bryan Spur 308 17-599-1	7500	7,969	235	3.13	5.00
Tamina IH 45 12-110-4	14180	17,117	1468	10.35	10.00
Conroe FM 1485 12-1062-35	810	935	62	7.72	10.40
Baytown Spur 330 12-508-7	11500	15,458	1979	17.21	10.00
Orange SH 12 20-499-3	3510	3,989	239	6.82	16.60
Bridge City IH 87 20-306-3	8930	9,763	416	4.66	12.00

TABLE 4 TRAFFIC

Part II

	Equivalent 18-K Single Axle Load Applications		Pavement		Design		
Test Section	One-Way	Two-Way	Type	Years	Thickness	Base	Using Average Distributions
Childress US 287 25-42-9	101,378	202,757	A	2	3	Flexible	No
Matador US 70 25-145-8	44,710	89,421	A	2	3	Flexible	No
Sherman SH 5 1-47-3	202,336	404,672	A	2	3	Flexible	No
Cooper SH 24 1-136-3	37,010	74,020	B	2	3	Flexible	No
Cumby IH 30 1-9-13	188,674	377,349	A	2			
Clifton SH 6 9-258-7	72,382	144,763	B	2	3	Flexible	No
Waco US 84 9-55-8	106,918	213,836	A	2	3	Flexible	No
Robinson US 77 9-209-1	156,859	313,717	A	2	8	Rigid	No
Milano SH 36 17-185-4	119,758	239,517	B	2	3	Flexible	No
Bryan Spur 308 17-599-1	164,851	329,703	B	2	8	Rigid	No
Tamina IH 45 12-110-4	497,734	995,469	A	2	8	Rigid	No
Conroe FM 1485 12-1062-35	24,444	48,888	C	2	3	Flexible	No
Baytown Spur 330 12-508-7	326,583	653,167	A	2	8	Rigid	No
Orange SH 12 20-499-3	121,164	242,327	B	2	8	Rigid	No
Bridge City IH 87 20-306-3	236,729	473,459	B	2	3	Flexible	No

TABLE 5 COMPACTION PROCEDURE
Part I

Test Section	Compaction Equipment							
	Breakdown Rolling				Intermediate Rolling			
	Passes/Section A	B	C	Type Roller/Size	Passes/Section A	B	C	Type Roller/Size
Childress US 287 25-42-9	4	3	6	3 wheel tandem, 12 ton; 5'-4' diameter	4	4	4	2 wheel tandem, 10 ton; 5'-4' diameter
Matador US 70 25-145-8	11	11	11	3 wheel, 10 ton	5	11	21	Tandem 10 ton
Sherman SH 5 1-47-3	6	12	24	3 wheel 10 ton	5	5	9	Tandem 8 ton
Cooper SH 24 1-136-3	3	5	9	3 wheel 10 ton	3	5	9	Tandem 10 ton; 4' diameter
Cumby IH 30 1-9-13	3	7	13	3 wheel 10 ton	3	5	9	Tandem 8 ton
Clifton SH 6 9-258-7	3	3	6	3 wheel 10 ton, 60"-42" diameter	4	8	16	Pneumatic 16.3ton 75 psi
Waco US 84 9-55-8	4	3	9	Tandem, 8 ton, 54" diameter	4	4	4	Tandem, 8 ton, 54" diameter
Robinson US 77 9-209-1	3	3	7	3 wheel, 10 ton, 60"-42" diameter	4	4	4	Tandem 8 ton, 54"-42" diameter
Milano SH 36 17-185-4	3	3	7	3 wheel, 10 ton, 60"-38" diameter	3	3	3	Tandem, 8 ton, 54" diameter
Bryan Spur 308 17-599-1	3	6	12	3 wheel, 10 ton, 60" diameter	3	3	3	Tandem, 8 ton, 54" diameter
Tamina IH 45 12-110-4	3	7	4	3 wheel, 10 ton, 42"-66" diameter	6	6	24	Penumatic, 10 ton, 85 psi
Conroe FM 1485 12-1062-35	3	7	14	3 wheel, 10 ton, 60" diameter	10	10	20	Pneumatic, 25 ton, 65-70 psi
Baytown Spur 330 12-508-7	3	6	12	3 wheel, 10 ton, 60" diameter	NONE			
Orange SH 12 20-499-3	5	7	13	3 wheel, 10 ton, 5'-3' diameter	NONE			
Bridge City IH 87 20-306-3	5	9	15		NONE			

TABLE 5 COMPACTION PROCEDURE
Part II

Test Section	Compaction Equipment				Temperature, °F			Field Initial Density (1 Day) Section B IWP
	Final Rolling							
	Passes/Section			Type Roller/Size	Air	Break-down	Final Roll	
	A	B	C					
Childress US 287 25-42-9	14	14	14	Pneumatic, 25ton 60 psi	51	145	125	8.69
Matador US 70 25-145-8	7	13	25	Pneumatic, 25 ton 75 psi	63	225	145	7.68
Sherman SH 5 1-47-3	10	20	40	Pneumatic, 12 ton 70 psi	80	200	135	8.26
Cooper SH 24 1-136-3	1	4	7	Pneumatic	82	155	75	10.85
Cumby IH 30 1-9-13	3	5	9	Pneumatic, 22.3to 102 psi	46	205	100	5.51
Clifton SH 6 9-258-7	3	7	14	Tandem, 8.8ton, 60"-48" diameter	96	220	150	9.89
Waco US 84 9-55-8	15	15	15	Pneumatic, 8 ton, 44-52 psi	101	180	135	7.39
Robinson US 77 9-209-1	12	12	18	Penumatic, 25 ton, 60 psi	98	160	130	8.53
Milano SH 36 17-185-4	3	7	13	Penumatic, 25 ton, 60 psi	95	160	145	20.79
Bryan Spur 308 17-599-1	4	4	8	Penumatic, 12 ton, 75 psi	95	170	135	18.76
Tamina IH 45 12-110-4	2	2	2	Tandem, 10 ton, 54" diameter	97	185	145	12.72
Conroe FM 1485 12-1062-35	3	3	6	Tandem, 8 ton, 60" diameter	95	155	135	12.34
Baytown Spur 330 12-508-7	3	3	3	Tandem, 8 ton, 54" diameter	108	180	100	25.88
Orange SH 12 20-499-3	3	5	11	Tandem, 12 ton, 4 1/2' - 3 1/2'	90	200	170	10.02
Bridge City IH 87 20-306-3	5	7	11	Tandem, 8 ton, 5'-4'	85	200	165	13.83

Laboratory Work

The loose mixture obtained from the laydown equipment in the field was transported to the central laboratory of the Texas Transportation Institute for future evaluation. Also samples of the mixture were obtained by the Texas Highway Department. The proposed purpose of the duplication of effort was to compare the results of the "field laboratory" and those of the research laboratory so that any recommendations resulting from the study could be translated to Texas Highway Department field conditions. However, this was a secondary objective of the study suggested by the Construction Division of the Texas Highway Department.

The laboratory measurements that were duplicated were those of making and testing job control specimens using the Texas motorized gyratory shear press. The compactive effort was a variable in the study and constituted an attempt to determine the optimum amount of laboratory compaction. The present recommended procedure according to test method Tex-206-F, Part II (tentative) (1) is to apply an initial gage pressure of 50 psi to the specimen to be compacted. The mold containing the loose mix is then tilted 1° and rotated three revolutions after which the mold is leveled and a check is made to determine whether or not the desired compaction has been reached. This is done by making one full stroke on the jack and (this deformation represents approximately 1 percent strain on the compacted specimen) if one stroke of the jack increases the gage pressure to 150 psi or more, the sample is considered to have been satisfactorily compacted. If one stroke on the jack does not increase the pressure to 150 psi, the gage pressure is adjusted to 50 psi, and another set of three gyrations is applied. The procedure is repeated until the designated end point has been reached, then the ends of the sample are leveled or made parallel by applying a leveling force equivalent to 1588 psi on the specimen. The leveling load is then immediately removed. The compacted sample is extruded from the mold and allowed to cool.

Variations in the compactive effort were obtained by changing the starting pressure and the end point pressure. The leveling procedure remained the same. The concensus was that the laboratory density obtained by the standard method (described above) was sufficient for normal roadway construction. Thus, it became necessary to reduce the compactive effort. This was accomplished by reducing the end point from 150 psi (gage) to 100 psi. This is termed for this report the medium compactive effort; whereas, the standard method described previously is called the high compactive effort. This so-called medium effort is the same as the procedure currently being used by the Texas Highway Department for the manual gyratory shear press (Tex-206-F Part I). (1) The low compactive effort was effected by reducing the starting pressure to 40 psi and the end to 50 psi, otherwise the procedure remained the same.

A second laboratory compactor was used at three different energy levels to aid in evaluating the compactibility of the asphaltic concrete mixtures. This compactor was the gyratory testing apparatus developed by the U. S. Army Corps of Engineers at Vicksburg, Mississippi and presently patented by the Engineering Development Company (EDCO). This apparatus is similar in design to the THD motorized gyratory shear press. The standard procedure or compactive effort requires 30 gyrations with the mold inclined at 1^{o} and a constant pressure of 100 psi on the specimen. The compactive effort was varied by changing the constant pressure to 50 and 150 psi and holding the 30 gyrations and 1^{o} of tilt constant.

A third type compaction procedure was used. This was the Marshall compaction procedure, which compacts by the impact of a dropped hammer. The standard procedure requires 50 blows per face and this was used as a medium effect for the laboratory study. The low Marshall compactive effort consisted of 10 blows per face while the high compactive effort consisted of 75 blows of the ten-pound hammer on each face of the specimen.

The California Kneading Compactor was the fourth type of compaction used. The high compactive effort followed the California specified procedure (Test Method No. Calif. 304-E) (2) which requires 150 tamps at 500 psi foot pressure. The medium compactive effort was set at 100 tamps while the low compactive effort was set at 25 tamps at 500 psi foot pressure.

<u>TTI and THD Comparisons</u>: All specimens compacted in the Texas Transportation Institute Laboratory were tested for density, percent air voids, stability, and cohesion. The samples compacted in the individual field laboratories were transported to the Texas Highway Department Materials and Test Laboratories in Austin, Texas for measurement of stability and density. Density and air void contents for the specimen compacted in the TTI laboratory were determined by weighing the specimens in air and water, and comparing this value with the Rice specific gravity obtained on the loose mix (the Rice method allows for absorption of the asphalt by the aggregates). These values are given in Table 6 while stability and cohesiometer values are given in Table 7. Density and air

void contents for the specimen compacted in the THD laboratory were determined by weighing the coated specimens in air and water, and comparing this value with the calculated theoretical maximum specific gravity of the components of the mix. Results obtained by the THD are shown in Table 8. The values shown in these tables are averages of three specimens.

The variations noted in density between the TTI and THD laboratory compacted specimens (Figure 8) are due to different methods of analysis as reported by Gallaway (3, 4), Gallaway and Harper (5) and explained above. However, even after corrections were made for the method of analysis, it was found that differences existed; but, on the average, the air void differences were less than 1.5 percent.

The Hveem stability values of specimens compacted in the TTI laboratory are compared with the specimen compacted in the THD laboratory in Figure 9. As shown the TTI values tended to be higher than the stabilities measured in the THD laboratory. These differences neglect the difference between the two stabilometers used in the measurements.

Comparison of Compaction Methods: Another objective of the laboratory portion of the study was to examine and compare methods of compacting specimens in the laboratory. The standard methods of compaction were used for these comparisons, i.e. 50-blow Marshall, 150-tamp California, 100 psi gyratory and 150 psi THD methods. A comparison between the THD and Corps of Engineers gyratory compaction method is shown in Figure 10 while comparisons between the THD and Marshall and THD and California method are shown in Figures 11 and 12.

TABLE 6 DENSITY AND AIR VOID CONTENT OF LABORATORY COMPACTED SPECIMEN-TEXAS TRANSPORTATION INSTITUTE LABORATORY

Test Section	Compactive Effort		THD	Marshal	Gyratory	California
Childress	Low	Density	2.343	2.250 (1)	2.331 (2)	2.317
		Voids	4.04	7.87	4.55	5.11
	Med	Density	2.367	2.386	2.391	2.364
		Voids	3.05	2.28	2.09	3.20
	High	Density	2.380	2.403	2.401	2.369
		Voids	2.52	1.60	1.66	2.99
Matador	Low	Density	2.389	2.312	2.388	2.273
		Voids	1.45	4.64	1.52	2.14
	Med	Density	2.386	2.350	2.389	2.391
		Voids	1.56	2.89	1.48	1.40
	High	Density	2.388	2.400	2.399	2.397
		Voids	1.49	1.04	1.07	1.13
Sherman	Low	Density	2.380	2.179	2.292	2.214
		Voids	3.54	7.82	3.01	6.32
	Med	Density	2.301	2.259	2.296	2.252
		Voids	2.65	4.43	2.85	4.72
	High	Density	2.305	2.288	2.316	2.270
		Voids	2.49	3.19	2.03	3.99
Cooper	Low	Density	2.347	2.238	2.374	2.320
		Voids	5.34	9.73	4.27	6.42
	Med	Density	2.365	2.360	2.381	2.352
		Voids	4.61	4.82	3.98	5.15
	High	Density	2.373	2.379	2.398	2.346
		Voids	4.28	4.04	3.29	5.39
Cumby	Low	Density	2.353	2.273	2.373	2.309
		Voids	3.94	7.11	3.04	5.67
	Med	Density	2.363	2.345	2.381	2.338
		Voids	3.45	4.21	2.73	4.46
	High	Density	2.371	2.361	2.398	2.344
		Voids	3.16	3.54	2.05	4.24
Clifton	Low	Density	2.376	2.302	2.344	2.336
		Voids	3.93	6.90	5.21	5.55
	Med	Density	2.385	2.333	2.348	2.355
		Voids	3.53	5.56	5.05	4.78
	High	Density	2.386	2.404	3.370.	2.379
		Voids	3.36	2.79	4.16	3.78
Waco	Low	Density	2.344	2.275 (1)	2.369 (2)	2.390 (3)
		Voids	4.92	7.73	3.91	3.05
	Med	Density	2.352	2.355	2.390	2.408
		Voids	4.59	4.20	3.07	2.32
	High	Density	2.373	2.379	2.407	2.407
		Voids	3.74	3.50	2.37	2.39
Robinson	Low	Density	2.349	2.270	2.359	2.333
		Voids	4.43	7.64	4.05	5.10
	Med	Density	2.350	2.346	2.379	2.367
		Voids	3.99	4.56	3.20	3.69
	High	Density	2.365	2.352	2.390	2.378
		Voids	3.78	4.91	2.78	3.25
Milano	Low	Density	2.205	2.045	2.093	2.102
		Voids	11.53	17.88	16.03	15.70
	Med	Density	2.230	2.194	2.134	2.153
		Voids	10.54	11.96	14.38	13.62
	High	Density	2.304	2.252	2.192	2.159
		Voids	7.59	9.67	12.07	13.39

(1) Compactive Effort
Low - 10 blows one face only

(2) Compactive Effort
Low - 25 psi

(3) Compactive Effort
Low - 100 tamps
Medium - 150 tamps
High - 200 tamps

TABLE 6 DENSITY AND AIR VOID CONTENT OF LABORATORY COMPACTED SPECIMEN-TEXAS TRANSPORTATION INSTITUTE LABORATORY (Cont'd)

Test Section	Compactive Effort		THD	Marshal	Gyratory	California
Bryan	Low	Density	2.168	2.009 (1)	2.062 (2)	2.248 (3)
		Voids	11.39	17.90	15.73	8.10
	Med	Density	2.190	2.120	2.121	2.269
		Voids	10.48	13.49	13.31	7.23
	High	Density	2.202	2.162	2.158	2.270
		Voids	9.98	11.60	11.80	7.20
Tamina	Low	Density	2.300	2.210	2.254	2.256
		Voids	7.4	11.0	9.2	9.12
	Med	Density	2.325	2.338	2.326	2.309
		Voids	7.4	11.0	9.2	9.12
	High	Density	2.378	2.351	2.349	2.281
		Voids	4.2	5.3	5.4	8.1
Conroe	Low	Density	2.331	2.266 (4)	2.316 (2)	2.371 (3)
		Voids	5.26	7.90	5.84	3.61
	Med	Density	2.352	2.357	2.360	2.376
		Voids	4.41	4.76	4.40	3.41
	High	Density	2.354	2.354	2.375	2.381
		Voids	4.28	4.29	3.43	3.20
Baytown	Low	Density	2.268	2.141 (1)	2.285 (2)	2.295 (3)
		Voids	7.09	12.30	6.40	5.96
	Med	Density	2.289	2.251	2.311	2.296
		Voids	6.23	7.77	5.33	5.96
	High	Density	2.294	2.278	2.237	2.281
		Voids	6.03	6.67	4.66	6.57
Orange	Low	Density	2.325	2.221	2.331	2.287
		Voids	5.89	10.09	5.61	7.40
	Med	Density	2.332	2.323	2.354	2.328
		Voids	5.57	5.95	4.69	5.75
	High	Density	2.374	2.319	2.364	2.332
		Voids	3.90	6.11	4.29	5.58
Bridge City	Low	Density	2.394	2.294	2.388	2.342
		Voids	4.38	8.36	4.63	6.48
	Med	Density	2.423	2.431	2.430	2.392
		Voids	3.81	5.02	3.32	4.78
	High	Density	2.423	2.431	2.430	2.392
		Voids	3.21	2.92	2.95	4.44

(1) Compactive Effort
Low - 10 blows, one face only

(2) Compactive Effort
Low - 25 psi

(3) Compactive Effort
Low - 100 tamps
Medium - 150 tamps
High - 200 tamps

(4) Compactive Effort
Low - 20 blows

TABLE 7 STABILITY AND COHESIOMETER VALUES OF LABORATORY COMPACTED SPECIMEN - TEXAS TRANSPORTATION INSTITUTE LABORATORY

Test Section	Compactive Effort		THD	Marshall	Gyratory	California
Childress	Low	Stability	40.0	21.2	32.8	34.8
		Cohes.	275	78.8 (1)	126.8 (2)	136.4
	Med	Stability	40.0	47.3	46.8	40.9
		Cohes.	209.5	274.0	338.7	242.0
	High	Stability	40.9	44.4	50.0	38.0
		Cohes.	240.0	277.3	525.0	255.5
Matador	Low	Stability	0.00	0.00	0.00	0.00
		Cohes.	375.0	185.0	447.0	260.0
	Med	Stability	16.4	0.00	0.00	0.00
		Cohes.	245.0	691.0	331.0	328.0
	High	Stability	17.9	0.00	0.00	0.00
		Cohes.	404.0	275.0	375.0	297.0
Sherman	Low	Stability	54.3	42.1	51.8	47.5
		Cohes.	558.0	345.0	493.0	271.0
	Med	Stability	57.8	60.3	56.0	49.3
		Cohes.	496.0	590.0	370.0	389.0
	High	Stability	57.3	58.2	58.5	42.2
		Cohes.	548.0	534.0	689.0	476.0
Cooper	Low	Stability	42.8	31.6	45.5	38.6
		Cohes.	177.0	289.0	167.0	128.0
	Med	Stability	44.2	44.8	48.3	41.4
		Cohes.	202.0	342.0	276.0	175.0
	High	Stability	44.8	40.5	49.5	44.0
		Cohes.	253.0	338.0	274.0	244.0
Cumby	Low	Stability	37.8	23.4	39.4	31.5
		Cohes.	102.0	100.0	284.0	153.0
	Med	Stability	36.6	39.7	39.8	36.3
		Cohes.	149.0	152.0	325.0	199.0
	High	Stability	39.0	44.1	41.0	39.0
		Cohes.	185.0	203.0	416.0	310.0
Clifton	Low	Stability	43.4	23.8	32.4	35.4
		Cohes.	516.7	153.6	367.0	325.2
	Med	Stability	48.4	45.2	39.7	44.7
		Cohes.	535.7	331.6	391.0	457.3
	High	Stability	43.8	51.1	49.2	42.0
		Cohes.	495.7	517.6	432.0	513.0
Waco	Low	Stability	37.2	17.4	28.4	35.3
		Cohes.	67.0	36.0 (1)	119.5 (2)	229.0 (3)
	Med	Stability	38.2	49.8	34.5	33.1
		Cohes.	83.0	122.0	163.0	295.0
	High	Stability	38.6	46.0	36.7	20.7
		Cohes.	128.0	172.0	230.0	242.0 (3)
Robinson	Low	Stability	40.9	27.6	43.5	34.1
		Cohes.	263.9	162.4	425.6	184.6
	Med	Stability	45.6	45.1	45.6	35.3
		Cohes.	303.2	226.3	512.1	325.4
	High	Stability	45.8	46.2	47.9	31.1
		Cohes.	238.9	303.7	526.0	360.6
Milano	Low	Stability	33.8	23.3	22.8	27.9
		Cohes.	130.9	41.2	54.2	39.4
	Med	Stability	34.4	31.0	24.5	29.3
		Cohes.	103.7	143.3	57.8	85.8
	High	Stability	34.7	38.0	29.8	28.1
		Cohes.	148.8	147.3	68.9	104.6

(1) Compactive Effort
Low - 10 blows, one face only

(2) Compactive Effort
Low - 25 psi

(3) Compactive Effort
Low - 100 tamps
Medium - 150 tamps
High - 200 tamps

TABLE 7 STABILITY AND COHESIOMETER VALUES OF LABORATORY COMPACTED SPECIMEN - TEXAS TRANSPORTATION INSTITUTE LABORATORY (Cont'd)

Test Section	Compactive Effort		THD	Marshall	Gyratory	California
Bryan	Low	Stability	26.0	19.2	18.2	28.4
		Cohes.	65.0	33.0 (1)	27.0 (2)	88.0 (3)
	Med	Stability	28.3	28.7	23.6	29.5
		Cohes	67.0	83.0	39.0	98.0
	High	Stability	27.7	32.7	27.9	34.9
		Cohes.	63.0	109.0	50.0	126.0
Tamina	Low	Stability	50.1	40.0	40.5	45.4
		Cohes.	110.0	51.5	118.5	124.6
	Med	Stability	53.4	54.1	59.5	57.3
		Cohes.	177.2	145.5	216.0	179.6
	High	Stability	57.7	58.4	66.0	55.9
		Cohes.	321.0	176.0	262.0	397.3
Conroe	Low	Stability	45.7	60.5	36.8	40.7
		Cohes.	351.0	168.0 (4)	311.0 (2)	387.0 (3)
	Med	Stability	45.6	45.8	41.5	44.6
		Cohes.	323.0	535.1	542.0	418.0
	High	Stability	40.9	46.7	48.6	53.1
		Cohes.	280.0	646.5	523.0	303.0
Baytown	Low	Stability	36.2	17.4 (1)	40.1 (2)	53.3
		Cohes.	30.0	specimen too weak	specimen too weak	70.0 (3)
	Med	Stability	37.7	48.8	46.0	51.2
		Cohes.	33.0	28.0	31.0	spec. too weak
	High	Stability	38.7	50.3	49.1	57.8
		Cohes.	28.0	32.0	45.0	58.0
Orange	Low	Stability	48.3	34.8	46.3	38.9
		Cohes.	61.0	22.35	61.7	41.1
	Med	Stability	47.4	54.5	53.7	37.2
		Cohes.	70.0	70.3	112.0	87.3
	High	Stability	47.5	52.3	56.8	39.1
		Cohes.	101.5	104.0	133.1	90.6
Bridge City	Low	Stability	44.8	34.2	45.0	41.8
		Cohes.	166.1	60.5	141.0	162.4
	Med	Stability	44.9	45.2	50.8	44.3
		Cohes.	183.8	149.0	315.0	285.0
	High	Stability	42.7	44.7	55.1	44.4
		Cohes.	171.2	299.0	357.0	314.1

(1) Compactive Effort
Low - 10 blows, one face only

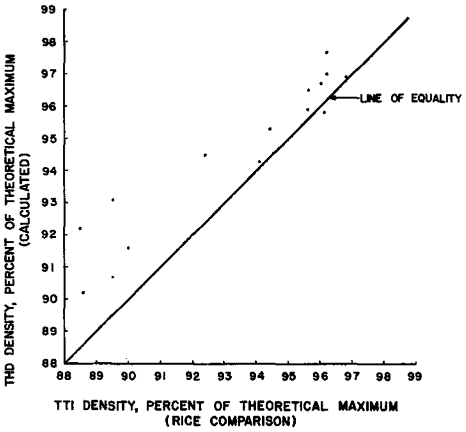
(2) Compactive Effort
Low - 25 psi

(3) Compactive Effort
Low - 100 tamps
Medium - 150 tamps
High - 200 tamps

(4) Compactive Effort
Low - 20 blows

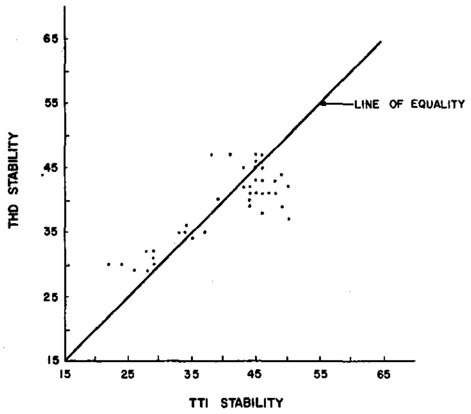
TABLE 8 DENSITY AND STABILITY VALUES OF LABORATORY COMPACTED SPECIMEN - TEXAS HIGHWAY DEPARTMENT LABORATORY

Test Section	Air Voids, Percent			Stability Values		
	Low Compactive Effort	Medium Compactive Effort	High Compactive Effort	Low Compactive Effort	Medium Compactive Effort	High Compactive Effort
Childress US 287 25-42-9	3.2	1.9	1.9	42	38	33
Matador US 70 25-145-8	3.1	2.5	1.9	42	33	22
Sherman SH 5 1-47-3	5.8	5.2	4.7	41	41	49
Cooper SH 24 1-136-3	7.6	5.5	4.8	41	44	46
Cumby IH 30 1-9-13	4.9	3.8	3.1	30	34	37
Clifton SH 6 9-258-7	1.9	1.0	0.9	50	47	45
Waco US 84 9-55-8						
Robinson US 77 9-209-1	4.1	3.3	2.3	44	44	46
Milano SH 36 17-185-4	7.8	6.9	5.5	35	35	35
Bryan Spur 308 17-599-1	9.8	9.3	8.4	30	30	32
Tamina IH 45 12-110-4						
Conroe FM 1485 12-1062-35						
Baytown Spur 330 12-508-7						
Orange SH 12 20-499-3	5.7	4.7	4.2	38	41	43
Bridge City IH 87 20-306-3	3.5	3.0	3.1	41	43	43



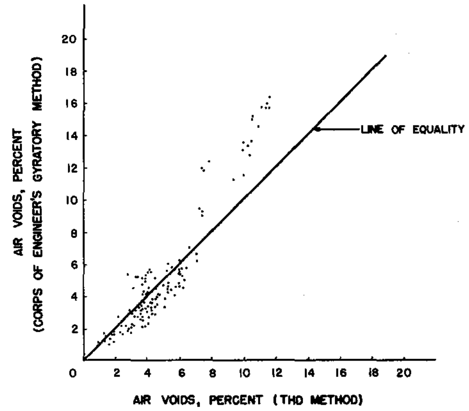
TEXAS TRANSPORTATION INSTITUTE'S VERSUS
TEXAS HIGHWAY DEPARTMENT'S
COMPACTIVE PROCEDURE

FIGURE 8



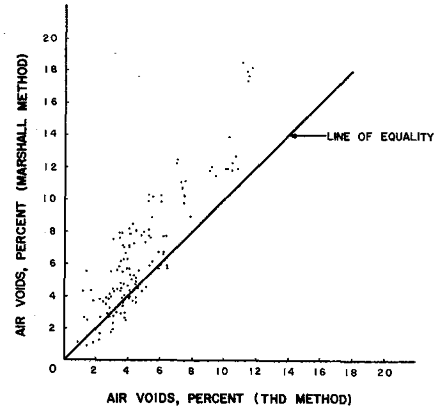
COMPARISON OF STABILITY VALUES
OBTAINED IN THE TEXAS TRANSPORTATION
INSTITUTE'S LABORATORY WITH THOSE
OBTAINED IN THE TEXAS HIGHWAY
DEPARTMENT'S LABORATORY

FIGURE 9



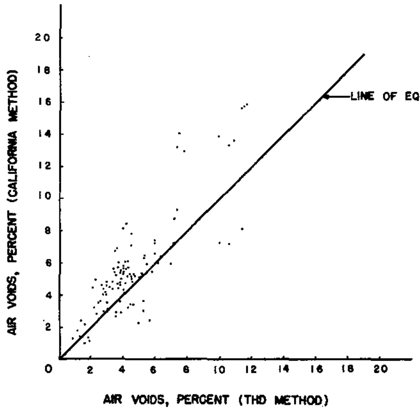
RELATIONSHIP BETWEEN THD AND CORPS OF
ENGINEERS GYRATORY METHOD OF COMPACTION

FIGURE 10



RELATIONSHIP BETWEEN THD AND
MARSHALL METHOD OF COMPACTION

FIGURE 11



RELATIONSHIP BETWEEN THD AND CALIFORNIA
METHOD OF COMPACTION

FIGURE 12

A regression analysis on the density data collected from the various compaction methods suggests that

$$D_G = -0.97 + 1.40\ D_T$$

where:

D_G = density of specimen compacted in the Corps of Engineers gyratory testing machine (100 psi pressure)

D_T = density of specimen compacted in the Texas gyratory shear press (150 psi end point pressure).

This relationship has a coefficient of determination equal to 0.74. Similarly a linear relationship was found to exist between the Marshall and THD method.

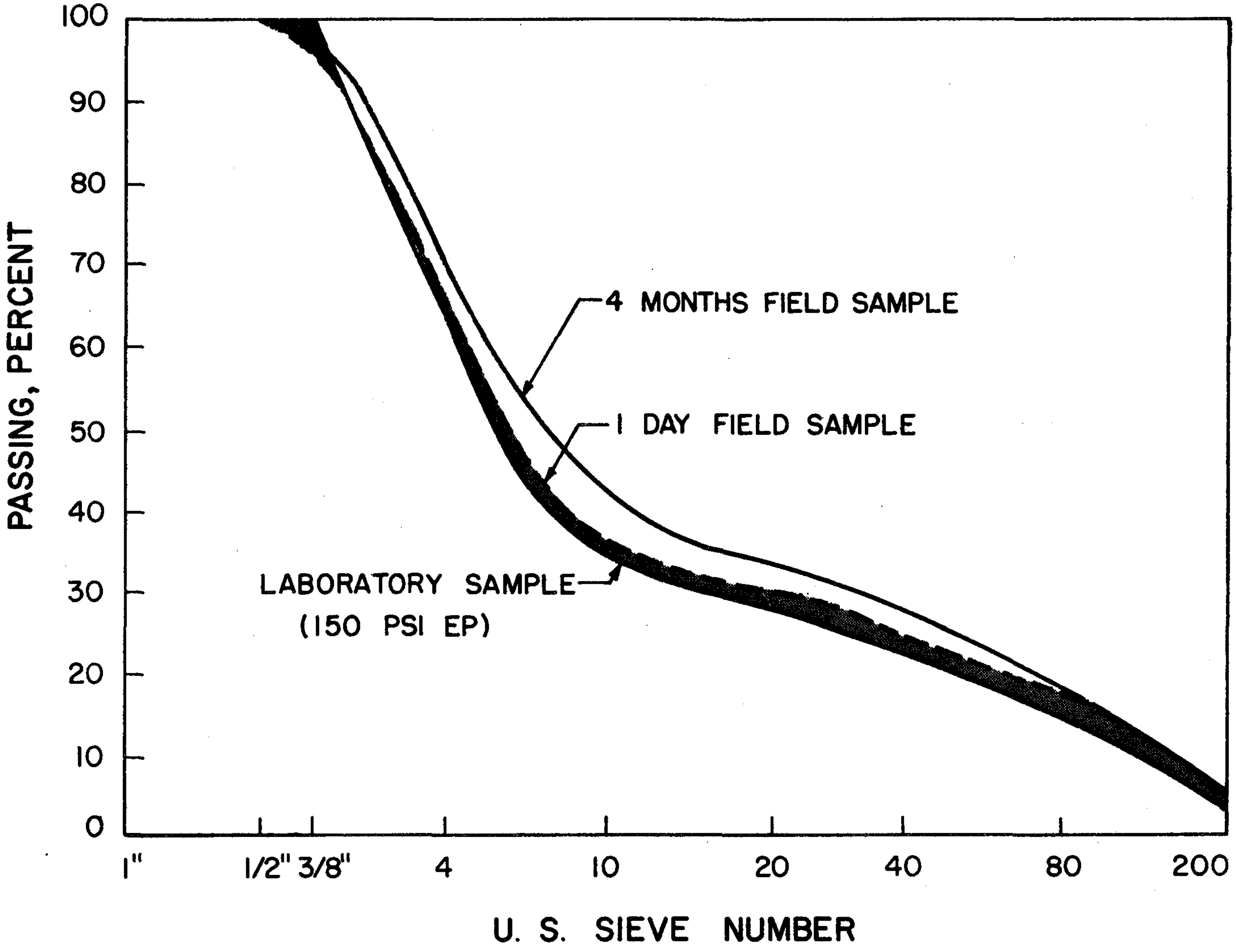
$$D_M = -0.67 + 1.27\ D_T$$

where:

D_M = density of specimen compacted by the Marshall method (50 blows) per face 10-pound hammer 18-inch drop).

These figures suggest that the Texas method produces a more dense mix than any of the other three compaction methods investigated for the majority of the mixtures under study.

Aggregate Degradation: Concern has been expressed by several investigators that aggregates degrade during the mixing and compaction process both in the field and the laboratory. Figure 13, which represents typical data collected in this project, suggests that little degradation takes place that cannot be explained by differences in sampling and degradations created by the coring operation. In particular, this figure shows the gradation of the aggregate after a sample of mixture obtained from the field has been compacted in the normal manner in the laboratory and the asphalt removed, the gradation from a core sample obtained from the field after construction and before traffic was allowed on the surface, and the gradation after 4 months of traffic. The original gradation determined from the THD samples of the hot bins is not shown; however, it falls in the shaded area between the laboratory and one-day samples. The gradation of the one-year sample is not shown; however, it also falls within the shaded region. Gradation curves showing degradation for all field sites are shown in Appendix B.



AGGREGATE DEGRADATION--WACO TEST SECTION

FIGURE 13

PURPOSE OF COMPACTION

The purpose of compacting asphalt pavements is to densify the asphalt concrete and thereby improve its mechanical properties as well as to provide a watertight segment for the underlying materials in the pavement structure. A properly designed paving mixture compacted to the optimum degree will, for selected types of aggregates, provide a smooth, skid-resistant pavement at minimum costs for its design life while being subject to traffic and environmental loading conditions.

The mixture properties that should be considered when selecting the optimum density compaction include stability, durability, flexibility, fatigue resistance, skid resistance, and fracture strength. By examining the density requirements for each of these mixture properties one can make an intelligent judgement as to the degree of compaction that is necessary to provide a long lasting economical pavement.

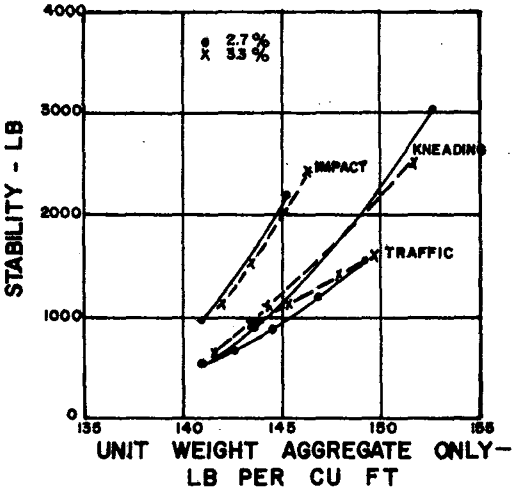
Stability

Stability, which can be defined as the resistance of a mix to deformation under load, has been shown to be dependent on density by numerous investigators including Monismith and Vallerga (6), McLeod (7), Kiefer (8), McRae (9), Bodell (10), and Bahie and Rader (11). As shown in Figure 14, the stability increases with increase in density and is mainly dependent on the type of compaction (6,9). In general, however, the stability increases with density until a critical air void content is reached, where upon the stability begins to decrease with increased density for certain asphalt contents. Air void contents below about 2 percent tend to produce mixtures with lower stabilities.

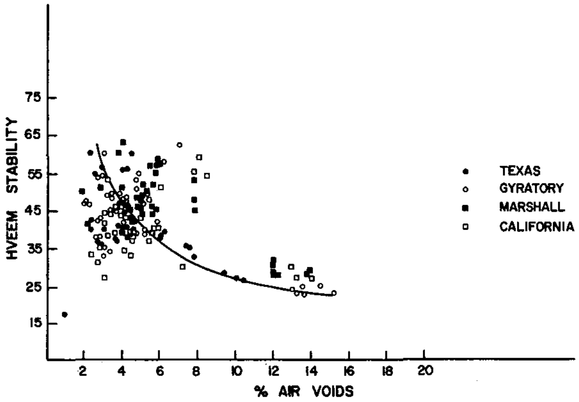
As mentioned previously and as shown in Figure 14, the stability is dependent upon, among other things, the type or laboratory compaction equipment used. These results suggest that samples compacted by static, impact and kneading procedures will have different stabilities at the same density. McRae (9) further suggests that the stability-density relationship for kneading laboratory compaction (gyratory) more nearly approximates the stability - density relationship brought about by traffic and environment.

Stability-air void curves are presented in Figure 15 for all projects included in this study. Figures 16 and 17 represent stability-air void curves for mixes from particular test sections. These figures reinforce the trends noted by other investigators in that the type of compaction influences the resulting stability. However, the trend is not as evident as that shown in Figure 14 (for the mixes investigated in this study). The above mentioned relationship should therefore be considered in selecting the type of laboratory compaction that is to be used for determining relative densities in field compacted pavements.

Goode and Lufsey (12) have presented data (Figure 18) which suggest that stability increases with air void content. This trend is, however, noted for specimens which have been subjected to oven curing at 140°F for 12 days. This apparent discrepancy of increased stability with increased air voids is due to the hardening of the asphalt when subjected to head and oxygen in specimens of various air void contents. These data suggest that air void content affects stability on a long term basis as well as initially and these effects may be opposite. Figure 19 illustrates

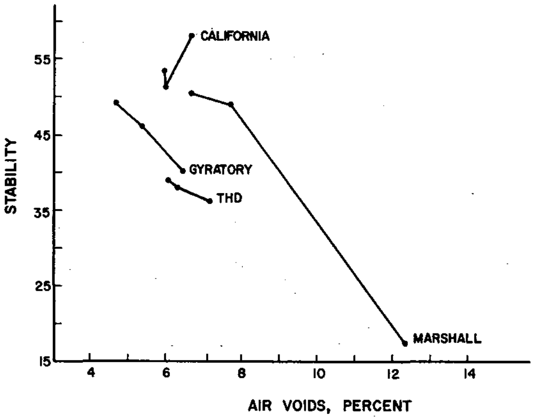


COMPARISON OF DENSITY
AND STABILITY FOR VARIOUS
TYPES OF COMPACTION
(AFTER McRAE (9))
FIGURE 14



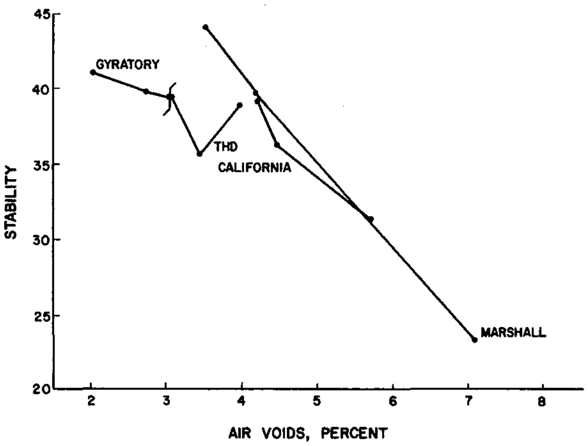
COMPARISON OF STABILITY AND AIR VOID CONTENT FOR VARIOUS TYPES OF COMPACTION ON ALL TEST SECTIONS

FIGURE 15



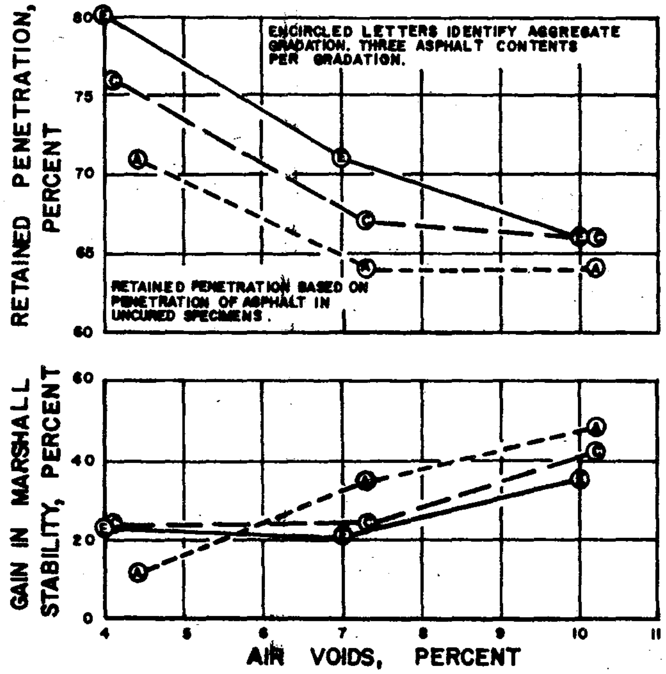
RELATIONSHIP BETWEEN HVEEM STABILITY AND AIR VOIDS CONTENT FOR VARIOUS TYPES OF COMPACTION (BAYTOWN TEST SECTION)

FIGURE 16



RELATIONSHIP BETWEEN HVEEM STABILITY AND AIR VOIDS CONTENT FOR VARIOUS TYPES OF COMPACTION (CUMBY TEST SECTION)

FIGURE 17



EFFECT OF AIR VOIDS ON DEGREE
OF ASPHALT HARDENING AFTER
12-DAY OVEN CURING AT 140°F.
(AFTER GOODE AND LUFSEY (12))

FIGURE 18

the effect of high density on stability for a mix compacted in the field. Stability and cohesiometer values for all mixtures compacted in the field are shown in Table 9.

