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FOREWORD

In spite of the importance of temperature on the behavior of bituminous materials, field information is not available for the many varied conditions in the United States. The paper by Rumney and Jimenez provides such information for Arizona. In addition, they assert that prediction of pavement temperature is possible with the use of information from weather reports of air temperature and solar radiation.

In the paper by Marek and Herrin a surface treatment design procedure based on measured voids content is presented. The procedure advanced contains a number of improvements over the methods advanced heretofore.

Nuclear engineering has had a pronounced influence on highway testing in the measurement of density. In the paper by Grey the conclusion is presented that the nuclear method of determining asphalt content by neutron moderation appears to be as reliable as the standard extraction process for the gage and samples utilized in the evaluation. He further concludes that, although the nuclear measurement can be made by those with limited experience, the preparation of the sample requires a high-level, well-trained technician.

The second paper by Grey deals with an evaluation of the compaction procedures used in bituminous construction. The study confirmed the previous view that the areas of lowest density were the joints and the pavement edges. Also he determined that the air gap density test was dependent on only the top $1\frac{3}{4}$ in. of the pavement.

Lee and Dutt examined many gradations to determine the influence of dense or gap graded on Marshall and Hveem design properties. They concluded that gap-graded aggregates can yield mixtures with equal or better physical properties than continously graded aggregates.

The shear susceptibility of asphalt cements is a much discussed subject and especially the appropriateness of various test temperatures to yield the best insight into asphalt properties. The paper by Schweyer and Busot entailed the evaluation of a capillary rheometer for both routine and research studies of asphalts. A test method is advanced that the authors feel is a simple procedure for both research and control work.

Test data on a variety of asphalts that have been in service since the mid-fifties is provided in the paper by Vallerga and Halstead. This is probably the most comprehensive study of the aging of asphalt cements ever undertaken. The asphalts were representative of the production of 130 asphalt cements of known source from about 100 refineries. This study should be of great interest to all highway agencies. They show as others have shown that the way in which the asphalt is used (voids in the pavement) is more influential in hardening than asphalt source. They note, for instance, that in pavements with less than 2 percent voids field aging is negligible. This comprehensive study provides information for all who are involved in the design of bituminous mixtures and those who must see to it that proper pavement properties are achieved in the field.

An abridged paper by McRae and Lagrone discusses the use of the gyratory testing machine to evaluate the effect of a modified reclaimed rubber and a ground vulcanized rubber on the physical properties of bituminous pavements.

—Jack H. Dillard

CONTENTS

PAVEMENT TEMPERATURES IN THE SOUTHWEST

Thomas N. Rumney and R. A. Jimenez, University of Arizona, Tucson

A study was conducted on a test section of asphalt pavement to produce needed temperature data from the desert portions of the Southwest. Solar radiation data were recorded concurrently for comparison with the temperature data. Typical temperature patterns for each month of the test period are presented and explained with the aid of heat transfer theory. Presentations of the data show the average minimum and maximum temperatures throughout the test period as well as the temperature durations at various levels. Evaluation of the statistics has led the authors to suggest possible additions to the present methods of testing asphaltic concrete for use in desert areas. Comparison between solar radiation and temperature substantiates the theory that shallow depths are greatly influenced by the sun's energy. Analysis of the data shows that prediction of pavement temperatures is possible with the use of information from weather reports of air temperature and solar radiation.

•THE DESIGN of asphaltic pavement has developed from an art, depending primarily on experience, to procedures based on analytical methods attempting to balance stress and strength. The return to the use of thick layers or many layers of asphaltic concrete in pavements has required that knowledge of surface and subsurface temperatures be known. The knowledge of temperature existing in the asphaltic structure is required because the stresses developed and strength of the asphaltic system are both temperature dependent.

In recognition of the need for the knowledge of temperature distribution in asphalt pavements, The Asphalt Institute has sponsored research to obtain such data. Kallas (5) and Straub et al. (12) have reported on measurements obtained from installations that served as guides for the present work. The Kallas data were obtained in Maryland and are assumed to represent average climatic conditions in the United States. The Straub measurements were made in Potsdam, New York, and represent conditions in a colder climate. It follows that data on pavement temperatures from the desert Southwest were required, and arrangements were made to obtain comparable measurements in Tucson, Arizona.

COLLECTION OF DATA

Temperature

The temperature data were recorded with the use of a Leeds and Northrup Speedomax G-12 point recorder on loan from The Asphalt Institute. The recorder printed temperatures from 0 to 200 F on a continuous roll of graph paper. Printing one temperature every 24 seconds, the recorder completed the 12-point cycle once every 4 minutes and 48 seconds or approximately every 5 minutes. The hourly temperatures at every depth and the maximum surface temperature for the day were taken from the printed sheets and recorded on prepared data sheets. A copy of a completed data sheet is shown in Figure 1.

The wires from the recorder and the wires from the thermocouples were joined together by a system of dual-point plugs. This system facilitated removal of the re-

Sponsored by Committee on Mechanical Properties of Bituminous Pavement Mixtures and presented at the 50th Annual Meeting.

PERIOD: 15 June 1969 THRU 21 June 1969
* INDICATES APPROXIMATE AVERAGE RATE AT THAT TIME
DATA RECORDED AS Btu/ft^2/hr AT THAT TIME

MONTH	June						
DAY	15	16	17X	18X	19X	20X	21X
TIME 0500	0	0	0	0	0	0	0
0600	22	18	22	22	25	22	25
0700	90	54*	88	88	90	88	92
0800	166	101*	157	157	160	160	164
0900	214*	124*	223	223	220	225	227
1000	286	214*	272	274	272	274	277
1100	319	180*	304	306	301	304	308
1200	326	326	322	324	319	322	331
1300	324	322*	324	322	319	324	333
1400	236*	306*	304	306	299	308	313
1500	292*	295*	268	268	265	274	277
1600	225*	70	218	218	216	223	225
1700	90*	216	146	151	148	155	155
1800	52	40	74	76	79	83	70
1900	11	16	16	16	16	16	18
MAX	328	360	326	324	324	328	335
TIME	1220	1130	1230	1230	1230	1245	1245

COMMENTS: X indicates a clear day

Figure 1. Temperature test data sheet.

corder from the test site whenever nevessary. Shortly after the pad was instrumented, the wires extending to the recorder were covered with a plastic sheath, and this proved to be a mistake. The purpose of the plastic was to protect the wires from the weather and keep them dry. Instead, condensation collected inside the shield, and the moisture on the wires seemed to affect the temperature readings. Forcing air through the protective shield might have solved this problem if air had been available. Because forced air was not available, experience showed that leaving the wires unprotected in the normally low relative humidity of southern Arizona gave the best results. Backing the low relative humdity is the fact that rainstorms in southern Arizona are rather infrequent and short-lived.

The temperature recording was started on June 30, 1969, and the recorder was turned on and off manually. As a trial, the recorder was run from approximately 8:00 a.m. to 5:00 p.m. daily with an additional 24-hour run once each week. It was not felt necessary to run the recorder 24 hours every day because the temperatures did not fluctuate greatly during the night and showed fairly uniform patterns from night to night. Therefore, it was decided to record for 24 hours once each week, and this proved to be quite sufficient. The 8:00 a.m. to 5:00 p.m. recording was barely sufficient and also very tiring to perform manually, so a 24-hour clock was used to turn the recorder on and off automatically. The timing sequence was set to turn the recorder on at 6:45 a.m. and off at 5:15 p.m. during the winter months. This sequence seemed optimum to record the minimum temperatures in the morning, the rise and fall patterns during the day, and the maximum temperatures during the afternoon. This time interval had to be lengthened as the days became longer.

Solar Radiation

The Institute of Atmospheric Physics at the University of Arizona keeps a continuous record of the solar radiation rate on a horizontal surface. The measuring equipment is on top of the Physics, Mathematics, and Meteorology Building on the campus. The measuring device is an Eppley Pyranometer, Model 50, made by Eppley Laboratories of Newport, Rhode Island, and consists of alternating black and white concentric circles mounted inside a glass bulb. The recording device, a Brown Electronik Model Y153XLLV-X-27V made by Honeywell, determines the radiation rate by interpreting the temperature difference between the 2 colored surfaces. The recorder prints a graph on a continuous roll of paper with radiation rates from 0 to 2 langleys. Besides printing the rate, the recorder is equipped with an integrating device to give the cumulative solar radiation received. The plotted graphs were read to give on-the-hour

Date: 5 August 1969

TC #	1	2	3	4	5	6	7	8	9	10	11	12
Loc	AIR	12"	10"	8"	6"	4"	2"	SUR	6"	4"	2"	SUR
0100	86	110	110	110	108	106	102	96	109	107	102	97
0200	84	109	109	108	106	104	100	95	107	105	101	96
0300	83	108	108	107	106	103	99	93	106	103	99	94
0400	82	108	107	106	104	102	98	92	105	102	98	93
0500	82	107	106	104	103	100	96	91	104	100	96	92
0600	82	106	106	104	102	99	96	91	103	99	96	92
0700	86	106	106	104	102	100	96	96	102	99	96	95
0800	92	106	106	104	102	100	102	106	104	102	101	105
0900	96	106	106	104	104	103	108	118	104	104	107	114
1000	100	106	106	105	106	107	115	130	106	108	114	127
1100	103	105	106	105	107	110	121	139	107	113	122	137
1200	106	105	106	106	110	116	128	148	110	118	130	145
1300	108	106	108	108	114	121	133	153	113	123	136	150
1400	110	108	109	110	118	126	138	157	116	127	140	155
1500	110	109	111	114	120	128	140	155	119	130	142	153
1600	104	109	111	114	121	128	138	143	120	130	140	143
1700	104	109	112	115	122	127	131	130	121	128	132	131
1800	102	109	112	115	121	124	127	125	120	126	130	130
1900	100	110	112	116	120	122	124	120	120	124	124	122
2000	91	111	113	116	119	120	117	108	119	121	119	110
2100	90	111	113	115	116	116	112	104	117	117	113	106
2200	88	111	112	114	113	113	109	102	114	114	110	104
2300	87	111	112	112	112	110	106	100	112	111	107	101
2400	86	110	111	111	109	108	104	98	110	109	104	99

Figure 2. Solar radiation data sheet.

rates and also the maximum rate for the day. The values in langleys were converted and recorded as Btu's per square foot-hour on prepared data sheets. Data have been taken daily from sunrise to sunset. A sample, completed data sheet is shown in Figure 2.

TEMPERATURE

Surface

Figure 3 shows temperature patterns for a typical July day. The surface experiences the greatest temperature variation by far. During the early morning hours, the temperature decreases slowly and reaches its minimum around 6:00 a.m. The surface temperature at night depends to a great extent on atmospheric conditions and usually averages about 5 F above the air temperature. After 6:00 a.m., the surface temperature increases very rapidly as the heating effects of solar radiation win over the cooling effects of the air. The increase in temperature tapers off around 12:00 noon, and the temperature remains fairly stable until about 3:00 p.m. during which time the maximum surface temperature usually occurs. After 3:00 p.m., the cooling effects of the air win over the radiation effects, and the temperature falls rather rapidly until around 7:00 p.m. when it again approaches the range of night air temperatures. From

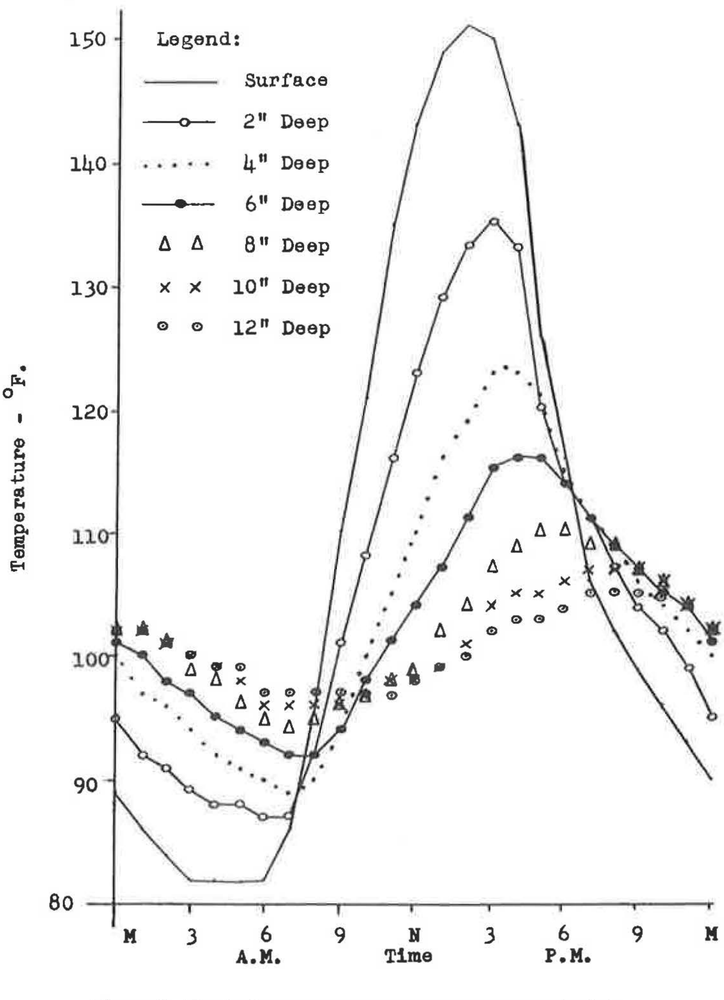

Figure 3. Typical pavement temperature patterns in July.

then until midnight, the surface temperature decreases slowly, still approaching the air temperature but remaining slightly above it.

The WASHO Road Test (13) and Galloway (4) both indicate that the surface temperature of asphaltic concrete is below the air temperature in the winter. In southern Arizona, the surface temperature did not fall below the air temperature. It is not expected that it would unless there happens to be a sudden, rare, large increase of air temperature during the night. The surface is kept warm by residual heat in the lower layers, and the pavement establishes a dynamic thermal equilibrium in the heat flow out of the warm underlayers.

Two-Inch Level

The 2-in. level follows much the same pattern as the surface except that the temperature differential is not so extreme. The 2-in. level cools during the early morning hours but stays warmer than the surface, taking its position in the thermal equilibrium. Normally between 7:00 and 9:00 a.m., the surface temperature surpasses the 2-in. level temperature as they both start their rapid increases almost simultaneously. The 2-in. level reaches its upper plateau slightly after the surface but holds it for a short period of time after the surface starts its decline. The surface temperature falls below that of the 2-in. level between 5:00 and 7:00 p.m. The temperature at the 2-in. level continues its decline into the night but stays above the surface temperature.

Deeper Levels

The deeper layers follow much the same pattern except that their respective daily differentials are much less severe. As depth increases, the extreme temperatures are delayed slightly from the layer just above. For example, the minimum of 6 in. occurs after the minimum at 4 in., and the maximum at 6 in. occurs after the maximum at 4 in.

Differentials

Knowledge of minimum temperatures is desired in design because asphaltic concrete loses flexibility with decreasing temperature. As opposed to this, knowledge of maximum temperatures is desired because asphalt loses stability with increasing temperature. Information on the daily temperature differentials is desired because of their effect on the physical properties of the asphaltic cement. Temperature differential may also be important because of the warping stresses it might induce in the pavement. However, warping stresses may not be important in asphaltic concrete because of its relatively low modulus of elasticity.

Table 1 gives surface temperature data. Before they are discussed, the column headings should be clarified.

1. The time period over which the averages were taken include the months from June through December, 1969, and January through May, 1970. Normally the time period is one calendar week in duration. One particular period covers 2 weeks, and a 4-week period is missing because of power failure problems in the area of the test site. Several other periods may have statistics for fewer than 7 days because of various small problems encountered with the recorder.
2. Minimum temperature indicates the lowest temperature recorded at each particular level during the specified time period. An idea of the time of day of these minimums can be obtained from the typical temperature plots discussed earlier.
3. Average minimum indicates the average of the daily minimum temperatures that occurred at each level during each week. These averages give a better understanding of the lower boundary of the temperature range than the minimum temperature.
4. Minimum peak is the lowest daily high temperature of each particular level for each time period.
5. Maximum peak is the maximum high temperature occurring at each level during the time period.

TABLE 1

SURFACE TEMPERATURE STATISTICS

Time Period	Minimum	Avg Minimum	Minimum Peak	Maximum Peak	Avg Peak	Avg Spread
6-30 to 7-15	—	—	149	160	155	—
7-6 to 7-12	87	88	134	160	148	60
7-13 to 7-19	84	86	183	158	146	60
7-20 to 7-26	82	83	139	151	146	63
7-27 to 8-2	85	87	142	158	152	65
8-3 to 8-9	91	93	146	156	151	58
8-10 to 8-16	82	83	115	155	142	59
8-17 to 8-23	82	83	147	149	148	65
8-24 to 8-30	85	87	136	146	143	56
8-31 to 9-13	84	84	121	145	132	48
9-14 to 9-20	72	76	116	135	130	54
9-21 to 9-27	70	76	128	131	130	54
9-28 to 10-4	74	74	116	132	125	51
10-5 to 10-11	62	68	117	125	119	51
10-12 to 10-18	56	59	109	116	113	54
10-19 to 10-25	54	59	90	110	103	44
10-26 to 11-1	52	55	101	108	104	49
12-7 to 12-13	41	45	71	88	81	36
12-14 to 12-20	42	43	82	88	86	43
12-21 to 12-27	45	46	80	88	85	39
12-28 to 1-3	31	39	64	78	73	34
1-4 to 1-10	34	38	72	81	75	37
1-11 to 1-17	42	47	81	89	84	37
1-18 to 1-24	44	46	85	93	87	41
1-25 to 1-31	34	49	81	91	87	38
2-1 to 2-7	38	44	80	94	88	44
2-8 to 2-14	50	54	96	109	102	48
2-15 to 2-21	44	51	92	105	98	47
2-22 to 2-28	50	53	98	104	102	49
3-1 to 3-7	49	54	58	106	92	38
3-8 to 3-14	47	52	95	112	106	54
3-15 to 3-21	45	52	100	114	108	56
3-22 to 3-28	52	55	108	117	114	59
3-29 to 4-4	51	60	98	118	112	52
4-5 to 4-11	62	67	105	122	114	47
4-12 to 4-18	55	67	76	126	106	39
4-19 to 4-25	58	63	107	117	112	49
4-26 to 5-2	55	61	102	126	110	49
5-3 to 5-9	58	67	100	134	120	53
5-10 to 5-16	70	81	126	138	130	49
5-17 to 5-23	80	84	123	138	131	47
5-24 to 5-30	76	80	119	127	121	41

6. Average peak designates the average of the daily peak temperatures at each level occurring during the time period. During periods in which the day-to-day weather was fairly constant, the minimum peak and the maximum peak close in on the average peak temperature.

7. Average spread is the numerical difference between the average peak temperature and the average minimum temperature during the time period. This number represents the average rise and fall in temperature that each level experienced during the course of one day.

For the surface and the 2-, 4-, 6-, 8-, 10-, and 12-in. levels, the minimum temperatures were 31, 37, 40, 42, 42, 42, and 43 F respectively. The temperatures all occurred within the same week and further demonstrate the upward transfer of heat as discussed earlier. Arena (1) presents tables showing that the binder course experienced lower temperatures than the wearing courses. In this study the surface minimum was always the lowest temperature. However, the minimum temperatures reported by Arena (1) in Louisiana are in the same range as those from this study. Minimum temperatures in the cooler climate of Maryland, reported by Kallas (5), were about 15 F less than those in Arizona. Straub, Schenck, and Przybycien (12) reported minimum temperatures from northern New York that were about 20 F below those in Arizona.

The maximum temperatures experienced at the surface and the 2-, 4-, 6-, 8-, 10-, and 12-in. levels were 160, 142, 132, 123, 116, 113, and 111 respectively. From the 2 sets of extremes, the pavement levels experienced long-term differentials of 129, 105, 92, 81, 74, 71, and 68 respectively. Fortunately these differentials occurred over a period of approximately 6 months as opposed to the average daily spread in which the heating and cooling cycles take approximately 6 hours and 18 hours respectively. Kallas (5) and Straub, Schenck, and Przybycien (12) reported maximum temperatures below those of Arizona, but these could be expected in the cooler climates.