Durability

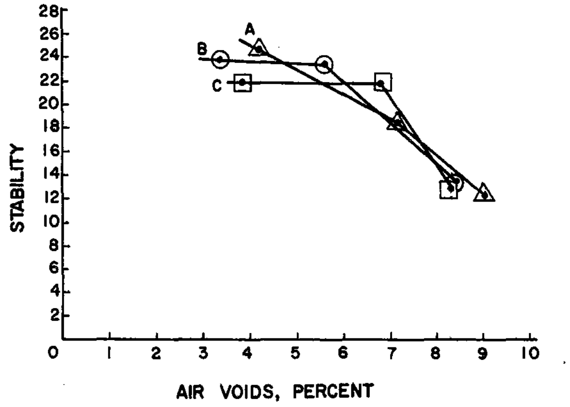
The durability of a paving mixture (resistance to weathering and the abrasive action of traffic) is dependent upon density (Figure 18) (7, 12, 13, 14). Although the absolute volume of air is not as important as the degree of interconnection of air voids, the dependence upon absolute density is nevertheless evident. The interconnected voids permit the intrusion of air and water into the pavement which in turn oxidizes the asphalt thereby creating a stiff and more brittle mix. These stiff and brittle mixes often fail as they can no longer withstand the repeated deflections imposed by traffic.

The increase in viscosity after four months of service expressed in terms of relative viscosity is shown in Figure 20 for several test sections. Although several asphalts were used which age at different rates and the pavements were subjected to various environments, the trend of increased relative viscosity with high air voids is evident.

If the volume and interconnection of voids in a pavement is such that water is transmitted to the base course, the pavement may fail due to loss of strength in the base material.

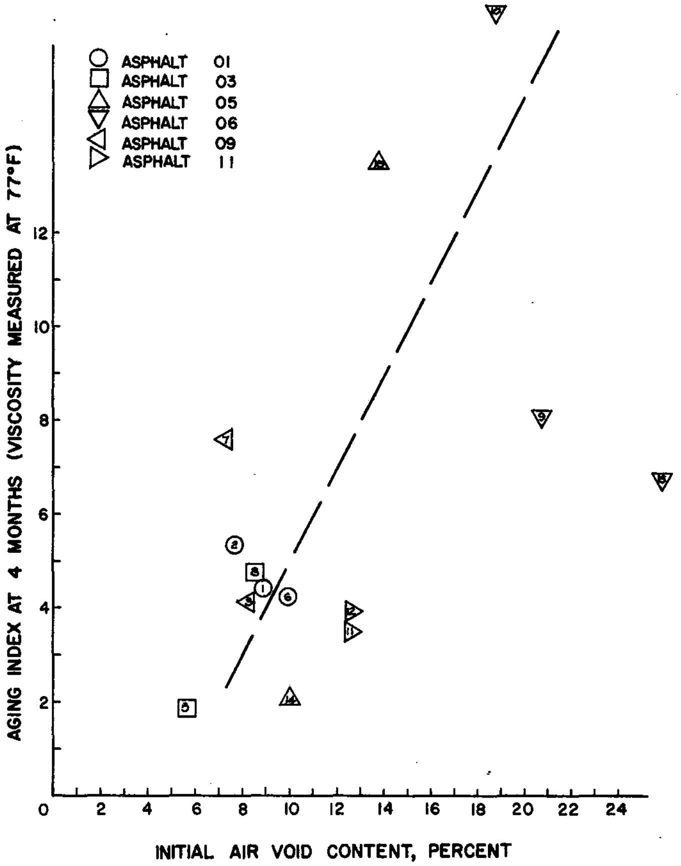
Tensile Strength

The presence of voids in asphalt concrete has essentially two effects on tensile strength. First, the presence of voids reduces the effective cross section of the stressed area and thereby reduces its potential strength; and second, the voids act as inducers of highly localized



RELATIONSHIP BETWEEN HVEEM STABILITY AND AIR VOID CONTENT FOR VARIOUS FIELD COMPACTION EFFORTS (CHILDRESS)

FIGURE 19



RELATIONSHIP BETWEEN ASPHALT HARDENING AND INITIAL AIR VOID CONTENT

FIGURE 20

TABLE 9 STABILITY AND COHESIOMETER VALUES OF MIXTURES COMPACTED IN THE FIELD

Part I

Test Section	Compactive Effort	One Day		One Week		One Month	
		Stab	Cohes	Stab	Cohes	Stab	Cohes
Childress US 287 25-42-9	A	12	23	12	42	17	52
	B	17	46	14	82	24	85
	C	10	93	13	74	22	66
Matador US 70 25-145-8	A	*	98	12	166	R	94
	B	11	85	*	180	R	91
	C	*	72	*	107	R	139
Sherman SH 5 1-47-3	A		87	*	56	*	90
	B	30	60	*	47	*	123
	C	29	65	32	75	32	185
Cooper SH 24 1-136-3	A	15	65	19	70	21	99
	B	17	66	21	68	24	66
	C	21	73	25	79	26	116
Cumby IH 30 1-9-13	A	25	51	28	43	24	48
	B	28	86	27	72	29	49
	C	30	73	28	55	27	71
Clifton SH 6 9-258-7	A	*	70	*	87	*	*
	B	*	60	*	105	*	*
	C	*	75	*	110	*	*
Waco US 84 9-55-8	A	*	70	*	*	*	130
	B	*		*	*	*	114
	C	*		*	*	*	154
Robinson US 77 9-209-1	A	*	*	*	*	*	*
	B	*	43	*	*	*	*
	C	*		*	*	*	*
Milano SH 36 17-185-4	A	*	*	*	*	*	*
	B	*	*	*	*	*	*
	C	*	*	*	*	*	*
Bryan Spur 308 17-599-1	A	*	W	*	W	*	100
	B	*	W	*	W	*	133
	C	*	W	*	W	*	101
Tamina IH 45 12-110-4	A	*	32	*	17	*	104
	B	*	106	*	100	*	156
	C	*	95	*	100	*	129
Conroe FM 1485 12-1062-35	A	17	160	*	165	*	129
	B	*	125	*	100	*	135
	C	*	102	*	135	*	122
Baytown Spur 330 12-508-7	A	*	W	*	W	*	W
	B	*	W	*	W	*	W
	C	*	W	*	W	*	W
Orange SH 12 20-499-3	A	*	12	*	50	*	55
	B	*	15	*	45	*	45
	C	*	18	*	41	*	52
Bridge City IH 87 20-306-3	A	*	*	*	*	*	*
	B	*	*	*	*	*	*
	C	*	*	*	*	*	*

* Cores too short to test and give significant values

W Too weak

R Damaged Cores

TABLE 9 STABILITY AND COHESIOMETER VALUES OF MIXTURES COMPACTED IN THE FIELD

Part II

Test Section	Compactive Effort	Four Month		One Year		Two Year	
		Stab	Cohes	Stab	Cohes	Stab	Cohes
Childress	A	25	180	17	210	19	314
US 287	B	24	180	19	309	18	338
25-42-9	C	22	230	21	271	16	260
Matador	A	15	98	14	*	R	*
US 70	B	14	227	15	*	R	*
25-145-8	C	11	*	10	*	R	*
Sherman	A	*	*	*	*	*	*
SH 5	B	*	*	*	*	*	*
1-47-3	C	*	107	*	*	*	*
Cooper	A	20	*	24	*	25	215
SH 24	B	26	*	22	*	25	227
1-136-3	C	27	*	20	*	26	240
Cumby	A	24	*	22	334	23	170
IH 30	B	30	*	27	312	32	290
1-9-13	C	33	*	32	279	24	270
Clifton	A	*	248	*	*	*	*
SH 6	B	*	266	*	*	*	*
9-258-7	C	*	485	*	*	*	*
Waco	A	*	*	17	168	*	*
US 84	B	*	*	19	195	17	407
9-55-8	C						
Robinson	A	*	69	23	278	20	190
US 77	B	*	52	*	*	*	*
9-209-1	C	*	96	*	*	*	*
Milano	A	*	110	*	*	*	*
SH 36	B	*	*	*	*	*	*
17-185-4	C	*	*	*	*	*	*
Bryan	A	*	*	*	*	*	*
Spur 308	B	*	43	*	75	*	*
17-599-1	C	*	47	*	53	*	*
Tamina	A	*	167	*	*	*	*
IH 45	B	*	250	*	*	*	*
12-110-4	C	*	340	*	*	*	*
Conroe	A	*	104	17	177	*	*
FM 1485	B	*	109	18	195	*	*
12-1062-35	C	*	111	*	160	*	*
Baytown	A	*	20	*	42	*	*
Spur 330	B	*	W	*	19	*	*
12-508-7	C	*	W	*	63	*	*
Orange	A	*	94	*	*	*	*
SH 12	B	*	108	*	*	*	*
20-499-3	C	*	72	*	*	*	*
Bridge City	A	*	*	*	*	*	*
IH 87	B	*	*	*	*	*	*
20-306-3	C	*	*	*	*	*	*

* Cores too short to test and give significant values
W Too weak
R Damaged Cores

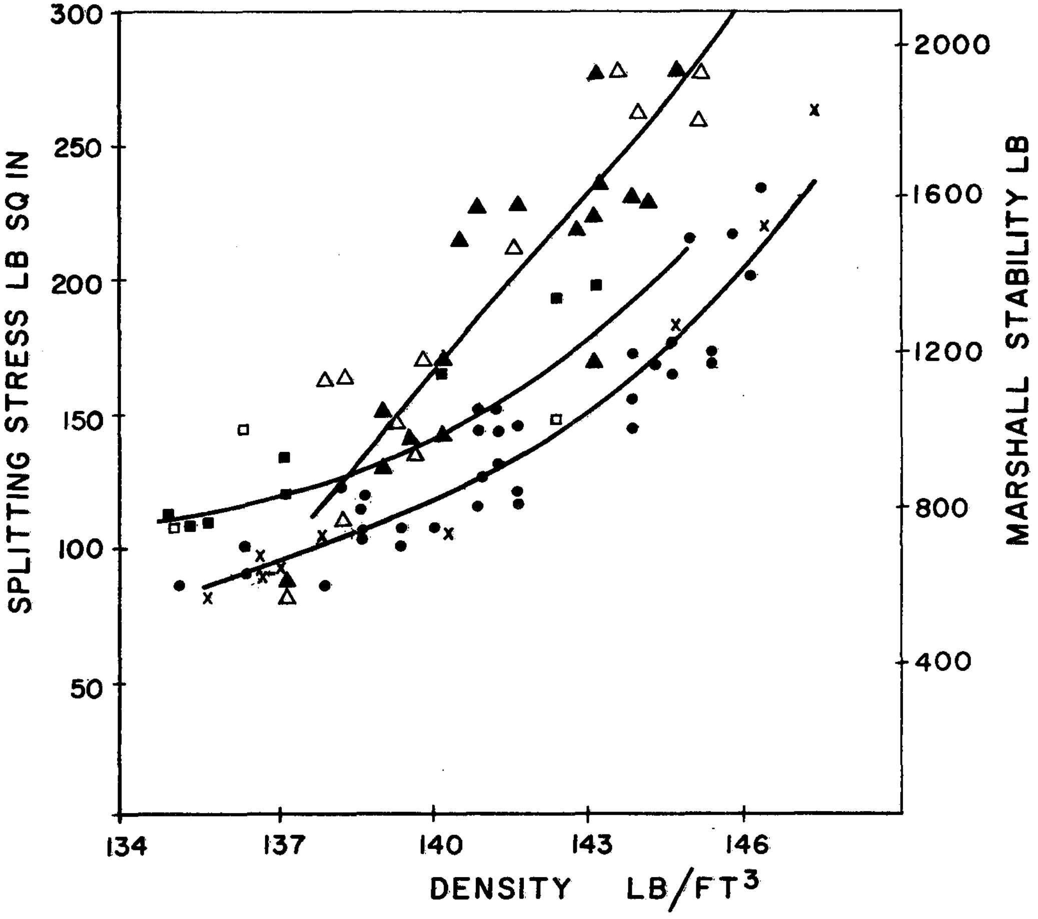
stresses (15). The magnitude of the increased stress is dependent upon the size and shape of the void which in turn is dependent primarily upon the type and amount of compaction.

Splitting tension tests performed by Livneh and Shkrlarsky (16) show that in general the strength increases with density and varies with the type of compaction (Figure 21).

Cohesiometer test results for selected projects are shown in Figures 22 and 23. These figures illustrate the trend of decrease in strength with increase in air voids. Differences in strength with different methods of compaction at similar air voids are also shown. Figure 24 illustrates the effect of air void content on the cohesiometer value for a mixture compacted in the field using various compactive efforts.

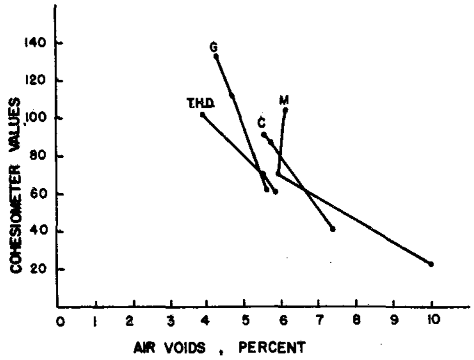
Fatigue Resistance

The importance of air void content on the fatigue behavior of asphalt concrete has been reported by Saal and Pell (17), Monismith (18), and Epps and Monismith (19) (Figure 25). These results show that high air void contents or low mixture specific gravities (densities) produce mixes with comparably short fatigue lives. These data suggest that variations in air void content create greater changes in fatigue life of coarse graded mixes than finer graded mixes. Thus, as is the case with tensile strength, both the structures or size and shape of the voids as well as their absolute volume influence the fatigue behavior of asphalt mixtures. It should be pointed out that the above description is based on results from constant stress fatigue tests. The influence of mixture density on asphalt mixture behavior under controlled strain fatigue tests is not well established.



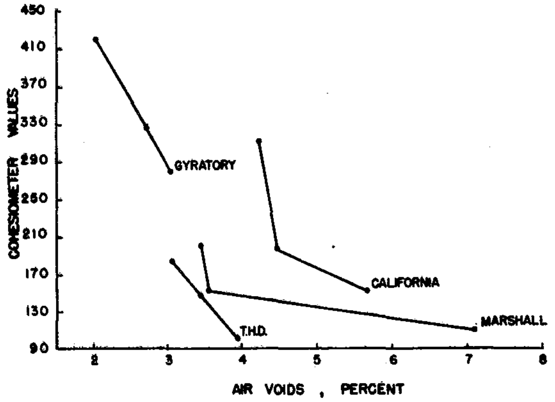
SPLITTING STRESS AND MARSHALL STABILITY OF SPECIMENS COMPACTED BY DIFFERENT METHODS (A LIUNEH AND SHKLARSKY [16])

FIGURE 21



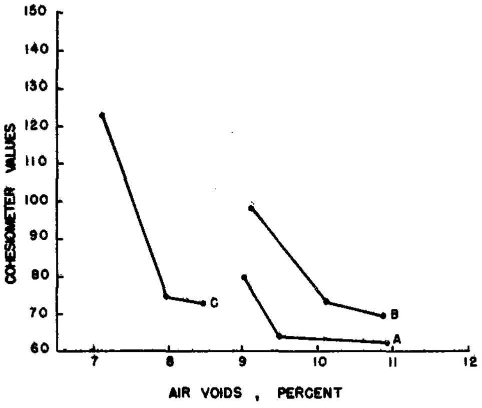
RELATION BETWEEN COHESIOMETER VALUE AND AIR VOID CONTENT FOR VARIOUS METHODS OF LABORATORY COMPACT

FIGURE 22



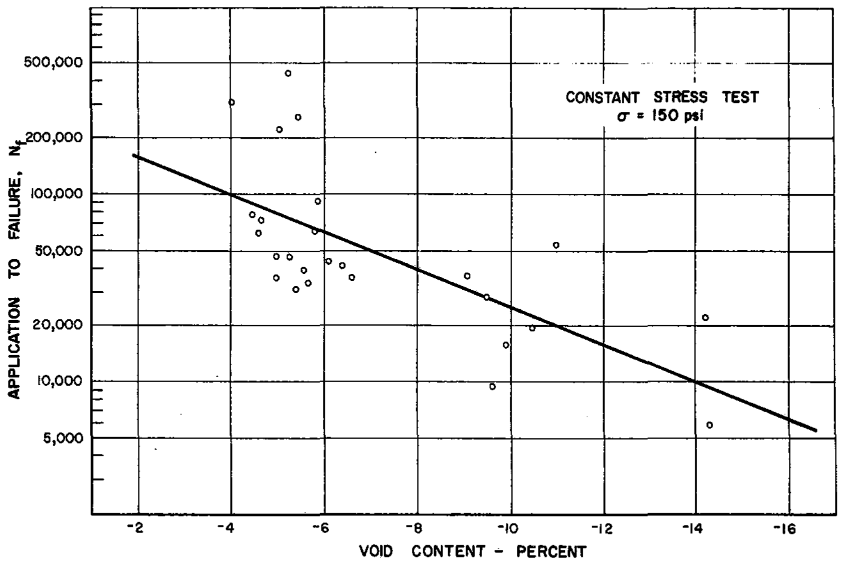
RELATIONSHIP BETWEEN COHESIOMETER VALUE AND AIR VOID CONTENT FOR VARIOUS METHODS OF LABORATORY COMPACTION-ROUNDED SILICEOUS

FIGURE 23

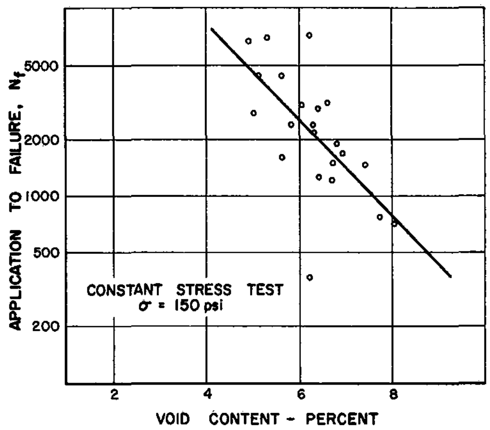


RELATIONSHIP BETWEEN COHESIOMETER VALUE AND AIR VOID CONTENT FOR VARIOUS AMOUNTS OF FIELD COMPACTION (COOPER TEST SECTION)

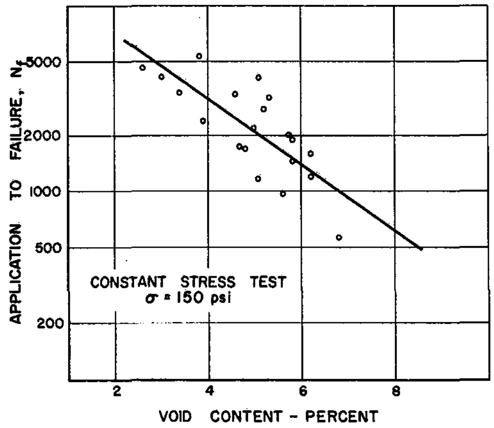
FIGURE 24



A. BRITISH STANDARD 594 GRADING - 7.9 PERCENT ASPHALT



B. CALIFORNIA FINE GRADING - 6 PERCENT ASPHALT



C. CALIFORNIA COARSE GRADING- 6 PERCENT ASPHALT

THE EFFECT OF VOIDS CONTENT ON FATIGUE LIFE (AFTER EPPS AND MONISMITH (19))

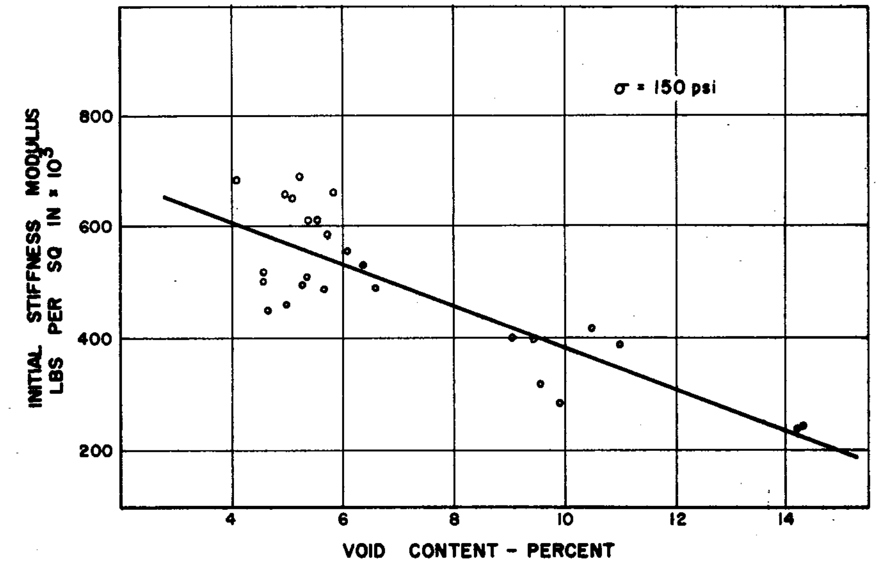
FIGURE 25

Stiffness

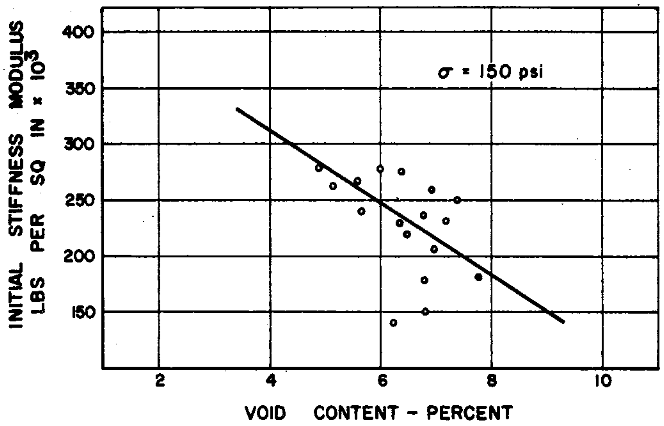
Stiffness, which is defined as the ratio of stress to strain at a particular temperature and time of loading, has been shown to be dependent upon density by Deacon (20) and Epps and Monismith (19) (Figure 26). As shown by these investigators, the stiffness increases with density suggesting that a more dense mixture results in greater load supporting capabilities of the material. Van Draat and Sommer (21) have presented an equation whereby the influence of air voids on stiffness may be estimated.

Flexibility

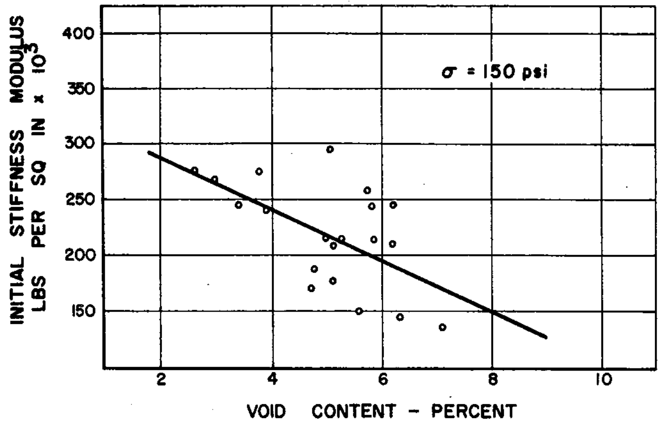
The flexibility of an asphalt paving mixture is defined as the ability of the mixture to conform to long-term variations in base and subgrade elevations. In general, those mixtures of acceptable stabilities with high asphalt contents and high air voids will produce mixtures with the greatest flexibility. This assumes the asphalt does not harden excessively.



A. BRITISH STANDARD 594 GRADING - 7.9 PERCENT ASPHALT



B. CALIFORNIA FINE GRADING - 6 PERCENT ASPHALT



C. CALIFORNIA COARSE GRADING - 6 PERCENT ASPHALT

RELATIONSHIPS BETWEEN INITIAL STIFFNESS MODULUS AND AIR VOID CONTENT - GRANITE AGGREGATE (AFTER EPPS AND MONISMITH (19))

FIGURE 26

FACTORS INFLUENCING INITIAL COMPACTION OF PAVEMENTS

The main purpose of this study is to define the factors which control the ultimate density of a pavement. The factors which control the ultimate term density have been separated for convenience into those variables which influence initial density and long term density. The factors which control the initial density will be discussed in hopes that the important variables can be recognized and separated from those variables which have a secondary effect on the compaction process.

Initial Density

The initial density of the pavement is dependent upon the compactability of the mix or the ease with which it can be compacted, the type of compaction equipment, the rolling sequence and procedure, and the timing of the compaction processes.

The compactability of a mix is dependent on material properties, mix design, subgrade support, thickness of lift, temperature of mix, weather conditions during placement, and moisture in the mix is to be determined.

Material Properties

Considerable information has been published concerning the effects of aggregates and asphalts on compaction. The effect of temperature on asphalt viscosity and therefore the influence of temperature on compactability has been reported widely. The effect of aggregate characteristics, however, will be discussed initially.

Aggregate Characteristics

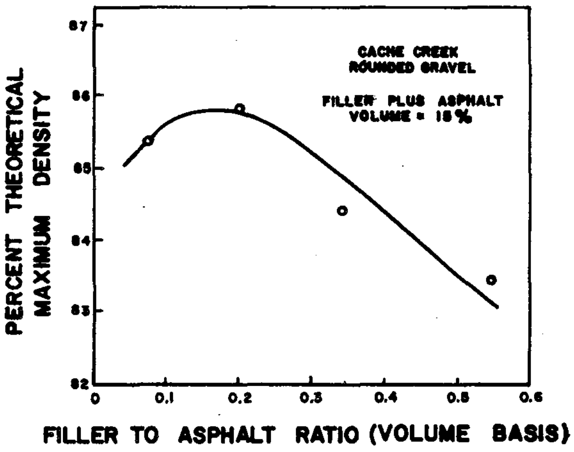
Santucci and Schmidt (22) in addition to Bahri and Rader (11) suggest that the filler-bitumen ratio influences the density of a mix for a given compactive effort. Furthermore, Santucci and Schmidt (22) suggest that an optimum filler-bitumen ratio exists for maximum density at a particular compactive effort (Figure 27).

In addition to the amount of filler present in a mix, Kallas and Krieger (23) have shown that the type of filler influences density (Figure 28). Therefore, not only the chemical characteristics of the filler or aggregate can influence compaction (24) but also its top size and grading (25).

Fromm (26) and Bright et al. (27) have reported that crushed materials are more difficult to compact than aggregates with smoother surface textures. This conclusion is supported by "pavement toughness" tests conducted by Santucci and Schmidt (24) which show that the angular rough surfaced textured granite is more difficult to displace than the rounded gravel mix compacted at the same temperature (Figure 29). Thus, as suggested by Schmidt et al. (28), mixes can be adjusted to give optimum compaction characteristics for particular compaction conditions by adjusting aggregate grading which includes filler content, size of filler and/or changing the amount of angular and/or rough textured aggregates in the mix.

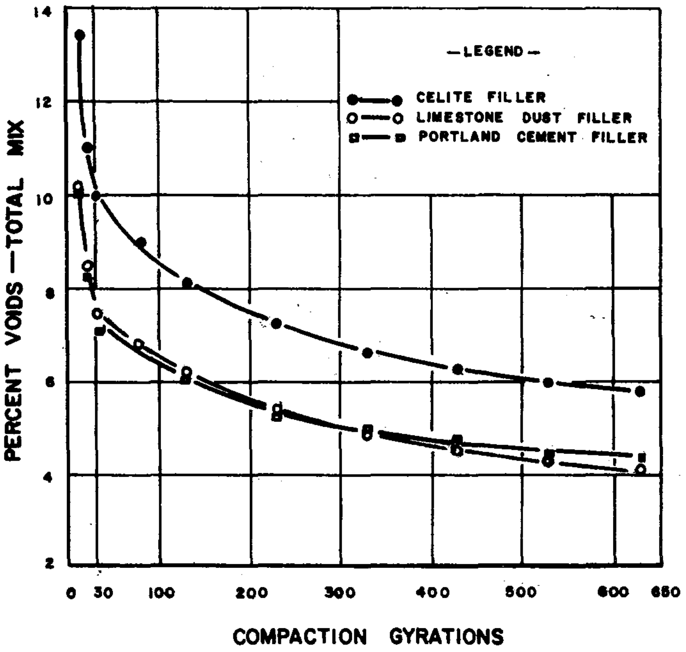
Tests performed using the Triaxial Institute Kneading Compactor suggest that aggregate gradation also influences the amount of compactive effort required to provide a given density in a mix of equal asphalt content and identical aggregates (29). These tests also illustrate the effect of aggregate surface characteristics on compaction.

Table 10 describes the type, source, grading, and maximum sizes of aggregates used in this study. Aggregate types included: siliceous

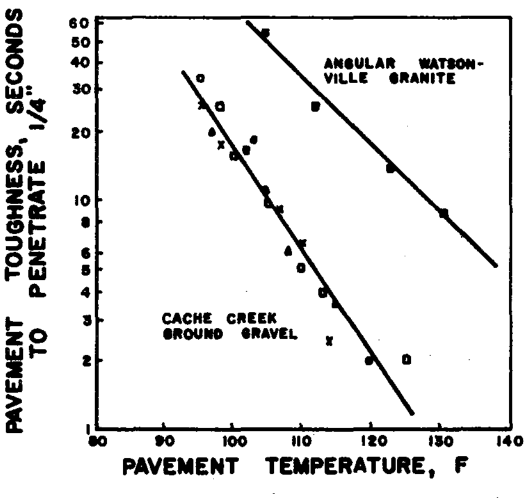


RELATION OF FILLER-TO-ASPHALT RATIO TO COMPACTIVE EFFORT.
(AFTER SANTUCCI AND SCHMIDT (22))

FIGURE 27



EFFECT OF FILLERS ON THE COMPACTION OF ASPHALT CONCRETE
(AFTER KALLAS AND KRIEGER (23))
FIGURE 28



TOUGHNESS OF CRITICAL MIXES INCREASED BY ANGULAR AGGREGATE
(AFTER SANTUCCI AND SCHMIDT (24))

FIGURE 29

TABLE 10 AGGREGATES

Test Section	Aggregate	Source	Grading (Plant)				Maximum Size (100% pass)
			+#4	-#4	-#40	-#200	
Childress US 287 25-42-9	Siliceous CRS - 65% Fines - 35%	Local - Tucker Pit	36%	64%	16.6%	1.2%	1/2
Matador US 70 25-145-8	CRS - 18.9% Int. - 23.2% Fine - 21.1% Sand - 36.8%	Local - Campbell Pit	45.5%	54.5%	23.1%	6.1%	7/8
Sherman SH 5 1-47-3	3/8 crushed LS. - 65% Field Sand - 25% Conc. Sand - 10%	Crushers Inc. Bill Ridenour	43%	57%	25.5%	2.2%	1/2
Cooper SH 24 1-136-3	L.S.S. - 18.1% Field Sand - 21.9% Pea Gravel - 55.2%	Bridgeport, Tex. Local - Backus Pit Van Pit Seegeville	38%	62%	31.1%	2.4%	1/2
Cumby IH 30 1-9-13	L.S.S. - 18.1% Field Sand - 21.9% Pea Gravel - 55.2%	Bridgeport, Tex. Local - Backus Pit Van Pit Seegeville	39%	61%	27.5%	3.9%	1/2
Clifton SH 6 9-258-7	D Rock - 32% Fine Sand - 28.3% Conc. Filler - 35%		41.8%	58.2%	19%	3.4%	1/2
Waco US 84 9-55-8	Flex Base with one Course SuF Treat	Neilson Pit	41.7%	58.3%	21.7%	2.7%	1/2
Robinson US 77 9-209-1	River Gravel - 65% Field Sand - 20% Conc Sand - 15%	Neelleys Pit Simons Pit	49.2%	50.8%	22.1%	7.2%	1/2
Milano SH 36 17-185-4	RSA - 70% RSA - 30% Rock Asphalt	Alcoa Uvalde Rockdale	5.4%	94.6%	18.4%	6.9%	3/8
Bryan Spur 308 17-599-1	RSA - 75% L.S. screen - 20% Field Sand - 5%	Alcoa - Rockdale Georgetown	1.3%	98.7%	22.8%	4.3%	3/8
Tamina IH 45 12-110-4	Iron Ore - 70% L. S. - 30%	Iron Ore Champion Pit (1-A)	44.8%	55.2%	26.0%	3.4%	1/2
Conroe FM 1485 12-1062-35	Iron Ore Field Sand	Gaylord Construction Company	42.0%	58.0%	26.1%	2.7%	1/2
Baytown Spur 330 12-508-7	Limestone - 33% Sand Coarse - 30% Sand Fine - 37%		46.0%	54%	28.6%	1.5%	1/2
Orange SH 12 20-499-3	L.S. - 35% Vido Field S. - 24% Helm's Screening- 41	Tex. Const. Mat. Burnet & Eagle Smith Pit-Vidor	35.7%	64.3%	25.9%	2.3%	1/2
Bridge City IH 87 20-306-3	L. S. - 35% Vidov F. S. - 24% Helm's Scr. - 41%	Tex. Const. Mat. Burnet & Eagle Smith Pit-Vidor	32.4%	67.6%	27.6%	3.3%	1/2 1/2

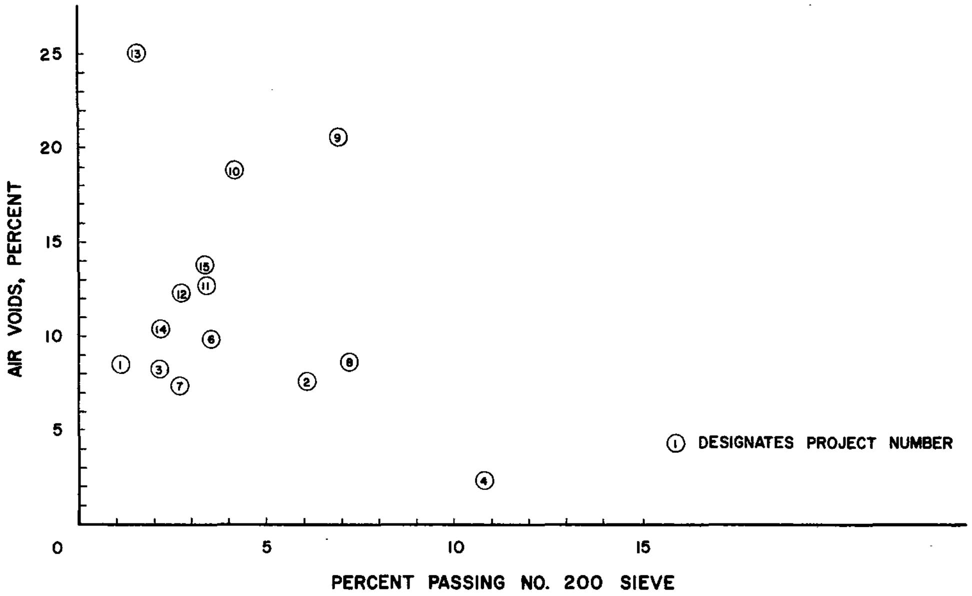
materials used on the Childress project, rock asphalt used on the Milano project (Rockdale slag aggregate plus rock asphalt), iron ore aggregate used on the Tamina and Conroe projects, slag used on the Milano and Bryan project, and limestone which was the predominant type of aggregate. A more complete description of the grading can be found in Appendix B.

Although the maximum size of aggregate ranged from 5/8-inch to No. 4, the majority of the aggregates had a maximum size of 3/8-inch. Since the literature suggested that the amount of filler and fine aggregate may affect the compaction of a pavement, these factors were plotted against initial density for the various projects (Figures 30 and 31). Although these figures do not present clear trends, the density is shown to decrease with increase in percent of the material passing the No. 4 sieve for the normal compactive effort on these particular projects (Figure 31).

Asphalt Characteristics

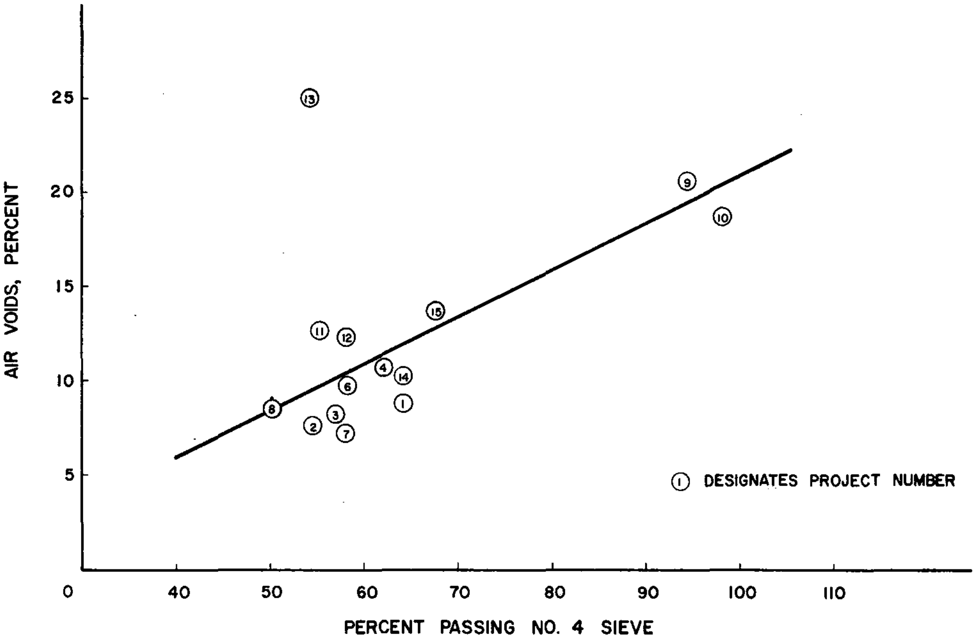
The characteristics of an asphalt that affect the compactability of asphalt concrete include the relationship between temperature and viscosity. Different asphalts can have widely different temperature-viscosity relationships and thus; although two mixes contain the same aggregate, aggregate grading, and asphalt content and are compacted at the same temperature with identical equipment, the resulting density can vary widely depending on the asphalts used.

The importance of compacting asphalt concrete at high temperatures has been advocated by numerous authors. Laboratory studies conducted by McLeod (7), Kiefer (8), Bahri and Rader (11), and Parker (30) show that density increases with the temperature of the mixture at the time of compaction. Studies conducted using field equipment, including those



RELATIONSHIP BETWEEN INITIAL AIR VOID CONTENT AND PERCENT PASSING NO. 200 SIEVE

FIGURE 30

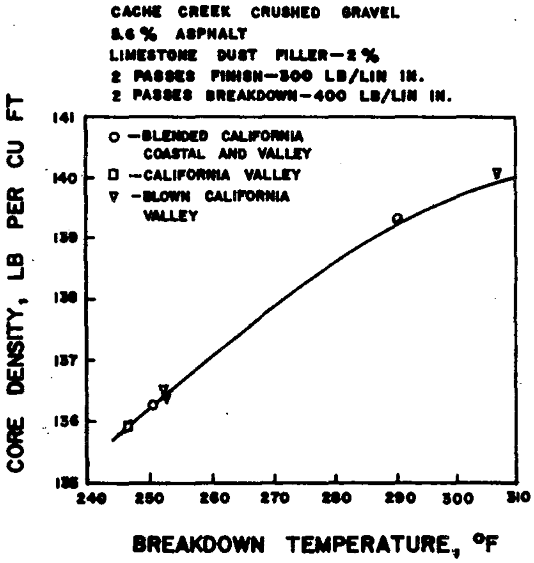


RELATIONSHIP BETWEEN INITIAL AIR VOID CONTENT AND PERCENT PASSING NO. 4 SIEVE

FIGURE 31

conducted by Santucci and Schmidt (24), Bright et al. (27), Schmidt et al. (28), and Swanson et al. (31) suggest that density increases with temperature at the time of compaction (Figure 32). Several of these reports, (7, 11, 24, 27) indicate that an optimum temperature exists at which a particular mix can be compacted to its greatest density with a given compactive effort. This observed behavior can be explained with the aid of terms used by Schmidt et al. (31). A mix is said to be "overstressed" when an increase in compactive effort causes a drop in density of the mix. Similarly, a mix is said to be "understressed" when an increase in compactive effort results in higher densities. Thus optimum compactive effort exists for a particular mix at a given temperature. If the temperature is reduced in the case of an overstressed mix the viscosity of the asphalt is increased and the mix can become understressed and compacted to a high density. Similarly, a temperature increase may aid compaction of an understressed mix provided the mix does not become unstable and behave as if it is overstressed.

The temperature of the mix, as suggested above, controls the viscosity of the asphalt which influences the deformation of the mix under load, thereby causing it to seek a stable dense arrangement and remain in a dense packing. Because of this as well as other factors, the State of Michigan requires the contractor to control the mix temperature as delivered to the construction site, within $\pm 20^{\circ}$F. This specification allows the engineer to select a viscosity of the asphalt within the range of 75 to 200 Saybolt Furol Seconds which the Michigan engineers believe results in optimum density for their particular mixes, compaction equipment, climatic conditions, and length of hauls (32).



RELATIONSHIP BETWEEN DENSITY AND TEMPERATURE OF ASPHALT CONCRETE DURING BREAKDOWN ROLLING (AFTER SCHMIDT ET AL (28))

FIGURE 32

The asphalt source, type, content, viscosity, and penetration for the various test sites is shown in Table 11. The viscosity and penetration of the asphalts recovered after service for various lengths of time are shown in Table 12.

Temperatures were recorded during breakdown and final rolling operations. The data presented in Table 5 indicate that breakdown rolling occurred at temperatures below 175°F on 6 of the 15 test sites and final rolling was started when the temperature was below 175°F on all projects. This information indicates that pavements in Texas are compacted at a temperature at which densification is not easily obtained.

The asphalt viscosity at the compaction temperature is given in Table 13. These data were obtained from temperature viscosity data on the original asphalt and therefore is not absolutely correct as some hardening occurred during the mixing operation. The relationship between air void content and asphalt viscosity at breakdown rolling for pavements studied in this project is shown in Figure 33. The general trend of increased density with low viscosity is not evident for these projects as too many other variables control the compactability of the mix. However, a comparison of the Cooper and Cumby projects suggests that the temperature of the pavement during compaction is very important. These pavements were constructed with the same asphalt type, approximately the same asphalt contents, aggregates, aggregate gradations, and compactive effort. Thus the major variable between these two projects is the pavement temperature. As shown in Figure 33 the Cooper project which was compacted at the lowest temperature resulted in the lowest density.

TABLE II ASPHALTS

Test Section	Asphalt Source	Asphalt Type	Asphalt Content, Percent		Original Viscosity			Original Penetration at 77°F, dmm
			Design	Extraction	megapoise 77°	stokes X10^3 140°	stokes 275°	
Childress US 287 25-42-9	01	AC-20	5.0	5.26	2.2	2.410	3.9	65
Matador US 70 25-145-8	01	AC-20	5.0	5.4	2.7	3.151	4.08	61.5
Sherman SH 5 1-47-3	09	AC-20	5.8	6.5	1.59	2.597	5.14	75
Cooper SH 24 1-136-3	03	AC-20	4.8	4.6	2.06	3.02	5.10	69.2
Cumby IH 30 1-9-13	03	AC-20	5.1	5.1	2.28	3.10	5.19	68.8
Clifton SH 6 9-258-7	01	AC-10	4.7	5.6	2.0	2.788	3.98	55
Waco US 84 9-55-8	09	85-100	4.8	4.95	0.87	2.28	5.12	75
Robinson US 77 9-209-1	03	AC-20	4.5	4.71	1.5	2.89	6.41	74
Milano SH 36 17-185-4	06	AC-20	6.75	6.77	3.6	3.349	4.32	39
Bryan Spur 308 17-599-1	06	AC-20	6.2	6.1	2.16	3.188	4.55	45
Tamina IH 45 12-110-4	06	AC-10	4.6	4.41	.77	1.672	3.1	85
Conroe FM 1485 12-1062-35	11	AC-20	4.75	5.2	1.8	3.888	5.12	53
Baytown Spur 330 12-508-7	11	85-100	5.2	5.4	.72	1.529	3.41	72
Orange SH 12 20-499-3	05	AC-20	5.0	5.2	5.4	4.186	5.15	37
Bridge City IH 87 20-306-3	05	AC-20	5.0	5.3	5.4	4.186	5.15	37

TABLE 12 PROPERTIES OF RECOVERED ASPHALTS

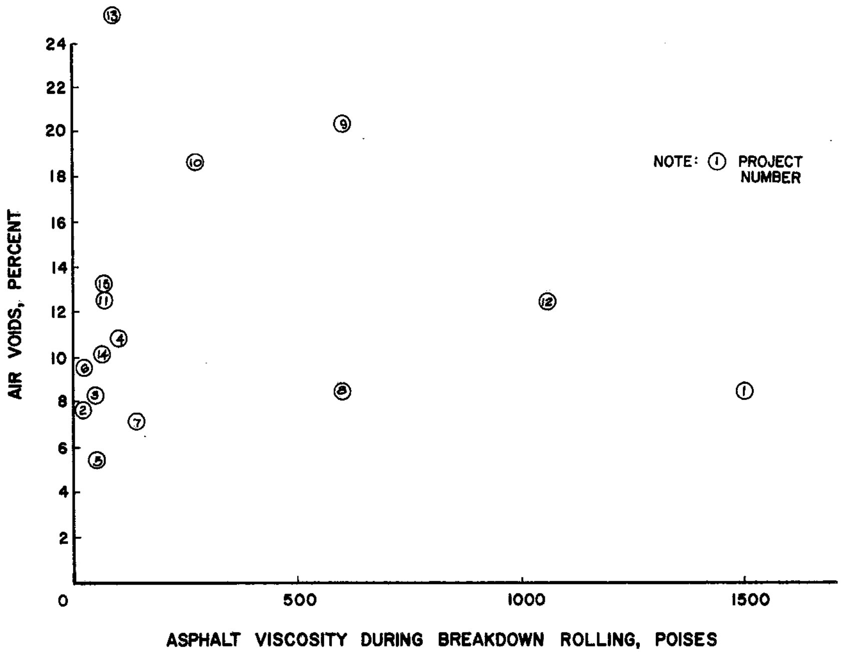
Test Section	Age of Sample	Viscosity			Penetration at 77°F, dmm
		77°F Megapoise*	140°F Stokes X10^3	275°F Stokes	
Childress US 287 25-42-9	original	2.2	2.4	3.9	57.3
	1 day	9.6			31.3
	1 week	9.8			35.0
	1 month	3.0			50.1
	4 months	9.6	16.5	6.45	29
Matador US 70 25-145-8	original	2.7	3.151	4.08	61.5
	1 hour	3.0			
	1 day	1.97			55.3
	1 week	2.32			80
	1 month	6.80			41.5
	4 months	14.20	14.7	6.46	29.0
Sherman SH 5 1-47-3	original	1.59	2.596	5.14	75
	1 hour	1.36			94
	1 day	3.20			54.3
	1 week	2.56			54.5
	4 months	6.60	6.784	6.68	42.3
Cooper SH 24 1-136-3	1 hour	3.60	4.37	5.56	45
Cumby IH 30 1-9-13	original	2.6	2.788	3.98	68.8
	1 hour	4.7	6.932	6.72	45.3
	1 day	7.8			
	1 week	7.8			
	1 month	10.6			
	4 months	10.7	14.527	6.55	39
Clifton SH 6 9-258-7	original	2.6	2.788	3.98	55.4
	1 day	7.8			38.0
	1 week	7.8			33.7
	1 month	10.6			30.5
	4 months	10.9	14.527	6.55	29
Waco US 84 9-55-8	original	.87	2.280	3.98	75.3
	1 day	2.5			33.5
	1 week	7.2			
	1 month	5.76			39.0
	4 months	7.0	9.82	7.99	35
	1 year	6.6			38.5
Robinson US 77 9-209-1	original	1.5	2.890	6.41	74.0
	1 day	3.79			51.5
	1 week	5.00			42.8
	1 month	6.8			41.0
	4 months	7.0	16.04	9.81	
Milano SH 36 17-185-4	original	3.00	3.349	4.32	39.6
	1 day	10.9			27.2
	1 week	8.7			30.1
	1 month	16.0			21.5
	4 months	24.0	13.86	7.06	
Bryan Spur 308 17-599-1	original	2.16	3.188	4.55	45.3
	1 day	19.6			18.8
	1 week	17.9			19.5
	1 month	20.0			17.3
	4 months	6.8	6.01	6.05	30
	1 year				
Tamina IH 45 12-110-4	original	.77	1.672	3.1	85.3
	1 day	2.36			55.6
	1 week	1.74			64.6
	1 month	6.00			51.7
	4 months	2.8	4.16	4.33	55
Conroe FM 1485 12-1062-35	original	1.8	3.88	4.53	52.6
	1 day	7.4			38.0
	1 week	5.7			43.5
	1 month	6.0			37.6
	4 months	6.9	15.68	10.7	31
	1 year	7.8			35
Baytown Spur 330 12-508-7	original	.720	1,529	3.41	72.0
	1 day	2.68			55.3
	1 week	4.26			36.2
	1 month	4.00			38.5
	4 months	4.84	4,230	4.81	38
	1 year	11.0			29.0
Orange SH 12 20-499-3	original	5.4			37.3
	1 day	8.70			20.8
	1 week	4.40			80.0
	1 month	15.6			22.5
	4 months	10.8	6,054	5.68	29
	1 year				
Bridge City IH 87 20-306-3	original	5.40	4,186	5.13	37.3
	1 day	24.0			15.3
	1 week	11.0			22.5
	1 month	52.0			12.0
	4 months	73.0	33,460	9.05	38
	1 year				

* Viscosity at shear rate of 5×10^{-2} sec^{-1}

TABLE 13 ASPHALT VISCOSITY DURING BREAKDOWN ROLLING*

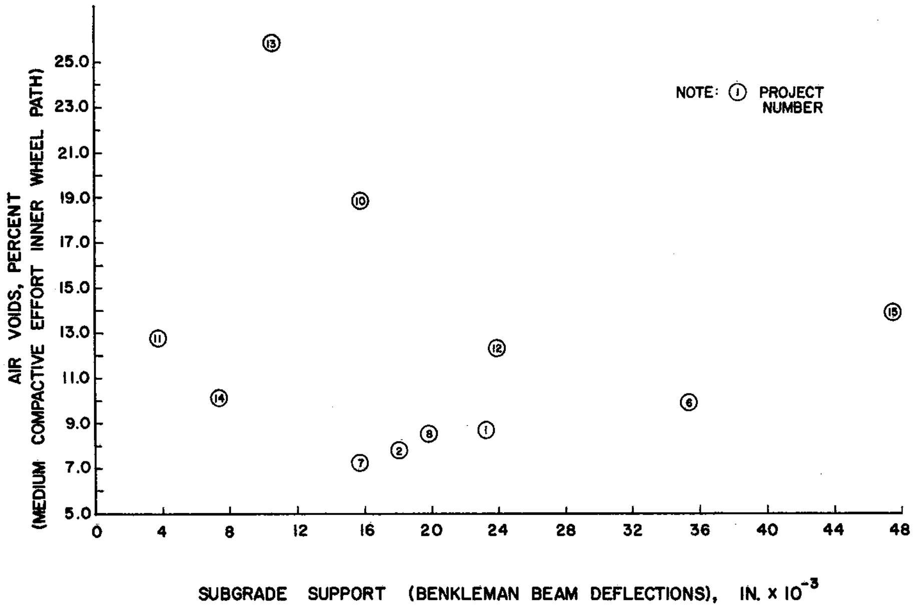
Test Section	Temperature, °F		Viscosity, Poises		Air Voids, Percent
	Breakdown Rolling	Final Rolling	Breakdown Rolling	Final Rolling	
Childress US 287 25-42-9	145	125	1,500	10,000	8.7
Matador US 70 25-145-8	225	145	17.3	1,370	7.7
Sherman SH 5 1-47-3	200	135	48	3,540	8.3
Cooper SH 24 1-136-3	155	75	92	$2.1x10^6$	10.9
Cumby IH 30 1-9-13	205	100	44	180,000	5.5
Clifton SH 6 9-258-7	220	150	18.8	1,060	9.9
Waco US 84 9-55-8	180	135	140	3,300	7.4
Robinson US 77 9-209-1	160	130	600	7,200	8.5
Milano SH 36 17-185-4	160	145	600	2,100	20.8
Bryan Spur 308 17-599-1	170	135	270	4,600	18.8
Tamina IH 45 12-110-4	185	145	50	1,700	12.7
Conroe FM 1485 12-1062-35	155	135	1,100	5,800	12.3
Baytown Spur 330 12-508-7	180	100	90	74,000	25.9
Orange SH 12 20-499-3	200	170	58	340	10.0
Bridge City IH 87 20-306-3	200	165	62	680	13.8

*Values extrapolated from Temp-Viscosity curve



RELATIONSHIP BETWEEN INITIAL AIR VOID CONTENT AND ASPHALT VISCOSITY DURING BREAKDOWN ROLLING

FIGURE 33



RELATIONSHIP BETWEEN INITIAL AIR VOID CONTENT AND SUBGRADE SUPPORT

FIGURE 34

From the above discussion it is evident that both the aggregate and asphalt influence the compactability of a mix. The gradation, shape, surface texture, and mineralogical composition of the aggregate as well as the type and amount of asphalt influence the resulting density of a particular mix for a certain compactive effort. The temperature of the mix during rolling affects the asphalt viscosity which affects the mixture compactability. Since both aggregate and asphalt characteristics vary widely, it becomes difficult to predict the compactability of a given mix before it is actually placed on the roadway under the prevailing environmental conditions.

Subgrade Support

Although the effect of subgrade support on compaction has been widely mentioned, only a few studies have considered its effect. Work performed by Swanson et al. (31) indicates that the effect of stiffness of base support on compaction of a two-inch bituminous concrete mat is small, although the harder bases give slightly higher densities and stabilities. On the other hand, work reported by Graham et al. (33) and discussed by Kari (34) and Marker (35) suggests that the supporting capacity of the material on which the asphalt concrete mat is compacted is important. Data collected in this project relating pavement density (Appendix A) with subgrade support (Table 3) do not indicate a trend (Figure 34).

Lift Thickness

A 1957 report (36) indicated that 35 out of 50 highway agencies specify a maximum lift thickness for surface course work. Specified maximum thickness for surface courses ranged between one inch and 3½ inches with the majority of the agencies reporting either 1½ inch or 2 inches.

A recent trend has been evolving whereby thick lifts are being placed and compacted successfully. The benefits claimed include better compaction due to greater heat retention of the mat (especially in cold weather) and the ability to compact on an otherwise marginal subgrade.

In specifying lift thickness the engineer must be aware of the maximum size of aggregate. Benson (37) has indicated that the maximum size aggregate permissible in a hot-mix asphalt pavement is a function of the thickness of the layer to be placed and the maximum size which can be efficiently handled by the lay-down equipment. The maximum size (smallest standard sieve or screen with 100 percent passing) should not exceed 2/3 of the layer thickness. For surface courses, since it is often desirable to obtain a smooth surface texture, the maximum size should not exceed ½ of the course thickness.

The ratio of the maximum size of the aggregate to lift or course thickness is given in Table 14. In all tests sites this ratio is below one-half, this is not considered to be an important variable in this project.

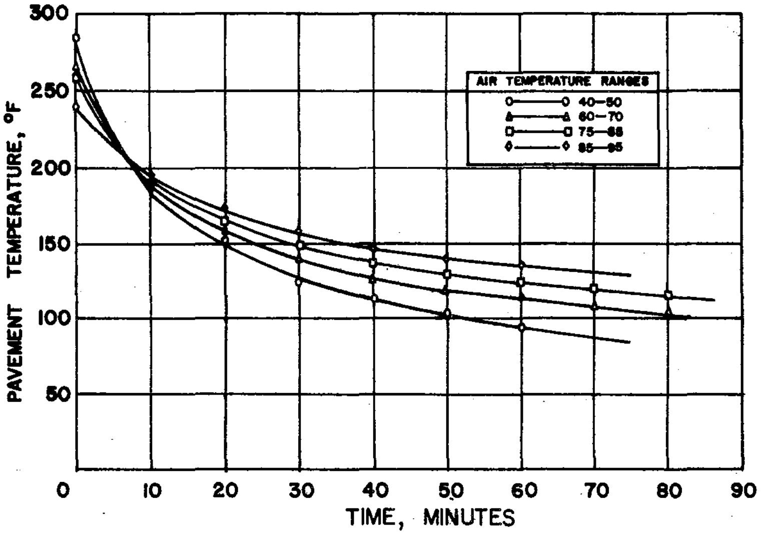
Weather Conditions

The effect of weather on compaction is primarily manifested in its effect on the cooling rate of the asphalt concrete. The cooling rate of the pavement as suggested in studies conducted by Barber (38) on pavements undergoing daily variations in temperature can be related to readily accessible meteorological data. These data include air temperature, wind velocity, and solar radiation.

Cooling curves for 1¼ inch lifts were reported by Serafin and Kole (39) for various air temperature ranges (Figure 35). As shown for these

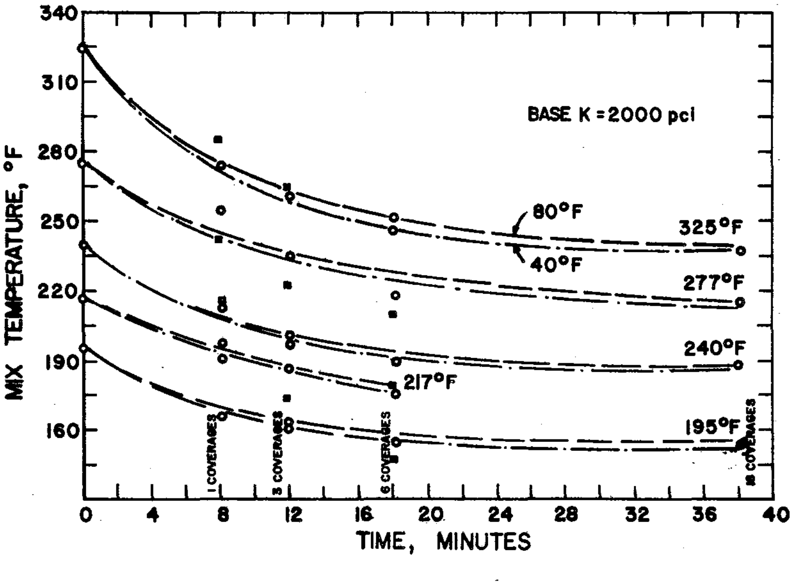
TABLE 14 RATIO OF MAXIMUM SIZE OF AGGREGATE TO LIFT THICKNESS

Test Section	Maximum Size of Aggregate	Range of Lift Thickness	Average Lift Thickness	Ratio of Maximum Aggregate Size to Lift Thickness	Pavement Density, Percent Air Voids
Childress US 287 25-42-9	3/8 inches	1.13-1.76	1.45	.26	8.7
Matador US 70 25-145-8	5/8 inches	0.99-1.69	1.34	.47	7.7
Sherman SH 5 1-47-3	3/8 inches	1.11-1.59	1.35	.28	8.3
Cooper SH 24 1-136-3	No. 4	1.05-1.69	1.37	.14	10.9
Cumby IH 30 1-9-13	No. 4	1.41-1.81	1.61	.12	5.5
Clifton SH 6 9-258-7	3/8 inches	0.92-1.36	1.14	.33	9.9
Waco US 84 9-55-8	3/8 inches	1.25-1.46	1.36	.28	7.4
Robinson US 77 9-209-1	3/8 inches	1.01-1.43	1.22	.31	8.5
Milano SH 36 17-185-4	No. 4	0.60-0.94	0.77	.24	20.8
Bryan Spur 308 17-599-1	No. 4	1.16-1.56	1.36	.14	18.8
Tamina IH 45 12-110-4	3/8 inches	1.06-1.51	1.29	.29	12.7
Conroe FM 1485 12-1062-35	3/8 inches	1.08-1.44	1.26	.30	12.3
Baytown Spur 330 12-508-7	3/8 inches	0.89-1.17	1.03	.36	25.9
Orange SH 12 20-499-3	3/8 inches	1.115-1.590	1.35	.28	10.0
Bridge City IH 87 20-306-3	3/8 inches	.460-1.325	0.89	.42	13.8



PAVEMENT COOLING CURVES OF BITUMINOUS CONCRETE WEARING COURSES MIXTURE FOR VARIOUS RANGES OF AIR TEMPERATURES.
(AFTER SERAFIN AND KOLE (39))

FIGURE 35



TEMPERATURE DECAY VERSUS TIME FOR STEEL WHEEL COMPACTED SPECIMEN
(AFTER SWANSON ET AL (31))
FIGURE
FIGURE 36

particular lift thicknesses the pavement temperature is below 200°F after 10 minutes regardless of the initial and air temperature. Cooling curves obtained in the laboratory on two inch mats have been reported by Swanson et al. (bl) Figure 36. Additional information obtained by the Texas Highway Department (40) reports temperature drops of from 40 to 70°F for 1 to 1.5 inch mat thicknesses after 3 to 7 passes of compaction rollers while Corlew and Dickson (41) indicate a 50°F loss may occur in as little as 4 minutes for a 1.5 inch mat.

As is evident by these cooling rates; it is important that compaction equipment follow the lay down machine as closely as possible for thin lifts, and cool weather construction; as the temperature drop increases the resistance to compaction by increasing the viscosity of the asphalt. Thus the rate at which a pavement cools becomes important with regard to the compactive effort needed to achieve a specified density.

The data collected in this study show that the pavements cooled a significant amount even though the air temperature was high (Table 5) in most projects. Therefore, this rapid cooling rate is primarily due to factors other than weather conditions (Table 5).

Since Parker (30) and Nijboer (42) suggest that rolling below a temperature of 175°F is not effective in obtaining adequate density for asphalts of 85-100 penetration or harder, the equipment should be compacting the pavement as soon as the mix will not shove under the weight of the equipment. As stated previously the speed of application of compaction equipment is especially critical for thin lift.

In addition to the previously mentioned air temperature and lift thickness, the cooling rate of asphalt concrete placed on a roadway is a function of the temperature of the "base" material on which the asphalt concrete is to be compacted and a function of the wind velocity. Thin lifts placed on cool base materials will cool rapidly and thus become difficult to compact. High wind velocities especially if the air temperature is low will decrease the asphalt concrete pavement temperature very rapidly. Humidity will also have an effect on the cooling rate of a pavement. This effect is, however, probably minor compared to the previously mentioned factors.

Equipment

Various types of rollers have been used to compact asphalt concrete pavements. The advantage of using intermediate pneumatic rolling in the sequence of rolling operation has been suggested to be beneficial by Schmidt, Santucci, and Garrison (43). Work conducted by Swanson et al. (31) indicates that no advantage exists in using the pneumatic rollers except that the surface appeared more dense. Furthermore, work conducted by Serafin and Kole (39) and by the California Division of Highways suggests that no advantage is obtained by using pneumatic rolling. Arena, Shah, and Adam (44) have presented data which they state emphasize the importance of compacting pavement with pneumatic rollers having contact pressures similar to that of the rolling stock on the highway. McLeod (7) also suggests that pneumatic rolling is beneficial and even more beneficial if the tire pressure is adjusted to match the resistance of compaction of the pavement. Bodell (45) indicates that a more dense pavement resulted when pneumatic tire equipment was being used.

Kari (46) has suggested that the reason for the differences noted is the difference in the bearing capacity of the mix. Pneumatic tire rolling does not increase density for high bearing capacity mixes. On the other hand low pressure pneumatic rolling contributes a great deal to compaction if the mix has a low bearing capacity.

Steel wheeled rollers with various types of roller configurations have been used extensively for many years (33). Recently sales of tandem rollers exceed those of the three-wheel rollers suggesting that the contractors are obtaining better results with this type of machine.

Foster (47) has compiled a report of information received from questionnaires sent out by the Highway Research Board. He indicates that the equipment used for breakdown rolling is predominantly steel wheel rollers while pneumatic tired rollers are used in the majority of cases when intermediate rolling is specified. Tire pressures range from 60-90 psi. Steel wheel rollers are used to smooth the pavement during final rolling.

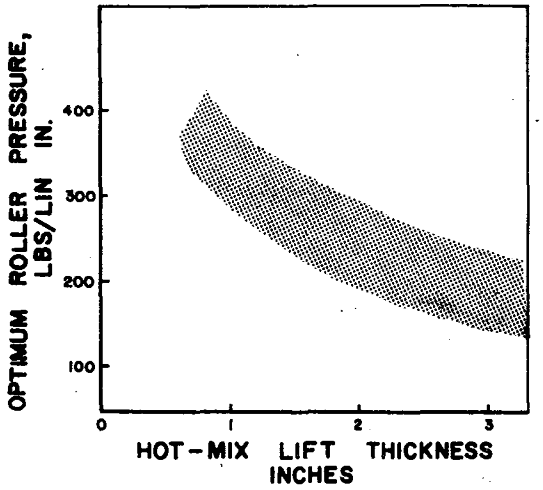
The weight of the roller as specified by the state highway departments runs from 5 to 12 tons for two-axle tandem steel rollers with 8 tons being the most popular weight. A three-axle tandem roller has been specified from 8 to 13 tons by these same highway agencies with the 10- and 12-ton rollers being the most popular. The 10-ton three-wheel roller appears to be the most popular three-wheel steel roller (47).

Rubber tired rollers have been specified over a wide range of tire pressures; however, the 60-90 psi range is the most popular among the state highway agencies.

Maximum roller speeds as specified by the state highway agencies in 1953 range from 1.5 to 3.0 m.p.h. The purpose of this requirement is to reduce the speed of the roller such that excessive displacement of the mix under the roller will be avoided. A significant number of agencies however, do not specify a maximum roller speed (36).

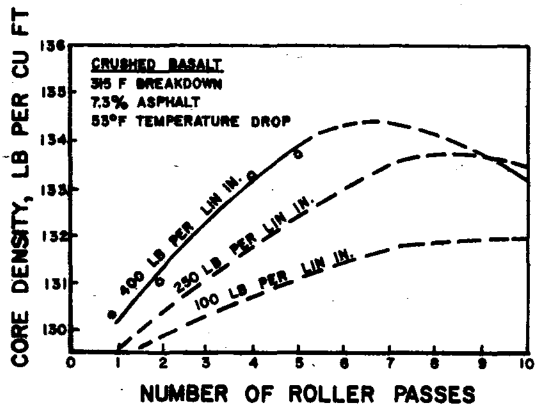
Schmidt, Kari, Bower, and Hein (28) suggest that each mix possesses an optimum roller weight and further indicate that stable mixes tolerate heavier rollers. Steel wheel diameters are also important in that large wheel diameters, allow higher pressures to be used and thus higher densities obtained before excessive shear deformation occurs. Wheels with small diameters cause excessive shear stresses at rather low loadings and will give low maximum densities. Additional data contained in this report indicate that an optimum roller pressure exits for a selected mix and a particular lift thickness (Figure 37).

The compaction equipment used for each test site is given in Table 5. The predominant type of roller used for breakdown rolling is the three-wheel ten-ton roller (14 projects). Intermediate rolling was conducted by using the eight- and ten-ton tandem roller on nine projects while pneumatic equipment was used on three test sites. Three test sites did not use intermediate rolling. Pneumatic rolling equipment was used for the final rolling operation on nine projects while steel tandem rollers were used on six projects. The roller weights and tire pressures on the pneumatic rollers and the roller weight of steel wheel rollers are representative of those used throughout the country.



RELATIONSHIP BETWEEN OPTIMUM ROLLER PRESSURE AND LIFT THICKNESS (AFTER SCHMIDT ET AL (28))

FIGURE 37



RELATIONSHIP BETWEEN CORE DENSITY AND NUMBER OF ROLLER PASSES FOR DIFFERENT WHEEL PRESSURES—STEEL WHEEL ROLLERS (AFTER SCHMIDT ET AL (28))

FIGURE 38

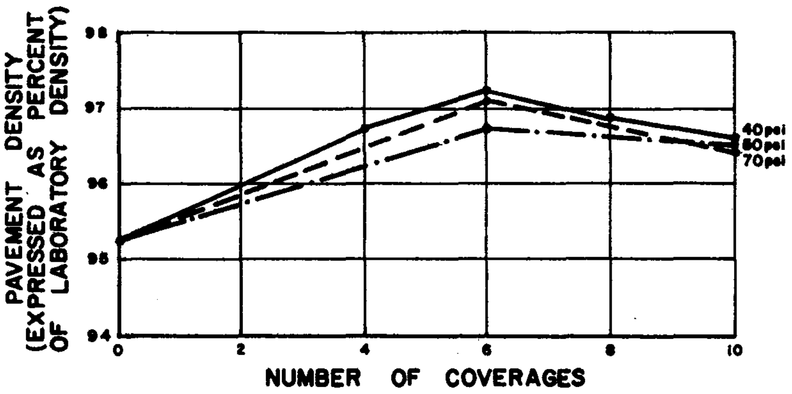
Rolling Procedures

Rolling procedures are fairly well established and an outline of procedures can be found in the Asphalt Institute Handbook (48).

As shown by Schmidt et al. (28) the optimum number of coverages depends on the individual mix and the type of rolling being used. Figure 38 shows typical density versus roller passes relationships for steel rolling and a particular mix. As shown the optimum number of passes varies with the roller weight expressed as lbs per linear inch. Data presented by Gartner et al. (49), Figure 39, on work conducted with pneumatic tired rollers suggests that optimum compaction is obtained at 6 coverages regardless of the pressures.

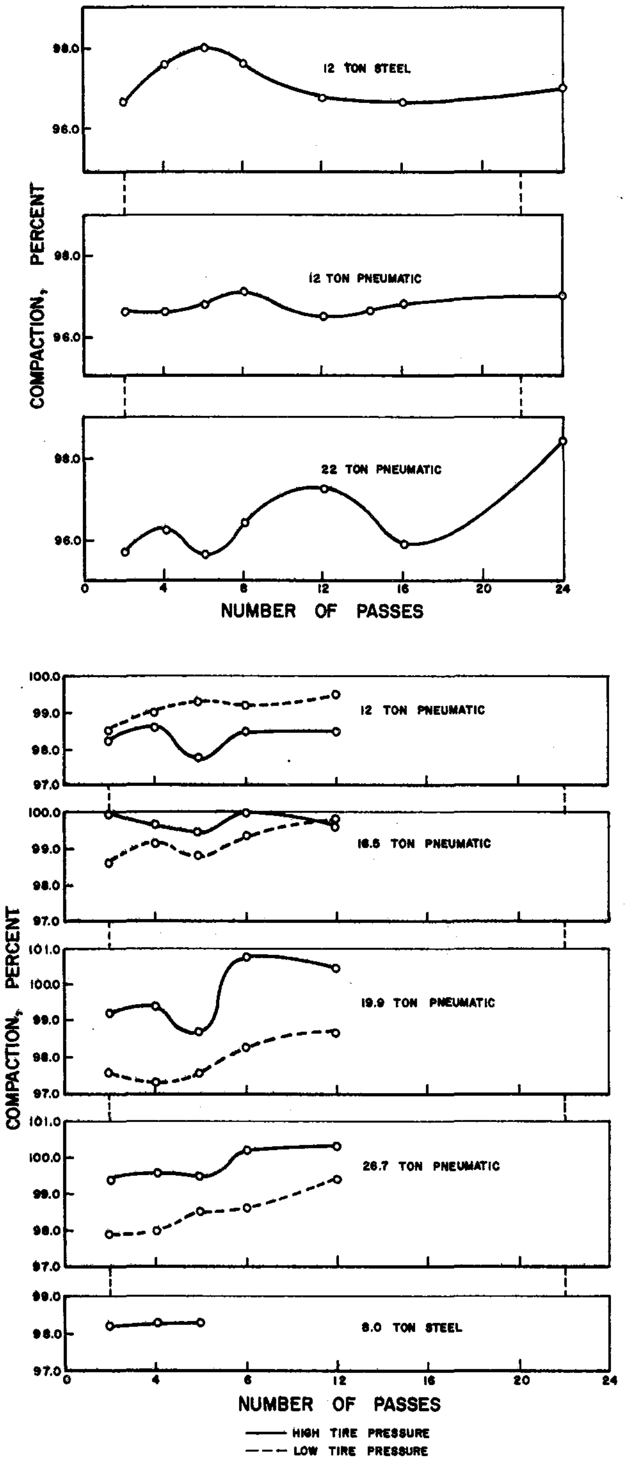
Data collected by Serafin and Kole (39) Figure 40, suggest that an increase in density up to certain numbers of passes is followed by a density decrease and then a density increase. This cycling effect which is most pronounced in the case of the 22-ton pneumatic roller is probably due to the cooling of the pavement which increases the viscosity of the asphalt and therefore increases the stiffness of the mix. Swanson et al. (31) suggest that (Figure 41) air voids continue to decrease for coverages up to 18 for the particular mix and roller used in their study.

For most mixes, 6 coverages seem to give an optimum density. However, it is recommended that field tests be performed to determine the optimum number of coverages for the particular mix, compacting equipment and rolling sequence that will be used by the contractor under weather conditions that are representative of those to be found on actual construction dates.



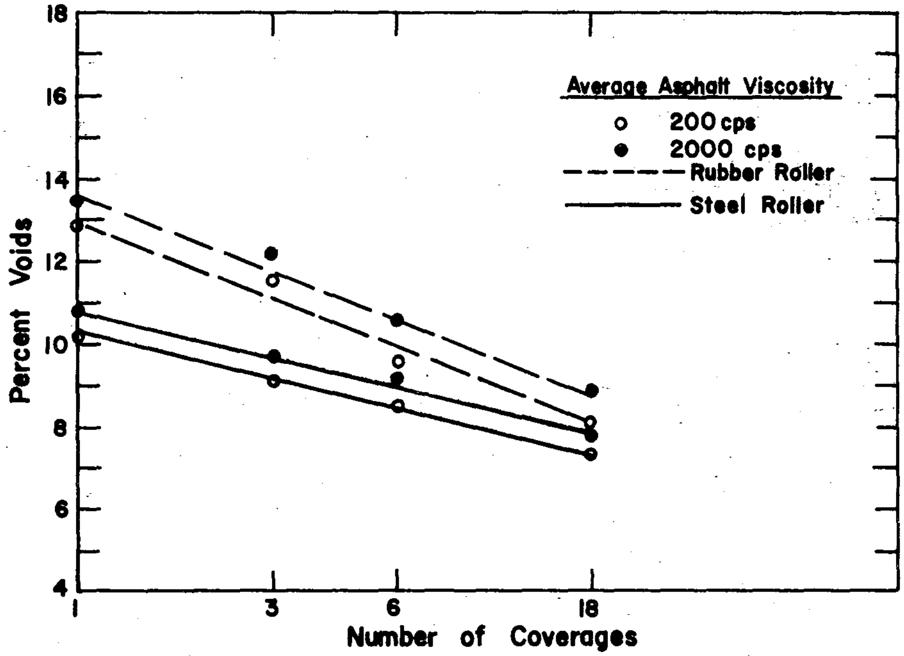
RELATIONSHIP BETWEEN PAVEMENT DENSITY
AND NUMBER OF COVERAGES FOR
PNEUMATIC-TIRED ROLLERS
(AFTER GARTNER ET AL (49))

FIGURE 39



CURVES SHOWING COMPACTION OBTAINED AT VARIOUS NUMBER OF PASSES FOR EACH TEST ROLLER USED ON 1960 AND 1961 PROJECTS WITH COMPACTION EXPRESSED AS A PERCENT OF MARSHALL DENSITY.
(AFTER SERAFIN AND KOLE (39))

FIGURE 40



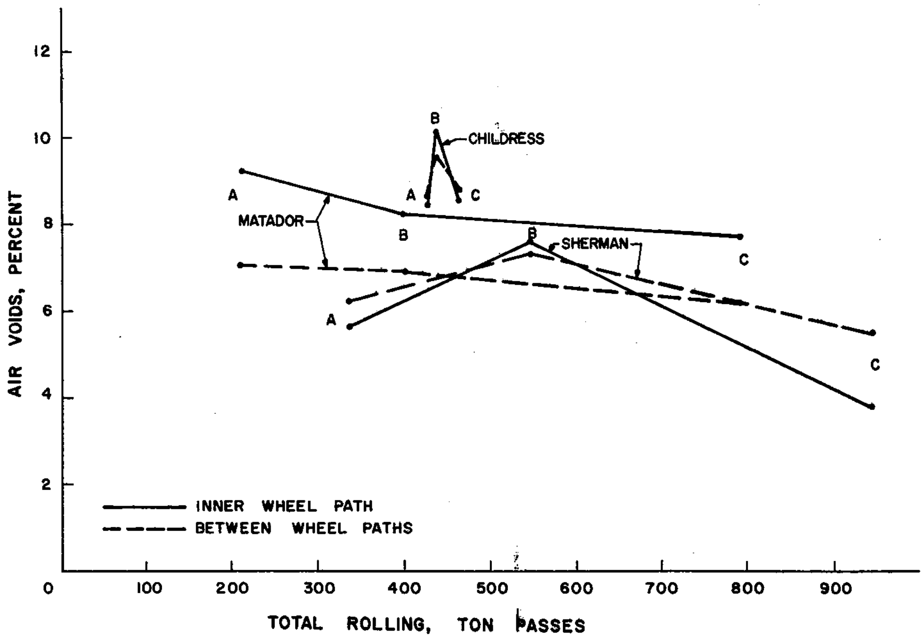
PERCENT VOIDS VERSUS NUMBER OF COVERAGES FOR STEEL AND RUBBER ROLLERS.
(SWANSON ET AL (31))

FIGURE 41

The predominant rolling procedure used in this study consisted of using steel three-wheel equipment for breakdown rolling, steel tandem rollers for intermediate rolling and pneumatic rollers for final compaction. This is quite different from the steel-pneumatic-steel rolling sequence used by most agencies. It is not known what effect this might have on the density of the pavement. As reviewed previously, the literature does not present a clear cut opinion on the proper sequence; but suggests that the sequence should depend on the individual mix. However, it is the authors' opinion that the steel-pneumatic-steel sequence gives best results on the majority of the paving mixtures.

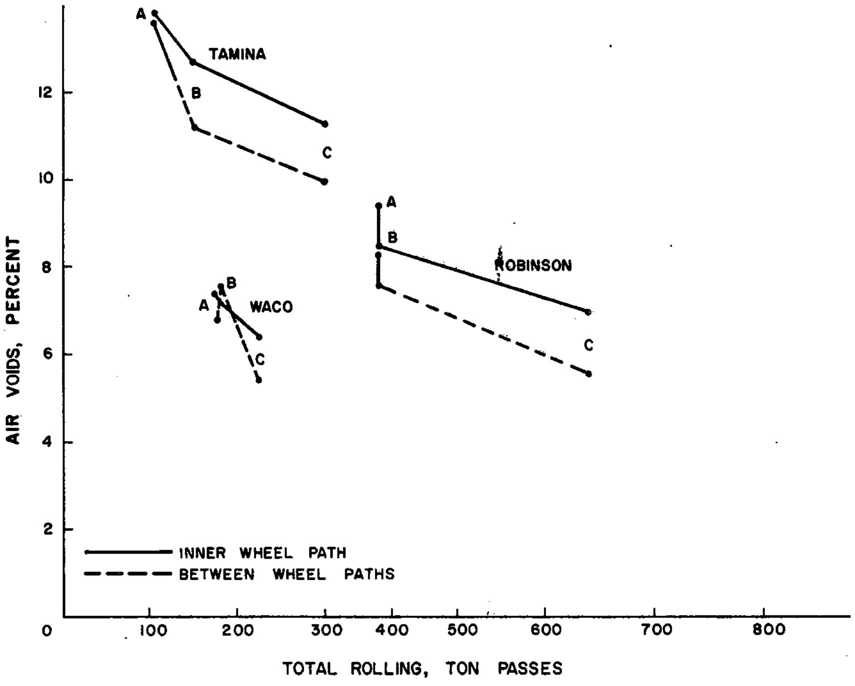
The number of passes during the breakdown rolling operation varied from 3 to 12, with 5 to 7 being the most common number of passes for normal compaction operation. Since the aggregates, asphalts, and the temperature of the paving mixtures varied so widely it is almost impossible to determine the optimum number of passes for these pavements.

In an attempt to access the effect of compactive effort on density, Figures 42 to 46 were prepared. These figures represent the air void content of the pavement as a function of total roller, ton-passes. These figures do not indicate a relationship between compactive effort and air void content among projects. Some projects indicate that optimum compaction was achieved using the normal compaction procedure while other projects indicate that additional compaction would increase the density of the pavement. Thus the compactive effort should be tailored to the individual project depending upon the characteristics of the aggregates, asphalts and conditions under which a pavement is placed.



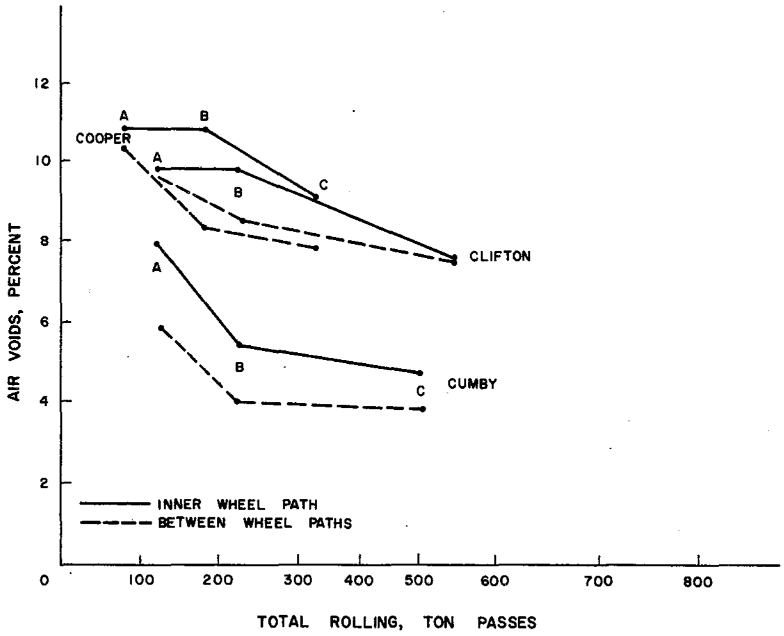
EFFECT OF ROLLING EFFORT ON AIR VOID CONTENT

FIGURE 42



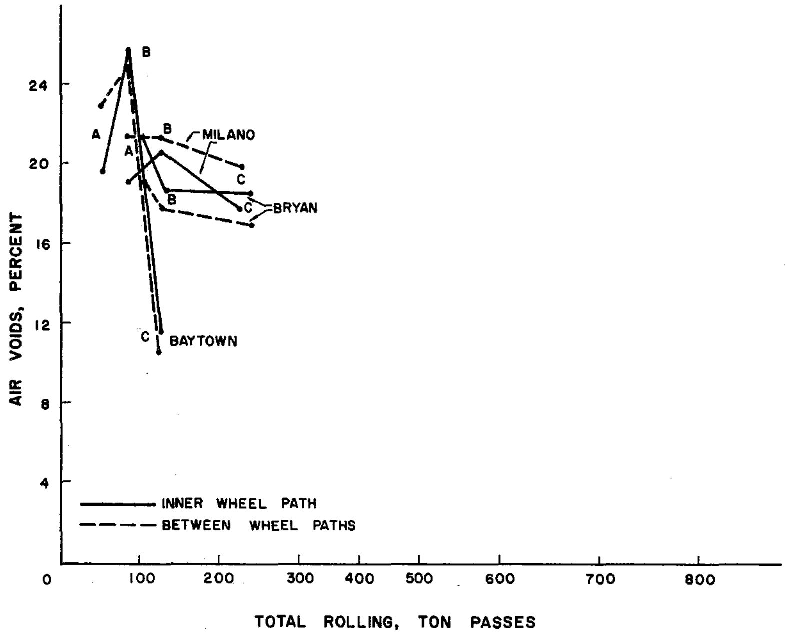
EFFECT OF ROLLING EFFORT ON AIR VOID CONTENT

FIGURE 43



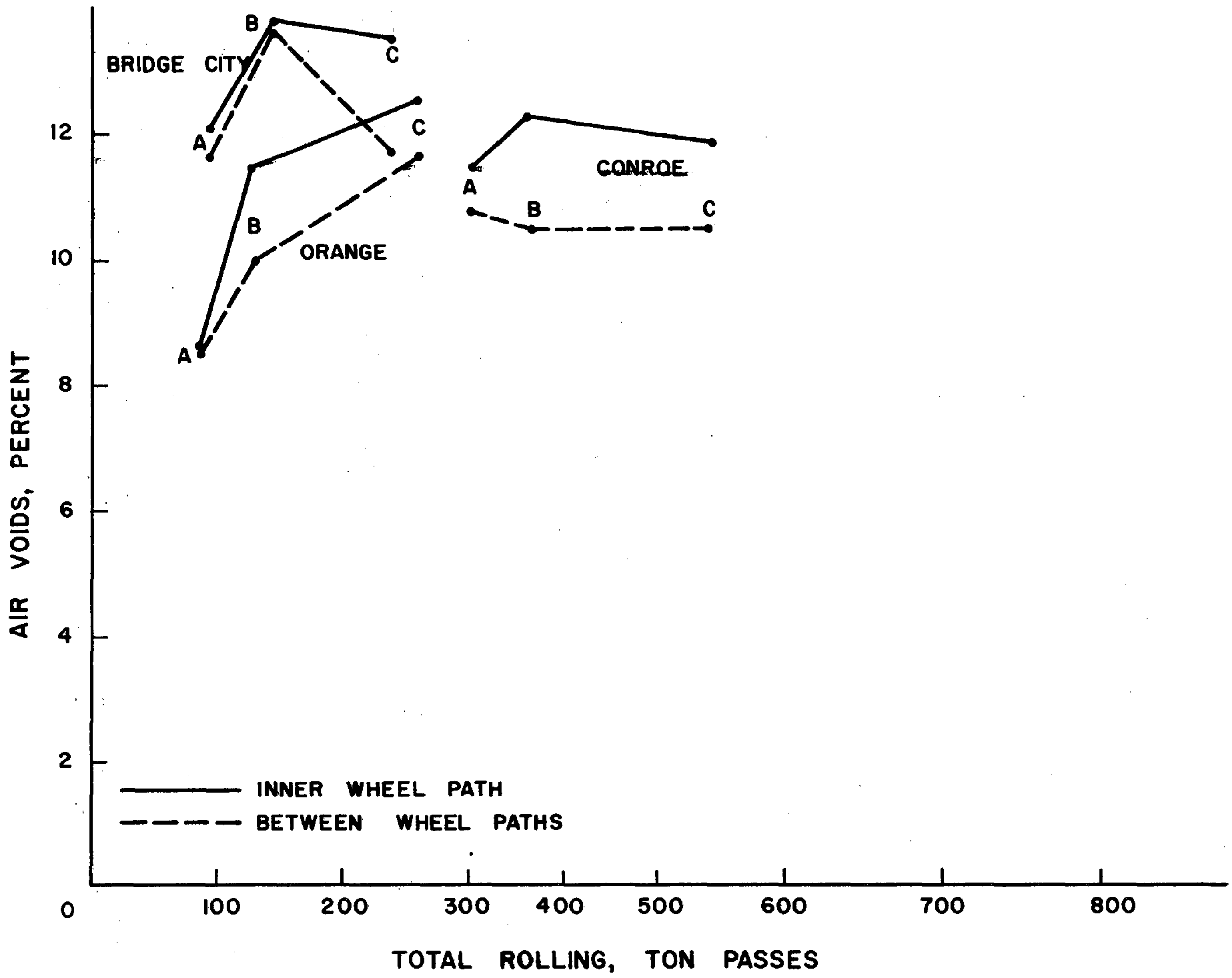
EFFECT OF ROLLING EFFORT ON AIR VOID CONTENT

FIGURE 44



EFFECT OF ROLLING EFFORT ON AIR VOID CONTENT

FIGURE 45



EFFECT OF ROLLING EFFORT ON AIR VOID CONTENT

FIGURE 46

Conclusions

1. The initial compactability of a mix has been shown, by data obtained in this study and data obtained from the literature, to be a function of aggregate surface texture, aggregate shape, aggregate mineralogical composition, aggregate gradation, asphalt type, asphalt content, subgrade support, lift thickness, temperature of the mix during compaction, subgrade temperature weather conditions, and the type and sequence of operation of the compaction equipment. All of these factors must be considered if the relative compactability of a mix is to be determined.