Because of the maximum peaks that occurred in August and through July, it might be suspected that higher maximum temperatures were experienced before the recording started. This has been neither proved nor disproved, but later evidence will substantiate the possibility.

Daily temperature differentials decrease with depth as observed earlier on the typical monthly temperature plots. The data also show that the daily temperature differential decreased during the colder months.

Level Duration

Data were obtained that give the relative amount of time during which the temperatures at the 7 depths were equal to or above a given temperature. The temperatures in the 6- and 12-in. sections were essentially the same and, therefore, the data are valid for both cases. Because the recording took place primarily in the daytime when higher temperatures were recorded in the shallow depths, the warmer temperature durations could be obtained with each, and a 10 F interval was used. Because the data collection was limited to one 24-hour run each week, the availability of data for lower temperature patterns was rather limited, and the interval was expanded to 20 F. This procedure may be justified by the fact that the desired temperature durations were in the upper layers with the warmer, more critical levels.

Table 2 gives data from the surface. The surface temperature stayed above 70 F 100 percent and above 90 F roughly 70 percent of the time during July, August, and September. During July and August, the surface remained above 110 F about 40 percent of the time, above 130 F roughly 25 percent, and above 140 F from 7 to 22 percent. The temperature stayed above 150 F from 3 to 8.5 percent of the time and reached 160 F on several days but did not remain there for any appreciable amount of time. The time that was spent above 160 F during June, if any, still remains a question here.

Also during July and August, the 2-in. level reached the 140 F mark on several instances but did not spend much time above this level. The durations above 130 F do indicate that the 140 F range was approached quite frequently. At any rate, it can be concluded that the asphaltic concrete between the 2-in. level and the surface does experience a significant amount of time above 140 F.

The asphalt between the 2- and 4-in. level approaches the 140 F temperature range; below the 4-in. level, it approaches the value of 130 F. Below the 6-in. depth, the asphalt pavement was above 110 F but did not exceed 120 F. In turn, below the 10-in. level, the asphaltic concrete occasionally did reach 110 F.

In many parts of the United States, pavement temperatures do not reach the levels recorded in Tucson, Arizona. Tests on asphaltic concrete have been developed in which the testing temperature is 140 F. The observations in Arizona raise questions concerning the use of only 140 F for stability testing in hot, desert regions. Upper layers of the pavement exceed this level, and lower layers never reach it. Before any changes are suggested, consideration must be given to the present, widespread acceptance and use of conventional test procedures. It is doubtful that it would be fair or necessary to change the entire test for a relatively small portion of the country.

On the basis of the data reported in this paper, the authors believe that minor modifications to test procedures might be an improvement for use in the Southwest. For asphaltic concrete to be placed above the 2-in. level, the standard 140 F stability test could be performed and several extra samples could be made and tested at 160 F. The mix would be considered acceptable if the 160 F stability did not fall below a specified

TABLE 2

PERCENTAGE OF TIME PERIOD WHEN SURFACE LEVEL TEMPERATURE WAS ABOVE GIVEN TEMPERATURE

Time Period	30 F	40 F	50 F	60 F	70 F	80 F	90 F	100 F	110 F	120 F	130 F	140 F	150 F
6-30 to 7-5	—	—	—	—	100	—	60	—	42	—	28	22	8.5
7-6 to 7-12	—	—	—	—	100	—	79	—	38	—	17	9	2
7-13 to 7-19	—	—	—	—	100	—	71	—	37	—	15	11	3
7-20 to 7-26	—	—	—	—	100	—	67	—	29	—	16	8	2
7-27 to 8-2	—	—	—	—	100	—	75	—	39	—	33	17	6
8-3 to 8-9	—	—	—	—	100	—	100	—	45	—	29	21	7
8-10 to 8-16	—	—	—	—	100	—	67	—	29	—	15	11	3
8-17 to 8-23	—	—	—	—	100	—	78	—	37	—	14	7	0
8-24 to 8-30	—	—	—	—	100	—	71	—	34	—	14	7	0
8-31 to 9-13	—	—	—	—	100	—	75	—	25	—	9	5	0
9-14 to 9-20	—	—	—	—	100	—	42	—	25	18	7	0	—
9-21 to 9-27	—	—	—	—	100	—	54	—	23	18	6	0	—
9-28 to 10-4	—	—	—	—	100	—	40	—	24	12	4	0	—
10-5 to 10-11	—	—	100	—	75	—	35	—	15	6	0	—	—
10-12 to 10-18	—	—	100	—	46	—	32	21	9	0	—	—	—
10-19 to 10-25	—	—	100	—	40	—	17	10	2	0	—	—	—
10-26 to 11-1	—	—	100	—	54	—	20	13	0	—	—	—	—
12-7 to 12-13	—	100	85	34	19	9	0	—	—	—	—	—	—
12-14 to 12-20	—	100	67	38	24	16	0	—	—	—	—	—	—
12-21 to 12-27	—	100	71	39	23	14	0	—	—	—	—	—	—
12-28 to 1-3	100	67	32	21	10	0	—	—	—	—	—	—	—
1-4 to 1-10	100	89	37	23	11	1	0	—	—	—	—	—	—
1-11 to 1-17	100	100	77	32	21	10	0	—	—	—	—	—	—
1-18 to 1-24	100	100	70	40	26	15	3	0	—	—	—	—	—
1-25 to 1-31	100	96	83	42	36	16	3	0	0	—	—	—	—
2-1 to 2-7	100	97	70	37	25	14	4	0	—	—	—	—	—
2-8 to 2-14	—	100	100	57	36	24	10	2	0	—	—	—	—
2-15 to 2-21	—	100	96	50	36	27	17	5	0	—	—	—	—
2-22 to 2-28	—	—	100	75	39	30	21	7	0	—	—	—	—
3-1 to 3-7	—	100	95	65	27	18	12	4	0	—	—	—	—
3-8 to 3-14	—	100	100	—	38	31	19	9	1	0	—	—	—
3-15 to 3-21	—	100	95	—	41	33	26	17	3	0	—	—	—
3-22 to 3-28	—	100	100	—	52	38	31	23	10	0	—	—	—
3-29 to 4-4	—	—	100	71	48	34	26	17	8	0	—	—	—
4-5 to 5-11	—	—	—	100	60	42	34	21	13	1	0	—	—
4-12 to 4-18	—	—	100	82	50	33	23	15	8	3	0	—	—
4-19 to 4-25	—	—	—	100	68	40	31	23	7	0	—	—	—
4-26 to 5-2	—	—	100	92	59	41	30	18	7	3	0	—	—
5-3 to 5-9	—	—	100	98	78	58	39	27	20	10	1	0	—
5-10 to 5-16	—	—	—	—	100	70	51	39	29	20	5	0	—
5-17 to 5-23	—	—	—	—	100	96	55	39	27	21	6	0	—
5-24 to 5-30	—	—	—	—	100	92	39	25	17	4	0	—	—

percentage of the 140 F value. For pavement between the 2- and 6-in. levels, the present 140 F testing temperature simulates the actual conditions experienced in the field. Asphaltic concrete below the 6-in. level could also be tested by the conventional procedure with extra samples molded for testing at 120 F. If the 120 F stability exceeded the 140 F stability by more than a specified amount, a weaker mixture might be justified for use at this level.

If these suggestions are followed, years of conventional testing procedure would not be thrown out; instead, the method would be modified to evaluate better the stability of mixes in southwestern desert regions. At the same time, problems may be avoided by eliminating a mixture that loses stability rapidly above 140 F. The majority of mixes probably would not weaken too much, but some might if the viscosity of the asphalt cement used was very sensitive to temperature in the 140 to 160 F range. On the other hand, for the pavement that will never experience in situ temperatures above 120 F, a substantial savings might arise by developing a less expensive mixture with adequate stability. Asphalt pavements are expected to have a certain degree of flexibility, and a pavement with too much stability may prove to be as harmful as one with inadequate stability.

SOLAR RADIATION AFFECTING TEMPERATURE

Solar radiation has a great deal of influence on the temperatures in asphaltic concrete. In his paper on pavement temperatures in Australia, Richards (9) states that the net radiation has a greater influence than air temperature on pavement temperatures. Straub, Schenck, and Przybycien (12) also reached the same conclusion and substantiated it by the use of graphs. One graph showed surface temperature plots for 2 days with similar radiation rates and different air temperatures with only a small effect on the surface. The other graph showed one clear and one cloudy day with similar air temperatures that produced a large difference in surface temperature. This section presents a new method of showing that solar radiation is more influential than air temperature on the upper layers of a pavement system. Figure 4 shows plots of solar radiation, air temperature, and pavement temperature against time.

Surface Temperature

For a typical clear July day, the first appreciable solar radiation is recorded at about 6:00 a. m. The rate increases steadily and rapidly until about 11:00 a. m., when it starts to level off. Between 11:00 a. m. and 1:00 p. m., the rate reaches its maximum, and about 2:00 p. m. it begins to decrease as rapidly as it increased during the morning hours. The radiation becomes hardly noticeable by 7:00 p. m. and completely nil by 8:00 p. m. For the scale at which the figure is drawn, the slope of the surface

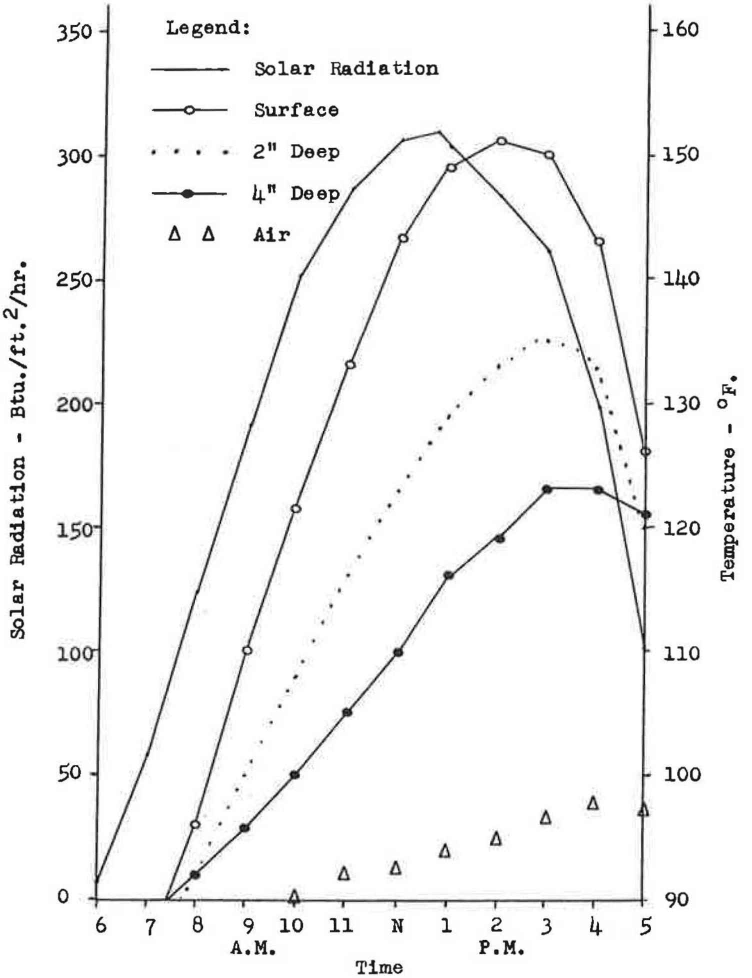

Figure 4. Solar radiation and temperature versus time for typical July day.

temperature curve is very similar to that of radiation curve in the morning hours. The surface temperature continues to increase while the solar rate passes through its upper plateau. As the radiation begins to taper off, the temperature peaks and then begins to fall as the cooling effects of the air win over the heating effects of the sun. The temperature falls with the decreasing radiation until it reaches the range of the air temperature where it remains until the next day.

Trying to correlate the air temperature curve with surface temperature is not nearly so successful. The air temperature curve increases slowly throughout the morning and early afternoon and usually reaches its maximum around 4:00 or 5:00 p.m. or 2 to 4 hours after the surface reaches its maximum. The peak temperature at the surface may be as much as 70 F above the air temperature at that time and, therefore, it becomes quite clear that, although surface temperatures may be dictated by air temperatures at night, they are definitely affected more heavily by solar radiation during the critical peak temperature periods.

Lower Level Temperatures

Temperatures for the 2- and 4-in. levels are also shown on the figures, and their correspondence with radiation is similar to that for the surface temperature. Their temperature increases are delayed by the time required for the heat conduction to their respective levels. Therefore, these temperatures could best be explained by the solar radiation effect and the heat transfer theory suggested earlier. The 2- and 4-in. level temperatures rise well above the air temperature, and this indicates that they are influenced to a great extent by the solar radiation.

Lower levels do not experience large spreads in temperatures as do the upper layers, but their temperature increases are an indirect effect of the solar radiation rates. However, the mean temperatures at the deepest layers do seem to follow the average daily maximum temperatures throughout the year.

PREDICTION OF UPPER LAYER PEAKS

The critical temperatures in southern Arizona are the peak temperatures in the upper layers of the pavements. A simple method of predicting the peak temperatures might be a valuable asset. Barber (2) presented an equation for calculating pavement temperatures from weather reports and several properties of the bituminous material. The large number of variables makes his equation rather cumbersome, but it does yield temperatures with reasonable accuracy. Also, Southgate (11) presented 2 sets of figures for predicting temperatures. The first set gives temperatures at depths of less than 2 in. from the surface temperature for different hours of the day. The other set predicts temperatures for various hours of the day at depths up to 12 in. from the surface temperature and the air temperatures from the 5 previous days. This method may give approximate results, but difficulty may be encountered in measuring the true surface temperature. Temperatures given by simple thermometers resting on the surface do not compare well with thermocouples implanted at the surface. Southgate's method was developed primarily for prediction of temperatures to be used in deflection analysis. The method of predicting upper layer peaks from air temperatures and solar radiation data presented in this section gives the critical peak temperatures that are quite important.

Figures 5, 6, and 7 are used for the surface, the 2-in. depth, and the 4-in. depth respectively. The maximum air temperature for the day should be selected on the abscissa scale. The average of the solar radiation rates at 11:00 a.m., noon, and 1:00 p.m. and the maximum for the day should be selected on the ordinate scale. The intersection of these lines should fall in the range of the given curves. Interpolating between the curves should give a good approximation of the temperature peak that can be expected at that level. The maximum air temperature supplied by the U.S. Weather Bureau should give the best results because the curves were constructed from its temperature data.

The curves were constructed solely on the basis of recorded data and, therefore, are purely empirical in nature. Their validity cannot be checked by data recorded at

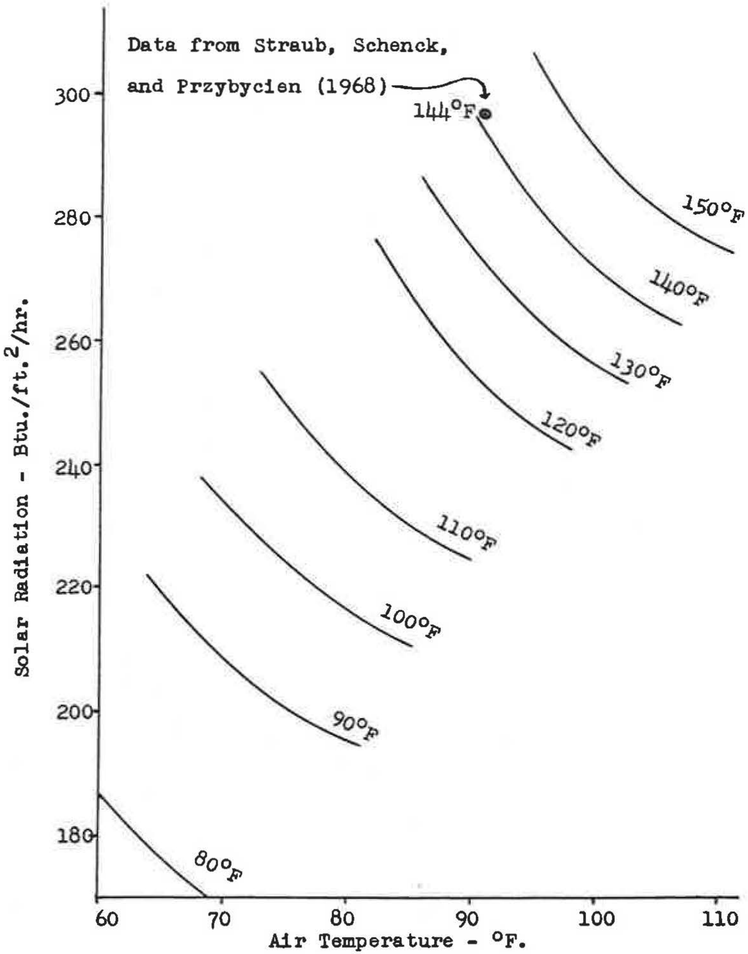

Figure 5. Prediction of surface temperature on clear days from solar radiation rate and air temperature.

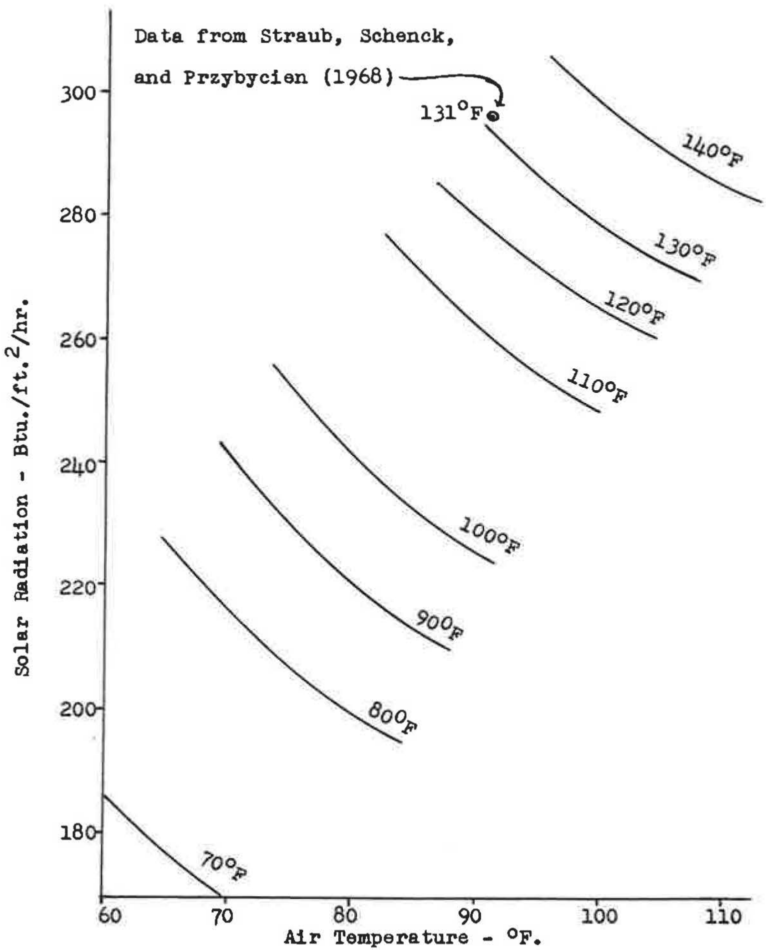

Figure 6. Prediction of 2-in. level temperature on clear days from solar radiation rate and air temperature.