2. Comparison of results obtained from the Milano and Bryan sites with those of the other projects suggest that the finer mixes, or those mixes with large percentages passing the No. 4 sieve, do not compact to a high density when normal procedures are used.

3. A comparison of results obtained from the Cooper and Cumby projects indicates that pavement temperature is an important variable.

4. Many pavements constructed in Texas are compacted at a temperature at which densification is not easily obtained.

5. The cooling rates of thin lifts of asphalt concrete are very rapid; thereby making it necessary to start rolling immediately after placement and complete most of the rolling operation within a matter of minutes.

6. The rolling sequence most frequently used in Texas (steel-steel-pneumatic) differs from the procedure generally used throughout the rest of the country (steel-pneumatic-steel).

7. Initial air void contents are high for the majority of these test sections.

FACTORS INFLUENCING THE LONG TERM DENSITY OF PAVEMENTS

Variation in pavement densities with time has been studied by various groups including Kenis (5) (Figure 47) Bright et al. (27) (Figure 48), Palmer and Thomas (51) (Figure 49), Campen et al. (52), Gallaway (53), and Pauls and Halstead (54). These and other studies have suggested that the following factors influence the long term density of the pavement.

1. Amount of initial compaction

2. Material properties
   a. Aggregate absorption
   b. Aggregate surface characteristics
   c. Aggregate gradation
   d. Asphalt temperature-viscosity relationship
   e. Asphalt susceptibility to hardening

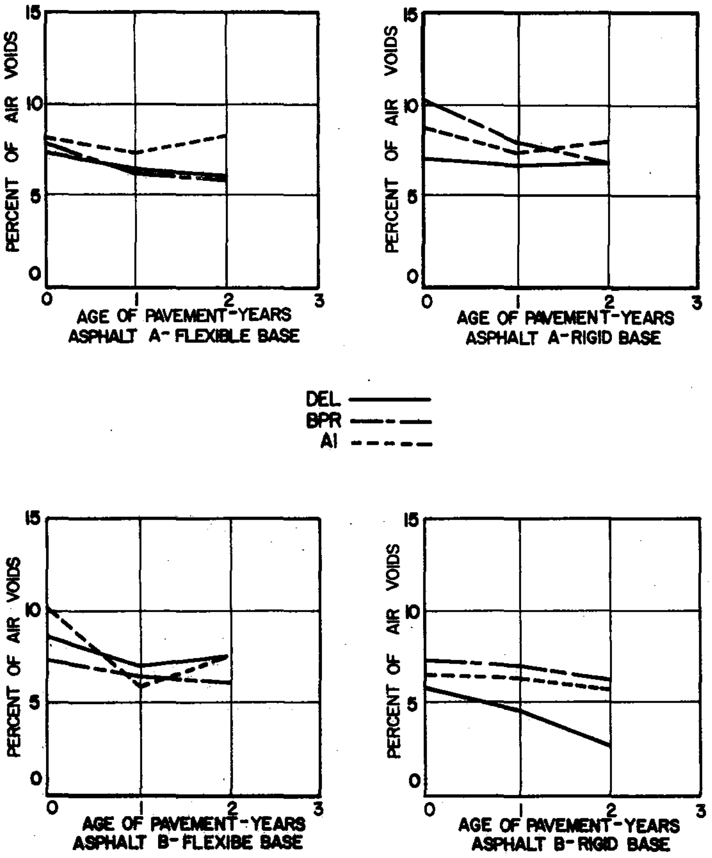
3. Mix Design
   a. Asphalt content (film thickness (12)
   b. Voids in mineral aggregate

4. Weather conditions
   a. Air temperature variations (daily and seasonly)
   b. Date of construction

5. Traffic
   a. Amount
   b. Type
   c. Distribution throughout the year
   d. Distribution in lanes

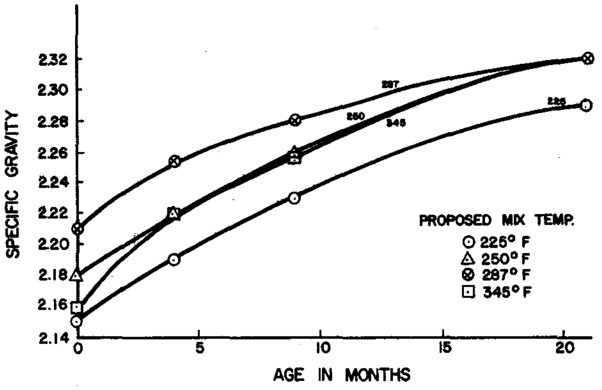
6. Pavement thickness

Results in fifteen test sites in this study are presented in Figures 50 to 79. Figures 50 to 64 represent air void contents for normal compactive efforts between the wheel path as well as the inner and outer wheel path. Figures 65 to 79 represent air void contents at the inner wheel path for the three compactive efforts. Appendix C contains information in graphical form at the inner and outer wheel path as well as



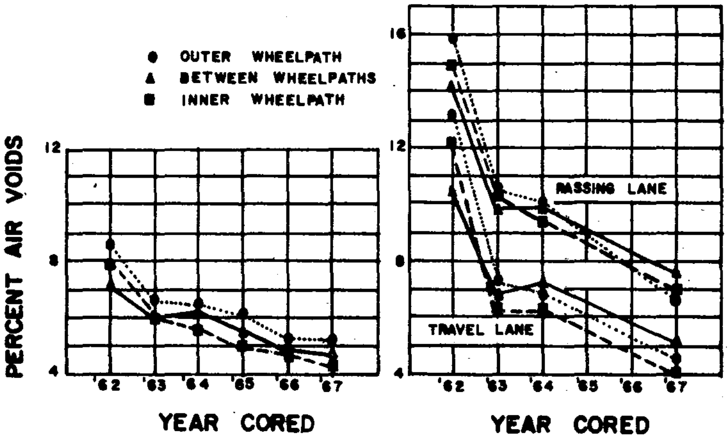
PERCENT OF AIR VOIDS, WITH YEARS OF SERVICE
(AFTER KENIS (50))

FIGURE 47

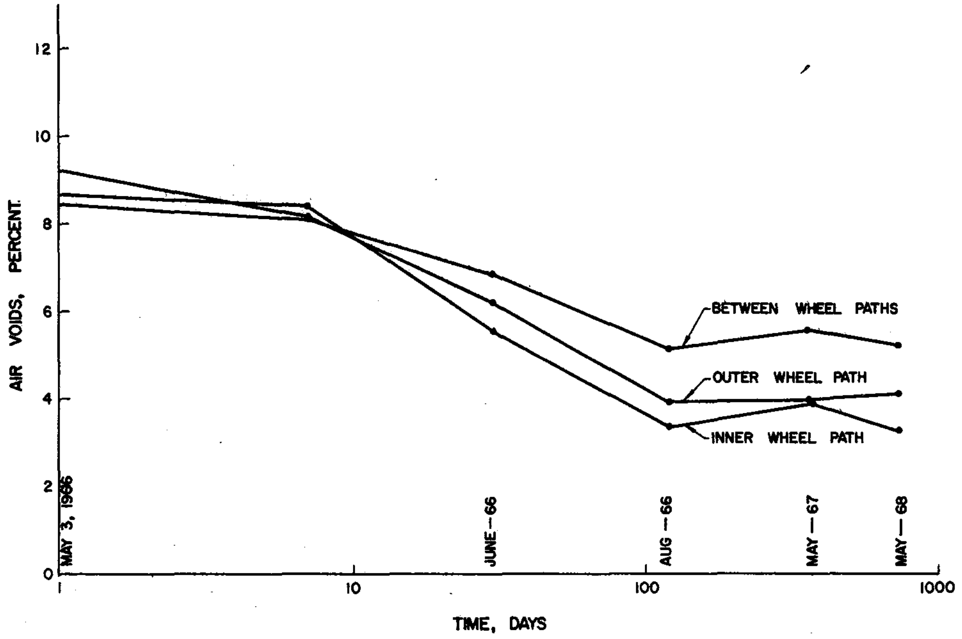


CHANGE IN SPECIFIC GRAVITY WITH AGE — SET I.
(AFTER BRIGHT et al (27))

FIGURE 48

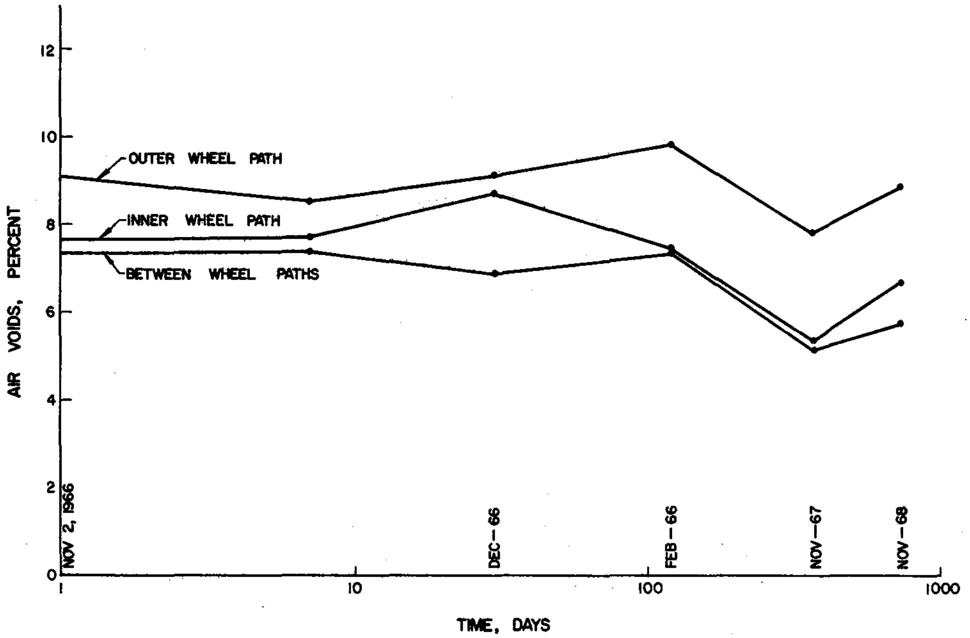


AIR VOID CONTENT CHANGE WITH TIME AVERAGING BY WHEELPATH FOR 10 STATE HIGHWAY JOBS (LEFT), AND BY LANE AND WHEELPATH FOR THRUWAY JOB 13 (RIGHT); CALCULATIONS BASED ON 1962 ASPHALT CONTENTS.
(AFTER PALMER AND THOMAS (51))
FIGURE 49



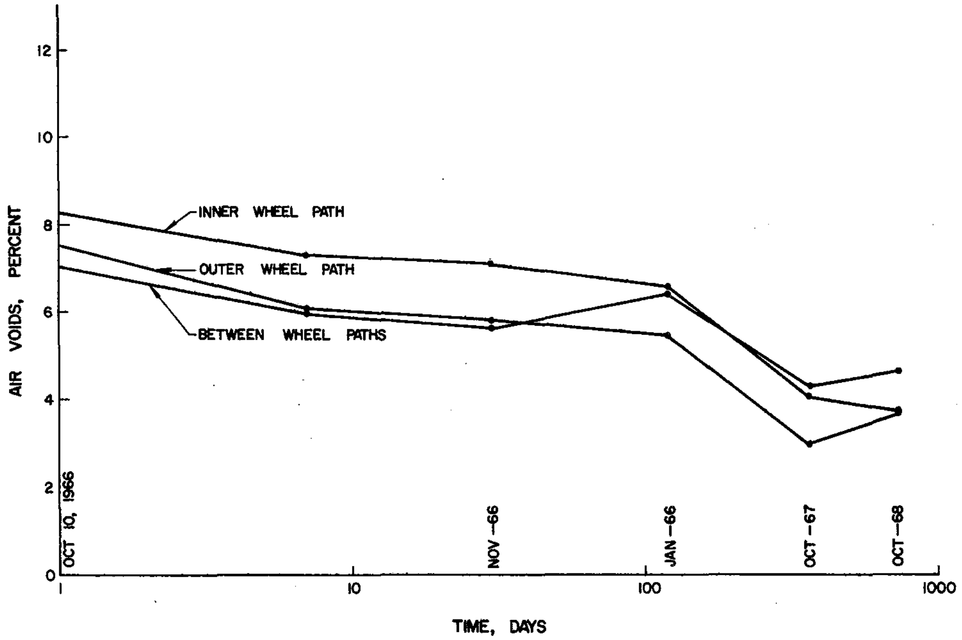
PAVEMENT DENSIFICATION WITH TIME AT VARIOUS LOCATIONS IN THE PAVEMENT
(CHILDRESS TEST SECTION)

FIGURE 50



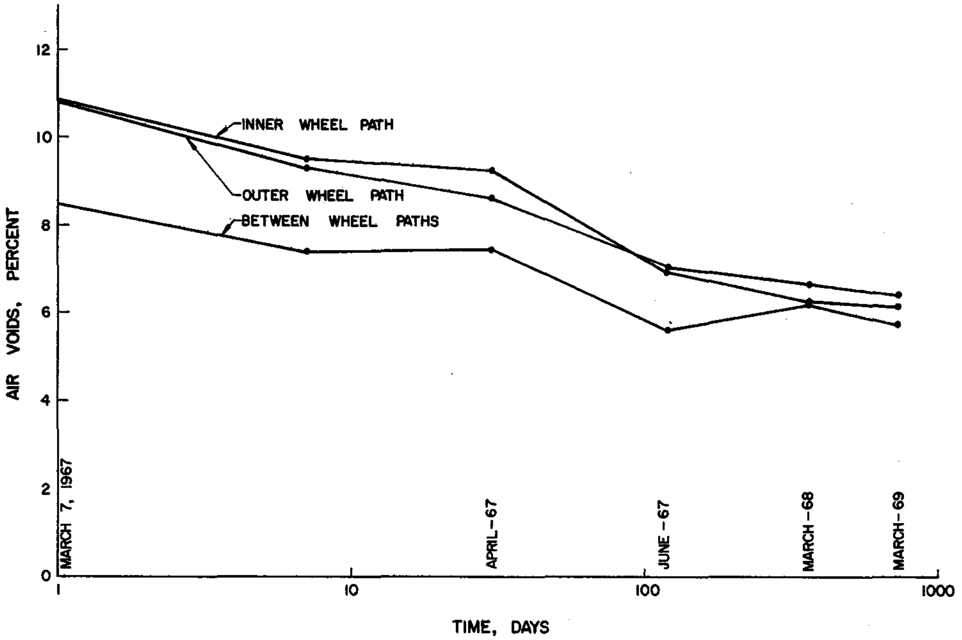
PAVEMENT DENSIFICATION WITH TIME AT VARIOUS LOCATIONS IN THE PAVEMENT
(MATADOR TEST SECTION)

FIGURE 51



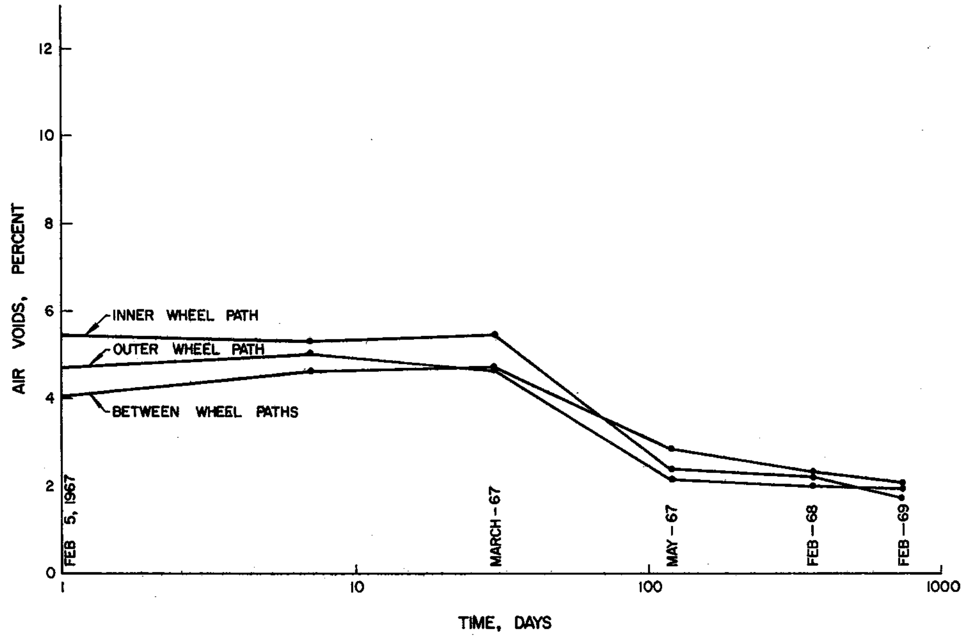
PAVEMENT DENSIFICATION WITH TIME AT VARIOUS LOCATIONS IN THE PAVEMENT (SHERMAN TEST SECTION)

FIGURE 52



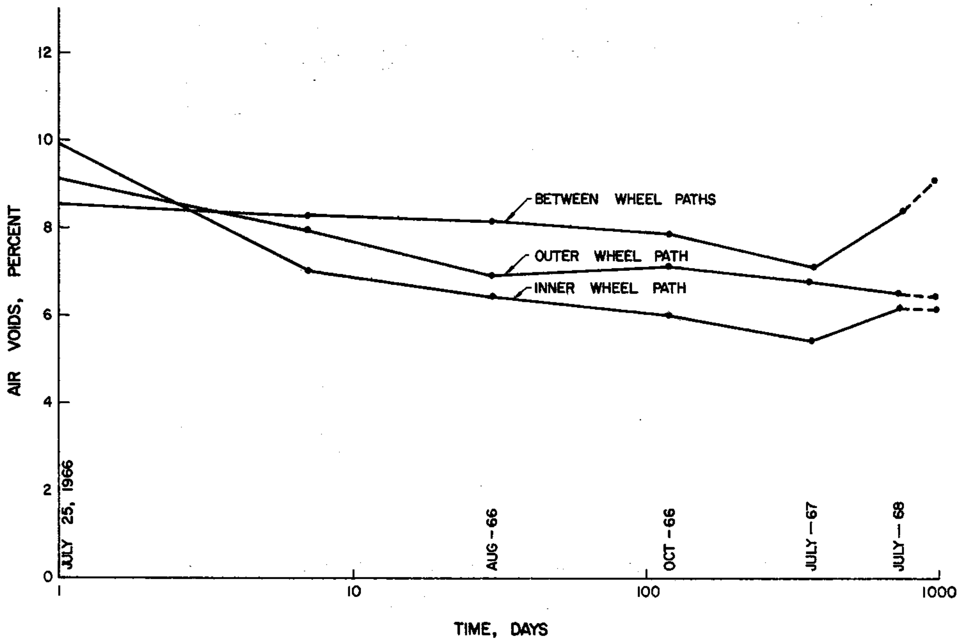
PAVEMENT DENSIFICATION WITH TIME AT VARIOUS LOCATIONS IN THE PAVEMENT (COOPER TEST SECTION)

FIGURE 53



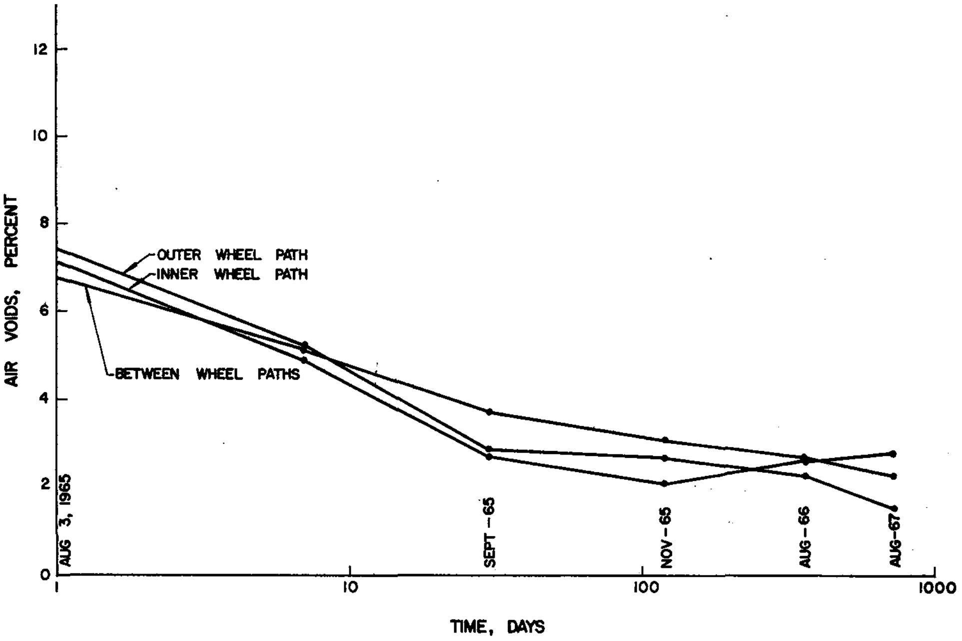
PAVEMENT DENSIFICATION WITH TIME AT VARIOUS LOCATIONS IN THE PAVEMENT (CUMBY TEST SECTION)

FIGURE 54



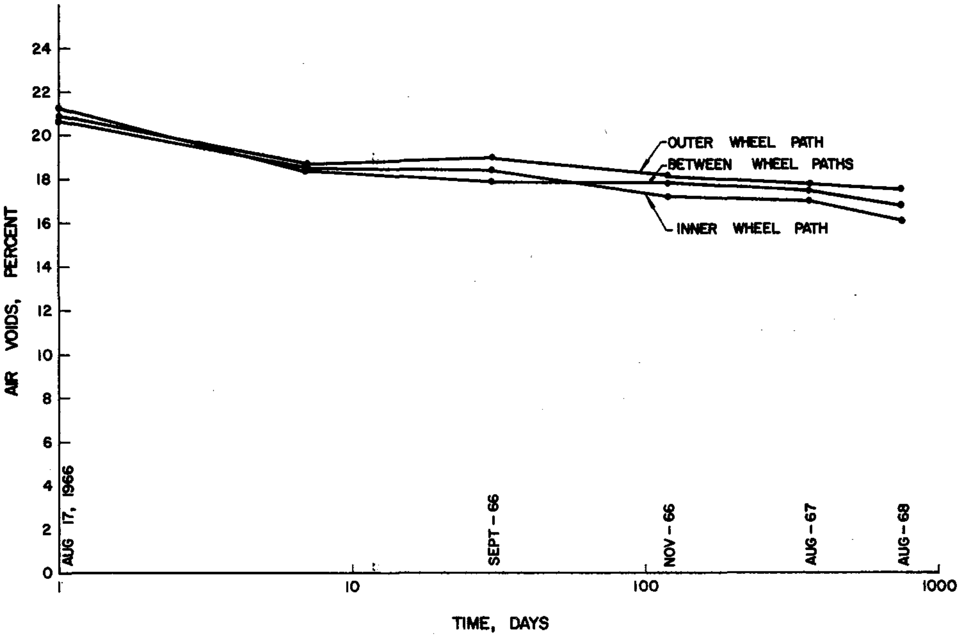
PAVEMENT DENSIFICATION WITH TIME AT VARIOUS LOCATIONS IN THE PAVEMENT (CLIFTON TEST SECTION)

FIGURE 55



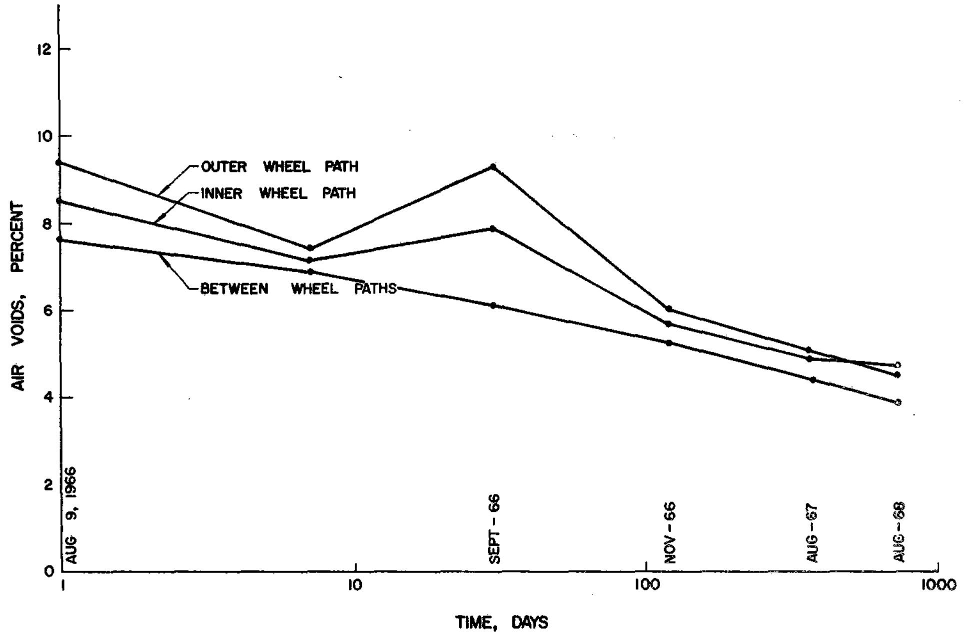
PAVEMENT DENSIFICATION WITH TIME AT VARIOUS LOCATIONS IN THE PAVEMENT
(WACO TEST SECTION)

FIGURE 56



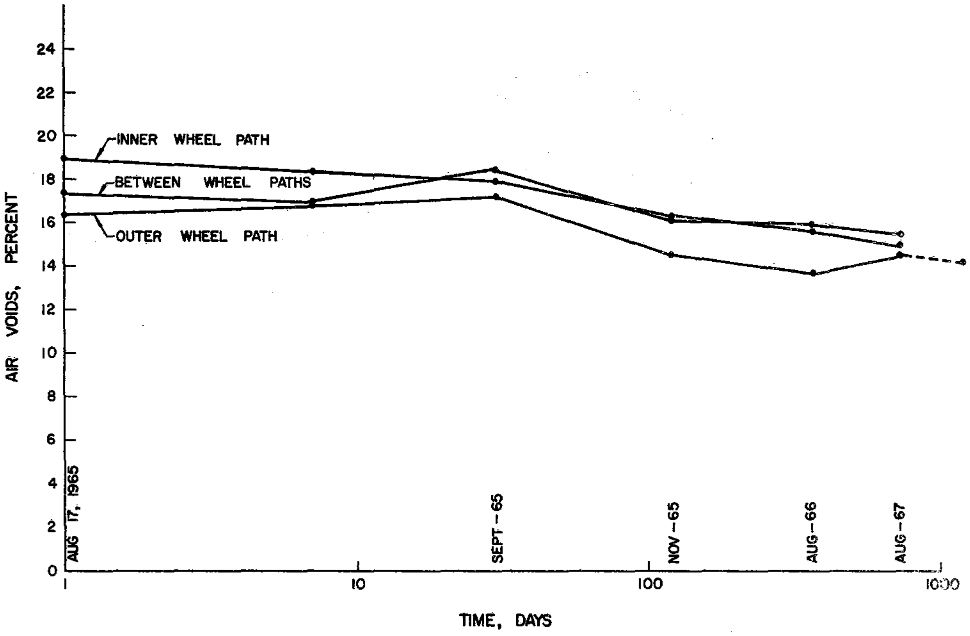
PAVEMENT DENSIFICATION WITH TIME AT VARIOUS LOCATIONS IN THE PAVEMENT
(MILANO TEST SECTION)

FIGURE 57



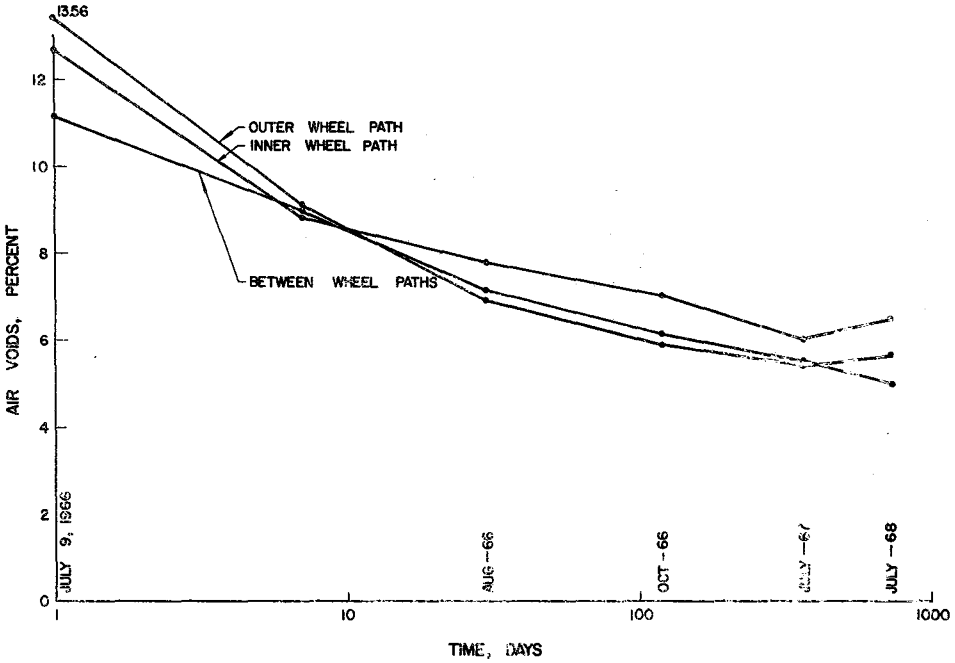
PAVEMENT DENSIFICATION WITH TIME AT VARIOUS LOCATIONS IN THE PAVEMENT (ROBINSON TEST SECTION)

FIGURE 58



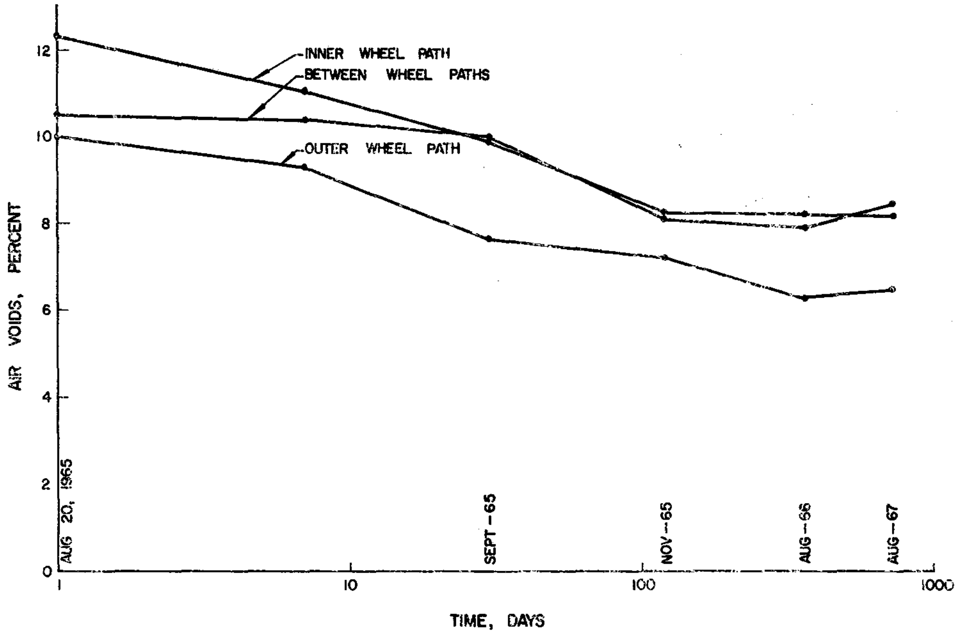
PAVEMENT DENSIFICATION WITH TIME AT VARIOUS LOCATIONS IN THE PAVEMENT (BRYAN TEST SECTION)

FIGURE 59



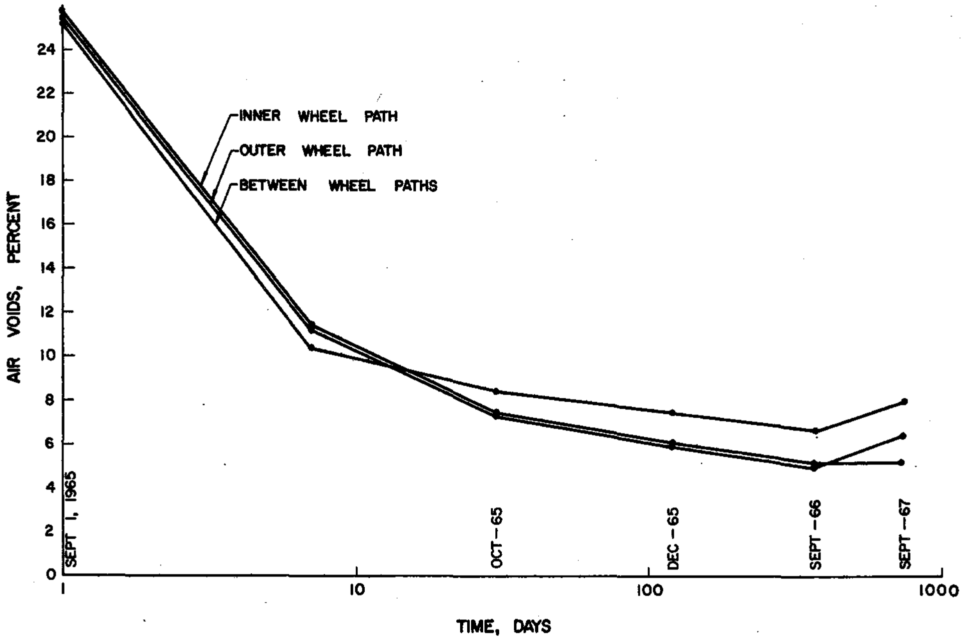
PAVEMENT DENSIFICATION WITH TIME AT VARIOUS LOCATIONS IN THE PAVEMENT
(TAMINA TEST SECTION)

FIGURE 60



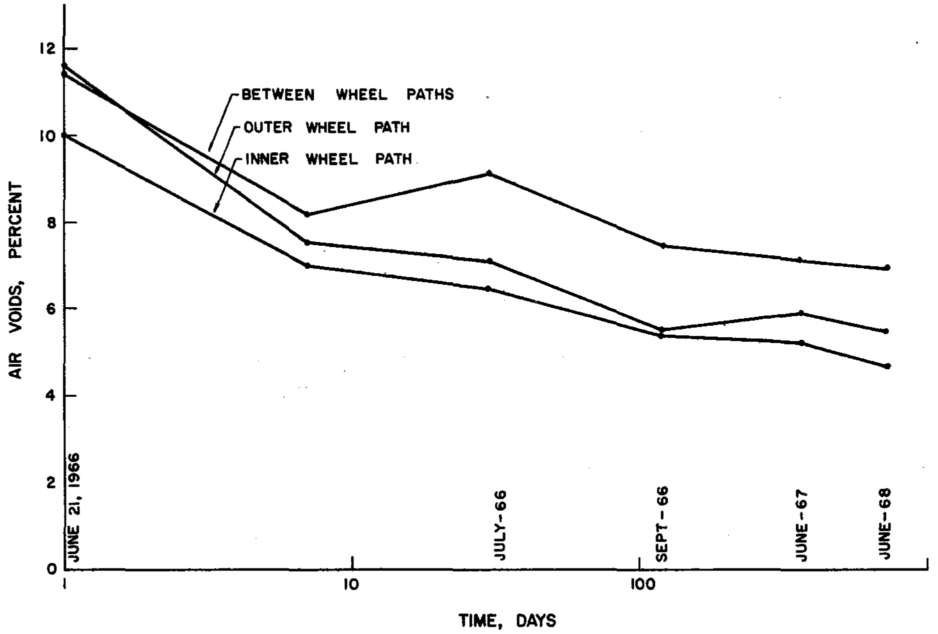
PAVEMENT DENSIFICATION WITH TIME AT VARIOUS LOCATIONS IN THE PAVEMENT
(CONROE TEST SECTION)

FIGURE 61



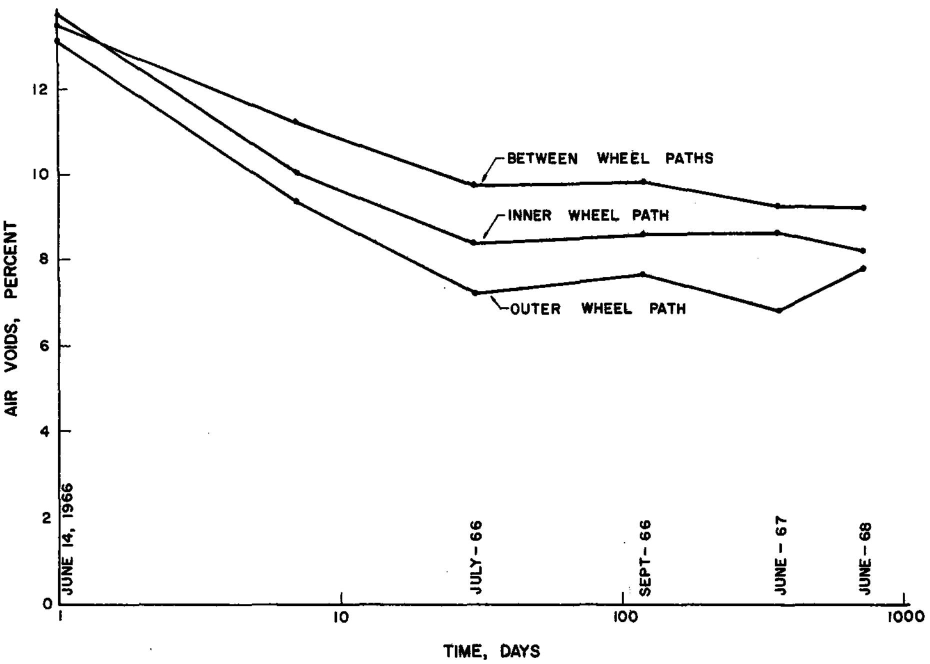
PAVEMENT DENSIFICATION WITH TIME AT VARIOUS LOCATIONS IN THE PAVEMENT
(BAYTOWN TEST SECTION)

FIGURE 62



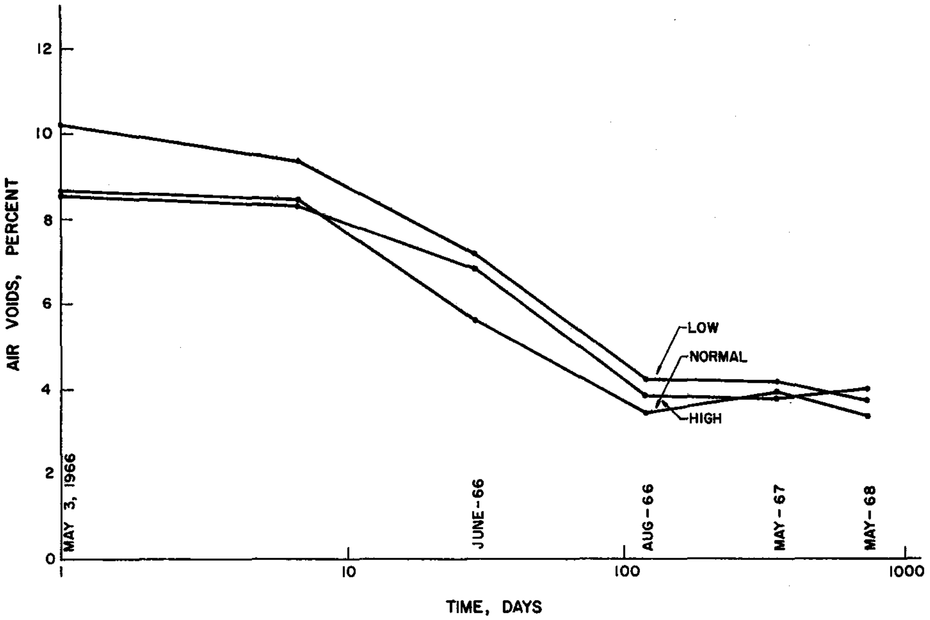
PAVEMENT DENSIFICATION WITH TIME AT VARIOUS LOCATIONS IN THE PAVEMENT
(ORANGE TEST SECTION)