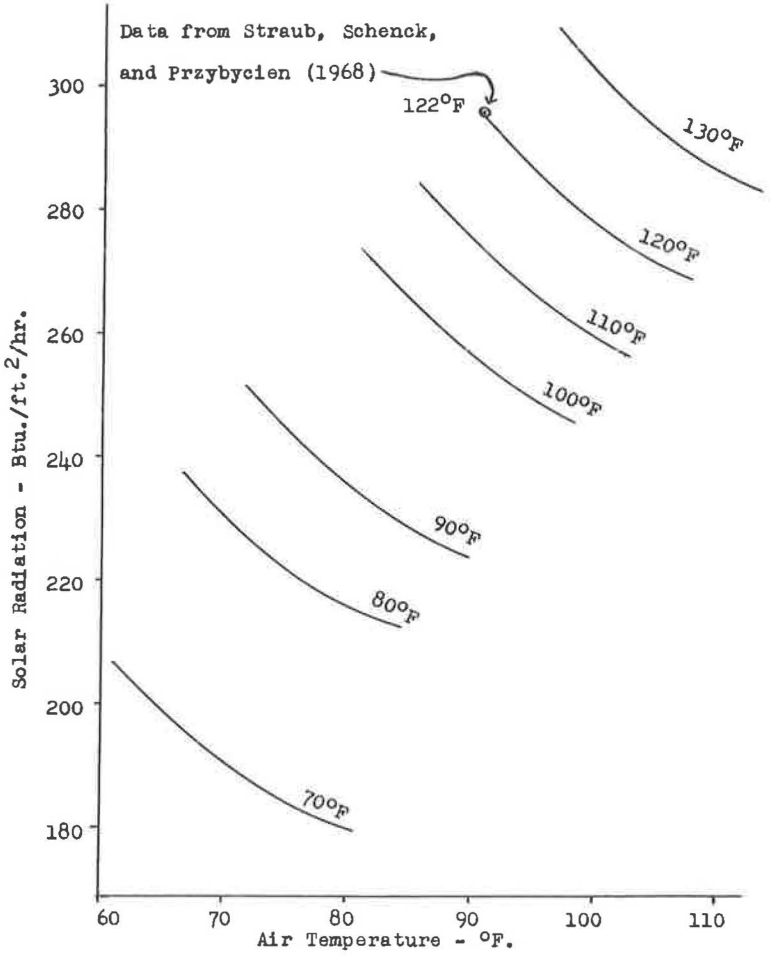

Figure 7. Prediction of 4-in. level temperature on clear days from solar radiation rate and air temperature.

the time of this report. However, compared with solar radiation values and the maximum air temperature from the report by Straub, Schenck, and Przybycien (12), these figures give their maximum temperatures to within 2 F. The points plotted on the figures give their maximum temperatures and show the close correlation. Because of the number of variables involved in predicting temperatures, it is doubtful that this accuracy is possible in all cases, and a standard error of ±5 F would probably be much more realistic.

Values for air temperature were taken from the U.S. Weather Bureau because it records under more standardized conditions with continuously moving air. The air temperature recorded at the test site varied from the Weather Bureau temperature primarily because of the effects of solar radiation.

Several values from the solar radiation data were tried on the ordinate scale before the average of 11:00, 12:00, and 1:00 and the maximum was chosen. The values tried included the maximum rate, different hourly rates taken singularly, and other averages; but the average mentioned earlier was the best for constructing the curves.

As mentioned earlier, a question arises as to whether higher temperatures occurred during June than were mentioned in July. Entering air temperatures as high as 105 F or more and computed solar radiation values as high as 320 Btu/ft^2/hr or more would cause the predicted temperature to be above the present range of the curves. The feeling of the authors is that higher pavement temperatures did occur before recording commenced.

CONCLUSIONS

The data included in this report cover the recording period from June 30, 1969, through May 30, 1970. The authors strongly recommend that the recording be continued to cover at least one full year.

The maximum temperatures recorded at the surface and at 2-, 4-, 6-, 8-, 10-, and 12-in. levels were 160, 142, 132, 123, 116, 113, and 111 F while the minimum temperatures were 31, 37, 40, 42, 42, 42, and 43 F respectively.

A statistical analysis has been run to determine the average minimum and maximum temperatures encountered throughout the testing period. This coupled with an analysis of the time durations at various temperature levels has led to suggestions for modifying conventional test procedures in hot, desert regions. Solar radiation data have been coupled with temperature data to support the theory that the former are much more influential than the latter on upper layer temperatures during daylight hours. A quick and easy method has been presented to predict the maximum upper layer temperatures that can be expected on clear days.

A large amount of data have been collected concerning pavement temperatures in the hot southwest portions of the United States. These data can be made available for further analysis if so desired.

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EFFECT OF MODIFIED RECLAIMED RUBBER AND GROUND VULCANIZED RUBBER ON THE PHYSICAL PROPERTIES OF BITUMINOUS PAVEMENTS AS EVALUATED BY THE GYRATORY TESTING MACHINES

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ABRIDGMENT

•THE purpose of this research was to develop a realistic model of field conditions for an asphaltic concrete pavement. A gyratory testing machine (GTM) was modified and programmed to impart cyclic loading at temperatures encountered during hot compaction and under traffic (Fig. 1).

Generally in line with the approach suggested by Ruth and Schaub (1), using the GTM compressible air roller system, specimens were subjected to 12 revolutions at 290 F to simulate field placement and then by 200 revolutions of cyclic loading at 140 F to simulate accelerated hot weather traffic. Following the densification process, GTM shear tests were conducted. Pertinent physical properties of the mixes containing small amounts of 2 rubber additives (derived from discarded tires) were measured and compared with a control mix of straight asphalt.

MATERIALS

The bituminous mix used in this investigation was a fairly well-graded sand asphalt with ⅜-in. top size gravel aggregate. The asphalt was 85 to 100 penetration grade. Two types of dry rubber additives were used: a minus 4 mesh powdered reclaimed and a minus 20 mesh ground vulcanized rubber.

DYNAMIC RECOVERY

Rubber treatment was found to improve the ability of a bituminous mix to recover from an imposed shear strain. To demonstrate this, specimens first densified by the gyratory compaction process were subsequently subjected to gyratory loading by using the air roller at reduced pressure so that any tendencies for the specimen to experience shear strain recovery could be measured by the gyrograph. Shear strain recovery would show up as a reduction in the gyrograph band width. Typical results are shown in Figure 2.

Figure 3 shows density versus revolutions of cyclic loading (hot compaction at 290 F followed by traffic simulation at 140 F) for a straight asphalt mix and 2 rubber-treated mixes. There is a significant difference in resistance to reduction in voids between treated and untreated mixes.

Figure 4 shows the gyrographs for the cyclic loading to simulate hot weather traffic. The gyrograph for the straight asphalt or control mix progressively widens, whereas neither mix containing rubber experiences this phenomenon. Because this is a laboratory simulation of traffic, it indicates that the addition of either of these rubber additives enhances the ability of the mix to resist failure caused by rutting and shoving; i.e., the rubber additive functions as a desensitizer in this regard.

Sponsored by Committee on Mechanical Properties of Bituminous Pavement Mixtures and presented at the 50th Annual Meeting.

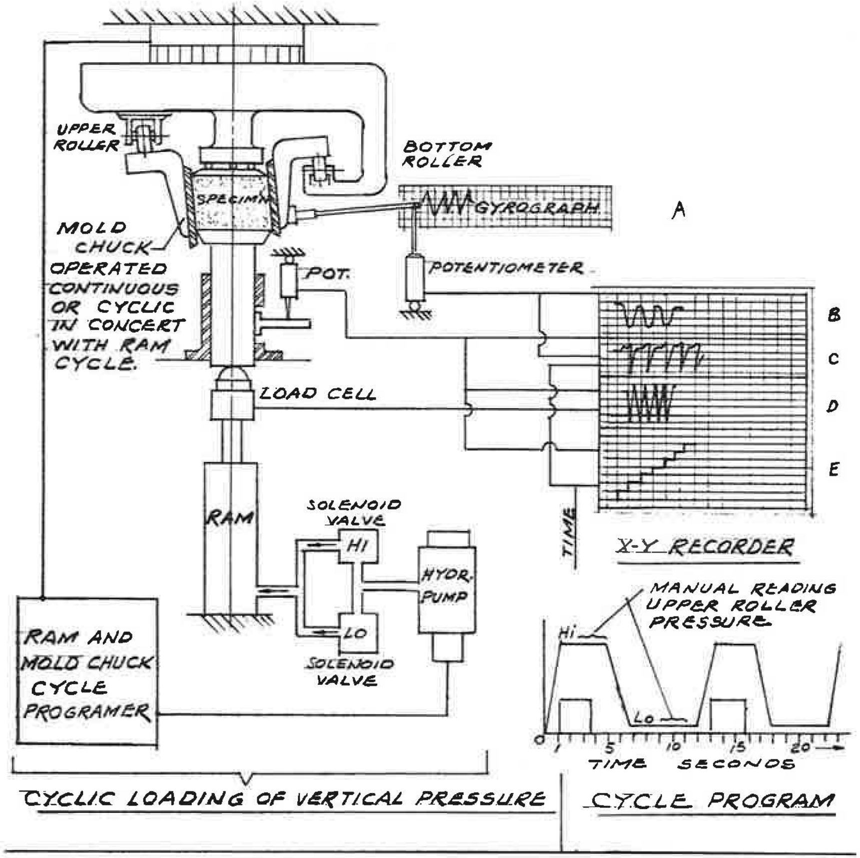
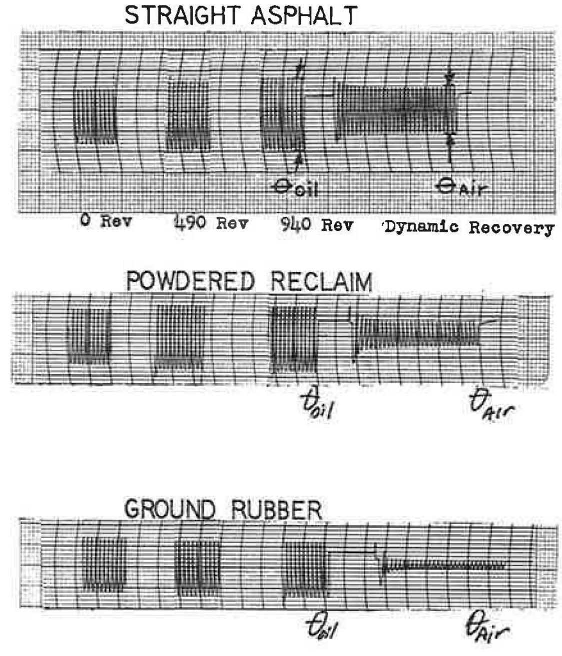


Figure 1. Schematic of GTM operation and instrumentation.



$$\text{RECOVERY INDEX} = \frac{\theta_{oil} - \theta_{Air}}{\theta_{oil}}$$

Vertical Pressure: 100 PSI
$\theta_o = 1^o = 1.75\%$ (Oil Roller)
Air Roller: 10 PSIG
60 Revolutions @ 290°
940 Revolutions @ 140°
Total Binder: 7.0%

Figure 2. Gyratory compaction and dynamic recovery from continuous kneading at 140 F.

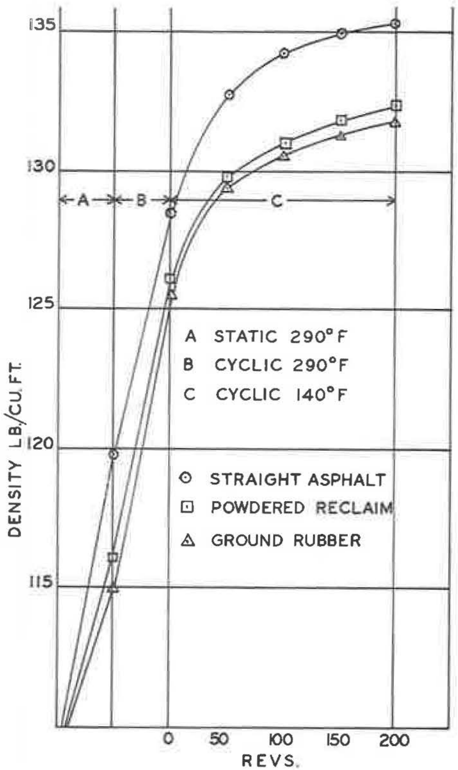
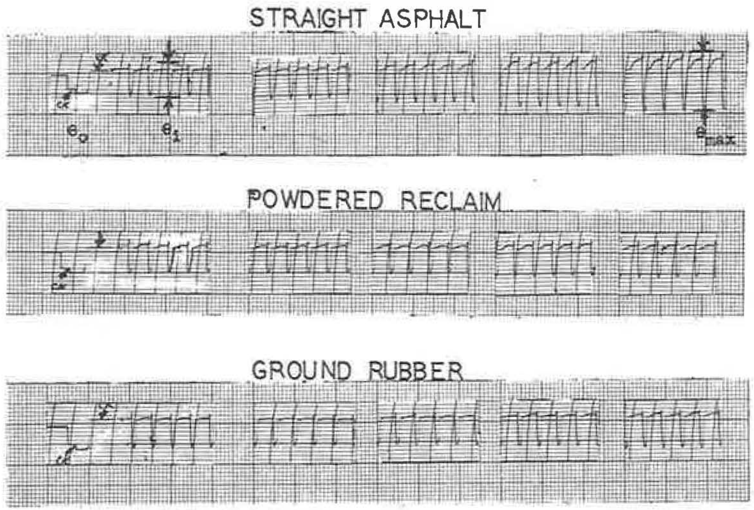


Figure 3. Density versus revolutions of cyclic loading.



Identification:
Total Binder: 7.0%
Rubber Treatment: 0-9%
Vertical Pressure: 75 psi
Air Roller Pressure: 10 psi-gage

CYCLIC GYRATORY LOADING @ 140° F.

Figure 4. GTM-simulated accelerated traffic at 140 F.

Figure 5 shows composite data for the 3 mixes investigated. The stability index (θ_{max}/θ_0) for the untreated asphalt mix rises rapidly for binder contents in excess of that which coincides with the normally recommended voids for this type of mix, while the recovery index, (θ oil - θ air)/ θ oil, shows a rapid loss in value over the corresponding range. The increase in the stability index indicates increased shear deformation, while the loss in the recovery index confirms that the strain is permanent plastic yield.

By using a combination of light vertical pressure and a small gyratory angle, it was possible to generate an exudation phenomenon in the GTM so that bitumen bled from the mix. This "bleeding" test demonstrates the beneficial effects of rubber in eliminating bleeding as shown in Figure 6. The specimen on the right containing powdered reclaimed rubber showed no evidence of exudation, while the specimen on the left showed considerable bleeding. Elimination of bleeding by use of reclaimed rubber additives has been demonstrated in actual practive (2, 3).

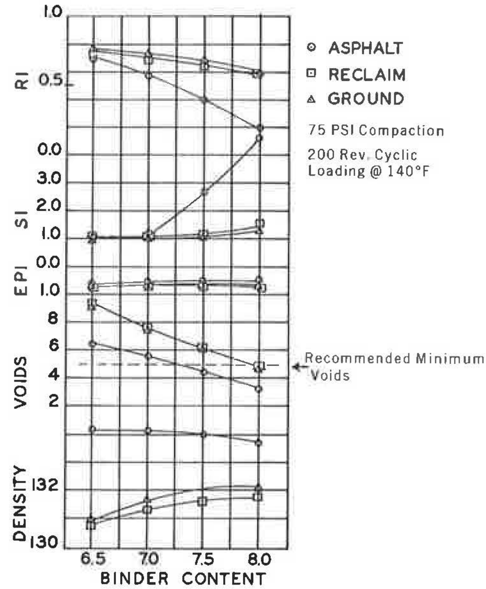
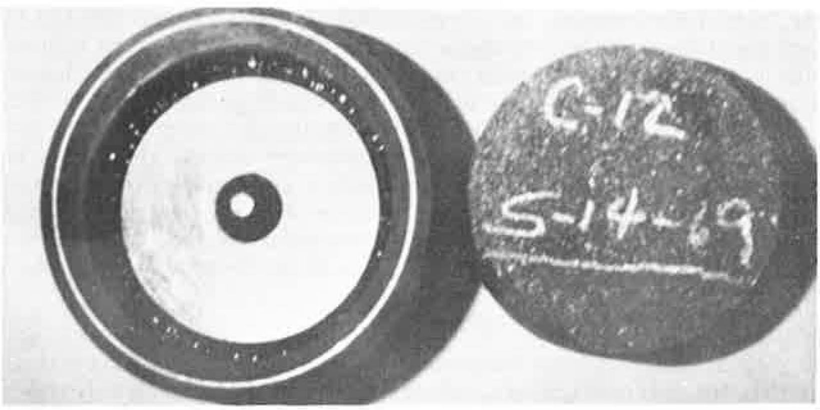


Figure 5. Binder content versus density and various indexes.

SHEAR TEST RESULTS

GTM shear was conducted on specimens subsequent to simulated hot placement and traffic compaction. In GTM shear, the matter of limited deflection is dealt with by finding the effective cohesion and internal friction values from Mohr's envelopes for various limiting degrees of shear strain. The Prandtl bearing value calculated from these values of C and ϕ thus becomes a bearing resistance associated with a limited permissible degree of shear strain that is in turn related to some limiting value of pavement deflection. Because the prime requirement in a modern pavement is a



Straight Asphalt Control

Powdered Reclaim Treatment

Figure 6. Test for bleeding.

TABLE 1

BEARING STRENGTHS OF MIXES

Mix	Binder Content (percent)	Density	Voids	Bearing Strength		
				Ultimate	1.75 Percent Strain	3.50 Percent Strain
Straight asphalt	6.5	134.1	6.50	533	210	510
	7.0	134.1	5.63	628	182	330
	7.5	134.0	4.32	873	20	71
	8.0	133.8	3.32	878	5	38
Powdered reclaimed	6.5	130.7	9.31	324	215	640
	7.0	131.3	7.62	550	160	390
	7.5	131.6	5.76	533	140	300
	8.0	131.8	4.98	745	160	375
Ground vulcanized	6.5	130.9	9.31	380	105	283
	7.0	131.6	7.62	425	160	250
	7.5	130.8	6.79	383	150	200
	8.0	132.1	4.57	410	125	240

smooth ride for high-speed vehicles making almost astronomical numbers of passes, it then becomes necessary to set these limiting strains at low values in order to avoid creep and fatigue phenomena that may eventually manifest themselves in ruts and cracks. The authors recommend that the limiting GTM shear strain be set in the range of 1.75 to 3.5 percent. The application of these suggested limiting criteria will be discussed presently.

It was found that there is no significant difference in stress-strain properties among the various mixes at binder contents equal to or less than the optimum for the regular asphalt mix; however, there was a marked difference between treated and untreated mixes where the binder content exceeds the standard optimum. Mixes containing the powdered reclaimed rubber show higher gyratory shear values than those of the straight asphalt mix below about 4 percent strain, while the straight asphalt mix shows higher gyratory shear for strain greater than about 4 percent. As already stated, the maximum working range of deflection for pavements is considered to place the shear strain below about 3.5 percent.

PRANDTL BEARING RESISTANCE

It is evident that the ultimate bearing resistance is generally highest for the untreated mix; however, the ultimate bearing for all mixes at all binder contents is more than the requirements for any normal pavement loading (Table 1). In other words, once a pavement has experienced the 5 to 6 percent strain required to develop its shear strength, it has failed as a normal pavement. On the basis of the recommended limiting strains (1.75 to 3.50 percent), the straight asphalt mix (control) and the mix treated with powdered reclaimed rubber are about equal at 6.5 and 7.0 percent bitumen content, while the mix treated with ground vulcanized rubber is equal to or lower than the other two. On the other hand, if sufficient binder is incorporated into the mix to cause the rubber treatment to become fully effective, then the treated mixes are enhanced by the properties as discussed earlier (greater recovery and elimination of bleeding) while they still retain essentially full bearing resistance. Should a ½ to 1 percent increase in binder occur on an untreated mix (because of faulty design or poor control, as the case might be), then the Prandtl bearing resistance at this limited deformation becomes extremely small, and failure of the surface due to shoving would be imminent.

SUMMARY

The rubber-treated mixes were consistently found to show greater resistance to densification than the untreated mixes. Probably the most significant finding was that the addition of these rubber additives tends to desensitize the mix with regard to flushing and bleeding. It was found that in order for the rubber to be effective in altering physical properties, other than resistance to densification, the binder level (asphalt +

rubber additive) must be somewhat greater than the optimum for the straight asphalt mix. Another significant finding was that rubber-treated samples show improved dynamic recovery properties. Data presented suggest that the utilization of a small amount of either powdered reclaimed or ground vulcanized rubber in the mix will allow the design engineer to desensitize the mix against bleeding or shoving or both. In addition, one can apparently improve the strength of the pavement at low strain levels as well as retain the original "life" (dynamic recovery) properties of the pavement. The amount of reclaimed rubber additive required to be effective appears to be very small: in the order of one half of one percent.

ACKNOWLEDGMENTS

The authors acknowledge the help of E. F. Sverdrup in initiating and monitoring this research program prior to his retirement from the U.S. Rubber Reclaiming Company, and to B. J. Huff in performing tests on asphaltic mixtures reported in this paper. We would also like to express our appreciation for the encouragement and wise counsel furnished by B. R. Wendrow and J. C. Elgin of the U.S. Rubber Reclaiming Company.

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VOIDS CONCEPT FOR DESIGN OF SEAL COATS AND SURFACE TREATMENTS

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A surface treatment design procedure based on the measured voids content of an aggregate layer is presented. The procedure can be used for the design of single-application surface treatments and seal coats. Relationships are developed for the quantities of cover aggregate and asphalt binder required to produce surface treatments that should exhibit good performance characteristics in the field. Factors taken into account in the determination of these quantities include aggregate size, aggregate shape, aggregate source, aggregate quantity, aggregate fines, percentage of solvent in binder, hardness of underlying surface, embedment of aggregate into the underlying surface, temperature, and compactive effort. A design example is also presented to illustrate the use of the method and the principles it incorporates.

•SEVERAL methods for the design of surface treatments have been developed by other engineers in the past, and some of these methods have been used for a number of years (3). While the design procedure suggested here utilizes some of the basic ideas in the other methods, it contains a number of improvements that have resulted from research concerning the amount of voids existing in surface treatment aggregate layers. These improvements are as follows:

1. More precise determination of the quality of bitumen needed by taking into account (a) the nonlinear variation in the volume of voids with depth within the aggregate layer, and (b) the depth of embedment of the aggregate into the underlying surface;
2. Simplified design procedure to account for the shape of the aggregate by use of different voids curves for crushed stone and gravel; and
3. An accurate method for adjusting the amount of bitumen to account for the fines in the aggregates.

FUNDAMENTAL CONSIDERATIONS IN DESIGN PROCEDURE

The bases of the design procedure are as follows:

1. The quantity of cover aggregate required is generally that amount needed to form a layer one particle in depth over the surface being treated. In no way is the required quantity of aggregate influenced by the amount of bituminous binder material that is to be used.
2. During construction and under subsequent traffic, the aggregate particles tend to reorient until they present their least dimensions in the vertical direction.
3. Because of reorientation of the aggregate, the quantity of bitumen and, to a certain extent, the quantity of aggregate needed are related to the average least dimension of the aggregate.
4. The basic quantity of bituminous material to be used is that amount required to fill the voids existing between the aggregate particles to an optimum depth. Therefore, the amount of binder needed is a function of the volume of the voids in the cover aggregate layer that, in turn, is influenced by factors such as aggregate gradation, maximum

Sponsored by Committee on Bituminous Surface Treatments and presented at the 50th Annual Meeting.

size, aggregate shape and surface texture, and aggregate embedment into the underlying surface.

5. In order to determine the spray quantity of bituminous material needed, the basic quantity of bituminous material must be adjusted for (a) aggregate absorption characteristics, (b) characteristics of underlying surface (rich or dry), (c) amount of volatiles present if a cutback or an emulsion is used, and (d) increase in the volume of the bituminous material when heated for spraying.

Prior to any mathematical calculations, two important decisions must be made. The type and size of aggregate and the type of bituminous material to be used in the surface treatment must be selected. In addition, basic identifying data in regard to the aggregate and bitumen must be determined by initial tests.

SELECTION OF TYPE AND SIZE OF AGGREGATE

Two gradations of aggregates are generally used for surface treatments: graded and one-size aggregates. Graded aggregates have a reasonable amount of aggregate in each sieve fraction and a sizable amount (greater than 10 percent) passing the No. 4 sieve. A one-size aggregate usually has little material passing the No. 4 sieve and probably has more than 60 percent of the aggregate of one size.

Although a well-graded aggregate is usually considered to be the best gradation for a compacted-asphalt mixture, it is not so desirable for surface-treatment construction. Research data indicate that the one-size aggregate not only performs better in surface treatments but also provides several advantages related to the design and construction of the surface treatments. The one-size aggregates usually develop good skid-resistance qualities. In addition, one-size aggregates are usually retained better by the bituminous binder because of the reduction in the fine fraction of the aggregate. Even more important, there is a good possibility that surface treatments constructed with aggregate containing large amounts of fine material will bleed. Excess fine material acts as a filler for the bituminous material and, in essence, will increase the amount of bituminous material in the aggregate.

It is often thought that one-size aggregate is economically unfeasible because of the added cost of producing the aggregate. However, the performance of surface treatments constructed with one-size aggregate appears to be quite superior to the performance of similar constructions using graded aggregate. (The service lives of surface treatments constructed with one-size aggregates are usually considerably longer than those of surfaces constructed with graded aggregate.) It may be better to expend more money initially, in order to obtain a one-size aggregate that will perform well, than to have lower costs initially but higher maintenance costs.

It is also desirable to use a fairly large-sized aggregate ($\frac{3}{4}$ in.) instead of a small-sized aggregate. The larger size of aggregate provides good skid resistance. In addition, there is less chance of bleeding occurring with the large-sized aggregate. A small variation in the amount of bituminous material in a small aggregate may completely fill the voids, while the same variation of the bituminous material in a larger size of aggregate may have almost no effect.

Some persons dislike surface treatments constructed with large-sized aggregates because of noise and roughness. However, with the increased performance and ~~long-lived~~ better, longer life skid resistance ~~with better and~~ that can be obtained by using the large-sized aggregate, the benefits of reducing the top size to $\frac{3}{8}$ in. should be seriously evaluated.

SELECTION OF TYPE AND GRADE OF BITUMINOUS BINDER

Consideration should be given to a number of factors when the type and grade of bituminous binder are selected. These factors include temperature, viscosity, atmospheric conditions, condition and temperature of surface to be treated, adhesion, and traffic type and speed. In order to satisfy one or more of these factors, asphalt cements, rapid-curing cutbacks, medium-curing cutbacks, and emulsions are normally used for surface treatment construction.

Based on observation and study, the authors recommend the use of a paving-grade asphalt cement for surface treatment construction, if the treatment can be placed satisfactorily. Of all of the bituminous binders that could be used for surface treatment construction, asphalt cement is perhaps the best. Primarily, asphalt cements harden quickly to provide a strong binder that holds the aggregate firmly in place. Thus, there is less chance of the aggregate being dislodged by early traffic. In addition, they provide a hard residue capable of developing a high cohesive strength. Asphalt cements also have less tendency to bleed, and their use provides a relatively impervious seal for the existing pavement surface. All of these desired qualities help ensure a surface treatment of long life.

In some instances, it may be difficult to get the aggregate suitably bonded with an asphalt cement binder. This may occur when the underlying surface is cold and causes chilling of the asphalt cement before the aggregate can be spread and compacted into the asphalt. In such cases, and in related instances, it may be desirable to use a cutback or an emulsion, but their use should be limited.

If the bituminous material to be used in the surface treatment construction is either a cutback or an emulsion, the percentage of volatiles must be determined. This should be done by distillation or, as a last resort, by estimation. In addition, the temperature to which the bituminous material must be heated in order to have adequate spraying viscosity must be determined.

AGGREGATE QUANTITY

The quantity of cover aggregate should be that amount needed to form a layer one stone in depth over the surface to be treated. This quantity is influenced by such factors as size, shape, and specific gravity of the aggregate. Usually 2 aggregate quantities are determined. One quantity is referred to as the basic quantity and is essentially the exact amount of aggregate needed to cover 1 sq yd of surface. The field-spread quantity, or normally the spread quantity, is the amount of aggregate that is to be spread on the road surface. This latter quantity is greater than the basic amount because varying allowances must be made for aggregate whip-off and construction inaccuracies. The basic aggregate quantity is usually determined first and then is modified to provide the spread quantity.

Average Least Dimension

Normally aggregate will tend to rotate under the rolling action of wheels until it is oriented with the least dimension in the vertical direction. Thus, the average of the least dimensions of the aggregate particles is an indication of the average thickness of the aggregate in the road surface. Consequently, the quantity of bituminous material needed and, to a certain extent, the quantity of aggregate needed is related to the average least dimension (ALD) of the aggregate.

The most accurate method of determining the ALD of either a one-sized or a graded aggregate is by direct measurement of the thickness of the aggregate particles after removal of all material passing the No. 4 sieve. The ALD of each aggregate particle is measured with a pair of calipers. From an average of the measured least dimension of a minimum of 100 particles, the average least dimension of the cover aggregate can be obtained.

An indirect method can be used to determine an approximate ALD of either a one-sized or a graded aggregate. Average values of the ALD of different-sizes and types of aggregates have been determined and are given in Table 1. Once the gradation of the aggregate has been determined, the weighed average of the ALD's for the various sieve fractions can be used as an estimate of the ALD of the cover aggregate.

TABLE 1

AVERAGE LEAST DIMENSION OF DIFFERENT SIEVE FRACTIONS

Sieve Fraction	ALD of Sieve Fraction	
	Crushed Stone	Gravel
>3/4 in.	0.57	0.60
1/2 to 3/4 in.	0.48	0.49
3/8 to 1/2 in.	0.36	0.37
No. 4 to 3/8 in.	0.25	0.26

The approximate least dimension is determined by multiplying the percentage of aggregate for each individual sieve fraction by the ALD for that fraction. The sum of the products is then divided by 100 minus the percentage passing the No. 4 sieve to obtain the desired estimate of the average least dimension.

Basic Quantity

The most direct method and probably the most accurate method for determining the basic quantity of cover aggregate is the test-board method. In this method, a board of known area is covered with a sufficient quantity of aggregate, with the least dimension of the aggregate particles placed upward, so that complete coverage of the board with the aggregate one thickness in depth is obtained. The weight of the aggregate on the board in pounds divided by the area of the board in square yards is the basic quantity of aggregate needed in pounds per square yard. This method is quite accurate and is especially good in that it indirectly accounts for all of the variables in the aggregate that influence the amount needed.

An estimate of the basic quantity of the aggregate can be determined by the following equation:

$$\text{Basic quantity} = 24 \times \text{ALD} \times G \tag{1}$$

where

ALD = average least dimension of the aggregate in in.; and
G = bulk specific gravity of the aggregate.

Use of Eq. 1 provides only a rough estimate of the basic quantity and is inferior to the test-board method. It only relates the quantity needed to the ALD of the aggregate and specific gravity of the aggregate and does not take into account the shape of the aggregate.

Field-Spread Quantity

Because of loss of aggregate due to whip-off and inaccuracy in spreading of the aggregate, the quantity of aggregate to be spread on the road surface should be greater than the basic quantity. The actual amount of increase in the basic quantity should be based on the construction techniques (quality control) and equipment to be used at the particular job site. However, when the actual increase is not known, an increase in the basic quantity of 6 to 15 percent (by weight) should be satisfactory.

CONDITIONS FOR DESIGN EXAMPLE

To illustrate the procedures outlined in this paper, a design example is included. For the design example, it will be assumed that a one-sized aggregate, single-layer surface treatment is to be placed on a nonporous bituminous surface. An MC-3000 cutback asphalt is to be used for the bituminous binder. The aggregate to be used is crushed stone. The "hardness" of the underlying surface is measured to be 0.40 in. at 100 F. The anticipated surface temperature at time of construction and early treatment life is 120 F. Forty percent of the aggregate (by volume) is to be exposed to provide adequate skid resistance.

The one-sized, crushed-stone aggregate to be used has the following gradation:

Sieve Size	Percent Passing
¾ in.	100
½ in.	98
⅜ in.	30
No. 4	9

The bulk specific gravity of the aggregate is 2.72. The MC-3000 was found to contain 19 percent cutterstock, and it was determined that the cutback should be heated to 220 F in order to have adequate spraying viscosity.

DETERMINATION OF AGGREGATE QUANTITY

For the design example, an approximate ALD, computed by using the data given in Table 1, is given in Table 2. Calculated estimate of ALD = 0.307/0.91 = 0.34 in.

The actual quantity of aggregate needed for the design example should be determined by using the test-board method. However, to illustrate the procedure, the basic quantity will be estimated from the ALD.

TABLE 2

ALD OF DIFFERENT SIEVE FRACTIONS USED IN DESIGN EXAMPLE

Sieve Fraction	Percent of Sieve Fraction in Aggregate	ALD of Sieve Fraction	Percent × ALD
½ to ¾ in.	2	0.48	0.009
⅜ to ½ in.	68	0.36	0.245
No. 4 to ⅜ in.	21	0.25	0.053
Total	91		0.307

$$\text{Basic quantity} = 24 \times \text{ALD} \times G = 24 \times 0.34 \times 2.72 = 22.2 \text{ lb/sq yd}$$

The basic quantity will be increased by 15 percent in the design example to allow for whip-off and waste.

$$\text{Spread quantity} = 22.2 \times 1.15 = 25.5 \text{ lb/sq yd}$$

DETERMINATION OF BITUMINOUS QUANTITY USING VOIDS CONCEPT

The required basic amount of bituminous material is that quantity needed to fill the voids existing between the aggregate particles to an optimum depth. Because the volume of voids between the aggregate particles is a function of (a) the size of the aggregate, (b) the shape of the aggregate, and (c) the amount of aggregate embedment into the underlying surface, the required amount of bituminous material is also related to these factors.

The first step in determining the desired basic binder quantity by the proposed procedure is to obtain a measure of the void space between aggregate particles to complete submergence when resting on a rigid (flat bottom) base. The total void space is then reduced by appropriate factors to account for (a) desired aggregate exposure to ensure a skid-resistant surface and (b) embedment of the aggregate into the underlying surface. Corrections are then applied to the resulting binder volume to account for (a) fines in the aggregate, (b) condition of the underlying surface, (c) amount of volatiles in the bituminous material used, and (d) volume change in the bituminous material due to temperature variation. The corrected volume relates the amount of binder to be applied by the distributor.

Percentage of Voids

Data obtained from an investigation (4, 8) of the void space between aggregate particles on a rigid, unyielding surface (flat bottom situation) indicated that the relationships between percentage of voids and depth (as a function of ALD) of the aggregate for both gravel and crushed stone with an ALD of 0.50 are as shown in Figure 1. The percentage of voids shown in this figure is defined as follows:

$$\text{Percentage of voids} = \frac{\text{volume of voids at depth d}}{\text{horizontal surface area} \times \text{depth d}} \times 100 \qquad (2)$$

where the volume is in cubic inches, the horizontal surface area is in square inches, and the depth is in inches. Use of Eq. 2 relates the void space as a percentage of the volume to depth d, not of the total volume to the top of the aggregate (complete aggregate submergence).

The information shown in Figure 1 is given for an ALD of 0.50 in. If an aggregate having a different ALD is to be used, the curves shown in Figure 1 must be shifted by

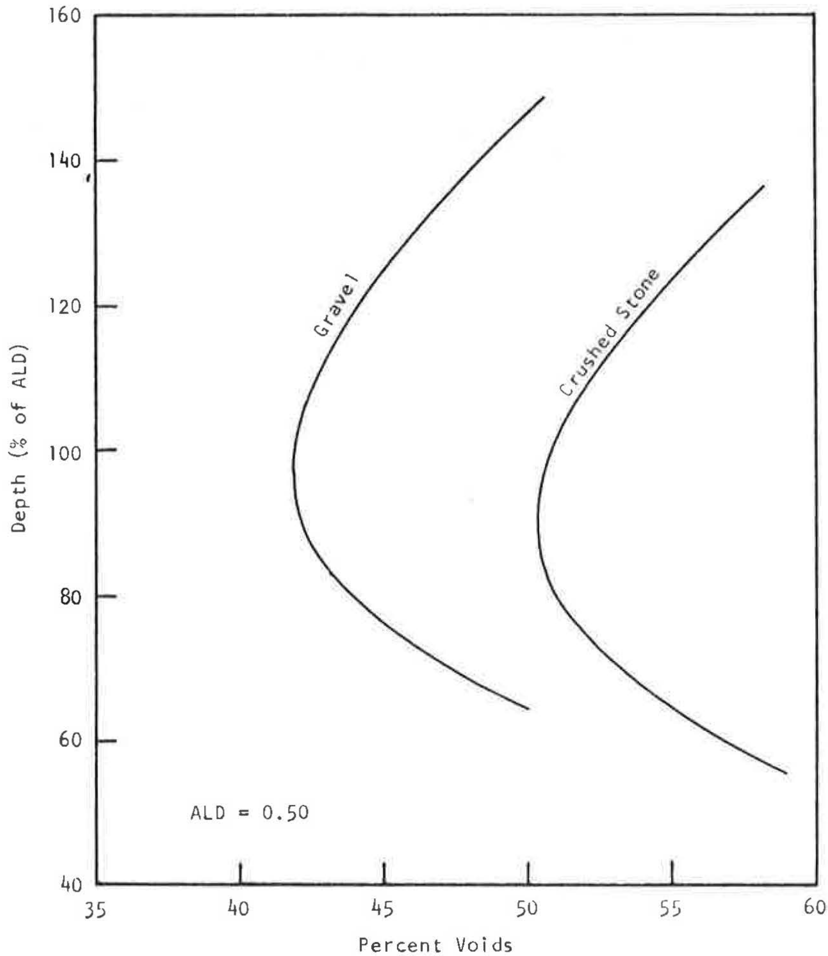

Figure 1. Variation in percentage of voids in a one-sized aggregate layer with depth as a function of the average least dimension of the aggregate (no embedment).

a factor determined by Eq. 3. This equation is applicable for both gravel and crushed stone.

$$\text{ALD factor} = 14.8 - 29.5\ (\text{ALD}) \tag{3}$$

where the ALD factor is a percent.

The data used to calculate the percentage of voids by Eq. 2 were also used to calculate the percentage of voids as a function of volume to complete aggregate submergence. The percentage of voids on this basis is defined as follows:

$$\text{Percentage of voids} = \frac{V_{v_{\text{flat bottom to depth d}}}}{\text{horizontal surface area} \times \text{depth D}} \times 100 \tag{4}$$

where $V_{v_{\text{flat bottom to depth d}}}$ is the volume of voids to any depth d measured by using a flat-bottom container with no aggregate embedment in cubic inches, and depth D is depth to a point of complete submergence of the aggregate when on a rigid base in inches. When the percentage of voids calculated by Eq. 4 is plotted against depth as a percentage of ALD, the curves for gravel and crushed stone shown in Figures 2 and 3 result. It can be observed from data shown in Figure 2 that the percentage of voids does not vary linearly with depth within the aggregate layer.

Total Percentage of Voids in Aggregate Layer

The uncorrected percentage of voids at any depth in a one-sized aggregate layer can be determined from the curves shown in Figure 1 and computed by Eq. 3 or from the curves shown in Figures 2 and 3. These were derived from data obtained in a previous investigation (4, 8).