FIGURE 63



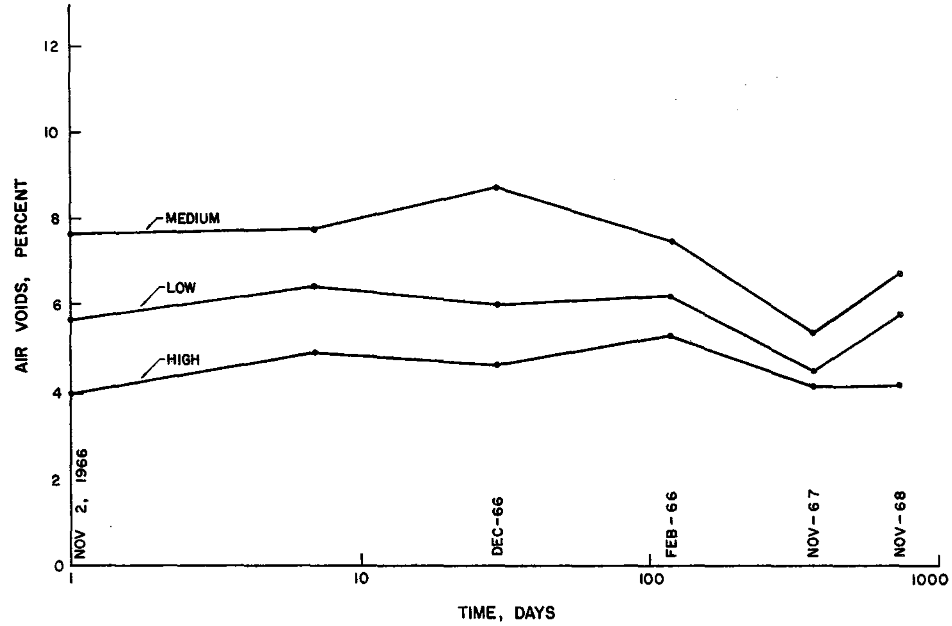
PAVEMENT DENSIFICATION WITH TIME AT VARIOUS LOCATIONS IN THE PAVEMENT
(BRIDGE CITY TEST SECTION)

FIGURE 64



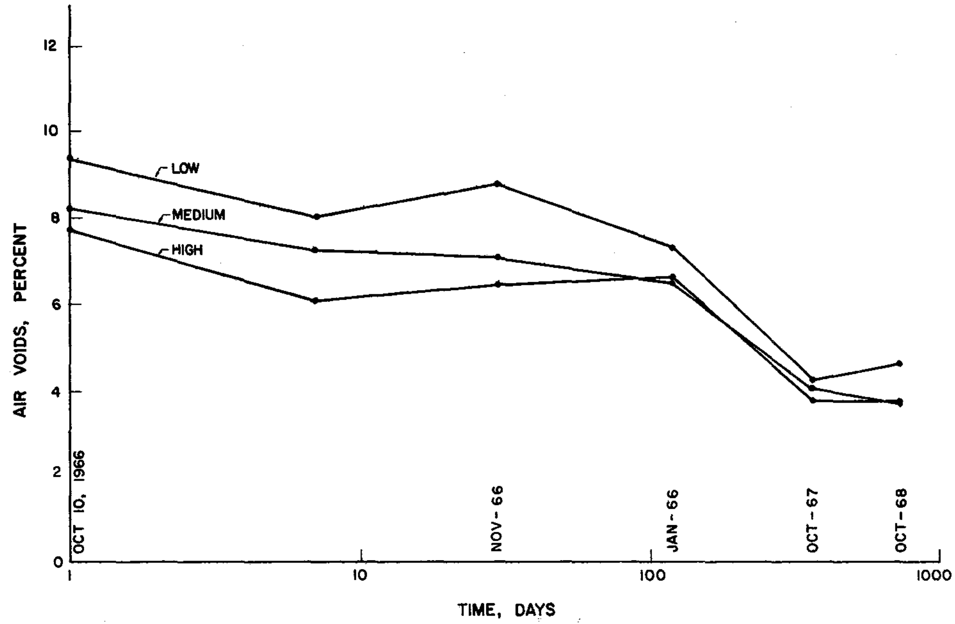
PAVEMENT DENSIFICATION WITH TIME FOR VARIOUS COMPACTIVE EFFORTS
INNER WHEEL PATH
(CHILDRESS TEST SECTION)

FIGURE 65



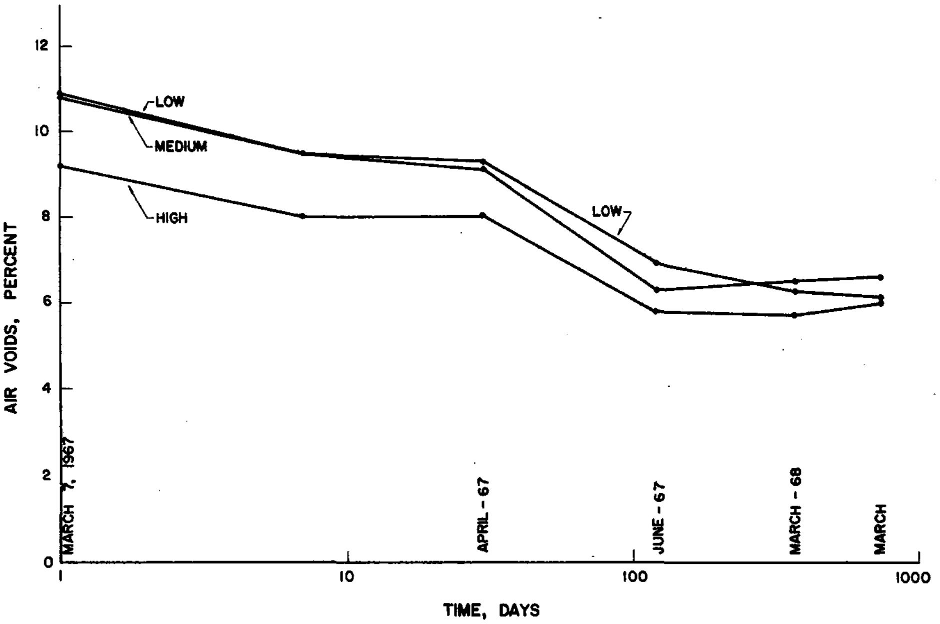
PAVEMENT DENSIFICATION WITH TIME FOR VARIOUS COMPACTIVE EFFORTS
INNER WHEEL PATH
(MATADOR TEST SECTION)

FIGURE 66



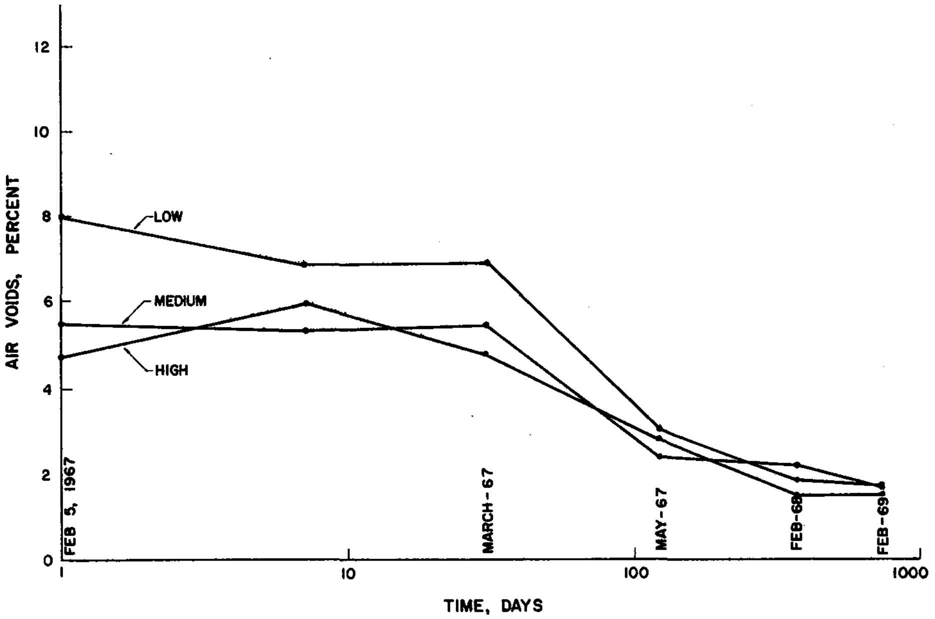
PAVEMENT DENSIFICATION WITH TIME FOR VARIOUS COMPACTIVE EFFORTS
INNER WHEEL PATH
(SHERMAN TEST SECTION)

FIGURE 67



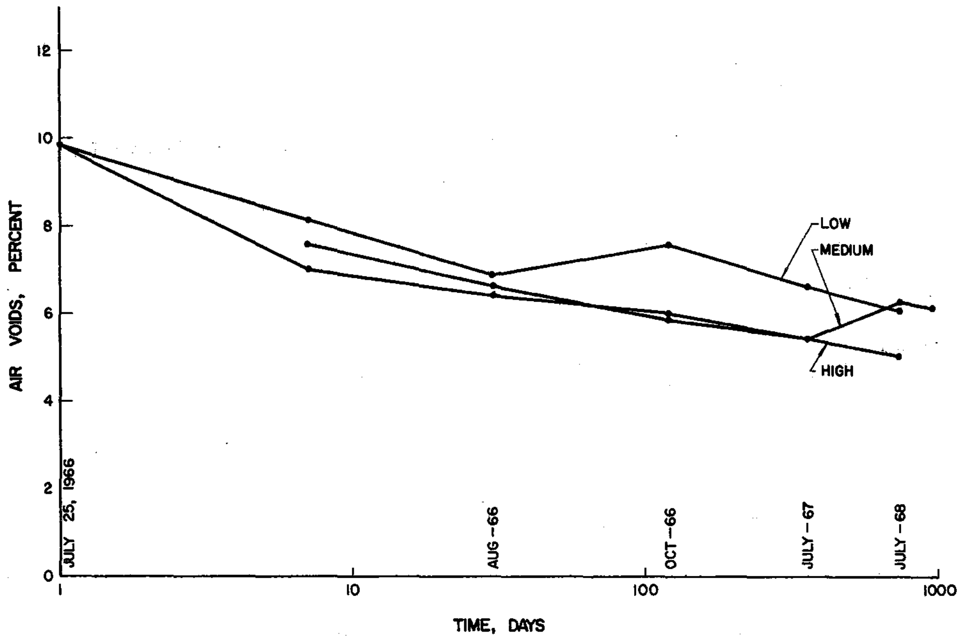
PAVEMENT DENSIFICATION WITH TIME FOR VARIOUS COMPACTIVE EFFORTS
INNER WHEEL PATH
(COOPER TEST SECTION)

FIGURE 68



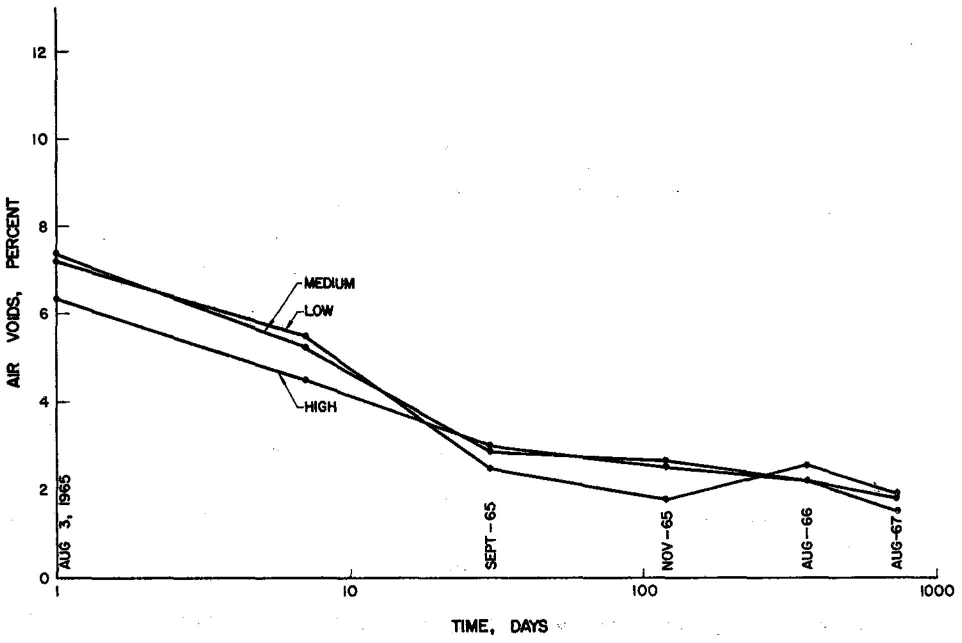
PAVEMENT DENSIFICATION WITH TIME FOR VARIOUS COMPACTIVE EFFORTS
INNER WHEEL PATH
(CUMBY TEST SECTION)

FIGURE 69



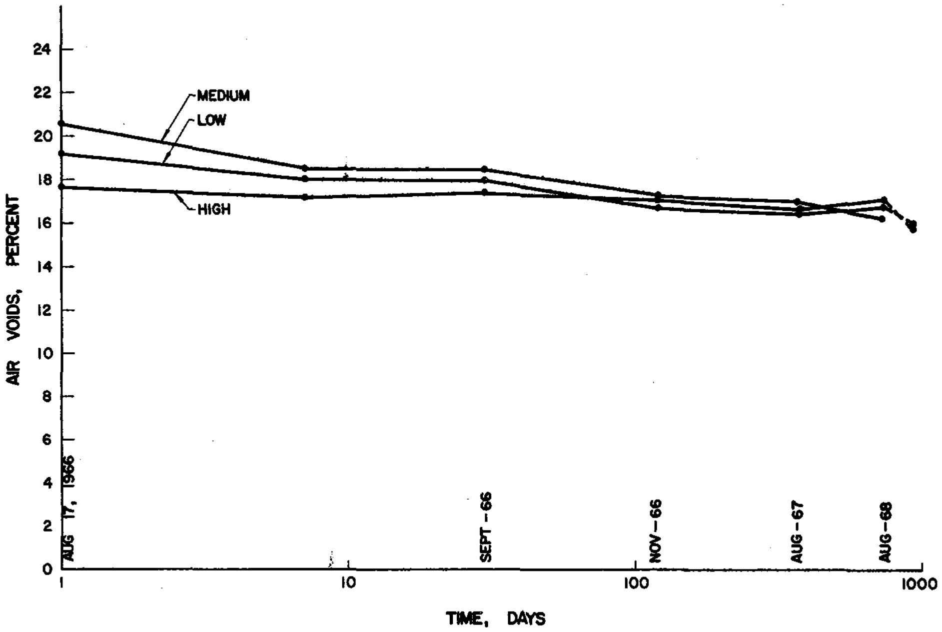
PAVEMENT DENSIFICATION WITH TIME FOR VARIOUS COMPACTIVE EFFORTS
INNER WHEEL PATH
(CLIFTON TEST SECTION)

FIGURE 70



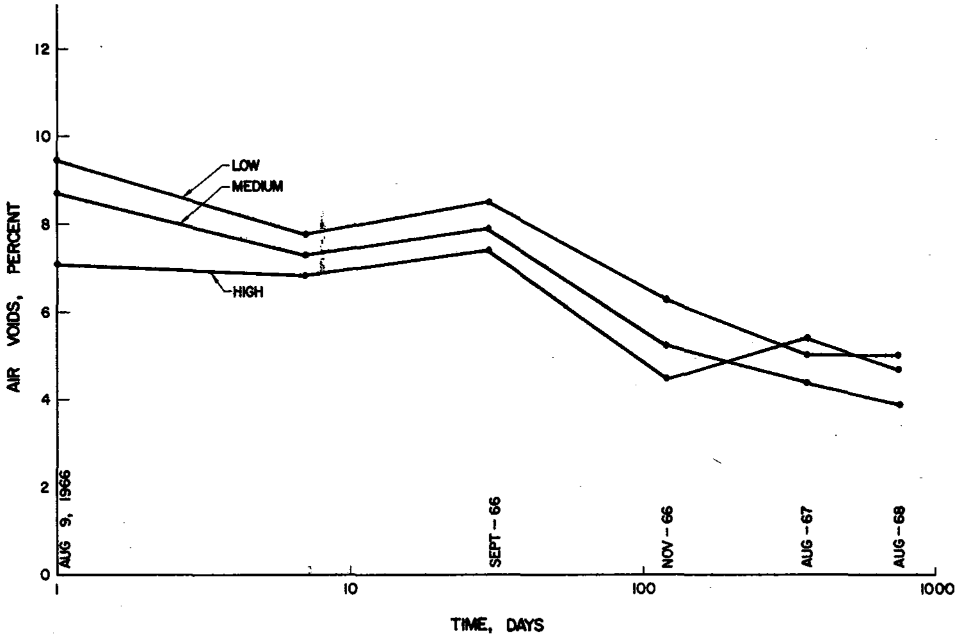
PAVEMENT DENSIFICATION WITH TIME FOR VARIOUS COMPACTIVE EFFORTS
INNER WHEEL PATH
(WACO TEST SECTION)

FIGURE 71



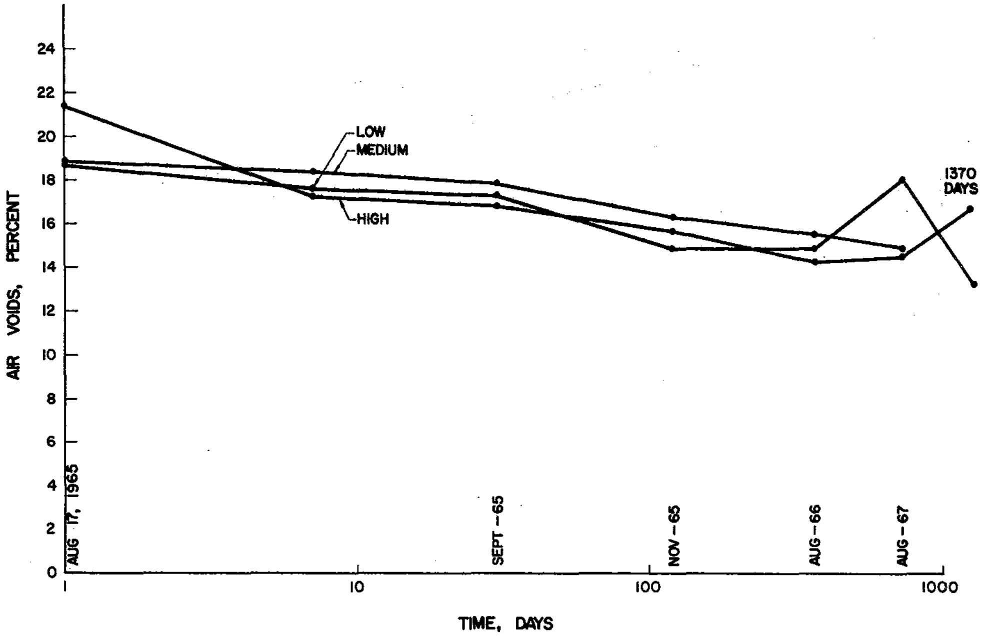
PAVEMENT DENSIFICATION WITH TIME FOR VARIOUS COMPACTIVE EFFORTS
INNER WHEEL PATH
(MILANO TEST SECTION)

FIGURE 72



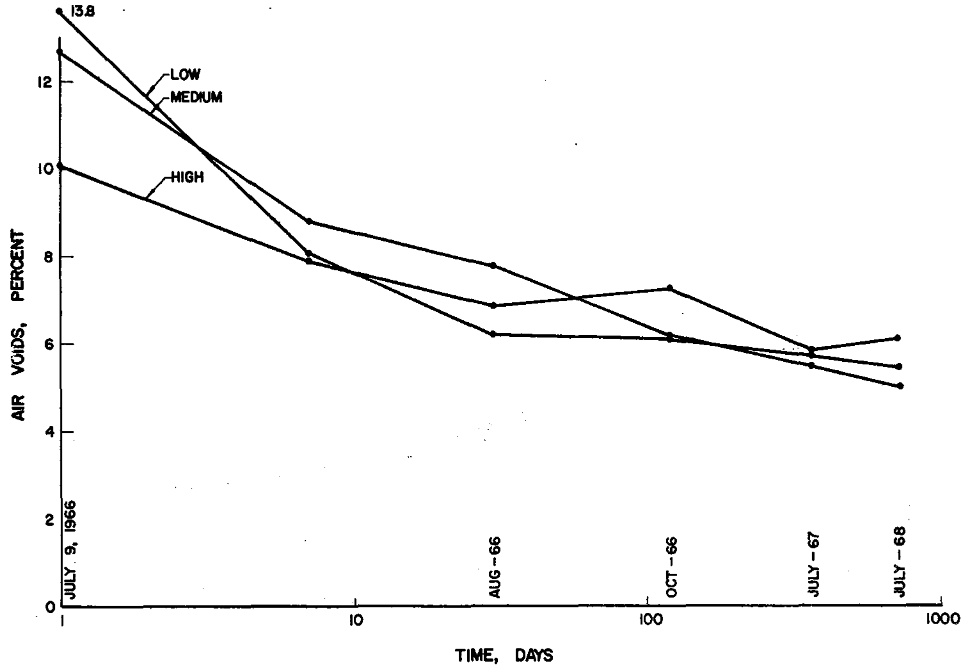
PAVEMENT DENSIFICATION WITH TIME FOR VARIOUS COMPACTIVE EFFORTS
INNER WHEEL PATH
(ROBINSON TEST SECTION)

FIGURE 73



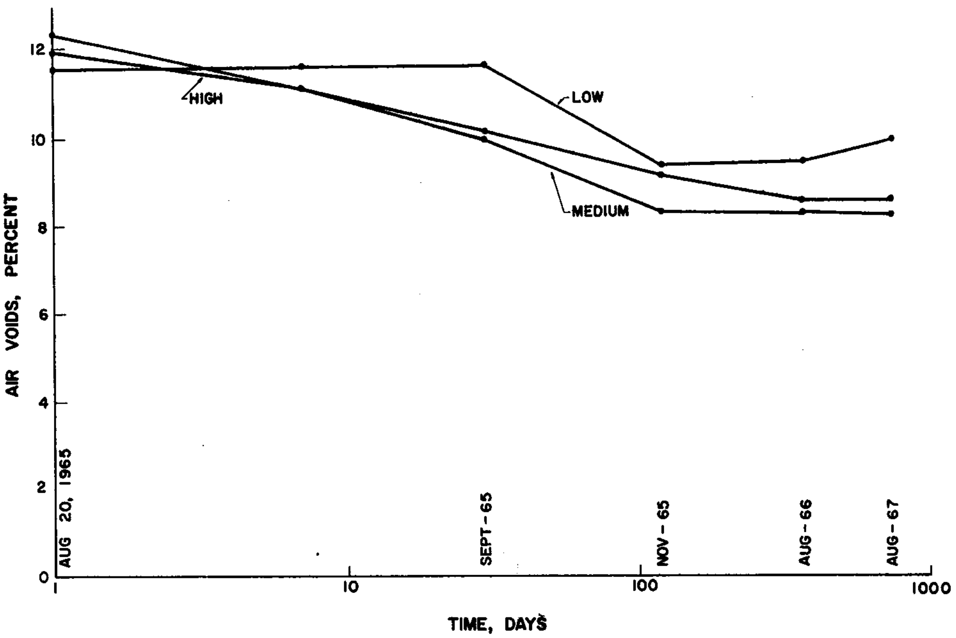
PAVEMENT DENSIFICATION WITH TIME FOR VARIOUS COMPACTIVE EFFORTS
INNER WHEEL PATH
(BRYAN TEST SECTION)

FIGURE 74



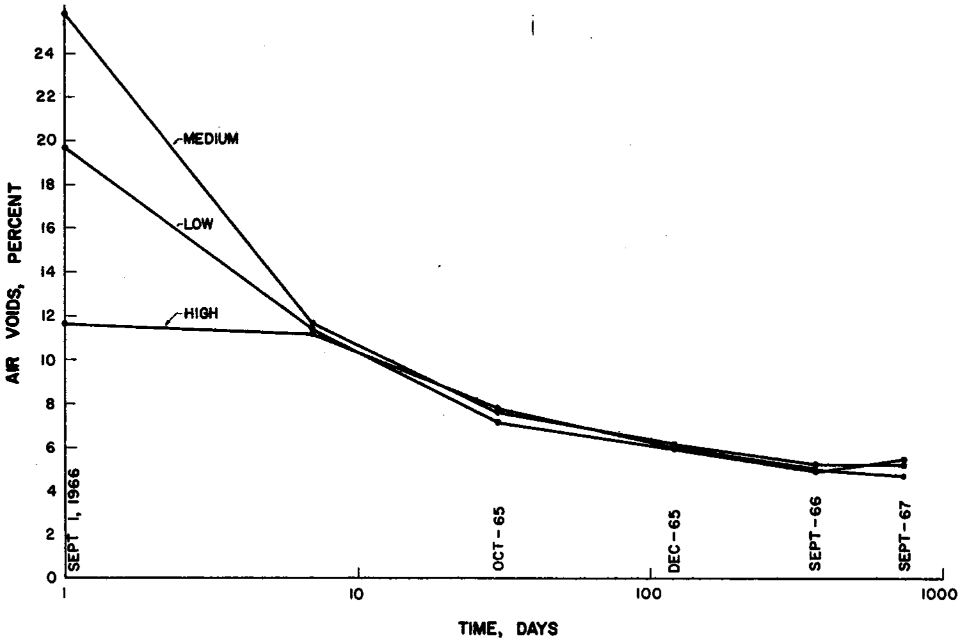
PAVEMENT DENSIFICATION WITH TIME FOR VARIOUS COMPACTIVE EFFORTS
INNER WHEEL PATH
(TAMINA TEST SECTION)

FIGURE 75



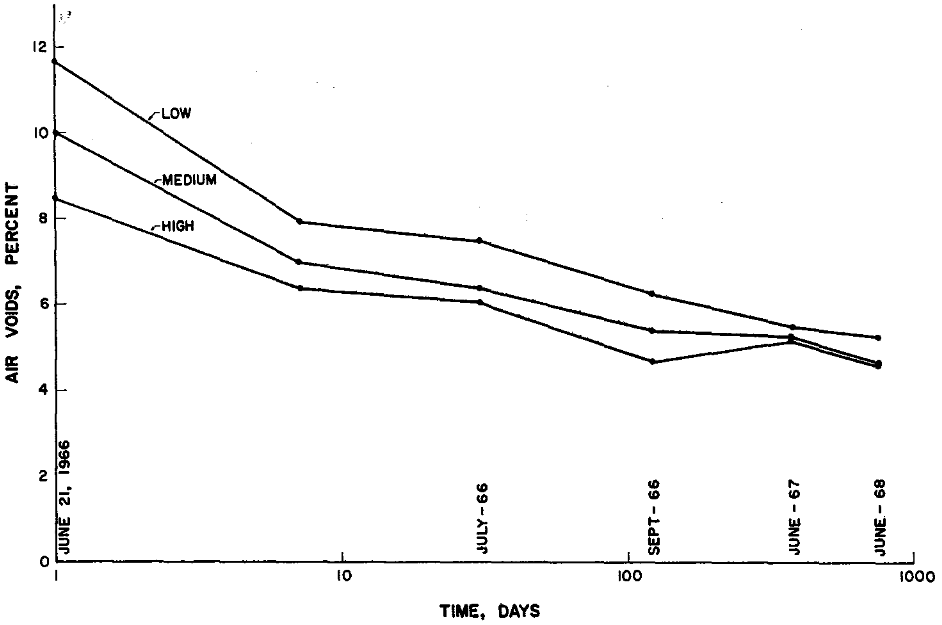
PAVEMENT DENSIFICATION WITH TIME FOR VARIOUS COMPACTIVE EFFORTS
INNER WHEEL PATH
(CONROE TEST SECTION)

FIGURE 76



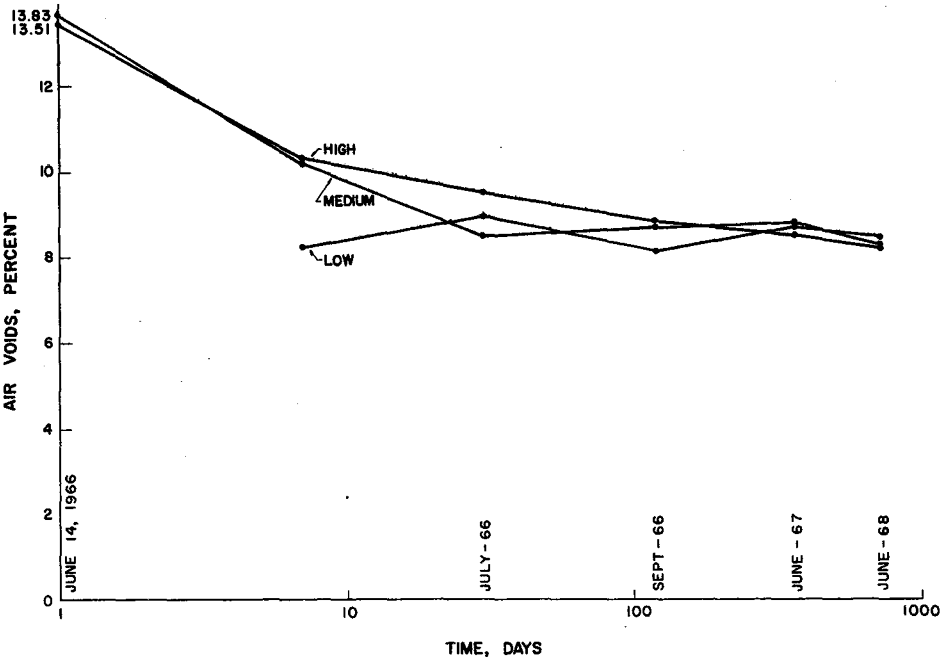
PAVEMENT DENSIFICATION WITH TIME FOR VARIOUS COMPACTIVE EFFORTS
INNER WHEEL PATH
(BAYTOWN TEST SECTION)

FIGURE 77



PAVEMENT DENSIFICATION WITH TIME FOR VARIOUS COMPACTIVE EFFORTS
INNER WHEEL PATH
(ORANGE TEST SECTION)

FIGURE 78



PAVEMENT DENSIFICATION WITH TIME FOR VARIOUS COMPACTIVE EFFORTS
INNER WHEEL PATH
(BRIDGE CITY TEST SECTION)

FIGURE 79

between the wheel paths for all three compactive efforts. In addition the laboratory densities for the standard Texas Highway Department, Marshall gyratory and California Highway Department Procedures are shown in Figures C1 to C15 of Appendix C. These results will be used subsequently to illustrate the dependency of long term compaction on the above mentioned factors. Detailed discussions follow.

Initial Compaction

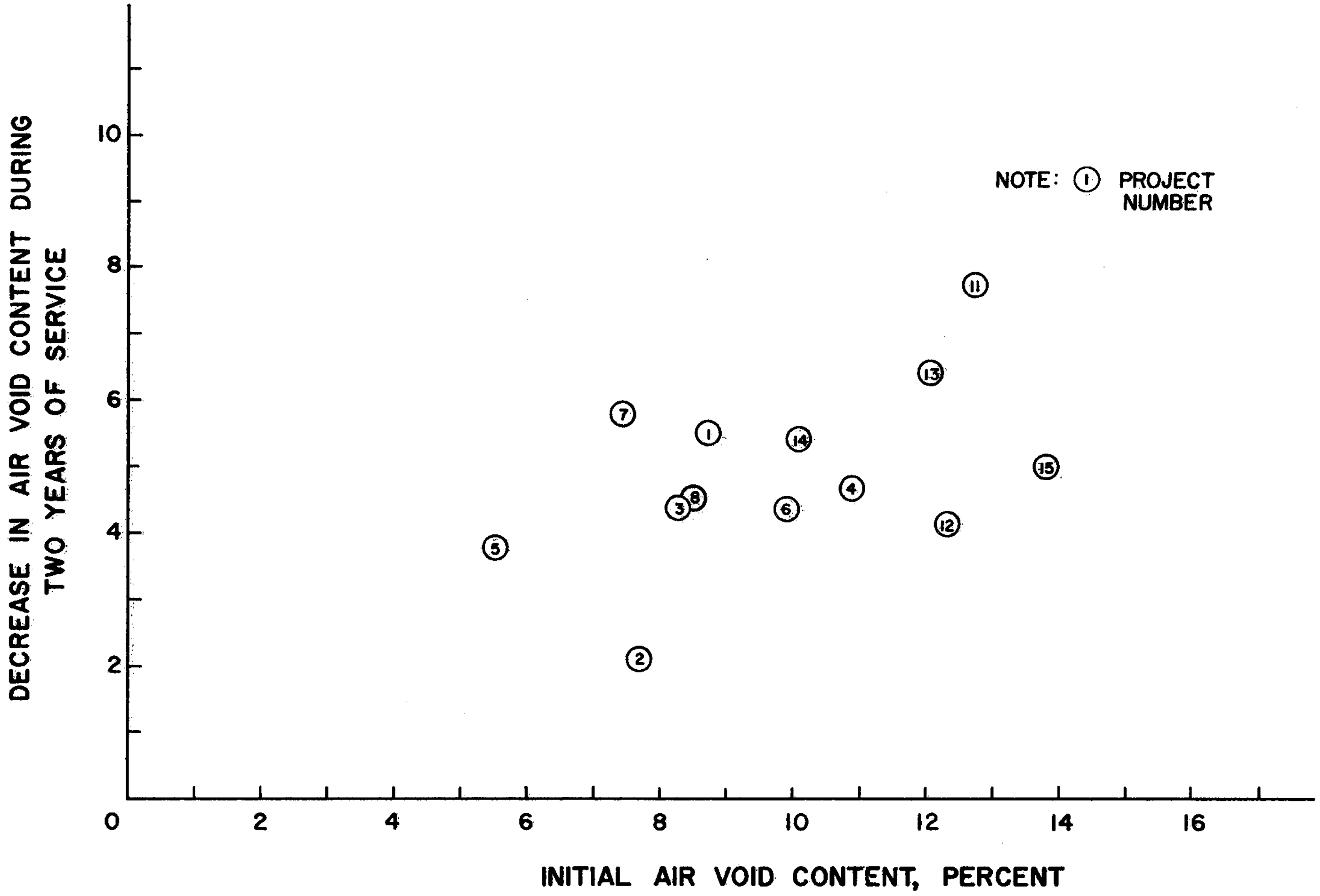
The degree of initial compaction of a pavement will determine to some degree the amount of densification that will occur due to mechanical and environmental loading during the life of a pavement. Figure 80 indicates the general trend of greater densification for those pavements with a low initial densification for thirteen test sites reported in the study. The two test sections containing slag aggregates are not included.

As noted previously, the individual test sites were subjected to different compactive efforts; however, little density variation was noted on most of the projects (Figures 42 to 46). Thus the density of the pavements after 2 years of service appears to be independent of initial compactive effort and initial density in most cases (Figures 65 to 79). As shown on these figures the majority of the pavements were compacted to a density within 2 to 3 percentage points of each other regardless of the compactive effort, and the resulting densities after 2 years of service fell within a range of 1 or 2 percentage points of each other.

An additional factor complicates the expected trend. Those pavements which exhibit low initial density usually age at a faster rate which in turn increases the viscosity of the asphalt and thereby increases the resistance of the pavement to further densification by traffic.

Material Properties

The properties of the asphalts and aggregates affect the long term densification



DENSITY CHANGE AS A FUNCTION OF INITIAL COMPACTION

FIGURE 80

of a pavement as well as its initial densification. Those material properties which tend to increase the resistance of a pavement to initial compaction behave in the same manner for the long term density increase due to mechanical and environmental loading.

Aggregate Absorption: Gallaway (52) has shown that, if aggregate absorption is not considered, asphalt densities can be calculated which result in values greater than that which is theoretically possible. The error in density measurements associated with absorption of asphalt by the aggregate can lead to high densities that are due neither to mechanical nor environmental loadings.

Aggregate Surface Characteristics: Although a wide variety of aggregates were used in terms of mineralogical composition, shape, surface texture, and maximum size; no conclusion can be drawn from data gathered on the fifteen test sites as to the effect of these variables on either the initial or long term compaction. However, it is well known that angular aggregates with rough surface textures will give high resistance to compaction.

Aggregate Gradation: The effect of aggregate gradations on initial compaction is shown in Figure 31. Two mixtures shown on the figure containing greater than ninety-five percent passing the number four sieve, were very difficult to compact. These same mixtures compacted very little with time (Figures 57, 59, 72, and 74) considering their high initial compaction and the relatively heavy traffic on the pavements (Table 4).

Asphalt Temperature-Viscosity Relationship: The asphalt temperature-viscosity relationship controls to a degree the compactability of a mix at a given temperature. Viscosity at various temperatures for the asphalts used in this project (Table 13) indicate that little difference exists in the initial temperature-viscosity relationship for these asphalts. Therefore, this variable is not considered important in this study.

Asphalt Susceptibility to Hardening: Asphalt hardening has been correlated to a certain degree with air void content and degree of interconnectability of air voids. Figures 81, 82, and 83 indicate that the greater the air void content, the faster the rate of hardening and thus the less likely the pavement will be compacted by traffic. In addition, data collected in this study suggest this same trend (Figure 20).

Asphalt viscosity at 4 months, which may be typical of the viscosity of the asphalt during its initial rapid densification due to traffic and environmental loading, is related to densification of the pavement with age in Figure 84. The general trend of low density gain with high viscosity asphalts is evident from this figure. Particular attention should be given the Milano and Bryan Projects (points 9 and 10 in Figure 84) which show low, long term densification and relatively high recovered asphalt viscosities after 4 months of service. It should be remembered that these two projects contained high initial air void contents which would contribute to high viscosities with age; and they also contained fine graded aggregates and relatively thin films of asphalt on the aggregate particles.

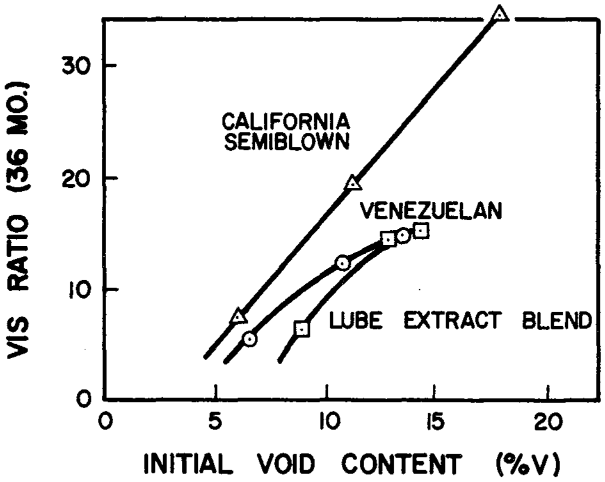
Mix Design

Mix design is important in that it is responsible for the selection of the asphalt content which controls the film thickness (22). It should be pointed out that mixture design quantities are dependent upon aggregate type, grading, surface texture, shape, asphalt viscosity, and other factors. Thus the influence of mix design on compaction has been discussed in part in the preceding section.

The effect of asphalt content (Table 11) on compaction is difficult to separate from the numerous variables which existed in this study.

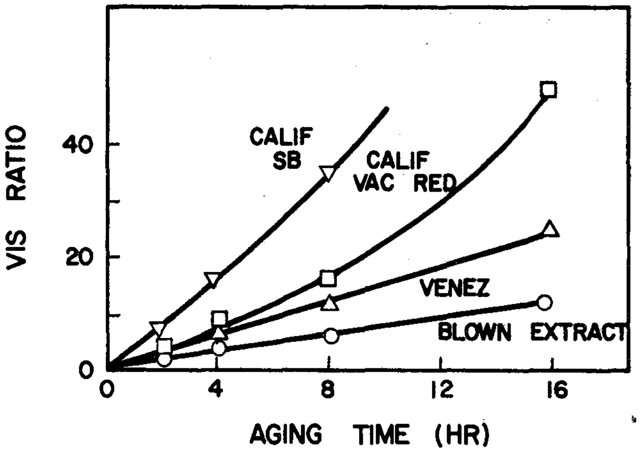
Weather Conditions

Density increases between the wheel paths have been noticed in several long term density studies (51, 54) (Figure 49) as well as this



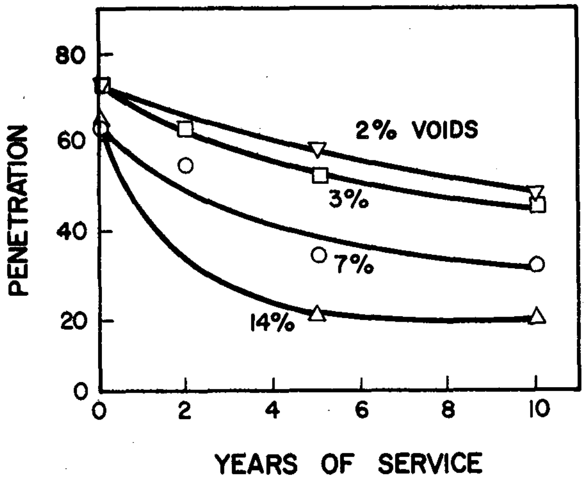
EFFECT OF VOID CONTENT ON HARDENING DURING 36 MONTHS' SERVICE.
(HEITHAUS AND JOHNSON (13))

FIGURE 81



EFFECT OF AGING TIME ON VISCOSITY RATIO
(HEITHAUS AND JOHNSON (13))

FIGURE 82



ASPHALT HARDENING IN SEVERAL MIDWESTERN PAVEMENTS
(HEITHAUS AND JOHNSON (13))

FIGURE 83

TABLE 15 ASPHALT AGING

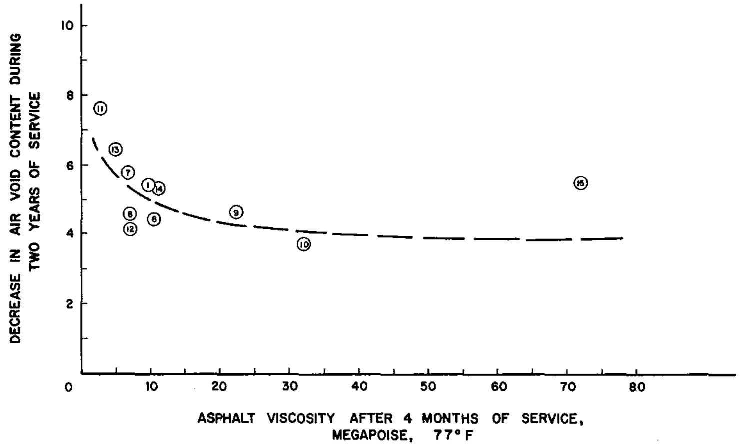
Test Section	Original Viscosity of Asphalt at '7°F, Megapoise	Viscosity of Asphalt After 4 Months In-Service at 77°F, Megapoise	Asphalt Source	Aging Index	Original Air Void Content, Percent	Air Void Content After 4 Months
Childress US 287 25-42-9	2.2	9.6	01 (AC-20)	4.4	8.7	3.4
Matador US 70 25-145-8	2.7	14.2	01 (AC-20)	5.3	7.7	7.4
Sherman SH 5 1-47-3	1.59	6.6	09 (AC-20)	4.1	8.3	6.6
Cooper SH 24 1-136-3			03 (AC-20)		10.9	6.9
Cumby IH 30 1-9-13	2.28	4.2	03 (AC-20)	1.8	5.5	2.4
Clifton SH 6 9-258-7	2.6	10.9	01 (AC-20)	4.2	9.9	6.1
Waco US 84 9-55-8	.87	6.6	09 (OA-90)	7.6	7.4	2.7
Robinson US 77 9-209-1	1.5	7.0	03 (AC-20)	4.7	8.5	5.3
Milano SH 36 17-185-4	3.0	24.0	06 (AC-20)	8.0	20.8	17.3
Bryan Spur 308 17-599-1	2.16	32.0	06 (AC-20)	14.8	18.8	16.2
Tamina IH 45 12-110-4	.77	2.84	11 (AC-10)	3.7	12.7	6.2
Conroe FM 1485 12-1062-35	1.80	6.90	11 (AC-20)	3.8	12.3	8.2
Baytown Spur 330 12-508-7	.72	4.84	06 (OA-90)	6.7	25.9	6.1
Orange SH 12 20-499-3	5.4	10.8	05 (AC-20)	2.0	10.0	5.4
Bridge City IH 87 20-306-3	5.4	73.0	05 (AC-20)	13.5	13.8	8.7

study (Figures 50 to 64). These data indicate that the density between the wheel paths is lower than either the inner or outer wheel path in most cases, however, this difference is usually less than two percentage points. Gallaway (55) has suggested that this increase in density between wheel paths may be due in part to thermal cycling.

With this in mind both the seasonal variations and daily cycling in temperatures were plotted for the projects. Figures 85 to 89 were prepared from U. S. Weather Bureau Station data obtained near the test sites. These figures suggest that the seasonal temperature extremes are greater in the northern part of the state (Childress, Matador, Sherman, Cooper, and Cumby) than in the more southerly and coastal projects (Tamina, Conroe, Baytown, Orange, and Bridge City). These seasonal variations amount to about 10^{o}F in the winter with the northern region the lower average monthly temperature. The summer average-monthly-temperatures are about the same for all locations.

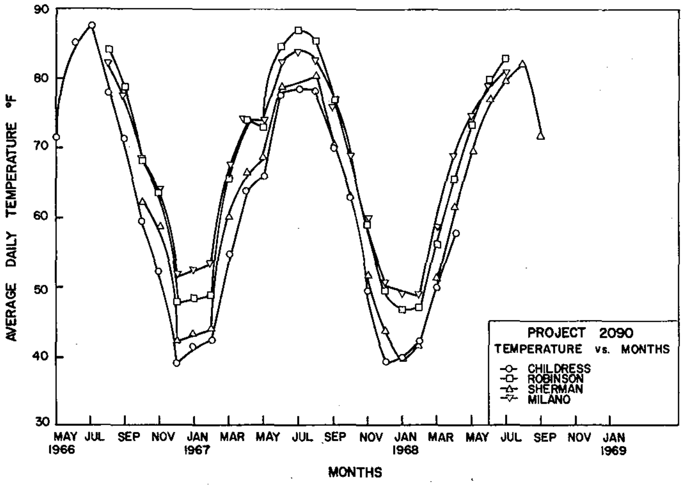
Daily temperature variation for selected weeks in the winter, spring, summer, and fall are given in Figures 90 to 95 for the various areas of the state. These figures indicate that daily temperature variation in the Panhandle region of Texas (Childress and Matador) has a greater cyclic temperature change than the more southerly coastal areas throughout the year.

These temperature data (daily temperature variation and seasonal temperature variation) do not satisfactorily explain the reason for densification between the wheel paths. In the majority of the projects the density between the wheel paths is less than the density in the wheel paths by approximately 1 to 2 percentage points. The amount of difference

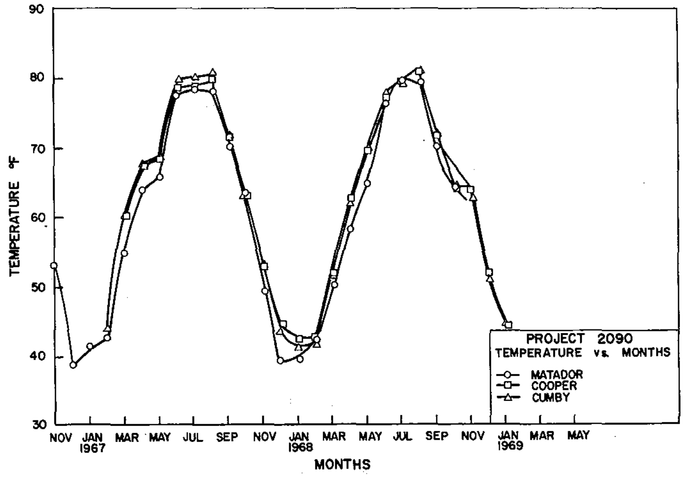


RELATIONSHIP BETWEEN DECREASE IN AIR VOID CONTENT AND ASPHALT VISCOSITY

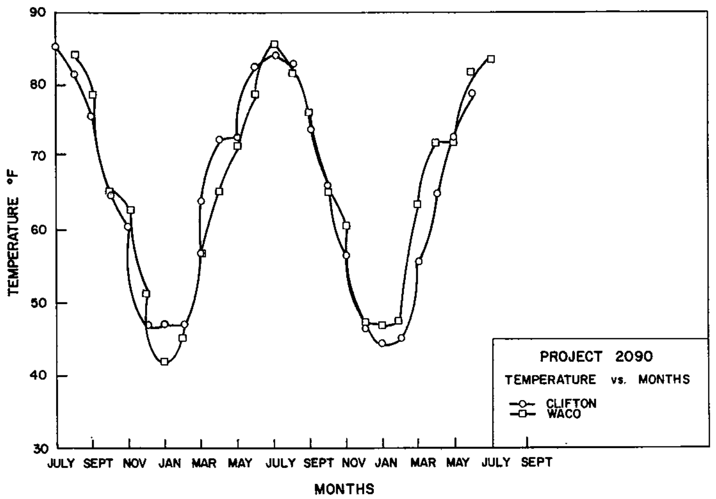
FIGURE 84



SEASONAL VARIATION IN TEMPERATURE
FIGURE 85

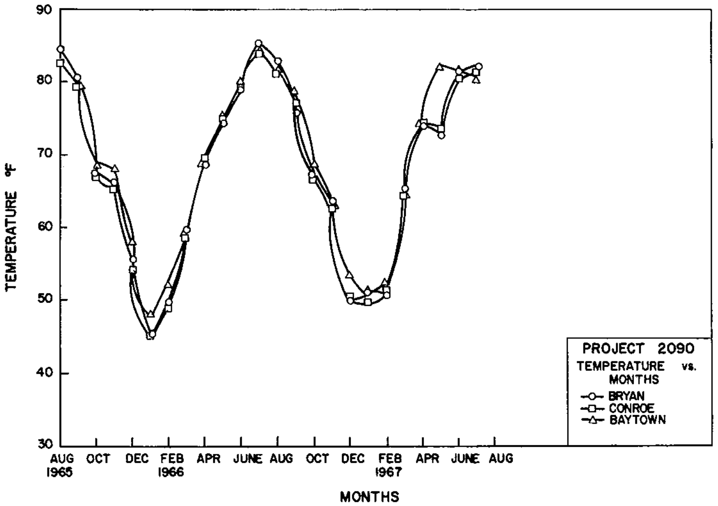


SEASONAL VARIATION IN TEMPERATURE
FIGURE 86



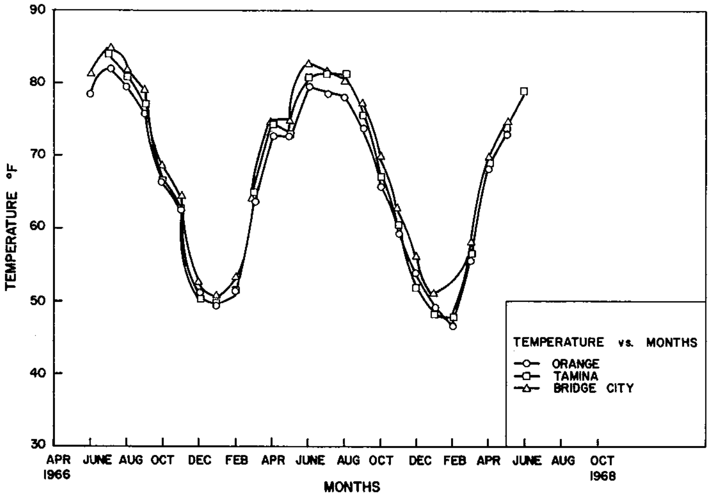
SEASONAL VARIATION IN TEMPERATURE

FIGURE 87



SEASONAL VARIATION IN TEMPERATURE

FIGURE 88



SEASONAL VARIATION IN TEMPERATURE

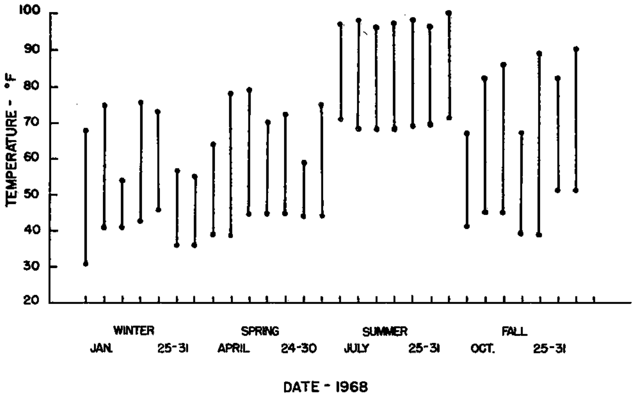
FIGURE 89

noted between the values of air void content in the various locations across the pavement cannot simply be related to the different seasonal and daily temperature environments (Figure 50 to 64). These data together with data published by Palmer and Thomas (51) on pavements in the state of New York (Figure 49) indicate that the entire pavement cross-section compacts to approximately the same degree of density and at approximately the same rate independent of initial compaction, seasonal variations in temperature, and daily variation in temperature for the range of traffic and environments to which these pavements have been subjected, but they do not suggest how the area between the wheel paths is densified.

The date of construction is important in that it determines the temperature of the pavement during its early life and thus its ability to be compacted by traffic. Three test sections were constructed in the late fall or winter in the northern part of the state (Matador, Sherman, and Cumby) (Figures 51, 52, 54, 66, 67, and 69). All of these pavements remained at essentially the same density until the warmer spring and summer months elevated the pavement temperature to a level sufficiently high for compaction to take place.

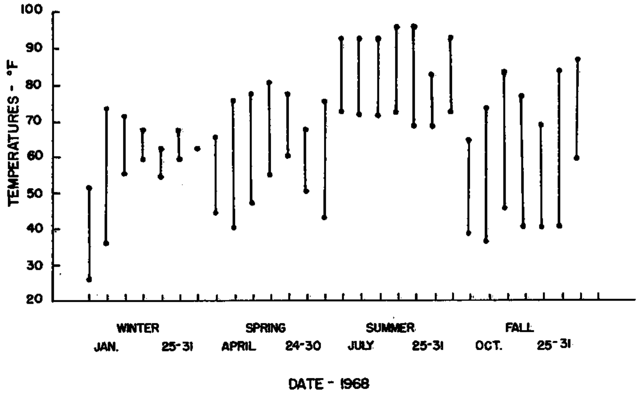
As shown above, little pavement densification occurred during the colder months. Thus if thermal cycling is a cause of densification between the wheel paths, it is not evident during the colder months on several of the projects.

The thermal strains in the pavement which are due to daily cycling in temperature should be slightly greater in the winter as the daily temperature change is greater (Figures 90 to 95). However, the stiffness



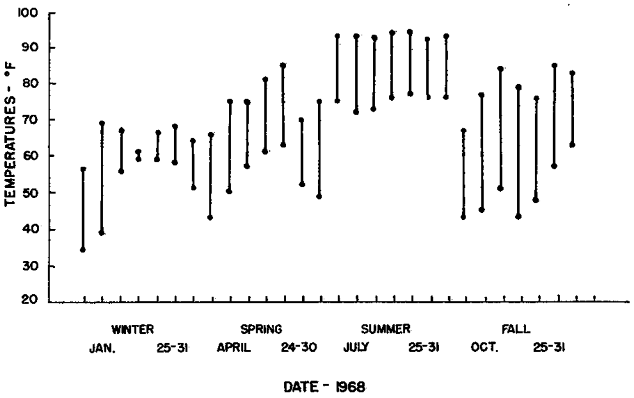
DAILY TEMPERATURE VARIATIONS
DISTRICT-25 (TEST SECTIONS CHILDRESS AND MATADOR) FOR 1968

FIGURE 90



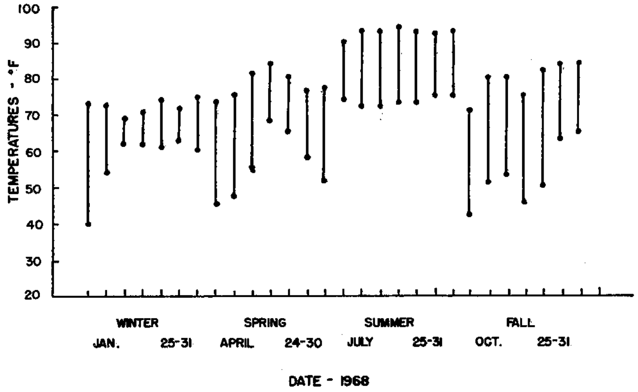
DAILY TEMPERATURE VARIATIONS
DISTRICT- (TEST SECTIONS-SHERMAN, COOPER AND CUMBY) FOR 1968

FIGURE 91



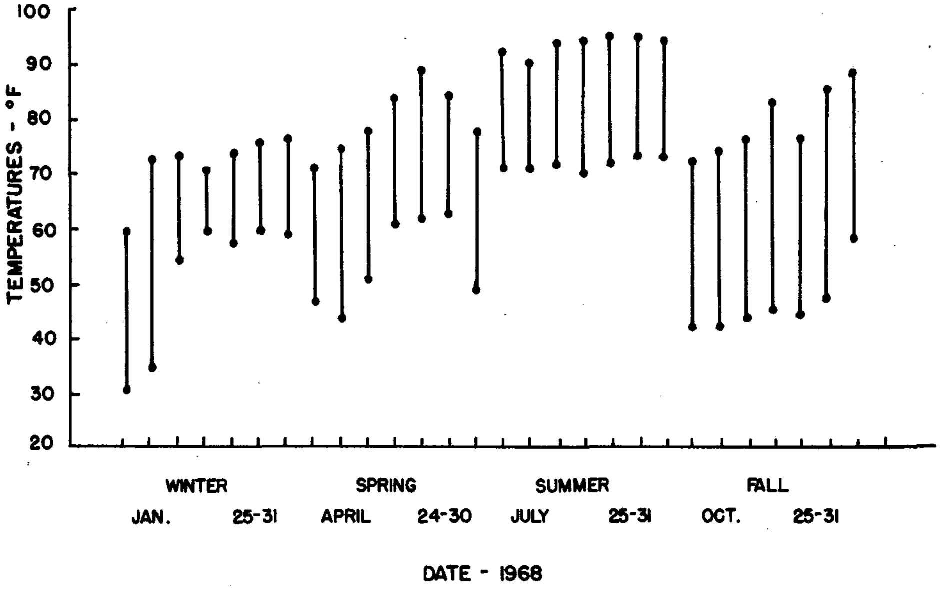
DAILY TEMPERATURE VARIATIONS
DISTRICT- 9 (TEST SECTIONS-CLIFTON, ROBINSON AND WACO) FOR 1968

FIGURE 92



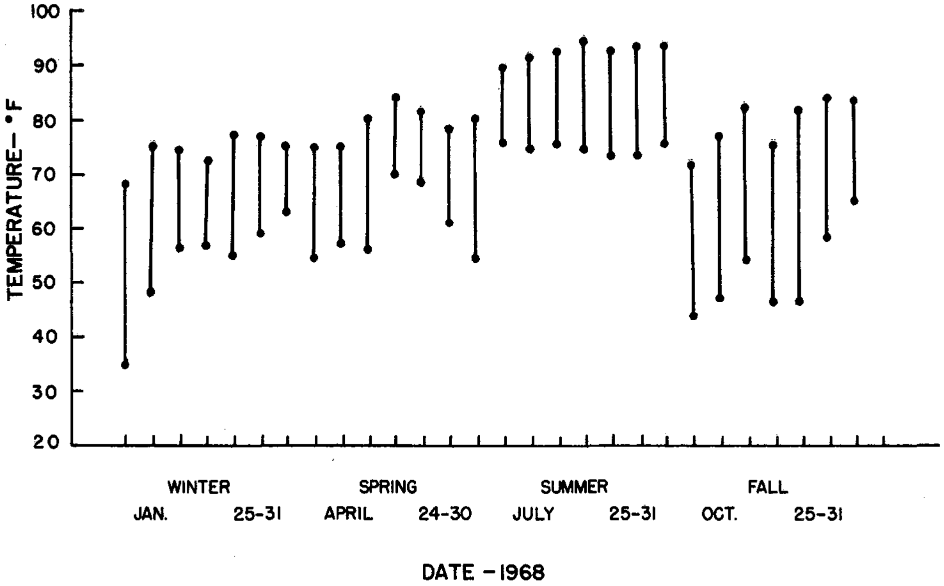
DAILY TEMPERATURE VARIATIONS
DISTRICT- 17 (TEST SECTIONS- MILANO AND BRYAN) FOR 1968

FIGURE 93



DAILY TEMPERATURE VARIATIONS
DISTRICT - 12 (TEST SECTIONS - TAMINA, CONROE AND BAYTOWN)
FOR 1968

FIGURE 94



DAILY TEMPERATURE VARIATIONS
DISTRICT-20 (TEST SECTIONS ORANGE AND BRIDGE CITY) FOR
1968

FIGURE—95

of the mix is much lower in the summer months and therefore it is easier for the aggregate particles to arrange themselves in a more dense arrangement. Unfortunately the pavements that were constructed during the warmer months were subjected to traffic immediately after construction and a check to determine if densification was due to a daily cycling in temperature during the warmer months, could not be made.

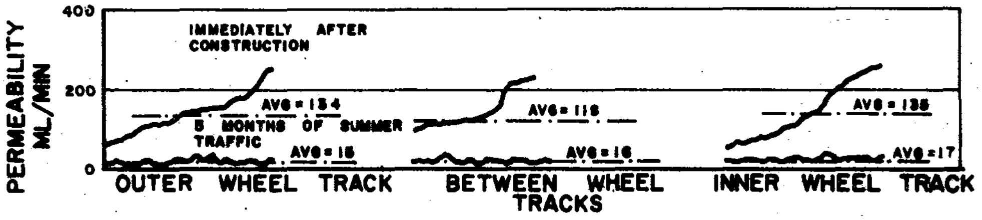
Traffic

The effect of traffic on long term pavement density has been established by a number of investigators including Zube (56) (Figure 96), Palmer and Thomas (51) (Figures 49 and 97), Campen (52), Arena et al., (44) (Figure 98), and McLeod (7). These figures suggest that the pavement densifies with axle load applications.

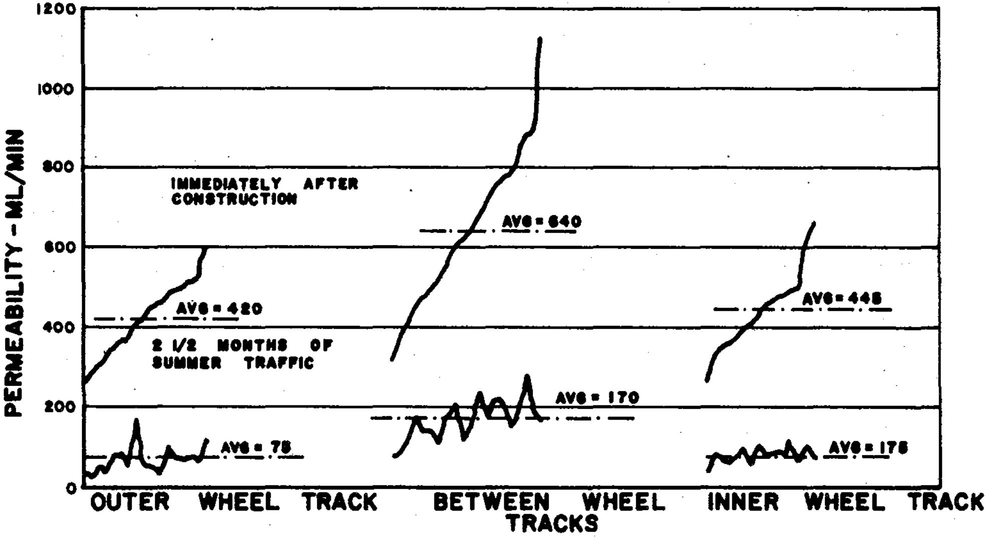
Volume of Traffic: Data reported in this study (Figure 99) indicate that pavements densify a greater amount with increased traffic independent of a number of other variables. This trend may explain the density increase in the wheel paths; however, it does not explain the density increase between the wheel paths.

Three test sites were not subjected to traffic for various lengths of time after construction. The Childress project was opened to traffic one week following construction and consequently the pavement did not densify (Figures 50 and 65) during this first week.

The Cumby project was not open for traffic for one month. Little density change is noted during this period (Figures 54 and 69).



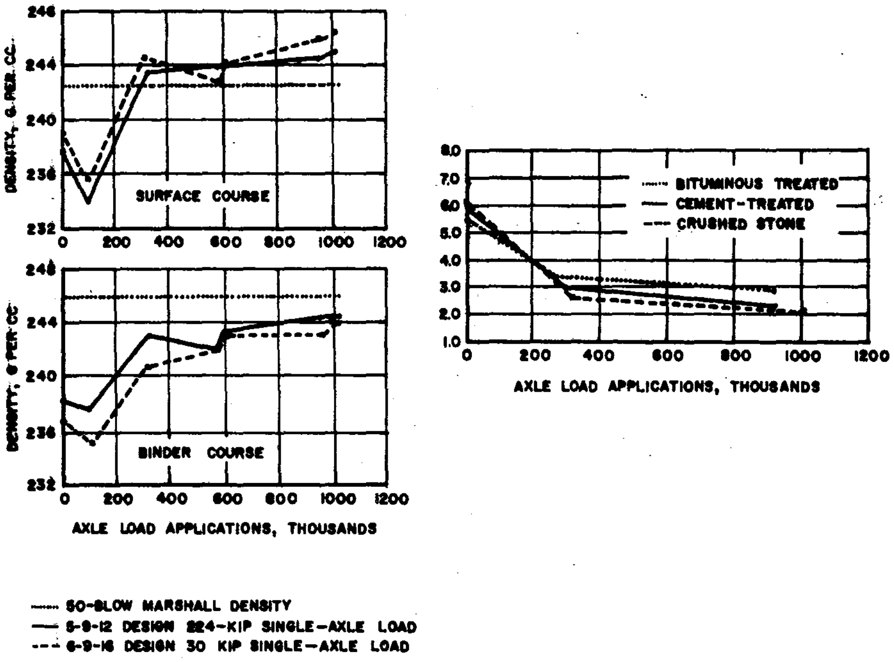
PROJECT G



PROJECT P

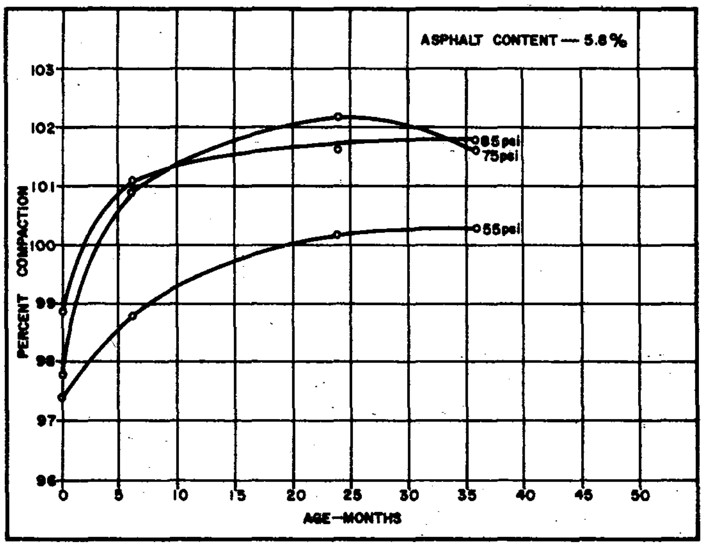
CHANGE IN PERMEABILITY VALUES AFTER SUMMER TRAFFIC, TRAVEL LANE (AFTER ZUBE (56))

FIGURE 96



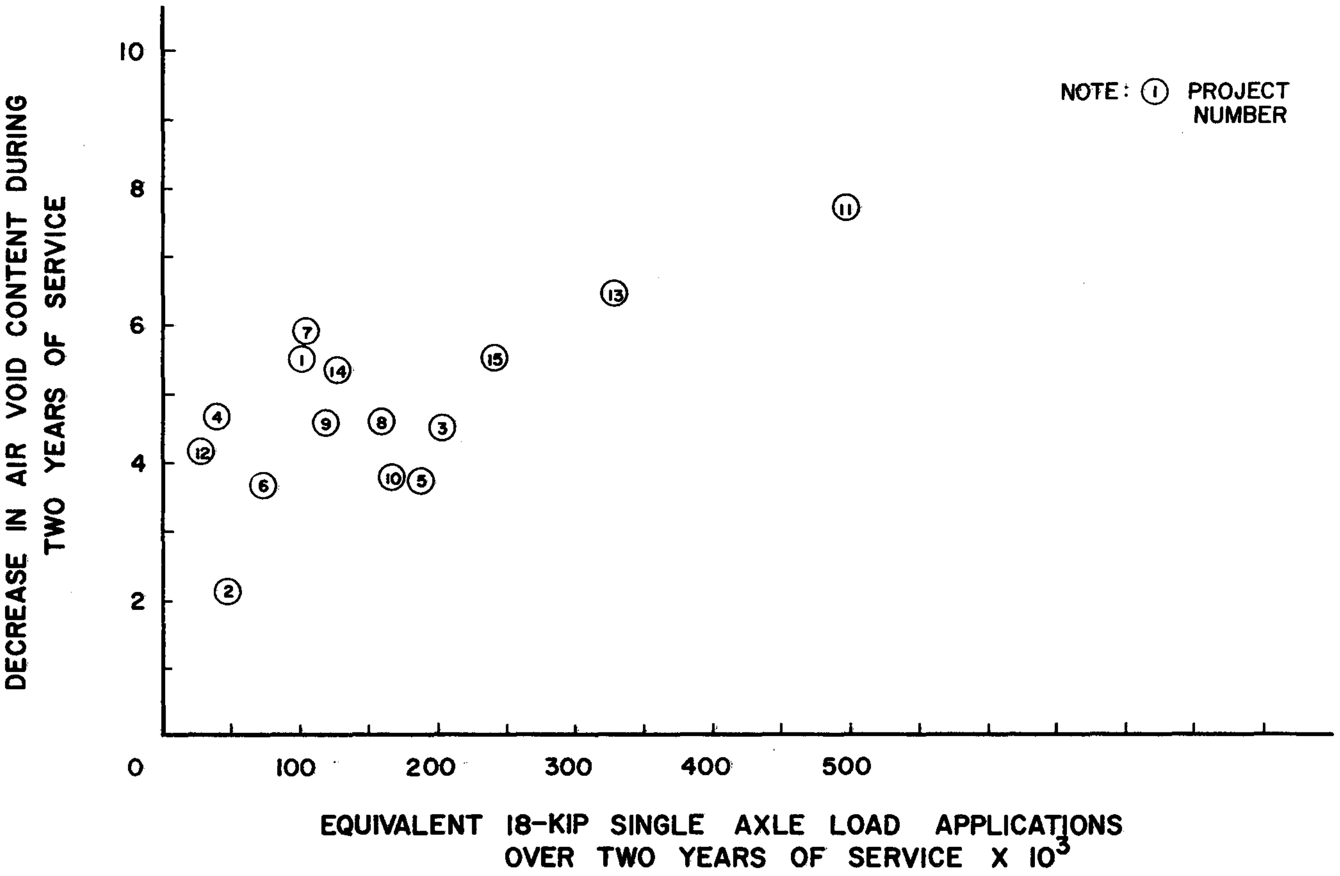
DENSITY AND VOIDS TRENDS, DEMONSTRATED ON THE
AASHO ROADS TEST
(AFTER PALMER AND THOMAS (51))

FIGURE 97



PERCENT COMPACTION VS. AGE FOR VARIOUS CONTACT PRESSURES AT OPTIMUM NUMBER OF PASSES OF PNEUMATIC ROLLER
(AFTER ARENA et al (44))

FIGURE 98



RELATIONSHIP BETWEEN INCREASE IN DENSITY AND TRAFFIC DURING TWO YEARS OF SERVICE

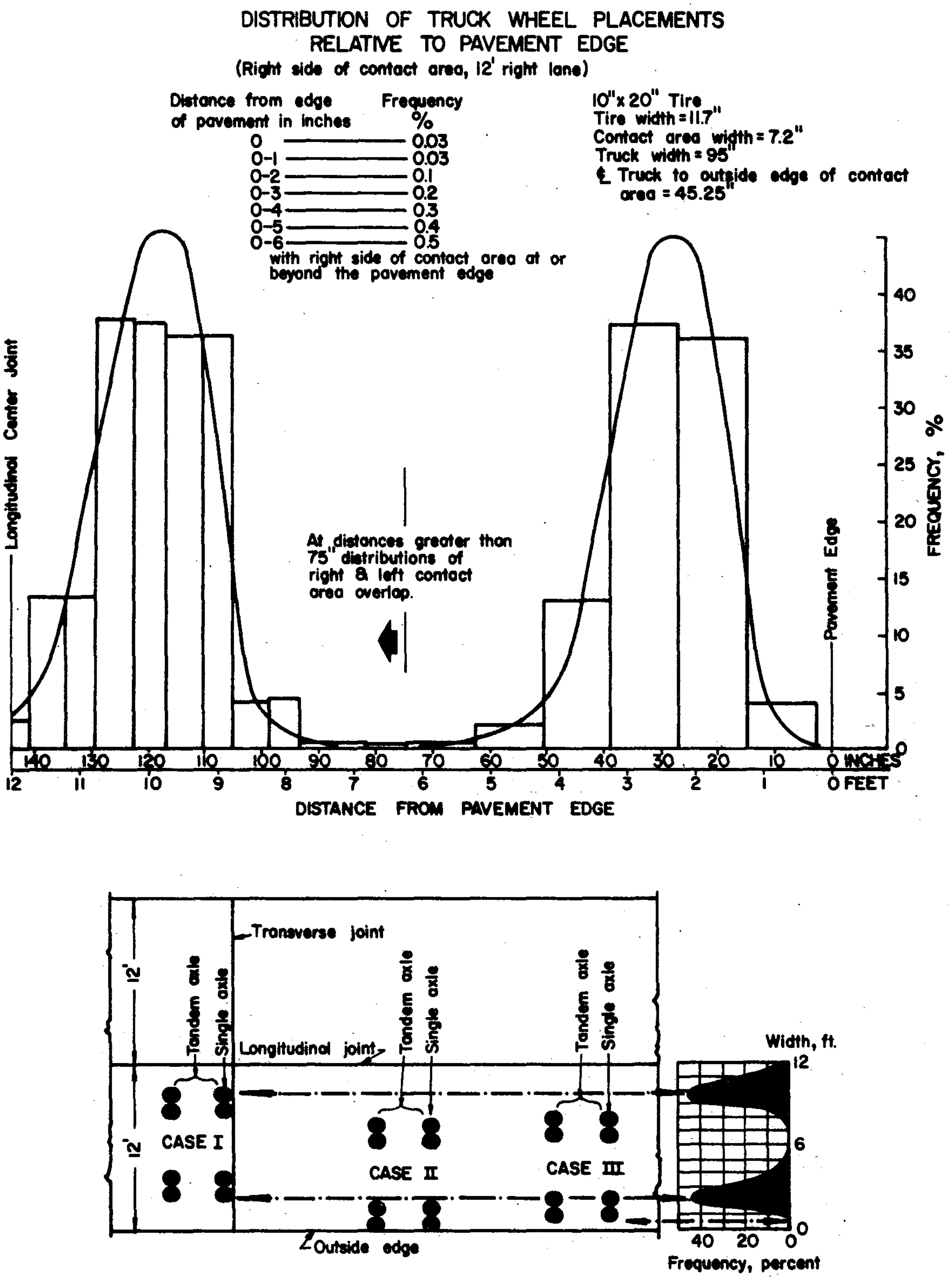
FIGURE 99

Baytown was opened to traffic one month after construction; however, a large amount of densification occurred during this period due to the fact that the contractor used this pavement as a haul road (Figures 62 and 77).

Type of Traffic: The distribution of wheel loads on a pavement will influence the density gain of a pavement with time. The greater the number of heavy axle loads the greater will be the density increase due to traffic. This suggests that not only the percent trucks must be considered but also the wheel load distribution. The equivalent 18-kip wheel load concept considers both of these factors.

Yearly Distribution of Traffic: The distribution of traffic throughout the year will influence the compaction of a pavement. If the heavy traffic is predominant during the warm months, a greater amount of densification will occur than if the heavy traffic used the highway in the cold months. This is primarily due to the susceptibility of the pavement to compaction when the asphalt viscosity is relatively low.

Traffic Distribution Across the Lane: The traffic distribution across the lane was investigated in hopes that it would explain the increase in density noted between the wheel paths. Data have suggested (57) that 10 to 16 percent of the wheel loads a pavement experiences may be in the center of the pavement. Figure 100 illustrates the distribution of truck wheel palcements relative to the pavement edge as used by the Portland Cement Association in their pavement design procedures. This distribution suggests that very little traffic uses the central part of the pavement. However, visual examination of several test



DISTRIBUTION OF TRUCK WHEEL PLACEMENTS RELATIVE TO PAVEMENT EDGE

(Right side of contact area, 12' right lane)

LOAD POSITIONS AND TRAFFIC DISTRIBUTION
(AFTER PCA (58) AND MONISMITH (59))

FIGURE 100

sections with thirteen feet wide lanes suggests that these data may be incorrect as the vehicles seem to wander in the lane a significant amount and therefore a larger portion of the wheel loads actually come in contact with the center portion of the pavement than would otherwise be expected.

Conclusions

1. Data collected in this study as well as others suggest that the long term density gain of asphalt concrete pavements is a function of many factors. The most important factors as indicated above are the degree of initial compaction, susceptibility of asphalt to hardening and thus increasing its viscosity, the volume of traffic and its nature, and the time of year of construction. It is evident that a pavement will densify with time provided it does not have high initial density and provided it is subjected to heavy wheel loads in warm weather.

2. Densification noted between the wheel paths closely paralleled the density gain in the wheel paths. The reasons for this trend are not clear. A possible explanation exists if we consider thermal cycling as a cause of densification. This however implies that the predominant forces creating compaction in all sections of the pavement are due to thermal changes and not traffic associated load. It is believed that the compaction between the wheel paths is due to wheel loads rather than thermal considerations as two pavements exhibited no density increase for periods of up to one month without traffic. If thermal stresses create densification with age, they should have been active during this period and a density increase should have been noted.

3. The air void content decreased by 2 and 8 percent during two years of service. The majority of the pavements were reduced 3 to 6 percent.

4. Eighty percent, of the anticipated two-year compaction due to traffic and environmental effects, was complete after one year of service on all of the projects studied.

SPECIFICATIONS

Hughes (66) presented a compilation of information concerning the control procedures in current use by the various state highway departments. This questionnaire suggests that the majority of states use end result specifications to control density. This type of specification states a particular density that must be achieved after the contractor has rolled the mix to his satisfaction.

The other type of specification disclosed is the method type specification which suggests the type and weight of rollers to be used and the number of passes that each roller should make. Seven of 46 replying agencies suggest that they used the latter type of specification.

Since end result specifications are the most commonly used, a short discussion will follow of the necessary data that must be obtained to enforce this type of specification.

Requirements for end result specifications include procedures for determining the standard reference density and test procedures for determining field density. A number of methods have been used to determine the standard reference density. These include maximum density methods based on 1) bulk specific gravity of the aggregate or vacuum testing methods. 2) standard laboratory or field densities which are the most common types and 3) maximum attainable field density.

Several methods have been used to measure field densities. These methods include destructive methods which include pavement cores or sawed specimen and split ring methods, non-destructive methods include

In addition, control strip techniques have been suggested (60) as an economical means of obtaining proper compaction of pavements. This method allows the engineer to compact a trial section using the equipment and materials that will be used on the job under similar environmental conditions. By compacting this test section with various number of passes of the compacting equipment and various sequences of operation of the equipment, the engineer can determine the optimum compactive effort required for the specialized conditions of the particular project.

These sections also allow the engineer to correlate rapid field measuring devices with cored densities. This information can then be used for additional job control.

Control strip techniques can produce well compacted pavements at minimum costs.

Texas Highway Department Standard Specifications (1962) require in-place densities between 95 and 100 percent of standard laboratory density for class AA hot-mix asphaltic concrete pavement. In addition the standard laboratory density must be within 94 to 99 percent of theoretical maximum density as measured by the Texas Highway Department method. This suggests the absolute air void content of a pavement may range from eleven to one percent and be within the specifications for class AA asphalt concrete. No in-place density requirements exist for the more popular class A mixes.

Other common specifications range from 95 to 100 percent of laboratory density or 85 to 100 percent of theoretical of mixtures. The Asphalt Institute recommends a field density of 97 percent of laboratory density.

nuclear air and water permeability methods. The majority of methods currently in use are destructive methods of test. Several reports have been written on the usefulness of these devices and will be discussed subsequently.

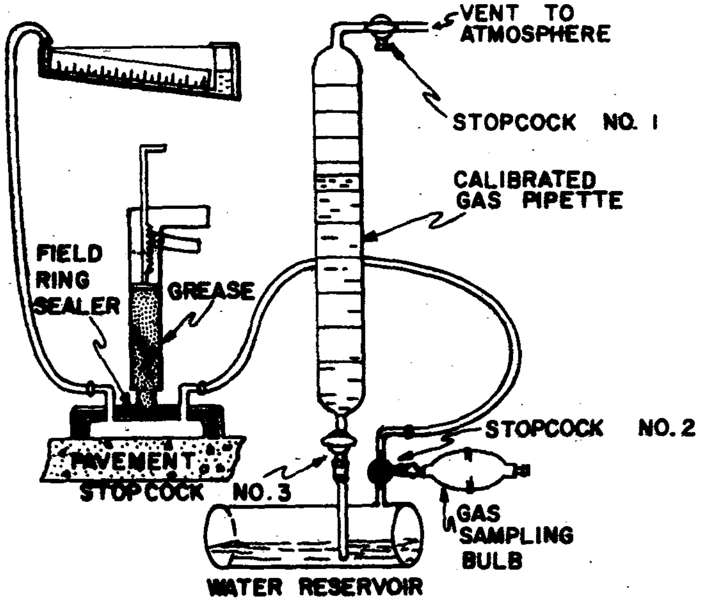
Core density tests have probably been used by more agencies than any other to determine the density of the pavement. Work by Kimble (61) suggests that densities can be obtained relatively quickly by use of portable laboratories. However, this type of test in addition to being destructive is also time consuming.

Comparison of air permeability devices (Figure 101) and density determined from coring operations have been made by Schmidt et al. (43) (Figure 102) and Kari and Santucci (62). Additional work by Ellis and Schmidt (63) has correlated core permeability with permeability of cores in place.

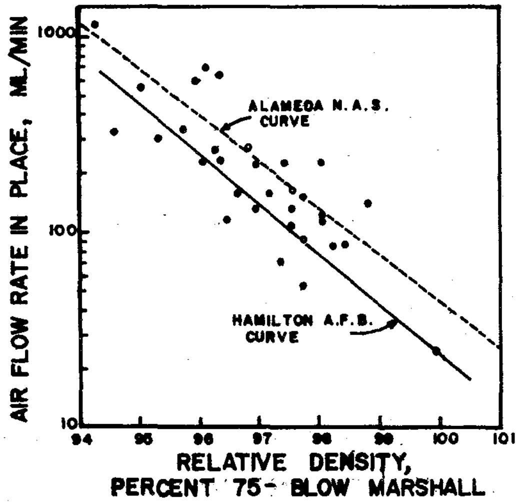
Goode and Lufsey (12) have shown that air permeability is a function of aggregate gradations as well as total air voids; therefore, the air permeability apparatus may have to be determined for each individual mix and compared with the standard for that mix.

Zube (56) has developed a water permeability deivce used by the California Division of Highways and has presented a correlation with field core measurements (Figure 103) for ten projects.