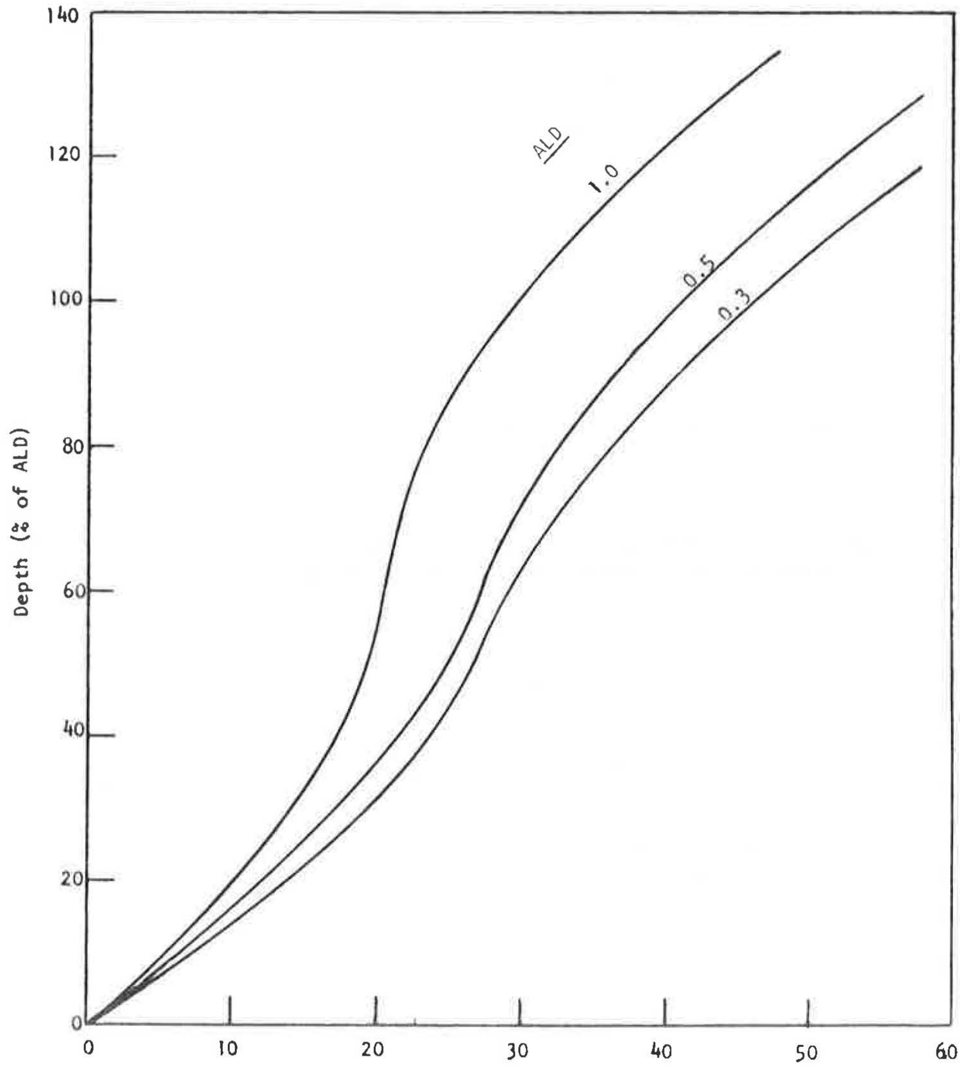

Figure 2. Relationship between percentage of voids and depth as a percentage of ALD for crushed stone.

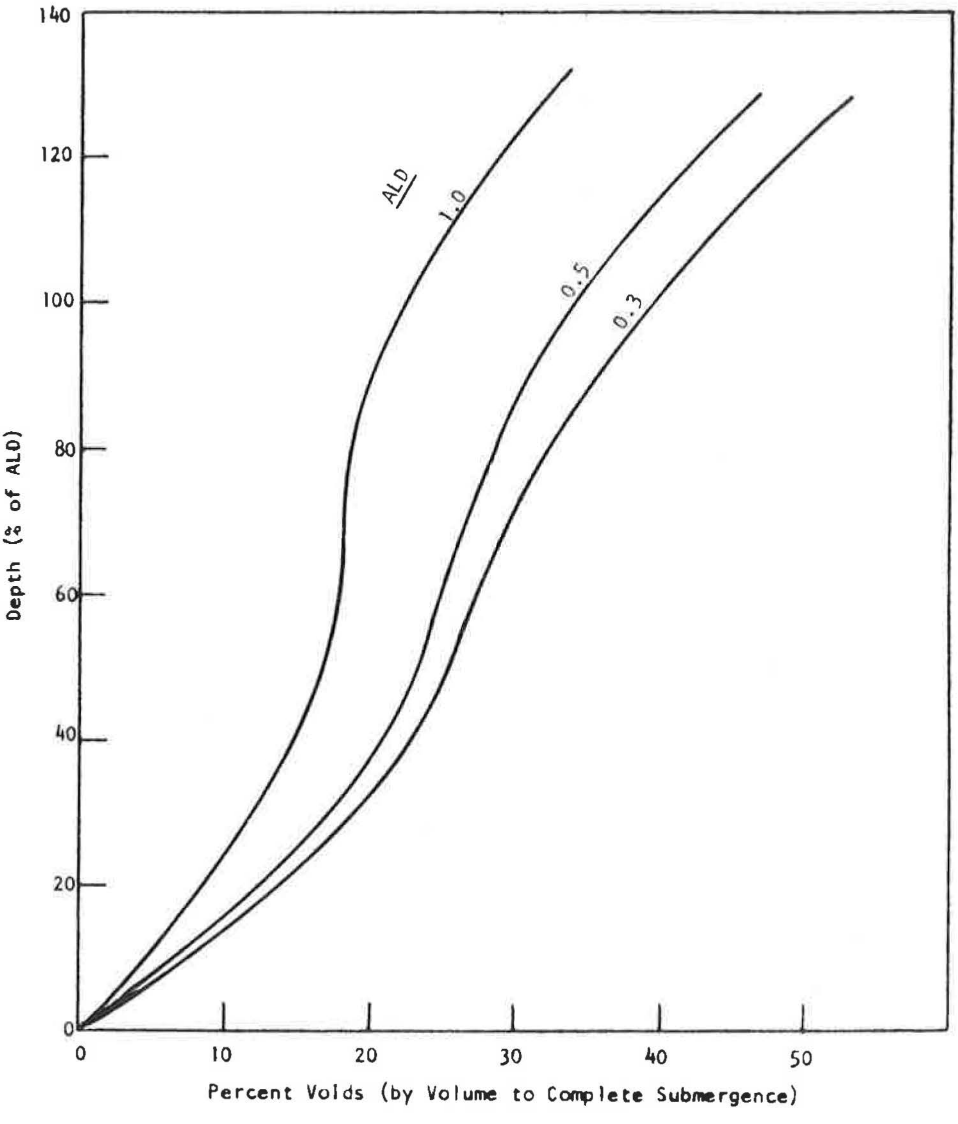

Figure 3. Relationship between percentage of voids and depth as a percentage of ALD for gravel.

The total percentage of voids at complete aggregate submergence (1.25 ALD) is determined from Figure 1 and the following equation:

$$V_{T(ALD)} = V_{T(0.5)} + 14.8 - 29.5\,(ALD) \tag{5}$$

where

$V_{T(ALD)}$ = total percentage of voids to complete submergence in an aggregate layer with a specific ALD; and

$V_{T(0.5)}$ = percentage of voids at complete submergence (125 percent of ALD) for an aggregate with an ALD = 0.50.

The total percentage of voids for the design example at complete aggregate submergence for a crushed stone with an ALD of 0.34 in. is as follows:

$$V_{T(ALD)} = 55.6 + 14.8 - 29.5\,(0.34) = 60.4 \text{ percent}$$

Reduction of Total Voids to Account for Aggregate Embedment Into Underlying Surface

The percentage reduction in void space to account for embedment of the aggregate into the underlying surface is a function of (a) the hardness of the underlying surface, (b) the temperature of this surface, and (c) the number of passes of the roller used to compact the aggregate. The relationship is shown in Figure 4. The percentage reduction shown in this figure was calculated by the following relationship:

$$\text{Percentage reduction} = \frac{V_{v_{\text{flat bottom}}} - V_{v_{\text{embedded}}}}{V_{v_{\text{flat bottom}}}} \times 100 \qquad (6)$$

where $Vv_{\text{flat bottom}}$ is the volume of voids to complete aggregate submergence measured when aggregate layer is on a rigid base (8); and Vv_{embedded} is the volume of voids to complete aggregate submergence measured after aggregate embedment (8).

Before Figure 4 can be used to determine the reduction in percentage of voids that is necessary to account for aggregate embedment, information concerning underlying surface type (series) must be obtained. This information is readily obtained by conducting a simple field test for hardness and by using data shown in Figure 5. The test for surface hardness at a given temperature is performed by using a standard Marshall compaction hammer and a 1-in. diameter hardened steel sphere. The sphere is placed in contact with the underlying surface to be tested and is subjected to 5 blows of a 10-lb weight falling through a fixed distance of 18 in. The vertical distance from the plane of the surface to the lowest point of the depression is measured with a standard penetrometer. This distance is obtained for a minimum of 5 locations on the surface being tested. The average of the 5 values is determined to give a measure of the surface hardness at the test temperature. As the hardness of the underlying surface increases, the depth of sphere penetration decreases.

For the design example, it was stated that the hardness of the underlying surface was measured to be 0.40 in. at 100 F. Thus, from Figure 5, the underlying surface is determined to be a Series 3 type.

The amount that the total percentage of voids must be reduced is obtained from data given in Figure 4. Enter the figure according to the highest temperature to which the underlying surface will be subjected; proceed upward to the type of underlying surface having the number of the roller passes to be used in compaction of the cover aggregate; and read the percentage reduction on the vertical axis.

The amount of reduction of the total voids to account for embedment into the underlying surface is computed by the following equation:

$$V_E = \frac{R}{100} \times V_{T\,(ALD)} \qquad (7)$$

where

V_E = amount (not percent) that the total percentage of voids must be reduced to account for embedment into the underlying surface; and

R = percentage reduction from Figure 4.

From Figure 4, the required percentage reduction for the design example for a Series 3 surface at 120 F and with 3 complete coverages by roller is determined to be 30

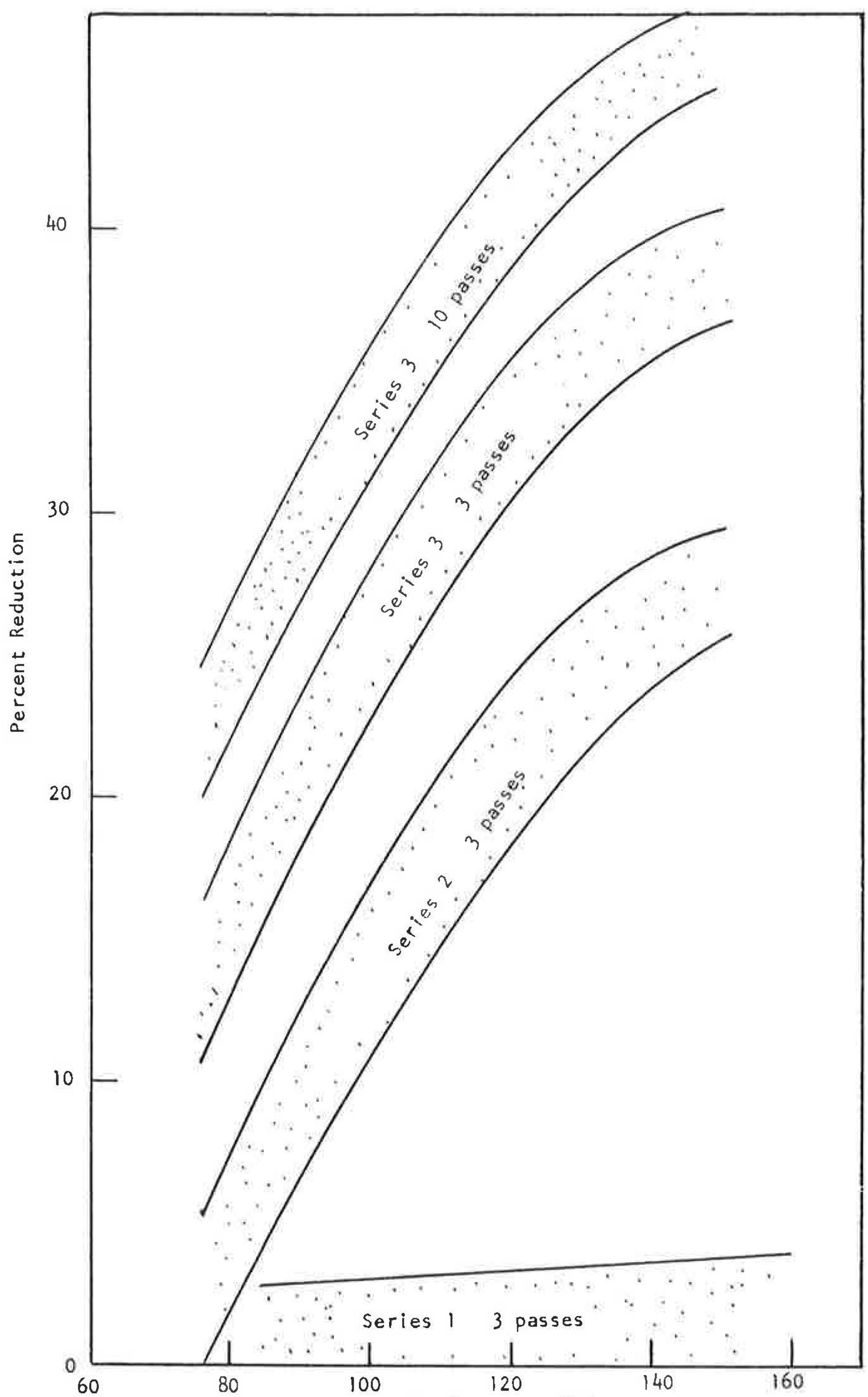

Figure 4. Percentage reduction in void space versus temperature of underlying surface.

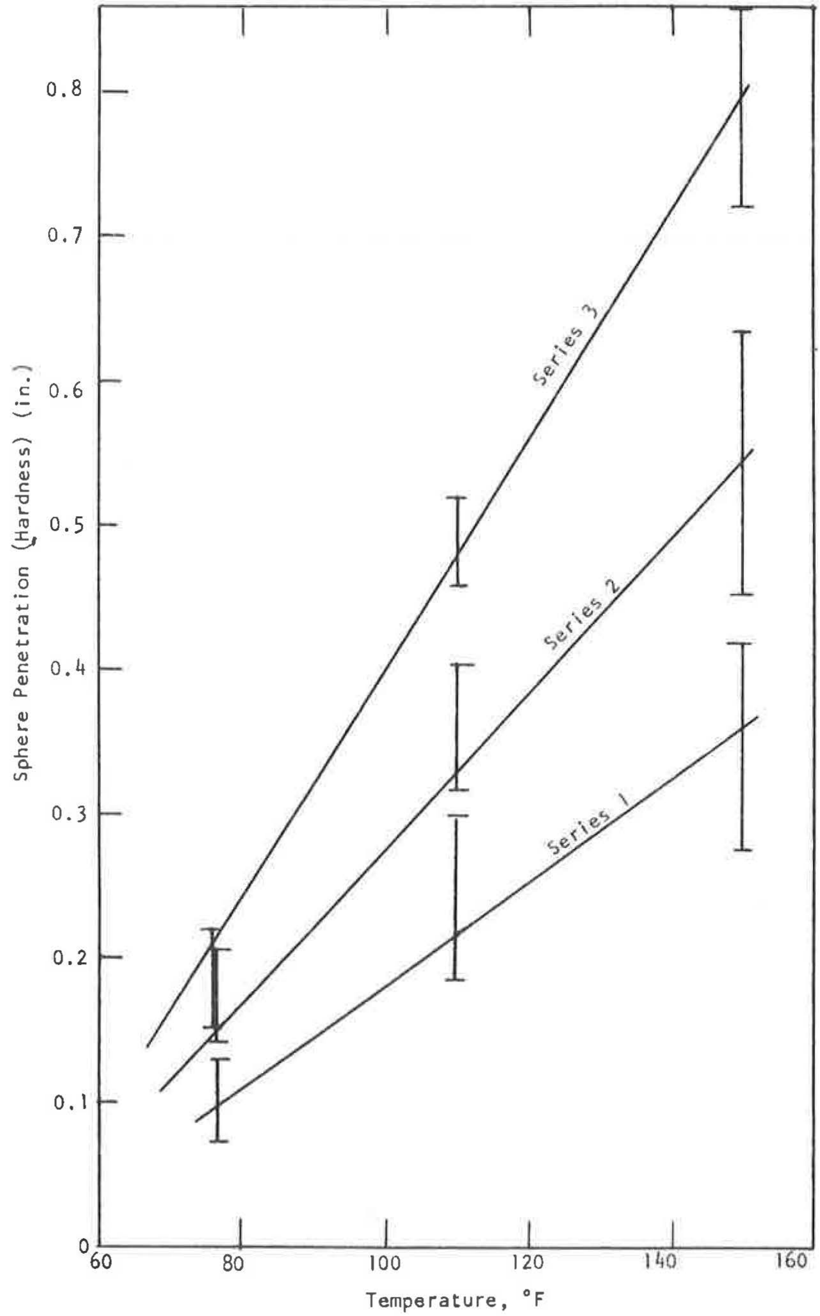

Figure 5. Relationship between hardness of underlying surfaces and temperature.

percent. That is, the void space to complete aggregate submergence for a no-embedment situation is reduced by 30 percent for the stated conditions.

The amount of reduction of the total percentage of voids is as follows:

$$V_E = \frac{R}{100} \times V_{T(ALD)} = \frac{30}{100} \times 60.4 = 18.1 \text{ percent}$$

Reduction of Total Voids to Provide Satisfactory Skid Resistance and Aggregate Retention

The voids within an aggregate layer should not be filled with binder to the depth of complete submergence of the aggregate. If this is done, the aggregate particles will be covered with binder. Under these conditions and especially when the temperature of the binder is increased, bleeding of the surface treatment may occur and the surface will have low skid resistance, especially when wet. Consequently, the depth of the binder in the aggregate layer should be less than the depth required for complete submergence of the aggregate.

Care should also be taken that the amount of binder is not reduced too much. If too little binder is used, the aggregate will not be held firmly in place, and there is a good probability that the aggregate will be easily dislodged by the forces produced by moving wheel loads.

At the present time, experimental data are not available to indicate critical values of these 2 parameters. Until more data are available, the following criteria are suggested:

1. In order to provide satisfactory skid resistance, about 40 percent (by volume) of the aggregate should be exposed; and
2. In all cases, the voids in the aggregate should be filled to a depth equal to or greater than 60 percent of the ALD of the aggregate. This should be done in order that sufficient bitumen is present to adequately bind the aggregate to the underlying surface.

Because the voids in the aggregate (and thus the quantity of bitumen needed) have been related to the depth in the aggregate layer, the percentage of aggregate exposed (for developing adequate skid resistance) will also be related to the depth. Such a relationship has been established and is shown in Figure 6. For design purposes, this figure may be entered with the percentage (by volume) of aggregate that must be exposed to provide adequate skid resistance to determine the depth to which the bituminous binder must be placed.

The curves shown in Figure 6 are applicable for all crushed stones or gravels of essentially one-sized gradation regardless of the ALD of the aggregate. A linear relationship does not exist between the 2 indicated variables. Further, 0 percent aggregate (by volume) is exposed when the depth of binder is 125 percent of the ALD of the aggregate.

The first step is to determine if the percentage of depth (percent ALD) is greater or less than 60 percent. If the percentage (of ALD) is greater than 60 percent, then sufficient bitumen is present to adequately bind the aggregate to the underlying surface. If it is less than 60 percent, an adjustment in the percentage of depth must be made to provide a balance between the maximum amount of bitumen needed to provide adequate skid resistance and the maximum amount of bitumen needed to provide satisfactory aggregate retention.

The amount that the total voids must be reduced to provide satisfactory depth of bitumen is computed as follows: Select either Figure 2 or 3 depending on the type of cover aggregate (gravel or crushed stone); enter the appropriate figure with the depth (percent ALD) previously determined and determine the percentage of voids to depth d based on complete submergence; and compute the amount of reduction of the total voids by the following equation:

$$V_S = V_{T(ALD)} - V_d \qquad (8)$$

where

V_S = amount (not percent) that total percentage of voids must be reduced to provide satisfactory depth of bitumen (so as to have adequate skid resistance and aggregate retention); and

V_d = percentage of voids to depth d determined from Figure 2 or 3.