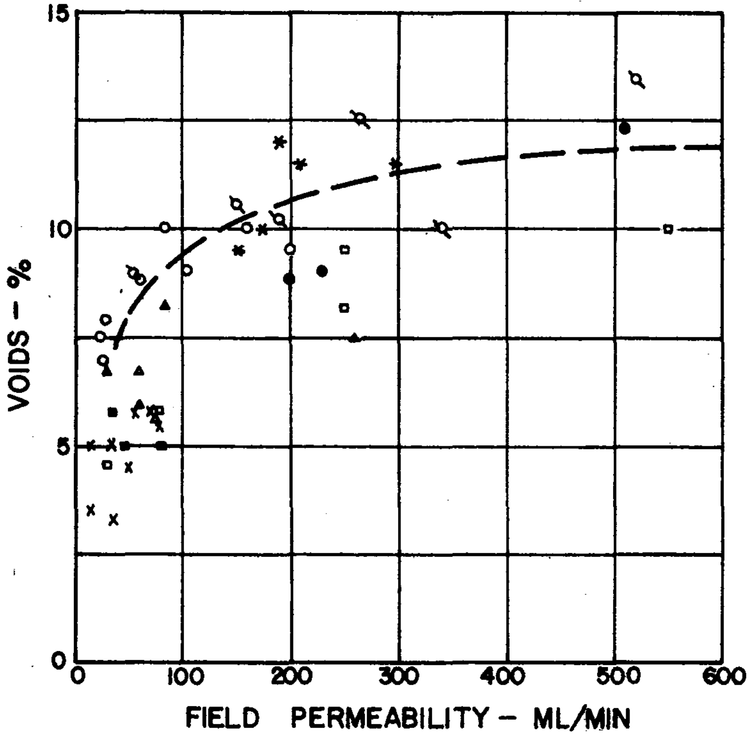
Nuclear gauges have been used in Oregon (64), Texas, and California (70). Work performed by Hughes (66, 67) and a number of factors including aggregate type, asphalt content, layer thickness, and surface texture affect the density as measured using this device be calibrated for use on a particular project.



SCHEMATIC DIAGRAM OF PERMEABILITY APPARATUS
(AFTER KARI AND SANTUCCI (62))
FIGURE 101



PAVEMENT AIR FLOW RATES
MAY BE USED TO PREDICT
RELATIVE DENSITY OF CORES.
TESTS CONDITIONS: 81. SQ.CM.
AREA, 0.25 IN. WATER PRESSURE.
(AFTER KARI AND SANTUCCI (62))
FIGURE 102



PERMEABILITY – VOIDS RELATION FOR TEN DIFFERENT PROJECTS.
(AFTER ZUBE (56))
FIGURE 103

If the data for all fifteen test sections are considered collectively, a histogram can be prepared and a cumulative frequency distribution chart can be plotted. The chart for the relative densities before traffic and after four months traffic is shown in Figure 104. From these curves it can be seen that, after construction and before any traffic is allowed on the pavement, 84 percent of the samples from the normal construction operations did not attain 95 percent of the laboratory density. After one week of traffic approximately 50 percent of the test sections had reached 95 percent laboratory density. After four months, 80 percent had reached 95 percent relative compaction. After two years of service 80 percent of the pavement reached 95 percent relative compaction while only 20 percent reached 100 percent relative compaction (Appendix D). Thus it is apparent that these test sites were not compacted initially to the expected density nor did their density increase to the desired 2 to 6 percent air void content after two years of service, in approximately one third of the sites.

Data collected in this project (Figure 105) indicate that the ease with which a mix can be compacted in the laboratory can be used as an approximation of how well the mix can be compacted in the field with normal construction techniques. In particular the laboratory test indicated that the slag aggregate mixes (Milano and Bryan Projects) could not be compacted to a high density.

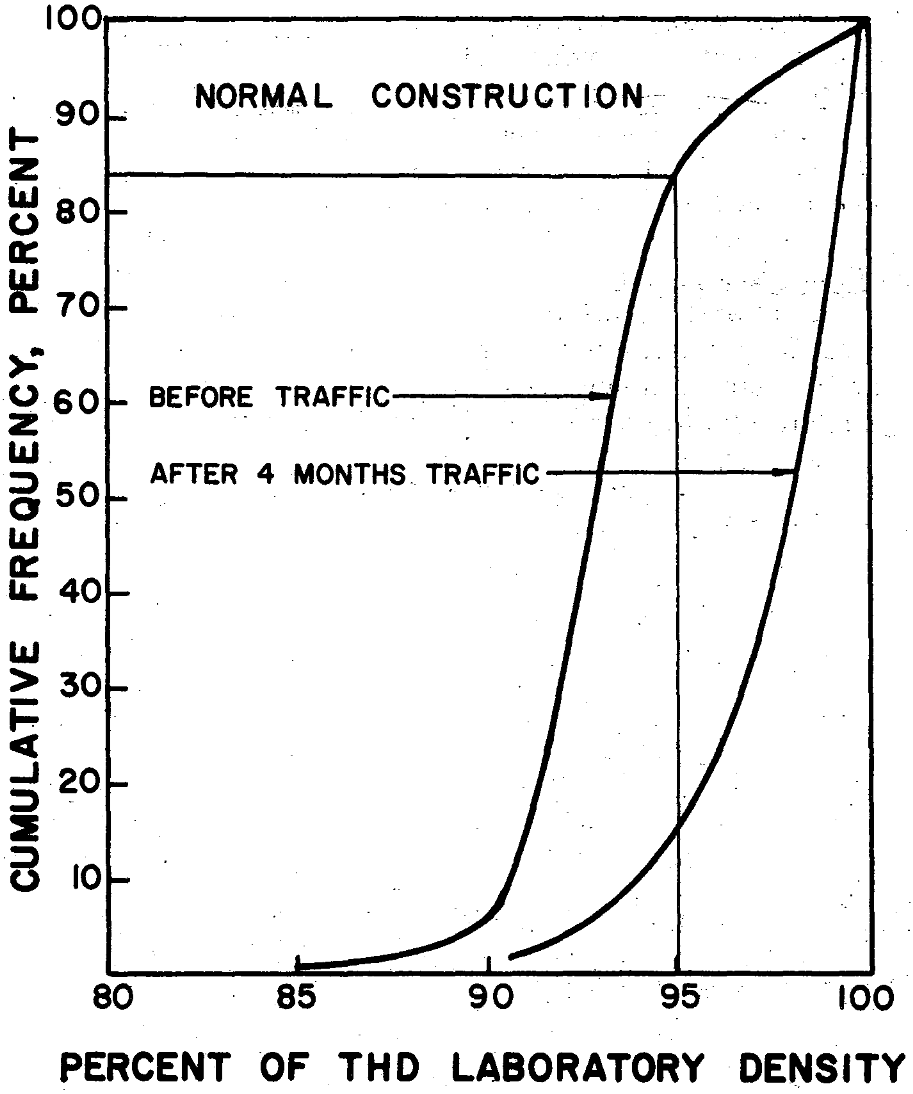
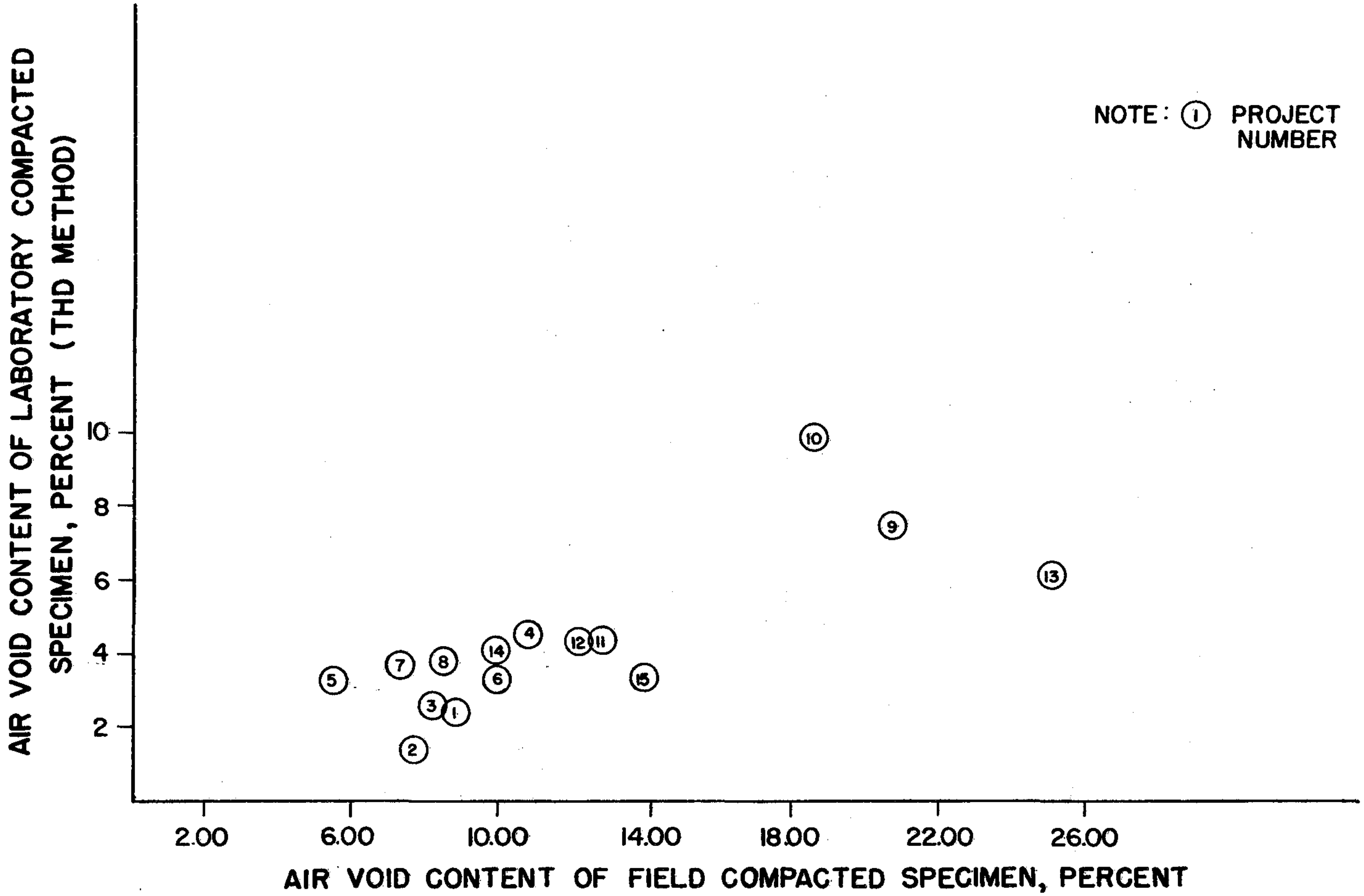


FIGURE 104 CUMULATIVE FREQUENCY RELATIVE DENSITY



RELATIONSHIP BETWEEN LABORATORY AND FIELD AIR VOID CONTENT

FIGURE 105

CONCLUSIONS AND RECOMMENDATIONS

Conclusion

1) The mechanical properties of asphalt concrete are largely determined by its density. Information presented in this study suggest that high densities are necessary if a pavement is to be durable and have adequate tensile strength, fatigue resistance, stiffness, and stability.

2) The engineer must consider a number of variables in order to obtain adequate initial compaction. These include:

a. The compaction characteristics of the particular asphalt-aggregate mixture which depend on aggregate absorption, aggregate surface texture, aggregate gradation, and asphalt type.

b. The environmental conditions under which the pavement will be placed.

c. The heat capacity of the mixture and its cooling rate for the particular geometry and environmental conditions under which it will be placed.

d. The timing and sequence of roller operations.

e. The type of equipment to be used.

f. The stiffness of the "base" material.

3) The long term density of a pavement is largely controlled by the following factors:

a. Amount of initial compaction

b. Amount of traffic

c. Type of traffic

d. Susceptibility of asphalt to harden

Daily temperature cycling may also contribute to the gain in density with age. However, the amount of densification due to this thermal cycling can not be determined from the information considered in this study.

4) The Texas Gyratory method of compacting specimen in the laboratory produces a more dense mix than any of the following methods:

  a. Corps of Engineers' Gyratory
  b. Marshall
  c. California Kneading

5) The majority of the pavements considered in this study were compacted to an air void content that ranged between 8 and 12 percent and obtained 95 percent relative compaction after 4 months. Decreases in air void content of 4 to 6 percent during two years of service were also noted.

Recommendations

As shown the initial density of a pavement is dependent upon a number of factors which are unique for any particular construction project. These factors are difficult to evaluate before actual construction. Therefore, it is recommended that trial compaction sections be included as a job requirement and constructed prior to the placement of the asphalt concrete. These trial sections would allow the engineer the opportunity to:

1) Determine the proper type of equipment needed to compact the particular mix.
2) Determine the timing and sequence of roller operation to compact the particular mix.
3) Determine the rate of heat loss for the particular mix and job geometry under conditions that will exist during full scale construction.

4) Determine the compaction characteristics of the particular mix.

5) Evaluate the "base" support for the particular job.

6) Calibrate non-destructive testing methods for use as compaction control devices during full scale construction.

These test sections should be constructed on the pavement structure that is to be paved and under environmental conditions that can be expected during the full scale construction.

The trial test section has been used successfully by several agencies and such usage resulted in optimum compaction with minimum compactive effort. This approach will be economically feasible for the contractor, as a minimum amount of effort will be used to obtain the desired density, and it will be economical to the owner as better control and higher densities will result.

As discussed previously, certain non-destructive testing methods afford the opportunity for rapid density determinations while the pavement is at an elevated temperature and further densification can take place. It is recommended that continued and widespread use be made of these techniques as they will allow the engineer to obtain higher densities.

Data reported herein suggest that breakdown rolling occurred in the majority of the test sections after the mat had cooled below 175°F. It is suggested that rolling be initiated after the laydown machine has placed the asphalt concrete or that thicker lifts be used so that the mat will retain its heat a longer period of time. Methods of obtaining greater densification during the laydown operation should be investigated. The mix is normally at an elevated temperature during this operation and thus can be easily compacted.

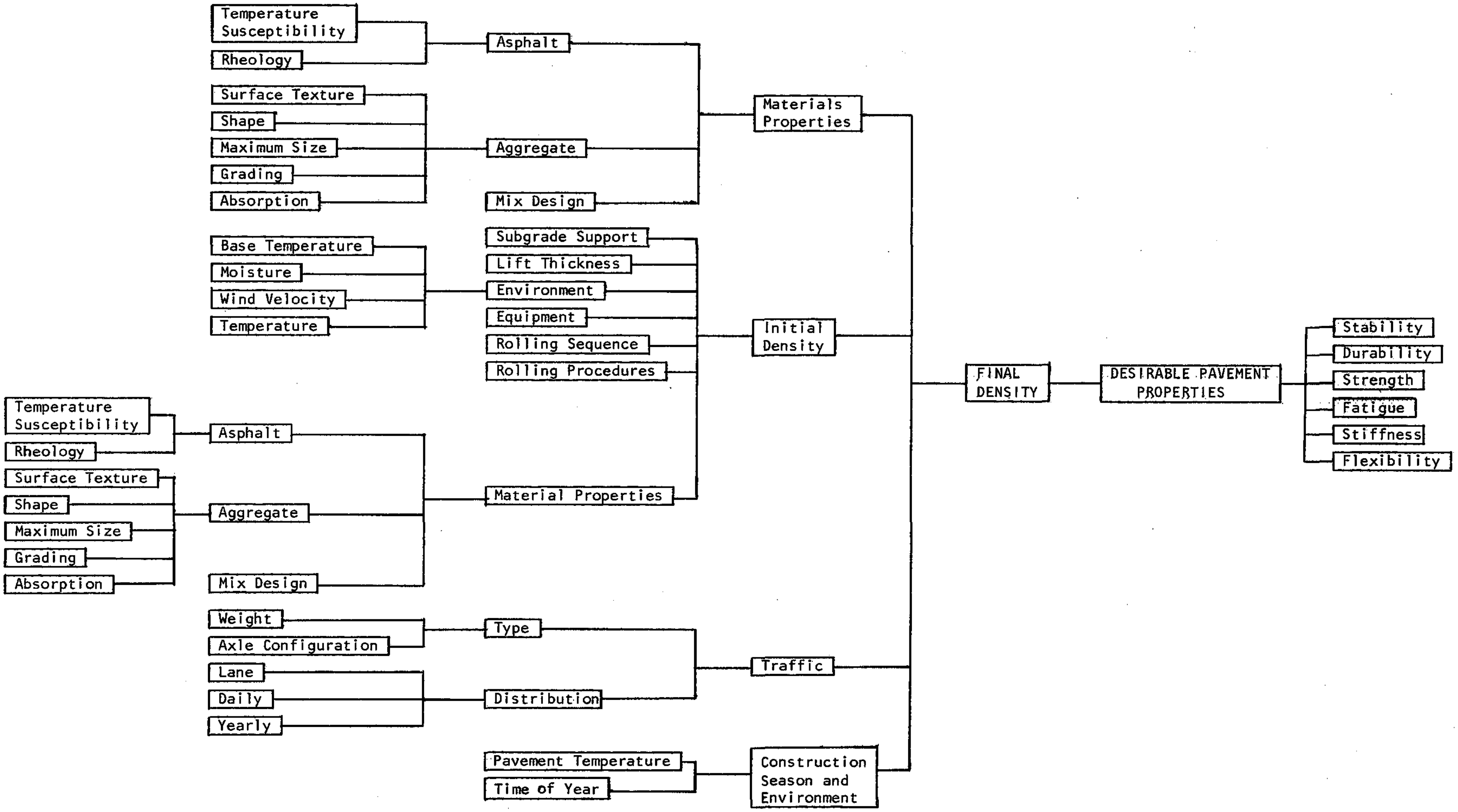


Figure 106. Factors Influencing Compaction of Asphalt Concrete Pavements

As suggested above considerable effort should be aimed at improving the initial density of the pavement as results contained herein indicate that only a 4 to 6 percent reduction in air voids occurs due to traffic over a two-year period. Less than half of the pavements studied reached the desired range in air void content of 2 to 6 percent.

This desired range in air void content is based on stability, durability, strength, fatigue resistance and stiffness requirements discussed previously. Furthermore, the rolling sequence most frequently used on these projects (steel-steel-pneumatic) should be compared with the more common (steel-pneumatic-steel) method used on particular projects to determine if the roller sequence is an important variable.

As an aid to the construction engineer, a flow diagram (Figure 106) has been prepared and it is suggested that it be referred to prior to the start of compaction operations. This diagram is intended to guide the engineer in order that he may consider the important items which control the compaction of asphalt concrete pavements. Details as to the relative importance of these factors and why these factors have been considered can be found in the report.

REFERENCES

1. ____________, "Manual of Testing Procedures-Volume I," Texas Highway Department, February, 1963.

2. ____________, "Materials Manual Volume I, Testing and Control Procedures," California Division of Highways, 1963.

3. Gallaway, Bob M., "Laboratory and Field Densities of Hot-Mix Asphaltic Concrete in Texas," Highway Research Board Bulletin 251, pp. 12-17, 1960.

4. Gallaway, Bob M., "A Limited Study of Laboratory and Field Densities of Hot-Mix Asphalt Concrete in Texas," Report for the Texas Highway Department, RP-13, 1960.

5. Gallaway, Bob M. and Harper, William J., "Interim Report on the Laboratory Considerations for the Use of Synthetic Aggregates for Hot-Mix Asphalt Pavements," Report for the Texas Highway Department, Research Report 51-3, 1966.

6. Monismith, C. L. and Vallerga, B. A., "Relationship Between Density and Stability of Asphaltic Paving Mixtures," Proceedings, Association of Asphalt Paving Technologists, Vol. 25, pp. 88-101, 1956.

7. McLeod, N. W., "Influence of Viscosity of Asphalt Cements on Compaction of Paving Mixtures in the Field," Highway Research Board Record 158, pp. 76-115, 1967.

8. Kiefer, R. W., "The Effect of Compaction Temperature on the Properties of Bituminous Concrete," American Society of Testing Materials, STP No. 294, pp. 19-26, 1960.

9. McRae, J. L., "Compaction of Bituminous Concrete," Proceedings, Association of Asphalt Paving Technologists, Vol. 26, pp. 206-213, 1957.

10. Bodell, R. S., "Field Compaction of Bituminous Concrete," Proceedings, Association of Asphalt Paving Technologists, Vol. 26, pp. 213-217, 1957.

11. Bahri, G. R. and Rader, L. F., "Effects of Asphalt Viscosity on Physical Properties of Asphaltic Concrete," Highway Research Board Record 67, pp. 59-79, 1965.

12. Goode, J. F. and Lufsey, L. A., "Voids Permeability, Film Thickness vs Asphalt Hardening," Proceedings, Association of Asphalt Paving Technologists, p. 430, 1965.

13. Heithaus, J. F. and Johnson, R. W., "A Microviscometer Study of Road Asphalt Hardening in the Field and Laboratory," Proceedings, Association of Asphalt Paving Technologists, Vol. 27, pp. 17-31, 1958.

14. Krchma, L. C. and Groening, T., "Influence of Pavement Voids, Asphalt Content and Asphalt Grade on Asphalt Performance," Proceedings, Association of Asphalt Paving Technologists, Vol. 28, pp. 34-47, 1959.

15. Tons, Egons and Krokosky, E. M., "Tensile Properties of Dense Graded Bituminous Concrete," Proceedings Association of Asphalt Paving Technologists, Vol. 32, pp. 497-528, 1963.

16. Livneh, M. and Shkrlarsky, E., "The Splitting Test for Determination of Bituminous Concrete Strength," Proceedings, Association of Asphalt Paving Technologists, Vol. 31, pp. 457-474, 1962.

17. Saal, R. M. J. and Pell, P. S., "Fatigue of Bituminous Road Mixes," Kolloid Zeitschrift (Darmstadt), Vol. 171, 1960.

18. Monismith, C. L., "Asphalt Mixture Behaviour in Repeated Flexure," Report No. TE65-9, ITTE.

19. Epps, Jon A. and Monismith, C. L., "Influence of Mixture Variables on the Flexural Fatigue Properties of Asphalt Concrete," Proceedings, Association of Asphalt Paving Technologists, Vol. 38, 1969.

20. Deacon, J. A., "Fatigue of Asphalt Concrete," Graduate Report, The Institute of Transportation and Traffic Engineering, University of California, Berkeley, 1965.

21. Van Draat, W. E. F. and Sommer, P. "Ein Gerat zur Bestimmung der dynamischen Elastizitates Moduln von Asphalt," Strasse and Autobahn, Vol. 35, 1966.

22. Santucci, L. E. and Schmidt, R. J., "Setting Rate of Asphalt Concrete," Highway Research Board Bulletin 333, pp. 1-8, 1962.

23. Kallas, B. F. and Krieger, H. C., "Effects of consistency of Asphalt Cements and Types of Mineral Filler on the Compaction of Asphalt Concrete," Proceedings, Association of Asphalt Paving Technologists, Vol. 29, pp. 152-171, 1960.

24. Santucci, L. E. and Schmidt, R. J., "Setting Rate of Asphalt Concrete," Highway Research Board Bulletin 333, pp. 1-8, 1962.

25. Nijboer, L. W., "The Compaction of Asphaltic Road Mixtures by Rolling," Proceedings, Association of Asphalt Paving Technologists, Vol. 33, pp. 241-268, 1964.

26. Fromm, H. J., "The Compaction of Asphaltic Concrete on the Road," Proceedings, Association of Asphalt Paving Technologists, Vol. 33, pp. 241-268, 1964.

27. Bright, Richard, Steed, Boyd, Steele, John, and Justice, Allan, "The Effect of Viscosity of Asphalt on Properties of Bituminous Wearing Surface Mixtures," Proceedings, Association of Asphalt Paving Technologists, Vol. 35, p. 164, 1966.

28. Schmidt, R. M., Kari, W. J., Bower, H. C., and Hein, T. C., "Behaviour or Hot Asphaltic Concrete Under Steel Wheel Rollers," Highway Research Bulletin 251, 1960.

29. Epps, Jon A., "Influence of Mixture Variables on the Flexural Fatigue and Tensile Properties of Asphalt Concrete," Thesis, University of California, Berkeley, 1968.

30. Parker, C. F., "Steel-Tired Rollers," Highway Research Board Bulletin 246, 1960.

31. Swanson, R. C., Nemec, Joseph, Jr., and Tons, Egons, "Effect of Asphalt Viscosity on Compaction of Bituminous Concrete," Highway Research Board Record 117, pp. 23-53, 1966.

32. Parr, Ward K., Serafin, P. J., and Humphries, Tom, "Michigan State Highway Experimental Bituminous Concrete Construction Project," Proceedings, Association of Asphalt Paving Technologists, Vol. 24, pp. 125-171, 1955.

33. Graham, M. D., Burnett, W. C., Thomas, Jerome J., and Dixon, W. C. "Pavement Density - What Influences It," Proceedings, Association of Asphalt Paving Technologists, Vol. 34, p. 286, 1965.

34. Kari, W. J., "Discussion to 'Symposium on Compaction of Asphalt Concrete,'" Proceedings, Association of Asphalt Paving Technologists, Vol. 36, pp. 288-294, 1967.

35. Marker, Vaughn, "Introduction, 'Symposium on Compaction of Asphalt Concrete,'" Proceedings, Association of Asphalt Paving Technologists, Vol. 36, pp. 288-294, 1967.

36. Chastain, W. E. and Burke, J. E., "State Practices in the Use of Bituminous Concrete," Highway Research Board Bulletin 160, Bituminous Paving Mixture, 1957.

37. Benson, F. J., "Survey of Use of Crushed Aggregates In Hot Mix Asphalt Pavements," National Asphalt Pavement Association, Publication Q1P-91, September, 1968.

38. Barber, E. S., "Calculation of Maximum Pavement Temperatures from Weather Reports," Highway Research Board Bulletin No. 168, 1957.

39. Serafin, P. J. and Kole, L. L., "Comparative Studies of Pneumatic Tire Rolling," Proceedings, Association of Asphalt Paving Technologists, Vol. 31, pp. 418-456, 1962.

40. Bituminous Pavement Investigation, Texas Highway Department, Report L1-14-68-F, October, 1968.

41. Corlew, J. S. and Dickson, P. F., "Methods of Calculating Temperatures Profiles of Hot Mix Asphalts Concrete as Related to the Construction of Asphalt Pavements," Proceedings, Assoication of Asphalt Paving Technologists, Vol. 37, 1968.

42. Nijboer, L. W., "Plasticity as a Factor in the Design of Dense Bituminous Road Carpets," p. 83, 1948.

43. Schmidt, R. J., Santucci, L. E., and Garrison, W. A., "High Temperatures Pneumatic Compaction," Highway Research Board Record No. 132, 1966.

44. Arena, Philip, Shah, S. C., and Adam, Verdi, "Compaction of Asphaltic Concrete Pavement with High Intensity Pneumatic Roller," Highway Research Board Record No. 178, 1967.

45. Bodell, R. S., "Field Compaction of Bituminous Concrete," Proceedings, Association of Asphalt Paving Technologists, Vol. 26, pp. 213-217, 1957.

46. Kari, W. J., "Mix Properties as They Influence Compaction," Proceedings, Association of Asphalt Paving Technologists, Vol. 36, pp. 295-309, 1967.

47. Foster, C. R., "The Current State of the Art of Compaction Procedure," Presented at the Highway Research Board Annual Meeting, January, 1969.

48. The Asphalt Handbook, The Asphalt Institute, 1965.

49. Gartner, W., Cobb, D. A., and Lindley, R. W., "Field Compaction Studies on Asphaltic Concrete," Highway Research Bulletin No. 358, pp. 38-53.

50. Kenis, W. J., "Progress Report on Changes in Asphaltic Concrete in Service," Highway Research Board Bulletin 333, pp. 39-65, 1962.

51. Palmer, Robert K. and Thomas, J. J., "Pavement Density - How It Changes," Proceedings, Association of Asphalt Paving Technologists, Vol. 37, pp. 542-574, 1968.

52. Campen, W. H., Smith, J. R., Erickson, L. G., and Mertz, L. R., "The Effect of Traffic on the Density of Bituminous Paving Mixtures," Proceedings, Association of Asphalt Paving Technologists, Vol. 30, pp. 378-388, 1961.

53. Gallaway, B. M., "Laboratory and Field Densities of Hot Mix Asphaltic Concrete in Texas," Highway Research Board Bulletin 251, pp. 12-17, 1960.

54. Pauls, J. T. and Halstead, W. J., "Progressive Alterations in a Sheet Asphalt Pavement Over a Long Period of Time," Proceedings, Association of Asphalt Paving Technologists, Vol. 27, 1958.

55. Gallaway, B. M., "Discussion of 'Pavement Density - How It Changes,'" Proceedings, Association of Asphalt Paving Technologists, Vol. 37, p. 577, 1968.

56. Zube, Ernest, "Compaction Studies of Asphalt Concrete Pavement as Related to the Water Permeability Test," Highway Research Board Bulletin 358, pp. 12-31, 1962.

57. ____________, " Road Test One - MD," Highway Research Board, Special Report 4, 1949.

58. Portland Cement Association, Thickness Design for Concrete Pavements, Chicago, Illinois, 1966.

59. Monismith, C. L., "Asphalt Mixture Behaviour in Repeated Flexure," Report TE 66-6, University of California, Berkeley, 1966.

60. McRae, J. L., "Compaction of Bituminious Concrete," Proceedings, Association of Asphalt Paving Technologists, Vol. 26, pp. 206-213, 1957.

61. Kimble, F., "A Field Method of In-Place Density Determination and Control of Hot Mix Asphaltic Concrete," Proceedings, Association of Asphalt Paving Technologists, Vol. 27, pp. 265-291, 1958.

62. Kari, W. J. and Santucci, L. E., "Control of Asphalt Concrete Construction by the Air Permeability Test," Proceedings, Association of Asphalt Paving Technologists, Vol. 32, 1963.

63. Ellis, W. H. and Schmidt, R. J., "A Method of Measuring the Air Permeabilities of Asphalt Concrete Pavements," American Society for Testing Materials STP No. 294, pp. 85-91, 1960.

64. Morgan, F. D., Decker, G. L., Yarbrough, F. W., and Fields, J. W., "Verification of Nuclear Moisture and Density Gauges," Oregon State Highway Department, 1966.

65. Handy, Henry and Baxter, C. W., "Evaluation of the Asphalt Density Gauge and Compaction of Bituminous Pavements," Texas Highway Department, Report No. SS6.0, July, 1967.

66. Hughes, C. S., "Investigation of a Nuclear Device for Determining the Density of Bituminous Concrete," Proceedings, Association of Asphalt Paving Technologists, Vol. 31, pp. 400-416, 1962.

67. Hughes, C. S. and Ralston, H. H., "Field Testing of a Nuclear Density Device on Bituminous Concrete," Proceedings, Association of Asphalt Paving Technologists, Vol. 32, pp. 106-142, 1963.

68. Stephens, Jack E., "Discussion of Symposium on Compaction," Proceedings, Association of Asphalt Paving Technologists, Vol. 36, p. 357, 1967.

69. Jones, George M., "Discussion of Symposium on Compaction," Proceedings, Association of Asphalt Paving Technologists, Vol. 36, p. 357, 1967.

70. Personal Correspondence with W. A. Garrison, 1969.

APPENDIX A

TABLE A1 PERCENT AIR VOIDS IN FIELD SAMPLES

Test Section	Sub-Section	Wheel Path	1 Day	1 Week	1 Month	4 Months	1 Year	2 Years
Childress US 287 25-42-9	A*	I**	10.20	9.34	7.16	4.20	4.15	3.70
		B**	9.62	9.66	8.73	5.92	6.23	5.23
		0**	10.53	9.84	7.60	4.55	4.17	3.90
	B*	I	8.69	8.45	5.60	3.39	3.90	3.32
		B	8.45	8.10	6.88	5.18	5.60	5.22
		0	9.20	8.15	6.26	3.97	3.93	4.08
	C*	I	8.58	8.33	6.81	3.81	3.84	3.98
		B	8.93	8.77	8.43	5.38	5.85	5.92
		0	9.03	9.72	7.80	5.11	5.11	4.70
Matador US 70 25-145-8	A	I	5.71	6.50	6.05	6.25	4.47	5.79
		B	6.25	6.42	6.02	5.94	4.62	4.57
		0	7.11	7.11	7.43	7.31	5.23	7.33
	B	I	7.68	7.79	8.77	7.47	5.36	6.77
		B	7.38	7.41	6.92	7.39	5.20	5.80
		0	9.06	8.58	9.19	9.88	7.90	8.92
	C	I	3.94	4.96	4.65	5.38	4.15	4.12
		B	5.56	6.40	6.16	6.19	3.50	4.11
		0	7.61	7.80	7.14	6.43	5.43	5.72
Sherman SH 5 1-47-3	A	I	9.32	8.05	8.80	7.39	4.29	4.66
		B	7.10	5.43	5.74	6.96	4.28	4.65
		0	7.19	7.21	5.62	5.64	2.70	3.33
	B	I	8.26	7.30	7.12	6.60	4.05	3.75
		B	6.99	5.96	5.63	6.45	4.36	4.75
		0	7.49	6.08	5.83	5.49	2.99	3.73
	C	I	7.77	6.14	6.49	6.72	3.80	3.79
		B	6.31	5.15	5.11	6.30	4.51	4.26
		0	6.82	5.78	4.66	5.09	3.13	4.31
Cooper SH 24 1-136-3	A	I	10.94	9.54	9.12	6.29	6.57	6.62
		B	10.43	9.30	9.23	6.70	6.33	6.30
		0	10.21	10.33	9.08	6.87	5.88	6.18
	B	I	10.85	9.54	9.29	6.92	6.32	6.17
		B	8.43	7.44	7.47	5.63	6.22	5.80
		0	10.82	9.34	8.63	7.04	6.72	6.50
	C	I	9.21	8.03	8.04	5.81	5.72	6.02
		B	7.93	8.00	6.91	5.54	5.33	5.72
		0	8.47	8.12	7.35	5.91	5.06	5.48
Cumby IH 30 1-9-13	A	I	8.02	6.92	6.94	3.06	1.95	1.79
		B	5.87	5.14	5.36	3.14	2.74	2.51
		0	7.11	6.27	6.22	3.40	1.88	1.59
	B	I	5.51	5.38	5.48	2.40	2.22	1.71
		B	4.08	4.64	4.77	2.87	2.31	2.05
		0	4.75	5.03	4.68	2.18	2.01	1.99
	C	I	4.77	6.01	4.84	2.85	1.56	1.60
		B	3.91	4.29	4.56	3.37	3.30	2.04
		0	4.50	4.28	4.59	2.20	1.64	1.52
Clifton SH 6 9-258-7	A	I	9.86	8.18	6.94	7.62	6.67	6.12
		B	9.65	8.97	8.90	8.45	7.39	8.45
		0	9.74	8.08	7.87	7.66	7.38	7.35
	B	I	9.89	7.02	6.47	6.07	5.46	6.23
		B	8.59	8.30	8.19	7.90	7.15	8.42
		0	9.13	7.98	6.96	7.19	6.82	6.59
	C	I	7.78	7.60	6.63	5.93	5.53	5.07
		B	7.68	7.62	7.21	6.93	6.60	6.61
		0	8.18	7.72	6.72	6.19	6.39	6.11
Waco US 84 9-55-8	A	I	7.23	5.52	2.48	1.80	2.59	1.95
		B	7.60	6.52	4.00	4.04	3.70	3.61
		0	7.58	5.89	2.53	2.57	3.01	2.36
	B	I	7.39	5.27	2.86	2.71	2.26	1.57
		B	6.77	5.18	3.75	3.07	2.76	2.30
		0	7.14	4.90	2.76	2.11	2.71	2.81
	C	I	6.35	4.55	3.05	2.57	2.22	1.89
		B	5.50	3.84	2.97	3.12	2.32	1.72
		0	5.55	3.85	2.34	1.97	2.34	2.25
Robinson US 77 9-209-1	A	I	9.41	7.79	8.56	6.34	5.08	5.03
		B	8.25	6.50	7.78	4.96	5.00	4.95
		0	8.71	7.34	6.75	6.07	4.06	5.05
	B	I	8.53	7.14	7.91	5.27	4.40	3.92
		B	7.62	6.90	9.31	5.72	4.91	4.77
		0	9.40	7.44	6.13	6.02	5.06	4.53
	C	I	7.01	6.86	7.43	4.50	5.42	4.73
		B	4.79	4.48	6.75	3.61	4.79	3.96
		0	6.39	5.89	4.99	4.62	5.66	3.73

TABLE A1 PERCENT AIR VOIDS IN FIELD SAMPLES
(Cont'd)

Test Section	Sub-Section	Wheel Path	1 Day	1 Week	1 Month	4 Months	1 Year	2 Years
Milano SH 36 17-185-4	A	I	19.23	17.96	18.00	16.77	16.31	16.85
		B	21.47	17.69	18.83	18.31	16.78	17.26
		0	19.71	18.55	18.10	17.55	17.14	17.35
	B	I	20.79	18.47	18.37	17.33	17.16	16.19
		B	21.39	18.36	17.97	17.89	17.41	16.80
		0	20.96	18.64	19.06	18.17	17.93	17.70
	C	I	17.85	17.46	17.60	17.21	16.59	17.12
		B	20.02	17.26	17.45	17.85	16.42	15.81
		0	18.54	16.75	16.93	17.83	15.42	15.17
Bryan Spur 308 17-599-1	A	I	21.46	17.38	16.90	15.91	14.22	14.48
		B	19.41	18.19	18.91	16.76	16.50	19.66
		0	17.31	16.21	17.37	15.61	13.85	17.98
	B	I	18.76	18.24	17.96	16.23	15.78	15.00
		B	17.40	16.94	18.45	16.16	15.98	15.60
		0	16.33	16.85	17.44	14.57	13.86	14.45
	C	I	18.73	17.68	17.45	14.95	14.97	18.11
		B	17.12	16.09	17.00	14.33	13.26	12.67
		0	16.69	15.59	15.79	14.06	13.94	13.67
Tamina IH 45 12-110-4	A	I	13.75	8.04	6.25	6.13	5.79	5.52
		B	13.07	9.32	7.48	6.39	6.39	5.90
		0	13.74	9.34	7.12	6.73	5.95	4.98
	B	I	12.72	8.80	7.80	6.18	5.55	5.00
		B	11.18	8.96	7.20	7.01	6.07	6.57
		0	13.56	9.12	6.94	5.92	5.44	5.68
	C	I	11.28	6.81	5.63	5.61	5.12	5.35
		B	10.01	7.91	6.89	7.36	5.84	6.14
		0	12.57	7.72	6.35	5.92	5.33	4.97
Conroe FM 1485 12-1062-35	A	I	11.53	11.59	11.65	9.38	9.44	9.96
		B	10.81	11.82	10.94	9.61	9.13	9.50
		0	9.81	10.28	8.93	8.88	7.89	7.51
	B	I	12.34	11.09	9.90	8.24	8.23	8.20
		B	10.54	10.42	10.02	8.13	7.94	8.50
		0	10.04	9.33	7.76	7.26	6.31	6.54
	C	I	11.87	11.09	10.10	9.08	8.58	8.56
		B	10.45	11.25	10.91	8.58	8.91	8.94
		0	9.68	9.03	8.38	7.52	7.08	6.33
Baytown Spur 330 12-508-7	A	I	19.71	11.45	7.18	5.97	4.91	5.52
		B	23.03	10.30	7.87	7.13	6.36	7.23
		0	23.55	11.94	7.64	6.34	4.73	5.42
	B	I	25.88	11.66	7.49	6.12	5.23	5.30
		B	25.12	10.35	8.53	7.60	6.78	8.05
		0	25.34	11.59	7.42	6.09	5.04	6.34
	C	I	11.70	11.31	7.77	6.00	4.97	4.75
		B	10.60	10.44	7.90	7.13	6.43	6.71
		0	11.38	10.80	7.71	5.90	5.15	3.48
Orange SH 12 20-499-3	A	I	11.69	7.97	7.58	6.28	5.49	5.27
		B	12.56	8.40	9.00	7.79	8.39	8.31
		0	12.33	7.67	6.08	6.06	6.49	5.70
	B	I	10.02	7.02	6.41	5.42	5.28	4.66
		B	11.50	8.20	9.11	7.53	7.16	6.95
		0	11.62	7.58	7.16	5.58	5.94	5.47
	C	I	8.51	6.43	6.09	4.68	5.27	4.63
		B	8.56	9.10	8.72	5.93	6.03	5.33
		0	7.92	7.24	6.78	4.92	4.71	4.48
Bridge City IH 87 20-306-3	A	I		8.22	8.97	8.11	8.76	8.45
		B	11.60	9.14	8.17	8.38	8.39	8.28
		0	12.10	9.49	7.83	7.95	8.27	7.58
	B	I	13.83	10.12	8.49	8.71	8.77	8.32
		B	13.63	11.36	9.83	9.96	9.36	9.32
		0	13.33	9.48	7.30	7.78	6.89	7.85
	C	I	13.51	10.30	9.47	8.80	8.54	7.28
		B	11.72	10.26	7.99		9.50	8.11
		0	14.12	9.27	7.36	8.26	7.43	9.10

*Subsection A - half as many roller passes as subsection B
*Subsection B - normal roller procedure for particular project
*Subsection C - twice as many roller passes as subsection B

** I - inner wheel path
** B - between wheel path
** 0 - outer wheel path

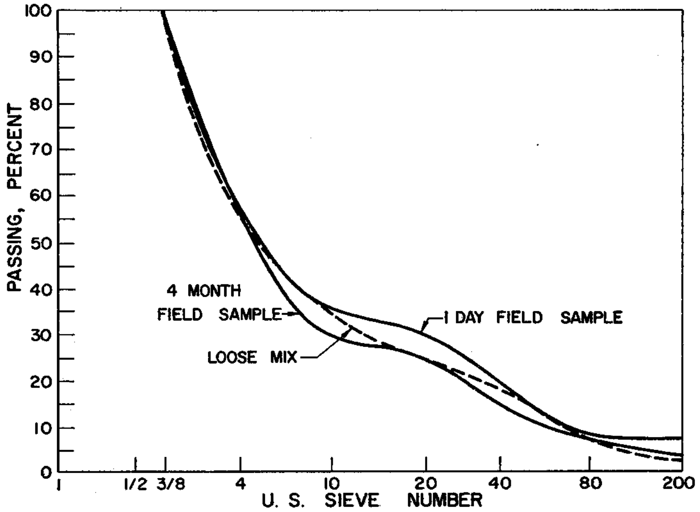
APPENDIX B

TABLE B1 RELATIVE COMPACTION (PERCENT OF STANDARD THD)