For the design example, 40 percent of the aggregate (by volume) is to be exposed to provide adequate skid resistance. From Figure 5 it is determined that a change in depth from 125 to 72 percent of ALD is required for the desired aggregate exposure. Therefore, the required depth change is 125 - 72 = 53 percent of ALD. A depth of 72 percent of ALD is greater than 60 percent. Thus, there should be sufficient bitumen present to adequately bind the aggregate to the underlying surface. Enter Figure 2

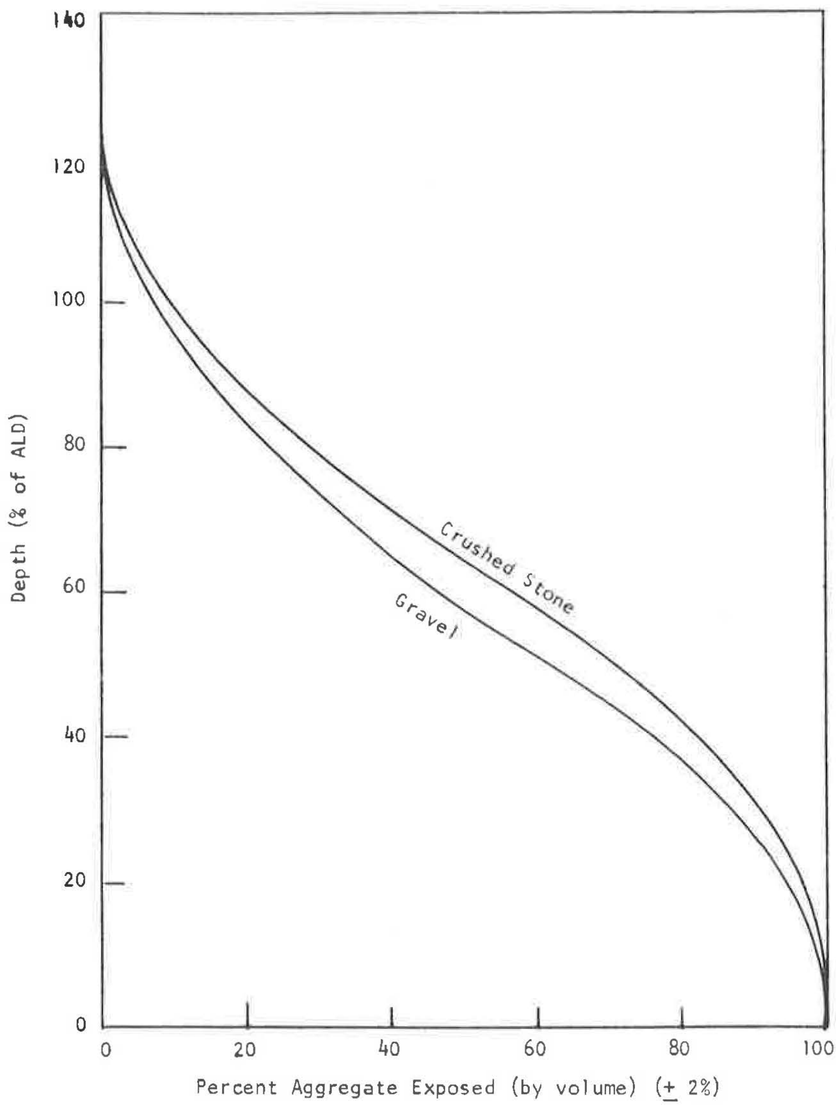

Figure 6. Relationship between percentage of aggregate exposed and percentage of depth regardless of aggregate size when the aggregate is on rigid base.

with a depth of 72 percent of ALD. For an ALD of 0.34, the value of V_d is read as 32.0 percent voids. The amount of reduction of the total percentage of voids computed by using Eq. 8 is as follows:

$$V_s = 60.4 - 32.0 = 28.4 \text{ percent}$$

Uncorrected Basic Quantity of Bitumen

In summary, the previous discussion has indicated the following:

1. The percentage of voids in an aggregate layer can be determined as a function of the specific size of the aggregate (ALD), the shape of the aggregate (gravel or crushed stone), and the depth to which the voids are determined.
2. The total percentage of voids in an aggregate layer must be reduced to account for the embedment of the aggregate into the underlying surface. The amount of this reduction is related to the hardness of the underlying surface, the temperature of this surface, and the number of passes of the roller used to compact the aggregate.
3. A further reduction in the voids in the aggregate must be made to allow adequate volume of the aggregate to be exposed above the bitumen layer so as to provide satisfactory skid resistance.
4. The percentage of voids to be filled with bitumen is determined by reducing the total percentage of voids by the amount of voids reduced by aggregate embedment into the underlying layer and by the amount of voids exposed above the top of the bitumen layer. This is expressed in equation form by

$$V_C = V_{T(ALD)} - (V_E + V_S) \tag{9}$$

where V_C = corrected percentage of voids.

5. The uncorrected basic quantity of bitumen is computed from the following equation:

$$Q_1 = \frac{V_C}{100} \times ALD \times 7.03 \tag{10}$$

where Q_1 = uncorrected basic quantity of bitumen in gal/sq yd. Equation 9 is used to compute the corrected percentage of voids for the design example.

$$V_C = 60.4 - (18.1 + 28.4) = 13.9 \text{ percent}$$

Thus, the uncorrected basic quantity of bitumen is

$$Q_1 = \frac{13.9}{100} \times 0.34 \times 7.03 = 0.33 \text{ gal/sq yd}$$

Bitumen Quantity Corrected to Account for Fines in the Aggregate

The fines in the aggregate that pass the No. 4 sieve materially influence the volume of binder needed. The fine material reduces the volume of voids in the aggregate and, if the bitumen quantity is not reduced, the fine material will, in effect, increase the depth of the bitumen and cause a flush surface condition to result. Thus, the quantity of the binder should be reduced to account for the fines in the aggregate.

The volume of reduction necessary to account for fines is equal to the volume of the fine material in the aggregate layer. This volume can be computed from the weight and the specific gravity of the fines. In other words, the quantity of binder needed can be determined from the following equation:

$$Q_B = Q_1 - 0.12 \times \frac{W}{G} \tag{11}$$

where

Q_B = basic quantity of bitumen in gal/sq yd; and
W = weight of minus No. 4 material in the aggregate in lb/sq yd of surface area.

For the design example, the weight of minus No. 4 material in the aggregate = 0.09 × 22.2 = 2.0 lb/sq yd. Thus, the basic quantity of bitumen is

$$Q_B = 0.33 - 0.12 \times 2.00/2.72 = 0.24 \text{ gal/sq yd}$$

Field-Spray Quantity

The amount of bituminous material that is to be sprayed from the distributor is not the same quantity as the computed basic amount. The basic quantity must be modified to adjust for conditions that exist at the construction site. These conditions include condition of the underlying surface, amount of volatiles in the bituminous materials, and change in the volume of bituminous material as it is heated for spraying.

Correction for Condition of Underlying Surface

In the determination of the basic quantity of the bituminous material, no allowance is made for that portion of the binder that will be absorbed into the existing road surface. Few data are currently available to indicate the exact influence of the absorptive condition of the underlying surface. Until these data are available, suggested approximate adjustments that may be used are given in Table 3.

For the design example, the underlying surface is nonporous. Thus, the bitumen content should be increased by 0.00 gal/sq yd.

$$Q = 0.24 + 0.00 = 0.24 \text{ gal/sq yd}$$

Correction for Amount of Volatiles in Bituminous Material

The basic amount of the bituminous binder is the amount of residual bitumen that must be used in the surface treatment construction. If the bituminous binder to be used is an asphalt cement, no adjustment in the bitumen quantity is needed. However, if the bituminous material contains volatiles, allowance must be made for loss of the cutter-stock from cutbacks or water from emulsions. This is normally accomplished by dividing the basic amount of residual bituminous material required by the percentage of the residual bituminous material in the cutback or emulsion to be used.

Because there is 19 percent solvent in the binder being considered in the design example, the amount of MC-3000 needed is

$$Q = 0.24/(1.00 - 0.19) = 0.30 \text{ gal/sq yd}$$

Thus, 0.30 gal/sq yd of the cutback will have the required basic amount of bitumen of 0.24 gal/sq yd.

TABLE 3

BITUMEN CORRECTION TO ACCOUNT FOR TYPE OF UNDERLYING SURFACE

Type of Underlying Surface	Increase in Bitumen Quantity (gal/sq yd)
Smooth, nonporous, hard	0.00
Slightly porous, hard	0.03
Dry, old, hard	0.06
Cracked, very dry, hard	0.09

Correction for Volume Change in Bituminous Material Due to Temperature Variation

Regardless of the type of bituminous binder to be used, asphalt cement or liquid asphalt, the binder is usually heated to a temperature suitable for spraying. Thus, a final correction must be made to the bitumen quantity in order to account for the volume change produced as the bitumen is heated to the spraying temperature. This can be done by using the common temperature-volume correction tables for asphaltic materials or by assuming that the coefficient of expansion of the material is approximately 0.00035 per 1 F.

The volume correction factor for the example design (spraying temperature of 220 F) is 0.9452. Thus, the spray quantity is

$$Q_S = 0.30/0.9452 = 0.32 \text{ gal/sq yd}$$

Thus, 0.32 gal/sq yd of the MC-3000 is needed at a spraying temperature of 220 F. The corrected amount, Q_S, is the quantity of bituminous material that the distributor should spray onto the underlying surface for the example given.

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DISCUSSION

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This is a very fine paper, and the authors should be commended for their effort. I am particularly pleased at their examination of the effect of depth of embedment of surface treatment aggregate into the underlying surface.

When I first read Hanson's paper (2) in the early 1950's, I noted that he estimated the percentage of voids in the aggregate as approximately 50 percent when first placed and as approximately 30 percent after rolling. It was my immediate thought that this reduction in voids was largely the result of increased embedment of the aggregate. Work that I did subsequently with a cone penetrometer indicated large differences in the rate of embedment of aggregate with temperature and type of pavement, and this appears to confirm the findings of the authors. I do not remember a previous paper that gave a method of quantitatively estimating the depth of embedment as this paper does, and for this reason I think that it deserves special recognition. However, the depth of embedment in this paper is based on 3 and 10 passes of a rubber-tired roller. This is considerably less than the depth of embedment that ultimately would be expected under traffic. Some form of correction is needed to properly estimate the ultimate depth of embedment.

My approach to estimating the binder requirements for surface treatments has been to estimate the voids in the mineral aggregate (VMA) based on the type of traffic, the condition of the underlying surface, and the type of aggregate. I have found that a good estimate of the ultimate voids in the surface treatment aggregate where the underlying bituminous pavement has heavy traffic is the VMA of the underlying pavement that presumably has reached equilibrium with the traffic. Under very heavy traffic, the VMA will usually be quite low. This means that there is not much room for binder at ultimate embedment and that the binder will not come very high on the aggregate until the aggregate is embedded. If surface treatment is attempted on a road under heavy traffic in cool weather, the embedment will be so slow that most of the aggregate will be thrown off. Those roads with heavy traffic should be surface-treated in the hottest weather so that the embedment of the aggregate can be accomplished in the shortest time possible. The authors state: "Of all the bituminous binders that could be used for surface treatment construction, asphalt cement is perhaps the best." If the authors are referring to laboratory or ideal field conditions, I would be inclined to agree. If they are referring to average field conditions, I would like to point out that in the eastern United States the first surface treatments were done with asphalt cements, and it was only because of the problems encountered that cutbacks and emulsions were developed.

The cost of surface treatment construction is greatly influenced by the percentage of the time that a surface treatment crew can work. Restricting the surface treatment crew to that portion of the time when conditions are ideal is not practical in the eastern United States. An indication as to why ideal conditions are important to the application of asphalt cement to surface treatment construction might be the following. I calculated the time for a $\frac{3}{8}$ in. piece of stone to penetrate through a $\frac{1}{2}$ gal layer of cutback, asphalt cement, and asphalt emulsion at 77 F with the following result:

Grade	Time in Seconds
RC-250	$\frac{1}{4}$
RS-2	$\frac{1}{4}$
150 to 200 penetration asphalt cement	1,250

ASPHALT CONTENT STUDIES BY THE NUCLEAR METHOD

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ABRIDGMENT

•A STUDY of asphalt content determination by neutron-thermalization techniques was performed at various bituminous batch plants with a prototype asphalt content gage. The gage had been specifically designed to provide a sensitive measure of bitumen content under field-testing conditions. The neutron-thermalization gage reported had previously been tested in the laboratory on controlled samples.

An investigation was made into gage response for different types of aggregates and also for similar aggregates of varying gradation. Tests were conducted on different batches of asphalt cement of a given manufacture in an attempt to study the variation of hydrogen content inherent in these samples. Results for the calibrated gage were compared to design content figures and extraction results.

This project was initiated to investigate the feasibility of the use of a nuclear device specifically designed to determine asphalt content of bituminous concrete. A previous report by the author (1) covered the initial studies in calibration of the gage and a statistical study of its reliability.

Studies by Lamb and Zoller (2), Varma and Reid (3), Howard and Covault (4), Walters (5), and Qureshi (6) have shown the feasibility of using a neutron gage for asphalt content determination. Very good reliability was achieved by the author in using the nuclear Chicago gage (1) due to several modifications that eradicated problems found by previous studies. Basically these were the inclusion of multiple slow neutron detectors, the achievement of a low background count, and the use of a fairly large source of fast neutrons. The most important of these appeared to be the low background count. Because the overall results are totally dependent on the statistics of radioactive decay, a gage with a low background count (count with no sample in the gage) will result in much better reliability than will a similar gage with high background. It is obvious that a low background count holds this variation to a minimum because any variation of background is included in any count of a material under test. Results indicated a standard deviation of 0.08 percent with this gage for 17 samples compared to carefully controlled design asphalt contents and 0.20 percent with extraction method of bitumen determination (1).

FIELD-TESTING PROCEDURE

The gage studied in this project and the pan that was used to hold the hot bituminous sample are shown in Figure 1. The gage with sample in the sample cavity prior to test is shown in Figure 2. Sliding the drawer shut positions the sample for test. The associated scaler-timer unit used to receive the raw counts is not shown.

The gage was used at 2 bituminous plants to check the feasibility of testing under field conditions. Samples of bituminous mixes were carefully prepared for calibration of the gage by weighing aggregates and bitumen to the nearest gram in a large mixing bowl and transferring the sample to the gage test pan. Fines and bitumen left in the mixing bowl after transfer to the gage were carefully measured, and the design asphalt content was corrected accordingly. In field practice with the gage tested, several samples of hot bituminous concrete should be prepared to establish a calibration curve for

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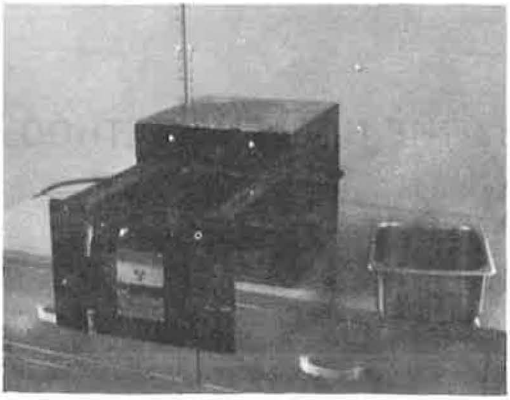

Figure 1. Gage with pan.

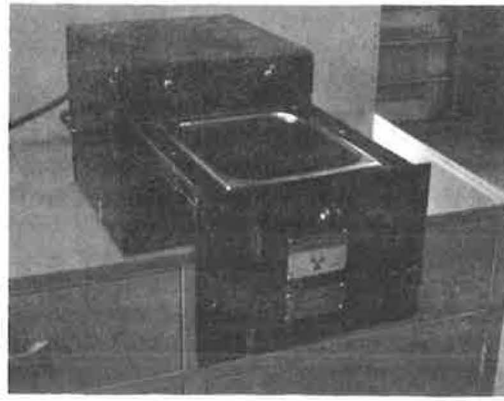

Figure 2. Gage with sample is cavity.

a particular material containing the aggregate and asphalt to be used in actual production. This calibration could then be used to check daily output of material used for construction so long as the mix design remained constant.

Extraction tests run on the materials after nuclear testing indicated a definite negative bias of extraction tests compared to design contents. The results were, therefore, fitted to design figures by least squares to more closely compare the nuclear and extraction results. Results of calibration tests at the 2 plants are given in Table 1.

The aggregates for plant 1 were river gravel and crusher sand with 100 percent passing the ½-in. sieve while those for plant 2 were river gravel and crusher sand with 100 percent passing the ⅜-in. sieve. Two different brands of asphalt were used in the two tests.

Actual testing each day required a 10-minute standard count on a standard material supplied with the gage in the morning and afternoon. The average daily standard count was used in calculating the nuclear ratio. The background count was the count with no sample in the gage. rms deviations from design content for the two jobs were ±0.12 percent (nuclear) and ±0.11 percent (extraction) and ±0.15 percent (nuclear) and ±0.12 percent (extraction) respectively. The mean deviations of extraction tests compared to design values prior to least squares fit were -0.19 percent and -0.48 percent respectively.

The gage response for different aggregates was evaluated by tests with the gage at 5 plants to substantiate the effect of aggregate type previously reported (1). Results are given in Table 2. Although different gradations of aggregate appeared to show no effect on the gage response, a definite effect was produced by different aggregate materials and possibly aggregate specific gravity, although this was not studied during this project.

Similarly, the effect of hydrogen content variation for different batches of the same

TABLE 1

CALIBRATION TEST RESULTS

Test	Design Asphalt Content (percent)		Nuclear Ratio		AC_t (percent)		AC_e (percent)		AC_e^1 (percent)	
	Plant 1	Plant 2	Plant 1	Plant 2	Plant 1	Plant 2	Plant 1	Plant 2	Plant 1	Plant 2
1	5.77	8.04	0.413	0.639	5.59	8.23	5.78	7.44	5.96	7.96
2	6.87	7.03	0.531	0.469	6.68	6.99	6.60	6.38	6.88	6.97
3	7.90	8.93	0.665	0.700	7.91	8.67	7.42	8.44	7.79	8.89
4	5.33	10.17	0.381	0.910	5.30	10.21	5.22	9.78	5.33	10.14
5	4.11	7.93	0.267	0.608	4.25	8.00	4.04	7.64	4.01	8.14

Note: Nuclear ratio = (material cpm - background)/(standard cpm - background); AC_t = 1.699 + 9.520 (nuclear ratio) by least squares; AC_e = extraction test results; and AC_e^1 = -0.5065 + 1.1188 (AC_e) by least squares.

TABLE 2

EFFECT OF VARIOUS AGGREGATES

Test	Aggregate	Maximum Size (in.)	Nuclear Ratio
1	Gravel	1½	0.247
2	Sand and gravel	½	0.226
3	Slag	½	0.200
4	Sand and gravel	⅜	0.268
5	Gravel	½	0.252

TABLE 3

EFFECT OF HYDROGEN CONTENT VARIATION

Test	Nuclear Ratio (pure asphalt cpm/std cpm)
1	3.703
2	3.693
3	3.691
4	3.557
5	3.567

bitumen was evaluated by 5 tests of the same liquid asphalt. Results are given in Table 3. Results indicated that there was a slight difference in hydrogen content for different batches of the same bitumen. A difference in hydrogen content for different brands of bitumen had previously been reported (1).

CONCLUSIONS

The results of this study indicate the following conclusions:

1. The nuclear method of determining asphalt content by neutron moderation appears as reliable as the standard extraction process for the gage used and the samples studied.
2. The test by nuclear methods can be performed by an individual with limited experience because of its simplicity; however, trained technicians are required to accurately prepare bitumen samples for calibration of the system.
3. After initial calibration, a test for asphalt content can be completely run in 15 minutes by the nuclear method employing the gage studied, compared to an hour or more for an extraction test.
4. There are definite effects on the nuclear test result due to the type of aggregate, but these may be accounted for by simple subtraction. Moisture trapped in the aggregates may similarly be accounted for.
5. There appears to be a slight difference in the hydrogen content of different batches of asphalt cement of given manufacture.
6. For limited samples, no gradation effect was noticed comparing one aggregate of two widely different gradations.
7. The possibility exists of an effect on count rate of slow neutrons due to aggregate specific gravity. This effect should be studied in more detail.

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EVALUATION OF BITUMINOUS COMPACTION PROCEDURES USING NUCLEAR GAGES

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A study was conducted by using nuclear gages to evaluate present bituminous construction procedures. Specifically, nuclear density tests were taken both during roller operations and after compaction had been completed. The density data were analyzed to study the feasibility of using nuclear gages to establish optimum rolling patterns for several different types of bituminous pavement materials for the rollers encountered. After final compaction, continued nuclear tests were taken in a study of any density variations in the compacted pavement. Density tests were taken transversely, on joints, along the pavement edges, along the longitudinal wheelpath areas, and in random locations along the pavement. Areas of low density appeared to be predominantly the joints and pavement edges. A separate study was conducted with 2 commercially available nuclear density gages to evaluate the effective depth of measurement. Both backscatter and air-gap techniques were analyzed. The air-gap density test was shown to be dependent on only the top $1\frac{3}{4}$ in. of material tested using the test method described.

•THE USE of nuclear gages for the determination of density and moisture has gained widespread acceptance in the past several years. Recent samplings show that nearly all states now use nuclear gages for specification control or are at least investigating the technique seriously prior to specification adoption. Early reluctance to accept nuclear devices revolved about the unwillingness to set aside techniques that had been in existence since the evolution of compaction control. Significant studies by Ballard and Gardner (1) evaluated nuclear gage techniques, and minimization of inherent errors has been accomplished as reported by McDougall, Dunn, and Gardner (7). Nuclear gages can no longer be questioned as to validity.

With the acceptance of nuclear gages for the determination of soil density and moisture, it was only natural that the technique be extended to bituminous compaction control. Because the theory of the nuclear gage on soil is completely analogous to bituminous materials, with only the temperature of the material being different, the techniques are readily applicable to compaction control of bituminous concrete construction. The advantage of nondestructively testing the compaction of hot bituminous material in situ is obvious.

Previous methods of determining bituminous concrete density usually consisted of removing cores or slabs of the finished bituminous mat after completion of compactive effort. These samples were then removed to the laboratory and tested by volumeter methods to ascertain their density and compare this to preestablished minimum control figures of optimum compaction. Inherent problems were that the samples left areas of the pavement to be patched; expensive equipment and trained crews were required for obtaining the samples; and, most significantly, samples were taken after pavement rolling had been completed. Thus, a lack of compaction as may have been shown by testing these samples could not be corrected because the pavement was already cooled and additional rolling usually accomplished nothing. The final result,

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nonetheless, was a bituminous concrete base course or pavement that was neither properly nor uniformly compacted, which often resulted in pavement degradation or failure after several years.

An additional problem prior to the advent of nondestructive testing was in the compactive effort itself, that is, the optimum number of passes to be made by the rollers approved for the job of compacting the hot mix to its optimum density. The number of passes to be made by a particular roller was usually decided by the roller operator who had developed a "feel" for compaction through years of experience. Operator judgment may have been surprisingly good in many cases, but compaction nevertheless was a hit-and-miss proposition dependent on the experience and "feel" of the equipment operator.

Because nuclear gages can rapidly test the hot mix nondestructively after each pass of the roller in operation, it was the intent of this project to evaluate current compaction procedures as to uniformity and overall effectiveness on hot bituminous materials and to investigate the validity of compaction control by nuclear density gages while the job is in progress.

FIELD-TESTING PROCEDURE

The primary objectives of this project were an evaluation of existing compaction techniques and the development of testing techniques by nuclear gages to control bituminous concrete compaction while the job is in progress.

Previous reports have shown the validity of nuclear testing applied to bituminous construction. An excellent list of references pertaining to the use of nuclear testing devices on bituminous materials is included in a report by Metcalfe, Averitt, and Larue (2). A recent study by Worona (3) entailed the use of several gage designs on various thicknesses and grades of bituminous mixes over several base courses. The final analysis consisted of thousands of tests by nuclear gages compared to 1,200 cores and 300 slab tests. Results showed that nuclear tests were compatible with conventional tests for the materials tested.

For this investigation several nuclear gages were used to gather data. These gages had previously been used in the study mentioned earlier so that no new gage variables or designs were introduced.

Data were gathered over a period of 2 construction seasons from 70 projects. Each job was designated by a code, and information was obtained for classifying the location of the project; the test dates; the model, manufacturer, and weight of rollers and weight ballast with each; the paver model; the base course; the material used for base and wearing course; and the design density and asphalt content of the design mix. All data were recorded on computer format forms for direct conversion to punch cards for analysis by computer.

On any individual project, a 2-phase study of compaction was carried out. First, readings of bituminous density were taken after placement of material by the paver but prior to any roller compactive effort and after each individual pass of any roller used on the job. The speed of the paver and each roller used and the temperature at the beginning and end of each roller phase were recorded. Gages were properly standardized and calibrated at the start of each day prior to testing, and all final density readings were corrected for photoelectric absorption effect by the air-gap technique after final compaction (4). A detailed explanation of the theory governing the determination of density, moisture, and asphalt content by nuclear methods may be found in another report (1). No attempt was made to control compaction by use of the gages; rather, compaction was left to the discretion of the roller operator and inspector and to the nuclear gages used to observe results of the methods of compaction employed. Each gage was initially outlined with talcum powder on the bituminous concrete so that it was placed in the same spot after each roller pass to minimize surface roughness errors and lateral positioning errors. Also, during testing, each gage was placed parallel to the direction of paving. This placement technique alleviated problems of seating the gage properly during the rubber-tire rolling.

Throughout this report, a roller pass means one traverse over a given spot in one direction.

After all rollers had made the number of passes deemed necessary based on the material temperature and the judgment of the roller operator and inspector, the area was considered to have reached final compaction. The second phase of the compaction study using nuclear gages was then carried out. This phase entailed a detailed study of the uniformity of compaction and sought to locate any cyclical variations in density if these proved apparent. Density tests conducted during this phase consisted of the following:

Test	Location
Transverse	From right edge to left edge of pavement in 1-ft increments
Right edge	Along right edge of pavement, usually about 1 ft in from edge, at 2-ft increments longitudinally for at least 100 ft
Left edge	Along left edge of pavement, usually about 1 ft in from edge, at 2-ft increments longitudinally for at least 100 ft
Longitudinal right lane	In middle of right lane at 2-ft increments for at least 100 ft
Longitudinal left lane	In middle of left lane at 2-ft increments for at least 100 ft
Longitudinal center joint	Along longitudinal center joint at 2-ft increments for at least 100 ft
Random	At locations decided by table of random x- and y-coordinates. Coordinates were measured by a 100-ft tape, and tests were taken at each coordinate position. Fifty random tests were taken in a 500-ft section.

Data for 4 of the largest jobs encountered are given in Table 1. Many thousands of additional tests were taken on approximately 70 separate construction projects. These four are representative of the whole in compaction trends and variation. Each reading given in Table 1 is an average of approximately 200 individual readings at selected stations.

Although overall tests were very near or above compaction requirements, many of the individuai tests fell far below an acceptable density figure. This appeared much more prevalent on the longitudinal center joint and on the left and right edges of the pavement where, it seems, roller compactive effort is neglected most as was visually noted in the many projects observed.

Poorest compaction was realized along the edges of the pavement. In many cases it was noticed that only one pass was made with each roller in these areas. Samples taken

TABLE 1

COMPACTION FOR FOUR REPRESENTATIVE JOBS

Test	Density (pcf)				Standard Deviation (pcf)				Compaction (percent)			
	Job 1	Job 2	Job 3	Job 4	Job 1	Job 2	Job 3	Job 4	Job 1	Job 2	Job 3	Job 4
Transverse	136.8	138.4	149.3	141.0	5.0	5.3	3.9	3.6	100.3	97.0	98.5	94.4
Longitudinal center joint	130.4	141.6	148.8	140.5	4.4	4.7	4.1	2.0	95.6	99.2	98.2	94.4
Left edge	129.1	129.0	144.2	137.2	3.1	3.7	2.9	3.2	94.6	90.4	95.1	91.8
Right edge	134.8	133.9	144.8	137.2	2.7	2.9	2.9	3.1	98.8	93.8	95.5	91.8
Longitudinal left lane	135.5	141.3	151.3	141.7	3.6	4.5	3.1	3.0	99.3	99.0	99.8	94.8
Longitudinal right lane	141.0	141.7	148.5	144.4	3.7	4.2	4.2	3.5	103.4	98.9	98.0	96.6
Random	138.2	138.3	149.0	141.2	5.5	5.3	4.2	4.2	101.3	96.9	98.3	94.5

Notes: Material for jobs 1, 2, and 3—ID2 wearing, 2 in. thick over ID2 binder; for job 4—bituminous concrete base course, 2 in. thick over base course. Marshall design density for job 1—136.4 pcf, 95 percent or 129.6 pcf required; for job 2—142.7 pcf, 95 percent or 135.6 pcf required; for job 3—151.6 pcf, 95 percent or 144.0 pcf required; for job 4—149.4 pcf, 90 percent or 134.5 pcf required.

TABLE 2

ROLLING PATTERNS

Job	Station	Breakdown Roller		Rubber-Tire Roller		Finish Roller		Temperature of Mat (deg F)		Final Density (lb/ft^3)
		Passes	Speed (mph)	Passes	Speed (mph)	Passes	Speed (mph)	Behind Paver	After Compative Effort	
1	035008	6	2.3	5	5.8	2	4.9	277	169	136.6
	035009	3	1.8	3	4.3	7	5.4	294	188	140.1
	103008	2	1.8	2	5.4	1	5.5	253	154	136.7
	103009	1	3.0	6	4.1	3	4.3	291	144	134.1
	121008	2	2.6	4	5.5	5	3.9	288	136	142.0
	121009	3	1.9	6	4.6	1	2.8	267	122	135.3
2	171008	2	1.0	4	5.5	2	1.4	260	180	151.0
	084209	2	4.0	3	6.0	—	—	290	110	144.0
	171009	3	3.6	5	4.5	3	3.9	285	191	152.2
	086108	1	2.2	4	5.5	5	4.0	272	170	147.3
	077308	3	3.0	3	3.9	6	3.5	254	168	141.8
	077309	5	4.1	—	—	1	5.1	293	132	142.8
	093408	2	2.6	5	5.3	5	4.8	287	148	151.4

as cores or slabs for compaction acceptance, however, are usually taken from the wheelpath area of the pavement (longitudinal left or right lane), which would indicate acceptable densities. This, of course, is not representative of actual compaction near the pavement edges. With nuclear testing, however, many more tests may be taken while a project is still progressing to indicate areas of low density so that the condition may be rectified by additional rolling.

As mentioned previously, no attempt was made to control the rolling operations on any of the projects. Data were taken, however, for densities versus the number of passes of each roller, including the speed of the roller and temperature of the mix at the time of each pass.

Data are given in Table 2 for two of the projects described earlier. Each set of data was taken at a different station on the pavement. The last digit of each station number, 8 or 9, indicates the right or left lane respectively. There was little, if any, methodology to rolling patterns. Speeds remained somewhat constant for a given roller, and the limited speed ranges are too narrow to attempt any analysis of roller speed versus compactive effort.

In many cases, there was a definite time lag before rolling operations proceeded after placement of the uncompacted mat and an even greater time gap between different rollers. During most of the jobs encountered, the ambient temperature was in the 70 to 90 F range. With cooler ambient temperatures such as the 40 F required minimum, such time lags would undoubtedly hamper compactive effort because rapid material temperature drop between successive rollers.

Because of the large variation in actual passes by each roller, it would be extremely difficult to set forth suggestions as to optimum number of passes for each roller. Also, each individual type of material requires a different roller pattern for optimum density. The only way to set forth optimum rolling patterns is to establish them at the beginning of each job on a given bituminous material. As long as the material remains resonably constant, the pattern set forth at the beginning should yield optimum compaction for the entire project.

However, decompaction can occur because of overrolling the bituminous mat. This occurs when too many passes are made, particularly with the breakdown roller, and the material is pushed out from under the roller, resulting in a thin mat of widely separated aggregate. Thus, care must be taken not only to establish an optimum minimum number of passes but also to see that decompaction does not result from too many roller passes. Either extreme is undesirable but may be excluded by taking nuclear density tests after each roller pass.

It appears further that much closer control is required so that rolling begins immediately after placement of the bituminous mat and progresses so that large time lags do not occur between start and finish of rolling over a given area as indicated by the rather large temperature differentials given in Table 2.

Wearing courses offering good gage seating predominated throughout the project because the variable intended to be studied was compaction as a function of compactive effort. Approximately 15 jobs were, however, coarse-graded mixes ranging from binder materials to bituminous concrete base courses. These usually had 100 percent passing the 1½- or 2-in. sieve compared with 75 to 100 percent passing the ⅜-in. sieve for the fine-graded wearing courses.

No serious problems of seating the nuclear gages on the course-graded bituminous mixes occurred as originally expected because of the open-surface texture. When the hot material was placed, the gage was easily seated. The soft nature of the material resulted in intimate gage-surface contact.

DEPTH OF PENETRATION STUDY

The depth of penetration of the nuclear gages was investigated at this time. Manufacturers of gages designed specifically for use on bituminous materials had usually claimed that the devices considered only the top 2½ in. of material, which is about the maximum to be expected in wearing course construction of bituminous pavement.

Tests were performed by inverting the nuclear gages—a nuclear Chicago asphalt density gage, model 5846, and a Troxler Electronics, Inc., asphalt density gage, model A-240F—and placing a 4-sided wooden box, which form-fitted the gage bottom area, directly over the gage bottom. This allowed material to be poured into the box so that the gage effectively "saw" only the material admitted to the box. Glass beads normally used for the reflectorization of traffic paint were then poured into the molds at ½-in. increments, and counts were taken after each successive layer. The glass beads had a density of approximately 98 lb/ft^3. Although this is much less than will be encountered in bituminous construction, uniformity of the sample was excellent and provided a clean material very easy to handle.

Both gages experienced a maximum count differential at 1½ in. (count at some depth minus count at immediately preceding depth), and both further exhibited 50 percent of final count achieved at very nearly 1 in. This is about ½ in. less than studies on soil density-moisture gages as shown previously by Weber (5).

On this low-density material, both gages further exhibited that 90 percent of the final count was due to approximately the first 3 in. of material. Use of a material of greater density, more closely approximating bituminous pavement material, would probably have decreased this depth to the 2½-in. range that was claimed by the manufacturers. Because most wearing courses are placed over a binder of similar properties and density as the top course, the sphere of influence of the asphalt density gages tested appeared satisfactory. If a thinner wearing surface is placed over a more dense base course such as concrete, however, allowances will have to be made for the scattering of gammas by the dense base resulting in erroneous density readings if the gage is used only in the Compton backscatter surface mode.

A "worst case" was included in the study to check the mechanics of such an occurrence and to examine the effect of an exaggeratedly dense base course. Glass beads were again placed in ½-in. increments, but now a ½-in. thick block of aluminum, approximately 170 lb/ft^3, was placed on top of each successive layer of beads. In this case, 62 percent of the final count achieved was due to the top 1-in. layer of beads compared to 50 percent found previously without the aluminum base. Also, 96 percent of the final count was due to 3 in. of beads now, rather than 90 percent found previously. A definite effect was produced by a heavier base material, even under 3 in. of material.

Thus, care must be taken to ensure that corrections are applied if thin wearing surfaces are tested over dense base courses by using only surface count readings. If one looks at only the minimum count method proposed by Virginia (6), this correction is unnecessary because the thickness of wearing course and the density of the base are constant.

Because all final readings were corrected for photoelectric absorption by the air-gap method, tests on depth of penetration were also conducted by incorporating an air-gap between the gages and the box. Material was then placed as before in ½-in. increments, and counts were taken at each depth interval. The air-gap space induced was that recommended by the manufacturer. Similar results were obtained for both gages. Inducing an air-gap resulted in 50 percent of the final count being due to approximately the first $\frac{1}{2}$ in. of material, while 90 percent of the final count was due to the first $1\frac{3}{4}$ in. of material. These depths would probably drop with the use of a more dense material. The finding is particularly significant because the inclusion of an air-gap test to obtain the final densities is now more heavily dependent on the wearing course and practically independent of the base course density for surface materials more than $1\frac{1}{2}$ in. thick.

CONCLUSIONS

Although only 4 major bituminous construction projects are listed in this report, similar data were taken on approximately 70 projects. The results listed are indicative of the majority of findings on all projects. Based on the results of this study, the following conclusions can be stated.

1. Present methods of compaction result in a lack of uniformity because rolling patterns are usually left to the discretion of the roller operators.
2. Nuclear devices can readily establish optimum rolling patterns while the job is in progress, and tests may be made at any time to check compaction.
3. Densities at edges are often lower than those for the rest of the roadway, indicating poor compaction control at edges.
4. Very large time lags are often prevalent between the times rolling begins and ends and between successive rollers. In many instances, rollers continued to attempt compaction when the material was so cool that any number of additional passes did not increase compaction. Greater control is necessary to see that rollers advance so that optimum compaction occurs.
5. Nuclear tests point out that there is a trend in transverse compaction.
6. No obvious cyclical trends appeared in any of the longitudinal density studies.
7. For the several different nuclear gages used in the study, the high temperatures had no effect on results, and no major gage malfunctions occurred so long as the batteries in the systems were maintained at proper charge level and the bottom surface kept clean.
8. From the similarity of standard deviations of density results as found by the nuclear gages, the method appears satisfactorily reliable and repeatable for compaction control on bituminous concrete construction projects.
9. Because the bituminous materials, when hot, allowed very good nuclear gage-material surface contact, no serious surface texture errors were prevalent in the use of nuclear gages on hot mixes of widely different gradations.
10. From the results of the compactive effort studies, it would appear that the best method of compaction control on bituminous construction is to establish optimum rolling patterns by the use of nuclear gages, to maintain this compactive effort for a given project with close inspection to see that the pattern is followed on all areas of the roadway at proper temperatures, and to take periodic tests of compaction results while the project is in progress to ensure that optimum compaction is being achieved.
11. Inclusion of an air-gap test to compensate for chemical effects results in density readings dependent more on the wearing surface and minimizes results due to backscatter from dense base courses. At wearing course depths greater than $1\frac{1}{2}$ in., corrections do not have to be applied for dense base effect.

ACKNOWLEDGMENTS

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EVALUATION OF GAP-GRADED ASPHALT CONCRETE MIXTURES

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Because of the increasing demand for high-quality and more durable paving mixture for modern traffic, the increasing costs for producing maximum density or well-graded aggregates in many parts of the country (especially near urban areas), and some possible advantages gap-graded asphalt paving mixtures may offer gap-graded aggregates in both portland cement and asphalt concretes have drawn some attention throughout the world. In this paper experimental results are reported on a comparative study involving 3 Fuller's curve gradings, 8 gap gradings, 2 aggregates, and 90 mixtures with varying asphalt contents. The physical properties of the mixtures were evaluated in terms of both Marshall design and Hveem design methods. Results have shown that gap-graded aggregates can produce mixtures with physical properties equal to or better than continuously graded aggregates at usually higher optimum asphalt contents.

•BITUMINOUS paving engineers generally agree that gradation of the aggregate in a paving mixture is one of the factors that must be carefully considered in a mixture design because it affects, directly or indirectly, the stability, durability, skid resistance, and economy of the finished pavement. Virtually all high-quality asphalt concrete used in the United States now contains a densely graded aggregate. However, there are differences of opinion in various localities as to what constitutes the "ideal" gradation for the densely graded aggregate.