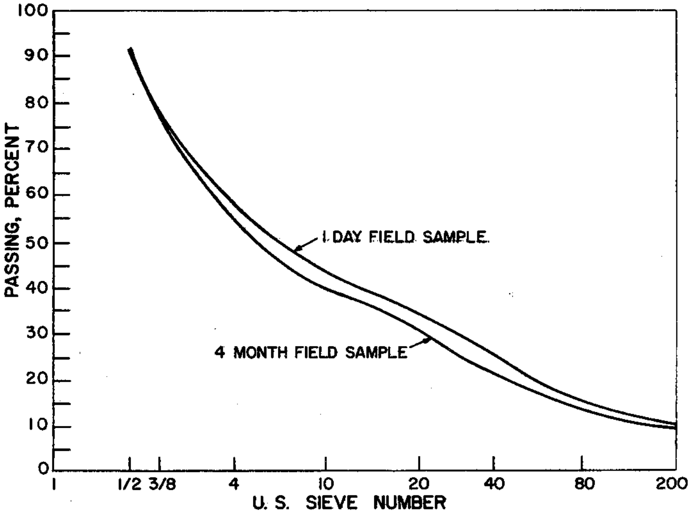
Location	Section	Wheel Path	1 Day	1 Week	1 Month	4 Months	1 Year	2 Years
Childress	A	I	92.2	93.0	95.2	98.3	98.3	98.8
		B	92.7	93.0	93.6	96.9	96.2	97.2
		0	91.8	92.5	94.8	97.9	98.3	98.6
	B	I	93.7	93.9	96.9	99.1	98.6	99.2
		B	93.9	94.3	95.6	97.2	96.8	97.2
		0	93.2	94.4	96.1	98.5	98.6	98.4
	C	I	94.4	94.0	95.6	98.7	98.7	98.5
		B	93.4	93.6	94.0	97.1	96.6	96.6
		0	93.3	92.6	94.6	97.7	97.4	97.8
Matador	A	I	95.7	94.9	95.4	95.2	97.0	97.7
		B	95.2	95.1	95.4	95.5	96.9	99.0
		0	94.3	94.3	94.0	94.1	96.2	96.1
	B	I	93.7	93.6	94.5	94.1	99.1	97.7
		B	94.0	94.0	92.7	93.9	95.8	96.7
		0	92.3	92.8	92.2	91.5	93.5	94.5
	C	I	97.4	96.5	97.8	96.1	97.44	99.4
		B	95.9	95.0	95.3	95.3	97.9	99.4
		0	93.8	93.6	94.3	95.0	100.2	97.8
Sherman	A	I	93.0	94.3	93.5	95.0	98.1	97.7
		B	94.2	97.0	96.7	94.4	98.2	97.7
		0	95.2	95.1	97.7	96.7	98.3	99.1
	B	I	94.1	95.0	95.2	95.8	98.4	98.7
		B	95.4	96.4	96.8	95.9	98.1	97.7
		0	94.9	96.3	96.6	96.9	99.5	98.7
	C	I	94.6	96.3	95.9	95.7	98.7	98.6
		B	96.0	97.3	97.2	96.1	97.9	98.2
		0	95.6	96.6	97.7	97.4	97.2	98.1
Cooper	A	I	93.0	94.5	94.9	97.9	97.6	97.9
		B	93.6	94.8	94.9	93.3	97.5	98.2
		0	93.8	93.7	95.0	97.3	98.4	96.2
	B	I	93.1	94.5	94.8	97.3	97.9	96.2
		B	95.7	96.7	96.7	98.6	98.0	98.7
		0	93.2	94.6	95.4	97.1	97.5	98.0
	C	I	94.9	96.1	96.1	98.4	98.5	98.5
		B	96.2	96.1	97.3	98.7	98.9	98.8
		0	95.6	96.0	96.8	98.3	99.2	99.1
Cumby	A	I	95.0	96.1	96.1	100.1	101.8	101.4
		B	97.2	98.0	97.7	100.0	100.4	100.6
		0	95.9	96.8	96.8	99.7	101.3	101.6
	B	I	97.6	97.7	97.6	100.8	100.9	101.5
		B	99.0	98.4	98.3	100.3	100.8	101.1
		0	98.4	98.1	98.4	101.0	100.3	101.2
	C	I	98.3	97.0	98.2	101.0	101.6	101.6
		B	99.2	98.8	98.2	99.7	99.8	101.1
		0	98.6	98.8	98.5	101.0	102.2	101.6
Clifton	A	I	93.4	95.2	92.6	95.8	96.9	97.3
		B	93.6	94.3	94.4	94.8	96.0	94.9
		0	93.5	95.2	95.5	95.7	95.9	96.0
	B	I	93.6	96.4	96.9	97.4	98.0	97.2
		B	94.7	95.0	95.1	95.4	96.2	95.1
		0	93.9	95.3	96.4	96.2	96.6	96.8
	C	I	94.9	96.1	96.8	97.5	97.9	98.4
		B	95.6	96.1	96.2	96.4	96.8	96.8
		0	95.1	95.6	96.7	97.2	97.0	97.3
Waco	A	I	96.4	97.1	101.4	101.3	100.8	101.9
		B	95.9	97.2	99.8	99.8	100.0	100.2
		0	96.0	97.8	101.3	101.3	100.8	101.5
	B	I	96.3	98.5	101.0	101.4	101.6	102.3
		B	96.9	98.6	100.0	100.7	93.7	101.6
		0	96.6	98.9	101.0	101.8	101.1	101.8
	C	I	97.3	99.2	100.8	101.3	101.6	102.0
		B	98.2	100.0	100.8	100.7	101.5	102.0
		0	98.2	100.0	101.5	101.9	101.5	101.6
Robinson	A	I	93.8	95.8	96.2	97.3	98.6	98.7
		B	95.3	96.4	95.8	98.8	98.7	98.8
		0	94.9	96.2	94.7	97.6	99.7	98.8
	B	I	95.0	96.5	95.7	98.4	99.4	99.8
		B	96.0	96.7	94.3	98.0	98.8	99.0
		0	94.1	95.8	93.7	97.7	98.6	99.2
	C	I	95.9	96.7	95.0	99.3	98.1	99.0
		B	98.9	99.2	96.9	100.1	98.9	99.8
		0	97.2	97.8	98.7	99.1	98.1	100.0

TABLE B1 RELATIVE COMPACTION (PERCENT OF STANDARD THD)
(CONT'D)

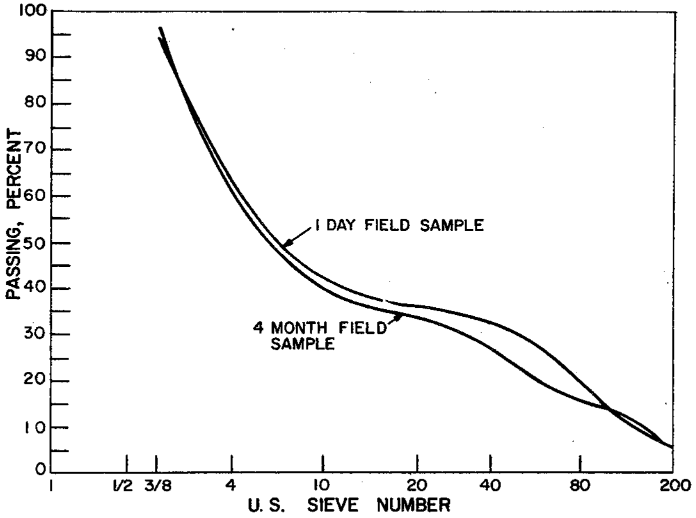
Location	Section	Wheel Path	1 Day	1 Week	1 Month	4 Months	1 Year	2 Years
Milano	A	I	87.4	88.7	88.7	90.4	90.5	90.0
		B	84.9	89.0	87.8	88.4	90.1	89.5
		0	86.8	88.1	88.6	89.2	89.7	89.4
	B	I	85.7	88.2	88.3	89.4	89.6	90.7
		B	85.0	88.3	88.7	88.8	89.4	90.0
		0	85.5	88.0	87.5	88.5	88.8	89.1
	C	I	88.9	89.3	89.1	89.6	90.2	89.7
		B	86.5	89.5	89.3	87.1	90.5	91.1
		0	88.1	90.1	89.9	90.0	91.5	91.8
Bryan	A	I	87.2	91.8	92.3	92.3	95.3	94.6
		B	89.5	90.9	90.1	92.5	89.7	94.9
		0	92.0	93.0	91.8	93.7	95.7	96.0
	B	I	90.2	90.8	91.1	93.0	93.5	94.4
		B	91.8	93.5	90.5	93.1	93.3	93.7
		0	93.0	92.4	91.7	94.9	95.7	95.0
	C	I	90.3	91.4	93.9	94.5	94.5	95.5
		B	90.9	93.2	92.2	95.1	96.4	98.2
		0	92.5	93.6	93.4	95.5	95.6	96.0
Tamina	A	I	90.0	96.0	97.9	96.7	98.4	98.7
		B	90.7	94.7	96.6	97.7	97.7	98.2
		0	90.0	94.6	96.9	97.3	98.2	99.2
	B	I	91.1	95.2	96.3	97.9	98.6	99.2
		B	92.7	95.0	96.9	97.1	98.1	97.6
		0	90.2	94.1	97.1	99.2	98.7	98.8
	C	I	92.8	97.3	98.5	98.5	99.1	98.8
		B	93.9	96.1	97.2	96.6	98.3	98.0
		0	91.2	95.7	97.8	98.2	98.8	99.2
Conroe	A	I	94.3	92.4	92.3	94.7	94.6	94.2
		B	93.2	92.1	93.1	92.6	94.9	94.6
		0	92.4	93.8	95.2	95.2	96.2	96.6
	B	I	91.6	92.9	94.1	95.9	95.9	95.9
		B	93.6	93.6	94.0	96.0	96.2	95.6
		0	94.0	95.0	96.4	96.9	97.9	97.7
	C	I	92.1	92.9	98.2	95.0	95.5	95.5
		B	93.2	92.6	92.7	95.5	95.2	95.2
		0	94.4	95.1	95.9	96.6	97.1	97.9
Baytown	A	I	85.4	94.2	98.8	100.0	99.8	99.8
		B	81.1	95.4	98.0	98.8	99.6	98.7
		0	81.3	93.7	98.3	99.6	101.3	101.5
	B	I	78.9	94.0	98.4	99.9	100.8	100.7
		B	79.7	95.4	97.3	98.3	99.2	97.9
		0	79.4	94.1	98.5	99.9	101.0	100.5
	C	I	94.0	94.4	98.1	100.0	101.1	101.4
		B	95.2	95.1	98.0	98.8	99.6	99.3
		0	94.3	94.9	98.2	100.1	100.9	102.7
Orange	A	I	94.8	97.3	97.2	97.5	98.3	98.5
		B	95.0	94.6	94.6	95.9	95.3	95.4
		0	95.6	96.5	97.1	97.7	97.1	98.1
	B	I	93.5	96.7	97.3	98.4	98.5	99.2
		B	92.1	95.3	94.6	96.2	96.5	96.8
		0	91.9	96.2	96.6	98.2	97.9	98.4
	C	I	91.9	95.3	94.9	97.8	97.5	98.5
		B	91.0	95.3	94.9	97.8	97.5	98.5
		0	91.0	96.0	97.0	99.0	99.1	99.5
Bridge City	A	I		94.8	95.1	95.0	94.3	94.6
		B	91.3	93.9	94.9	94.7	94.7	94.8
		0	90.8	93.5	95.3	95.2	94.8	95.6
	B	I	89.0	92.9	94.5	94.4	94.3	94.8
		B	89.2	91.6	93.2	93.1	93.7	93.8
		0	89.5	93.5	95.8	95.4	96.3	95.3
	C	I	89.4	92.7	92.8	94.3	94.4	95.9
		B	92.0	92.7	95.1		93.6	95.0
		0	88.7	93.8	95.7	94.9	95.7	94.3



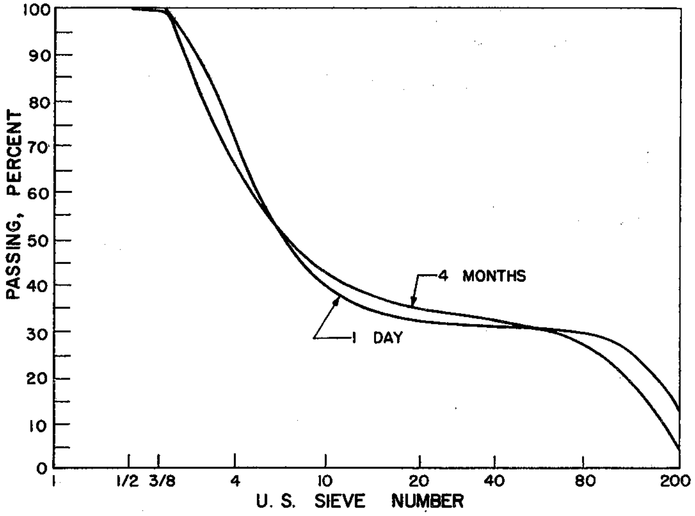
AGGREGATE DEGRADATION
(CHILDRESS TEST SECTION)
FIGURE B-1



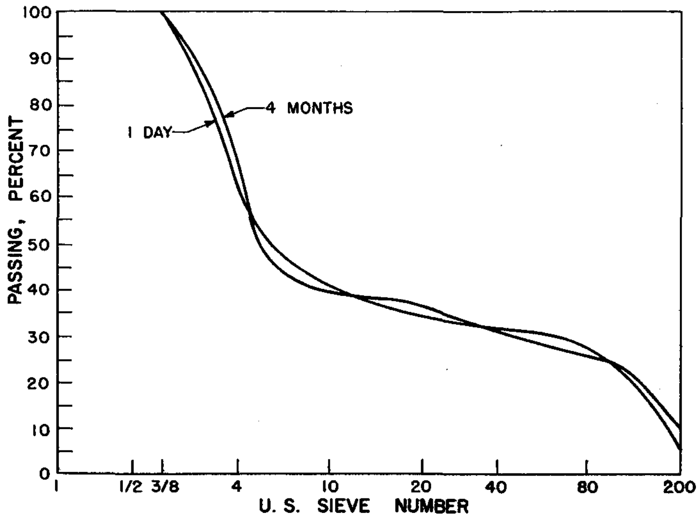
AGGREGATE DEGRADATION
(MATADOR TEST SECTION)
FIGURE B-2



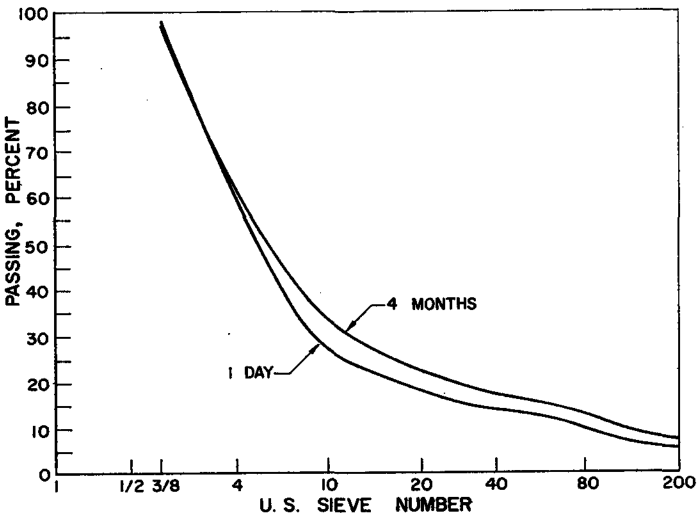
AGGREGATE DEGRADATION
(SHERMAN TEST SECTION)
FIGURE B-3



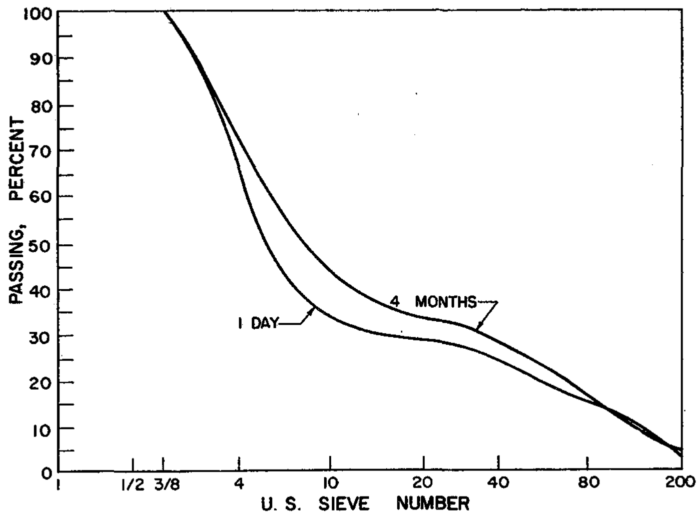
AGGREGATE DEGRADATION
(COOPER TEST SECTION)
FIGURE B-4



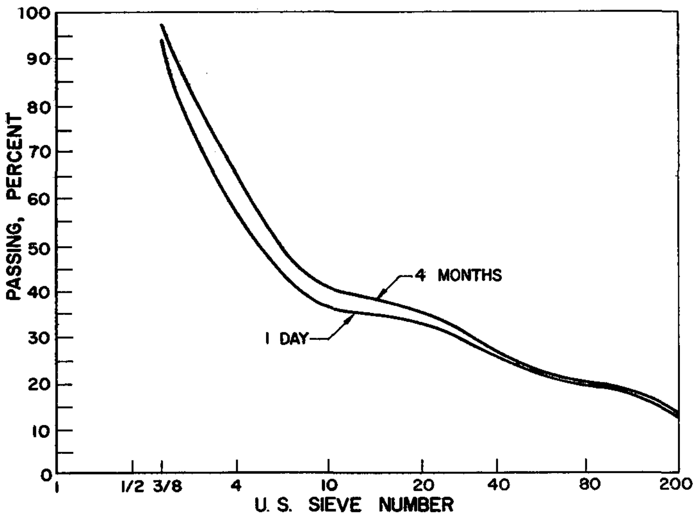
AGGREGATE DEGRADATION
(CUMBY TEST SECTION)
FIGURE B-5



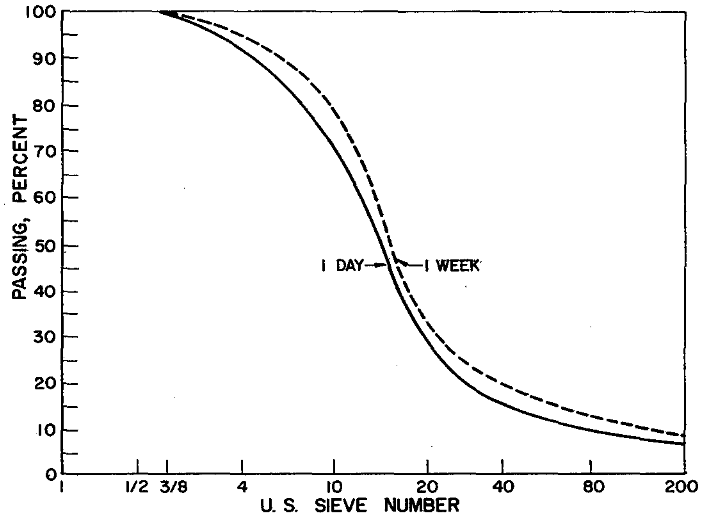
AGGREGATE DEGRADATION
(CLIFTON TEST SECTION)
FIGURE B-6



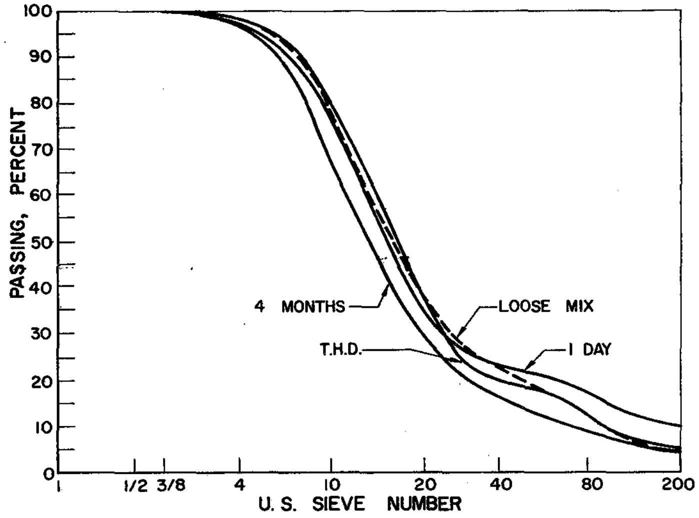
AGGREGATE DEGRADATION
(WACO TEST SECTION)
FIGURE B-7



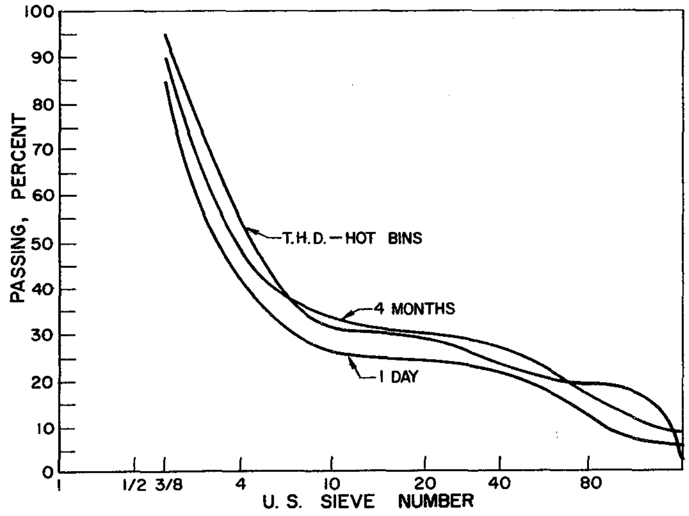
AGGREGATE DEGRADATION
(ROBINSON TEST SECTION)
FIGURE B-8



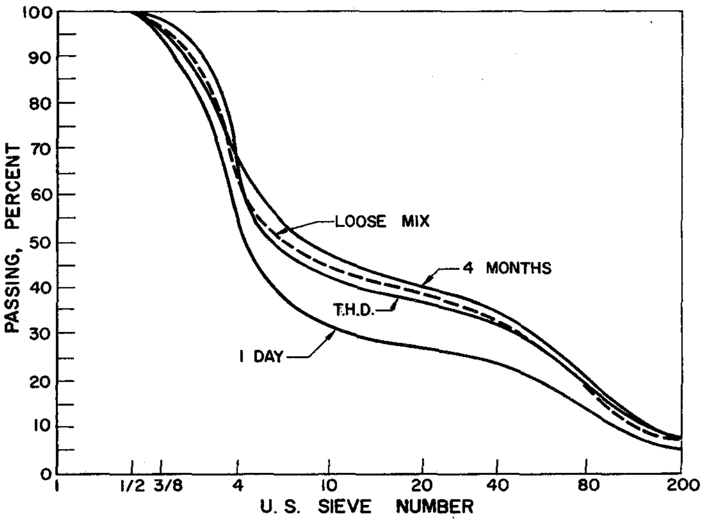
AGGREGATE DEGRADATION
(MILANO TEST SECTION)
FIGURE B-9



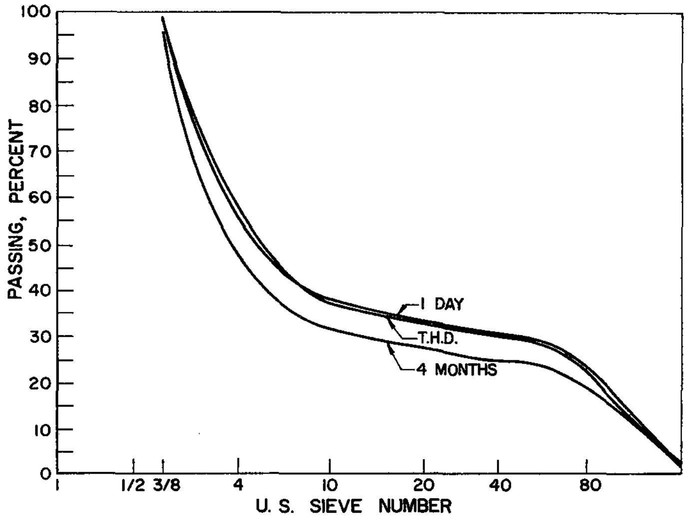
AGGREGATE DEGRADATION
(BRYAN TEST SECTION)
FIGURE B-10



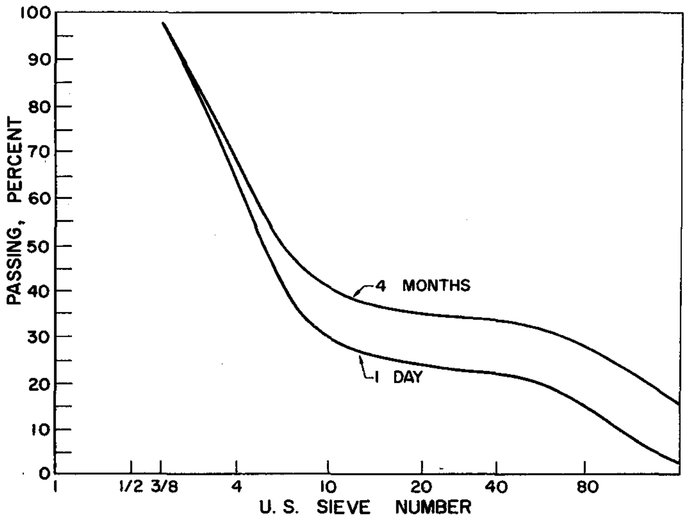
AGGREGATE DEGRADATION
(TAMINA TEST SECTION)
FIGURE B-11



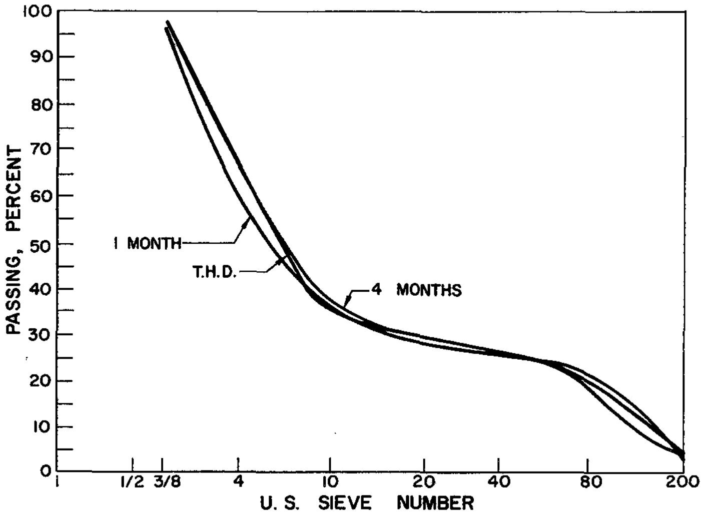
AGGREGATE DEGRADATION
(CONROE TEST SECTION)
FIGURE B-12



AGGREGATE DEGRADATION
(BAYTOWN TEST SECTION)
FIGURE B-13

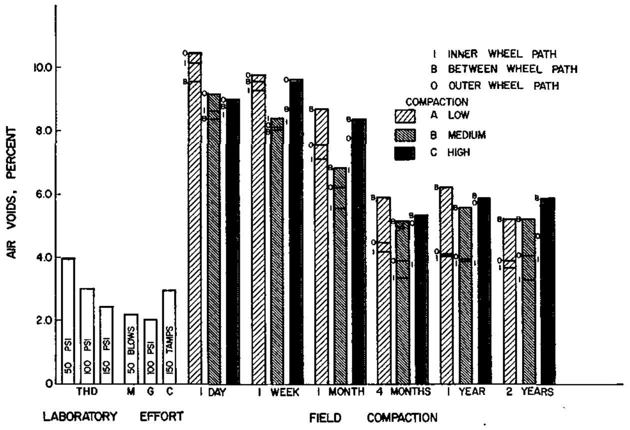


AGGREGATE DEGRADATION
(ORANGE TEST SECTION)
FIGURE B-14



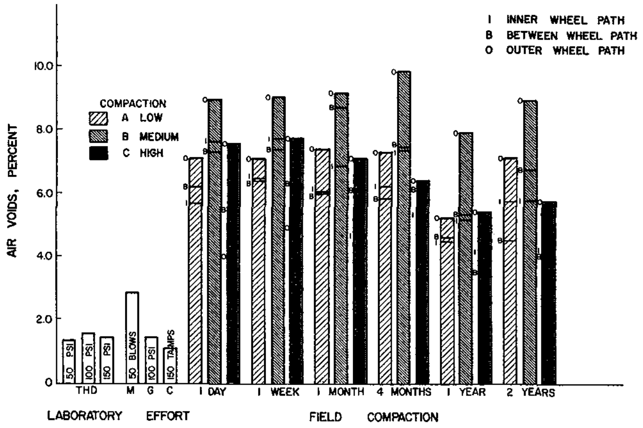
AGGREGATE DEGRADATION
(BRIDGE CITY TEST SECTION)
FIGURE B-15

APPENDIX C



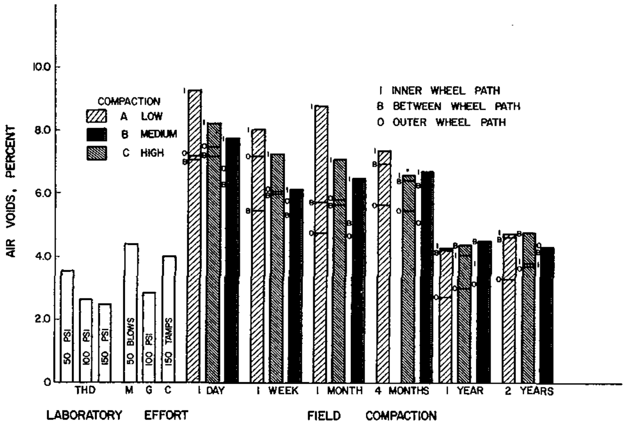
AIR VOID CONTENT FOR LABORATORY AND FIELD SAMPLES
(CHILDRESS TEST SITE)

FIGURE C-1



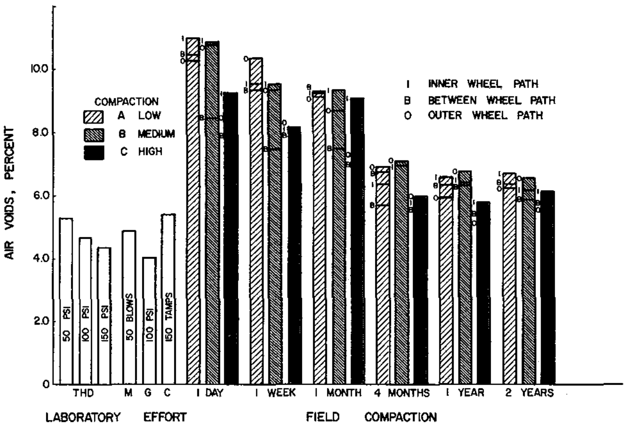
AIR VOID CONTENT FOR LABORATORY AND FIELD SAMPLES
(MATADOR TEST SITE)

FIGURE C-2



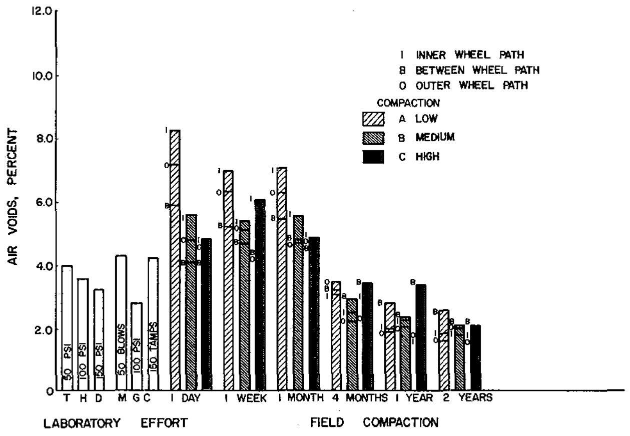
AIR VOID CONTENT FOR LABORATORY AND FIELD SAMPLES
(SHERMAN TEST SITE)

FIGURE C-3



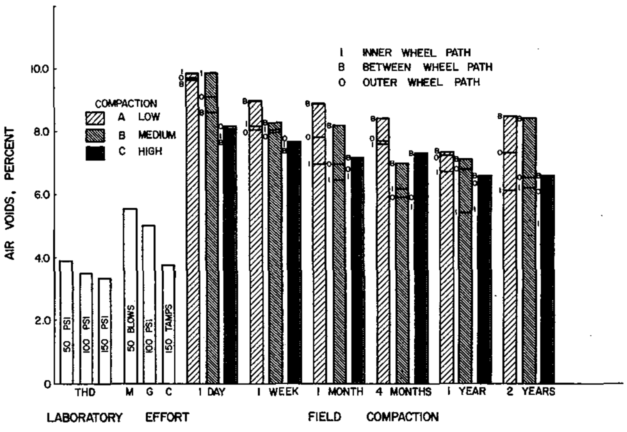
AIR VOID CONTENT FOR LABORATORY AND FIELD SAMPLES
(COOPER TEST SITE)

FIGURE C-4



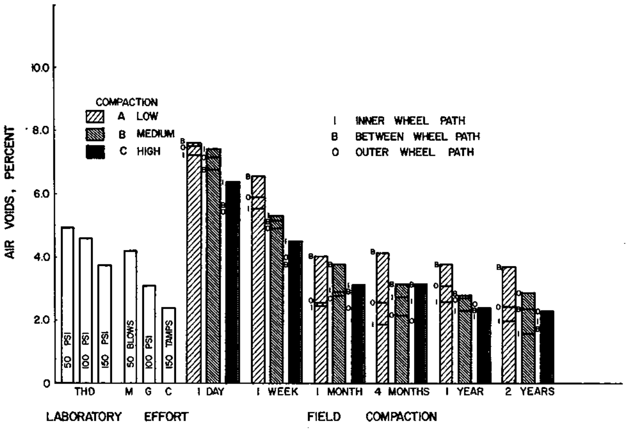
AIR VOID CONTENT FOR LABORATORY AND FIELD SAMPLES
(CUMBY TEST SITE)

FIGURE C-5



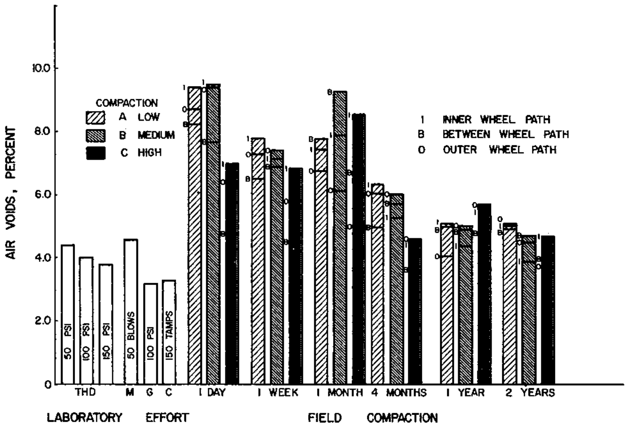
AIR VOID CONTENT FOR LABORATORY AND FIELD SAMPLES
(CLIFTON TEST SITE)

FIGURE C-6



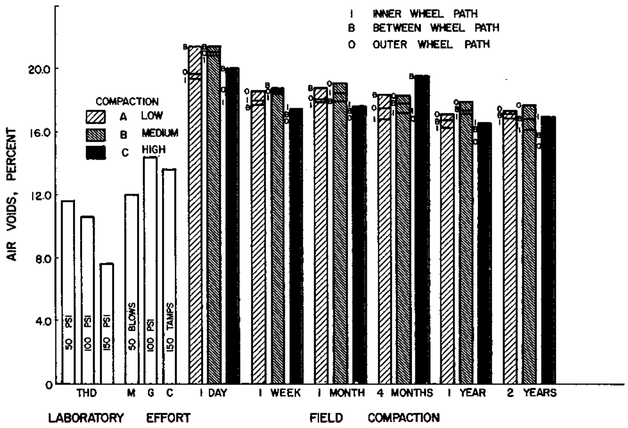
AIR VOID CONTENT FOR LABORATORY AND FIELD SAMPLES
(WACO TEST SITE)

FIGURE C-7



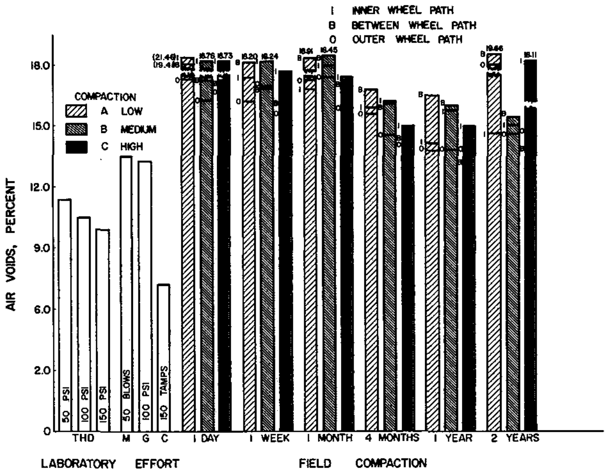
AIR VOID CONTENT FOR LABORATORY AND FIELD SAMPLES
(ROBINSON TEST SITE)

FIGURE C-8



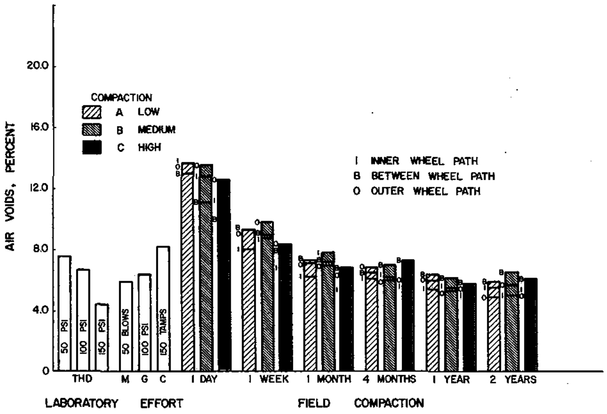
AIR VOID CONTENT FOR LABORATORY AND FIELD SAMPLES
(MILANO TEST SITE)

FIGURE C-9



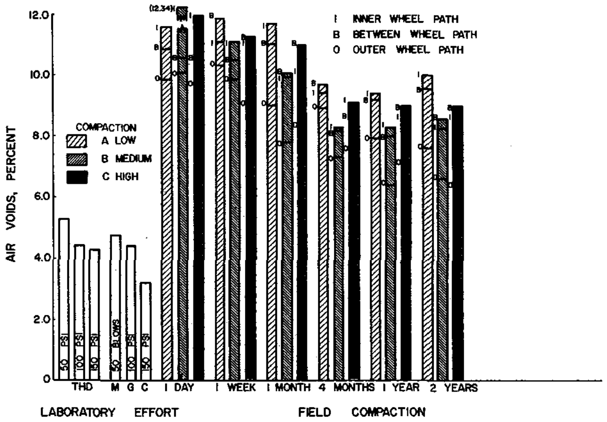
AIR VOID CONTENT FOR LABORATORY AND FIELD SAMPLES
(BRYAN TEST SITE)

FIGURE C-10



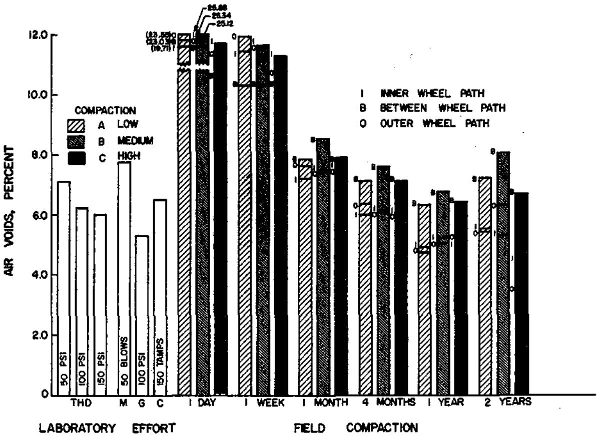
AIR VOID CONTENT FOR LABORATORY AND FIELD SAMPLES
(TAMINA TEST SITE)

FIGURE C-11



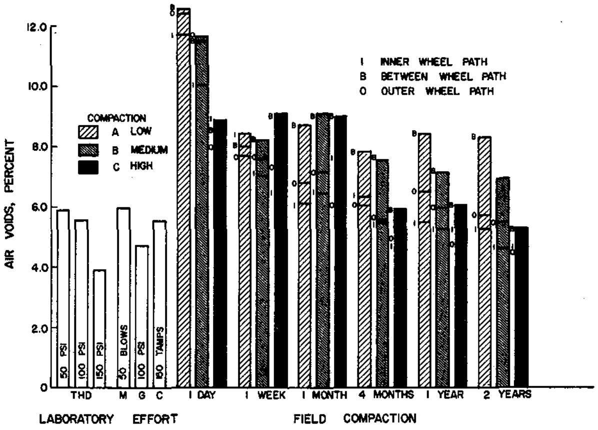
AIR VOID CONTENT FOR LABORATORY AND FIELD SAMPLES
(CONROE TEST SITE)

FIGURE C-12



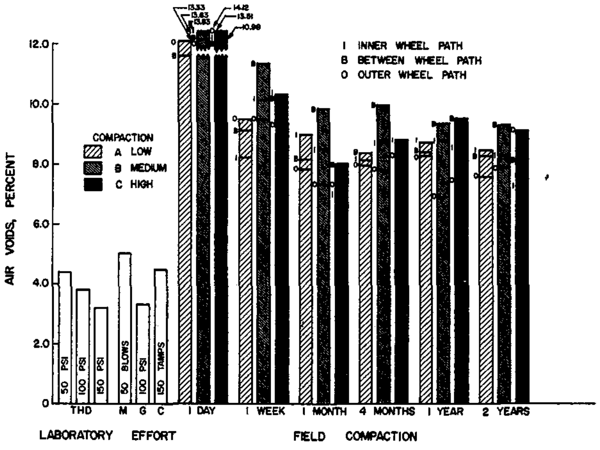
AIR VOID CONTENT FOR LABORATORY AND FIELD SAMPLES
(BAYTOWN TEST SITE)

FIGURE C-13



AIR VOID CONTENT FOR LABORATORY AND FIELD SAMPLES
(ORANGE TEST SITE)

FIGURE C-14



AIR VOID CONTENT FOR LABORATORY AND FIELD SAMPLES
(BRIDGE CITY TEST SITE)

FIGURE C-15