Examination of the gradation requirements of specifications used by various state highway departments and other agencies in the United States, Canada, and some European countries reveals that, with few exceptions such as British Standard 594, they are approximate to the Fuller's maximum density curves (1, 2). It can also be observed that (a) specifications on aggregate gradation differ greatly, and tolerance of gradation limits vary widely; (b) under certain sets of conditions, a number of gradations can produce satisfactory paving mixtures; and (c) present knowledge on aggregate gradation, when coupled with economic considerations, may not justify the application of narrow gradation limits.

Of special significance is the fact that there are also reported experiences (3) where successful paving mixtures were associated with the most unconventional and irregular grading curves, and failures identified with gradings complied nicely with the ideal maximum density curves such as those presented by Fuller.

The demand is increasing for high-quality and more durable paving mixture for modern traffic. Costs for producing maximum-density or well-graded aggregates are also increasing in many parts of the country because of rapid depletion of natural deposits that meet the specifications of continuous grading and because of increased costs of labor, transportation, and processing. It, therefore, appears desirable to examine other types of aggregate gradings as compared with generally required well-graded maximum density gradations so that more efficient use can be made of locally available

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aggregates. One of these gradings that has been used with success is a gap-graded mixture (2, 3, 4, 5).

Grading of aggregates to conform to a curve of maximum density, developed by Fuller and Thompson (6) and later modified and confirmed by a number of others, is generally accepted as the most desirable grading for the production of good, economical portland cement and asphalt concretes. These gradings are also referred to as well-graded or continuously graded and are generally expressed by the relationship $P = 100 (d/D)^{1/2}$ where P is the percentage by weight of the aggregate passing or finer than the sieve d, and D is the maximum size of the aggregate. On the other hand, an aggregate is said to be gap-graded (or skip or discontinuously graded) when certain particle sizes in the grading of aggregate are missing. The absence of certain particle sizes can be achieved either by using natural aggregates as they are obtained or by deliberately omitting them to obtain certain desired properties of the mixture.

Various investigators (7, 8, 9, 10, 11) have suggested the following possible advantages of gap-graded asphalt concrete mixtures over continuously graded aggregate mixtures: (a) It may be more economical to produce in some localities; (b) it may allow more asphalt to be used in the mixture, and thus, produce thicker asphalt films and more durable paving mixture; (c) it may have better flexibility, higher fatigue life, and higher value of strain at failure because of higher content of low penetration asphalt; (d) it is more skid resistant; and (e) it may tolerate more asphalt content variations.

On the other hand, the continuous grading has been criticized for at least 3 disadvantages that deserve reexamination. Some countries, such as Japan, that traditionally specify continuous grading for their high-quality asphalt mixtures have already been studying the feasibility of using gap-graded mixtures (5). The major disadvantages of well-graded mixtures are the following: (a) They are more expensive to produce, especially for some states where suitable aggregates sources are depleting and where narrow limits are specified; (b) they are more sensitive to variation of asphalt content change, leading to disintegration or slipperiness (12); and (c) they are difficult to handle and tend to segregate (7).

A large amount of literature, especially theoretical, can be found on the packing of aggregate particles and maximum density or minimum porosity gradings, including the classic work on concrete proportioning by Fuller and Thompson and more recent work on dense asphaltic mixtures by Lees (13). There is also abundant published information on gap-graded concretes as compared to the corresponding continuously graded concretes (14, 15, 16). However, reported data on gap-graded asphalt concrete mixtures are few and scattered.

The purpose of this investigation was to make a comparative study of the gap-graded and the well-graded asphalt concrete mixtures in terms of Marshall and Hveem design methods.

MATERIALS AND PROCEDURES

Eleven aggregate gradations involving 3 maximum sizes, 2 aggregate types, 1 asphalt cement, and a range of asphalt contents were studied.

Four aggregate gradings were examined for ¾ in. maximum size, including one grading following Fuller's maximum density curve and 3 gradings following Fuller's curve but with gaps between ½ in. and No. 4 sieves, between No. 4 and No. 8 sieves, and between No. 8 and No. 30 sieves (Fig. 1). Four aggregate gradings were examined for ½ in. maximum size, including 1 Fuller's curve grading, 2 gradings with gaps between No. 4 and No. 8 sieves and between No. 8 and No. 30 sieves, and 1

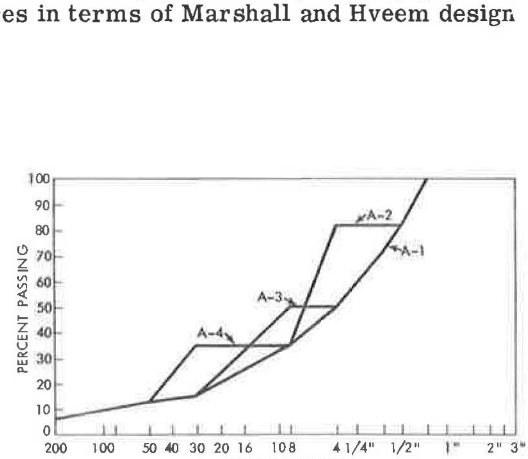

Figure 1. Grading curves for ¾-in. maximum size aggregates.

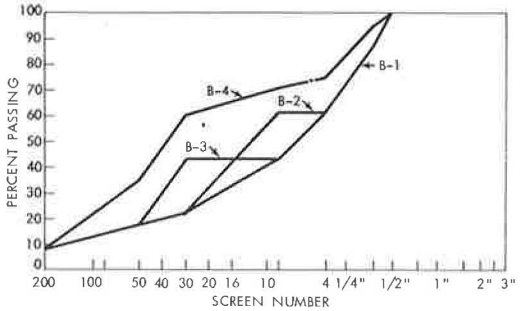

Figure 2. Grading curves for $^1\!/_2$-in. maximum size aggregates.

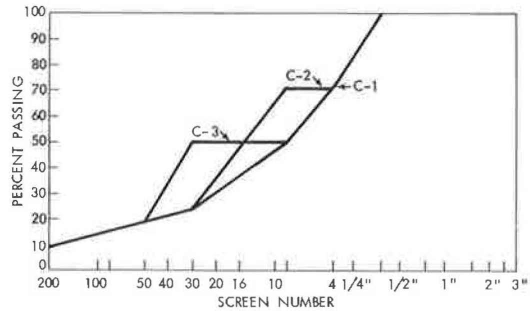

Figure 3. Grading curves for $^3\!/_8$-in. maximum size aggregates.

TABLE 1

PROPERTIES OF CRUSHED LIMESTONE AGGREGATE

Aggregate	Specific Gravity		Water Absorption (percent)
	Apparent	Bulk	
Menlo, Adair County			
Coarse	2.666	2.542	2.55
Fine	2.654	2.489	2.61
Alden, Hardin County			
Coarse	2.645	2.508	2.91
Fine	2.628	2.514	3.10

TABLE 2

PROPERTIES OF 120 TO 150 PENETRATION ASPHALT CEMENT

Property	Value
Penetration, 77/100/5, dmm	138
Specific gravity	1.023
Flash point, COC, deg F	605
Fire point, COC, deg F	685
Softening point, R and B, deg F	112.0
Viscosity	
140 F, poise	687
77 F, megapoise	0.52
Solubility in CCl_4, percent	99.87
Asphaltenes, percent[a]	15.4

Note: Data supplied by American Oil Company, Sugar Creek, Mo.
[a]Insoluble in 86 to 88 degree petroleum naptha.

grading corresponding to the British Standard 594 (2, 8) (Fig. 2). Three aggregate gradings were examined for $^3\!/_8$ in. maximum size, including 1 Fuller's curve grading, 1 grading with gap between No. 4 and No. 8 sieves, and 1 grading with gap between No. 8 and No. 30 sieves (Fig. 3).

Two crushed limestone aggregates and one 120 to 150 penetration asphalt cement were used in the investigation. The properties of the aggregates and asphalt are given in Tables 1 and 2 respectively.

Crushed aggregates were first separated by $^3\!/_4$ in., $^1\!/_2$ in., $^3\!/_8$ in., No. 4, No. 8, No. 30, No. 50, and No. 200 sieves. Required weights of each fraction were then combined to produce gradation curves shown in Figures 1 to 3. Twenty-pound batches of asphalt concrete mixtures were made in a 50-lb laboratory pug-mill mixer at asphalt contents from 4 to 9 percent. From each batch of mix, 3 Marshall specimens and 3 Hveem specimens were compacted following standard procedures (17). Maximum theoretical specific gravities of mixtures were determined on duplicate samples by Rice's method using aerosol solution. Bulk density of compacted specimens, Marshall stability and flow, and Hveem stability and cohesiometer values were determined by procedures recommended by The Asphalt Institute.

RESULTS AND DISCUSSION

Marshall Specimens Made With Menlo Aggregates

Results of Marshall specimens made with Menlo aggregates are shown in Figures 4, 5, and 6. Regardless of the maximum aggregate size and where the gap is in the grading, gap-graded mixtures behaved the same way as did the Fuller's curve mixtures.

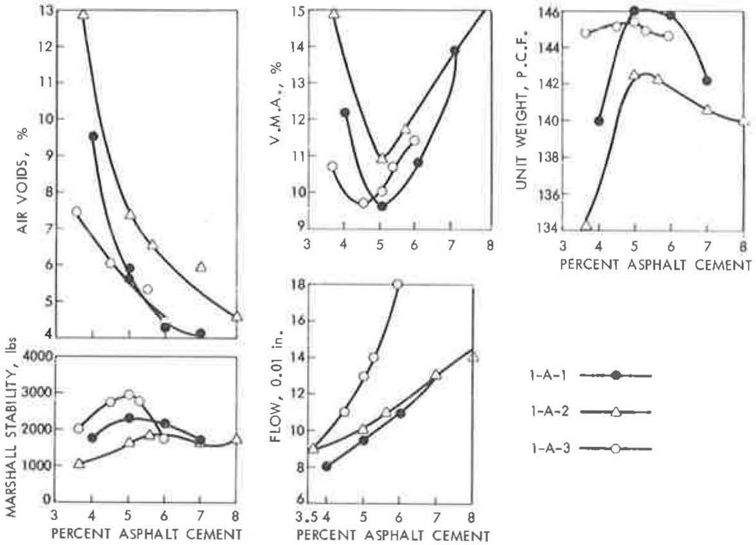

Figure 4. Mixture property curves by Marshall method, $^3/_4$-in. Menlo aggregates.

In all cases, Fuller grading produced mixtures of higher density, the differences between maximum densities being in the order of 1 to 3 pcf. The higher maximum density of Fuller gradings was also reflected in the percentage of air voids; i.e., they produced mixtures of lower voids at respective optimum asphalt contents. However, some of the gap-graded aggregates resulted in mixtures of higher maximum stability (1-A-3 and 1-B-4). Because gap gradings produced mixtures of higher VMA at equivalent asphalt content and stability was very much more than the design criteria of 500 to 750 lb for all mixtures, it was possible to arrive at optimum asphalt contents to meet all design

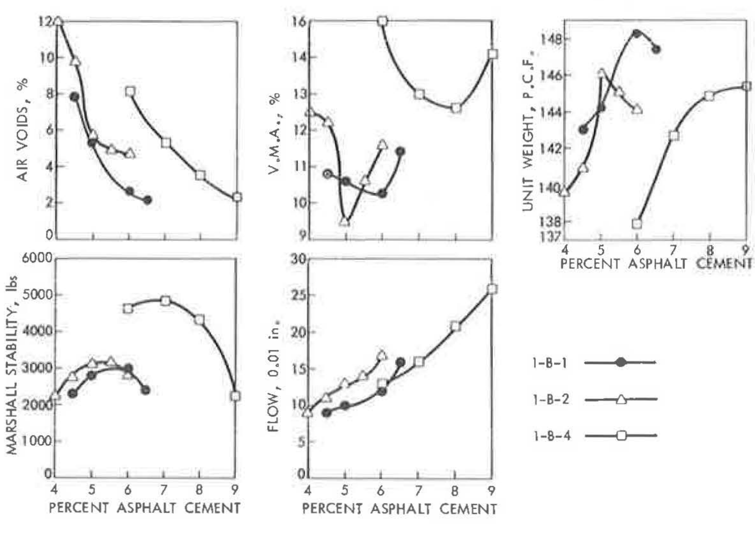

Figure 5. Mixture property curves by Marshall method, $^1/_2$-in. Menlo aggregates.

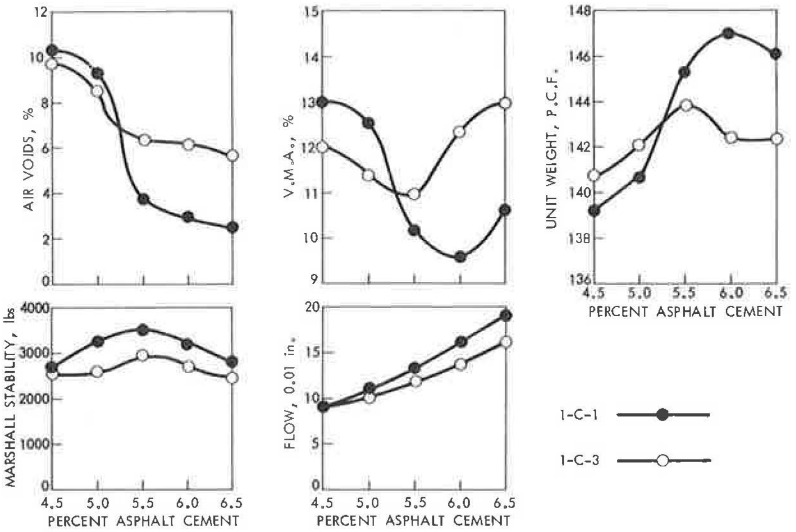

Figure 6. Mixture property curves by Marshall method, $^3/_8$-in. Menlo aggregates.

requirements for gap-graded mixtures. The optimum asphalt contents for gap-graded mixtures were in most cases higher than those for equivalent continuously graded mixtures. In any case, the unnecessary requirement for aggregates to meet the Fuller grading and the advantage of being able to use more binder in gap-graded mixtures (such as in 1-A-2 and 1-B-4) are obvious. Table 3 gives the optimum asphalt contents for all gradings and evaluation of design properties at optimum asphalt contents. Evalua-

TABLE 3

OPTIMUM ASPHALT CONTENT AND PROPERTY EVALUATION BY MARSHALL AND HVEEM METHODS

Mix	Aggregate	Marshall Method					Hveem Method			
		Optimum Asphalt Content (percent)	VMA	Stability	Flow	Voids (percent)	Optimum Asphalt Content (percent)	Stability	Cohesiometer	Voids (percent)
1-A-1	Menlo	5.7	10.3[a]	2,350	11	4.7	5.6	60	250	4.0
1-A-2	Menlo	7.0	13.9	1,600	13	5.3[b]	5.5	52	300	4.0
1-A-3	Menlo	5.7	11.1[a]	2,200	16	4.8	3.8	56	265	4.0
1-B-1	Menlo	5.4	10.5[a]	2,900	10	3.9	4.6	50	228	4.0
1-B-2	Menlo	5.8	11.1[a]	3,000	15	4.7	4.6	64	292	4.0
1-B-4	Menlo	7.5	12.7[a]	4,700	18	4.2	6.0	56	218	4.0
1-C-1	Menlo	5.7	9.7[a]	3,460	14	3.4	5.1	50	236	4.0
1-C-3	Menlo	5.9	12.0[a]	2,800	13	6.0[b]	5.4	55	248	4.0
2-A-1	Alden	5.2	10.6[a]	3,480	12	4.6	4.4	63	415	4.0
2-A-2	Alden	5.3	13.7[a]	4,060	15	6.8[b]	5.5	42	485	5.0
2-A-3	Alden	5.7	14.5	2,980	15	6.6[b]	5.3	53	510	4.0
2-A-4	Alden	5.1	10.0[a]	3,540	9	4.8	4.8	69	325	4.0
2-B-1	Alden	5.2	10.4[a]	4,650	13	4.0	4.9	65	395	4.0
2-B-2	Alden	5.7	12.4[a]	3,800	14	4.0	5.5	60	555	4.0
2-B-3	Alden	6.9	17.0	3,150	10	4.2	6.5	66	265	4.0
2-B-4	Alden	7.5	15.0	3,200	14	5.0	6.0	70	300	4.0
2-C-1	Alden	6.5	18.0	2,900	14	7.5[b]	6.8	35	350	7.0[a]
2-C-2	Alden	6.7	16.6	3,730	11	6.2[b]	7.0	50	345	4.0
2-C-3	Alden	6.8	17.8	2,940	16	4.5	6.2	50	360	4.0

[a]Low. [b]High.

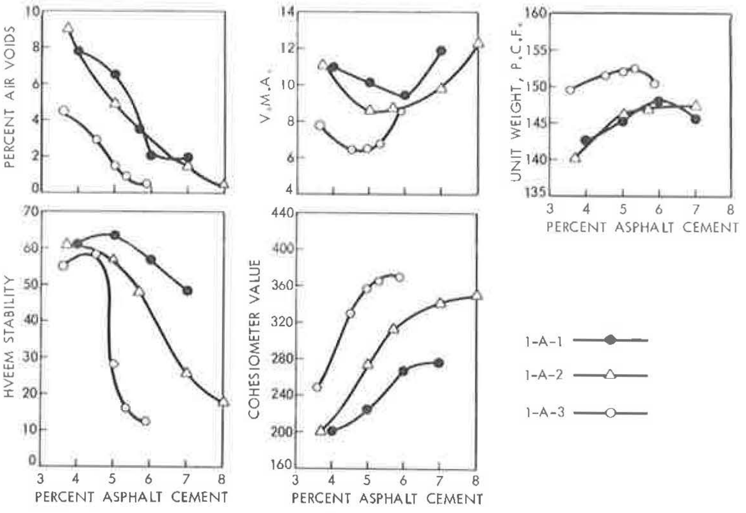

Figure 7. Mixture property curves by Hveem method, ¾-in. Menlo aggregates.

tion was based on The Asphalt Institute criteria for medium traffic by the Marshall method (17). In nearly all cases, the optimum asphalt contents for gap-graded mixes were higher than those for Fuller gradings and possessed properties equal to or better than the Fuller grading mixtures.

Hveem Specimens Made With Menlo Aggregates

Property curves for Hveem specimens made with Menlo aggregates are shown in Figures 7, 8, and 9. The optimum asphalt contents, determined by criteria recommended by The Asphalt Institute for medium traffic (17), and property evaluation at optimum asphalt contents are given in Table 3. The same statements made for Mar-

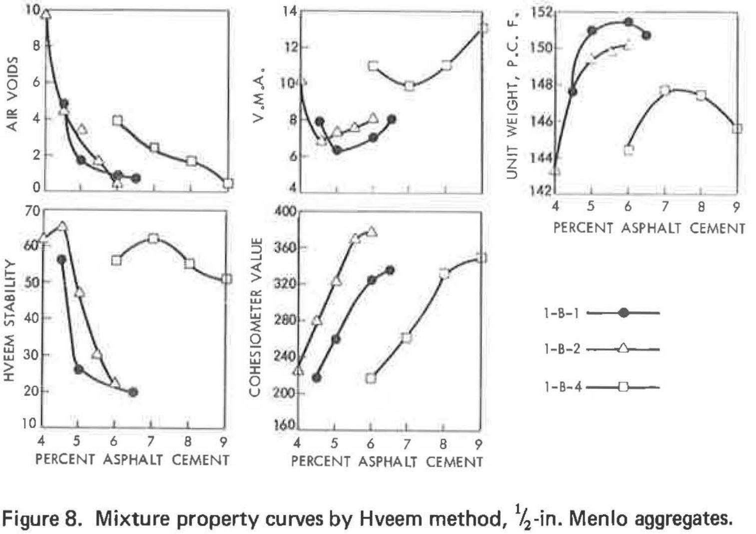

Figure 8. Mixture property curves by Hveem method, ½-in. Menlo aggregates.

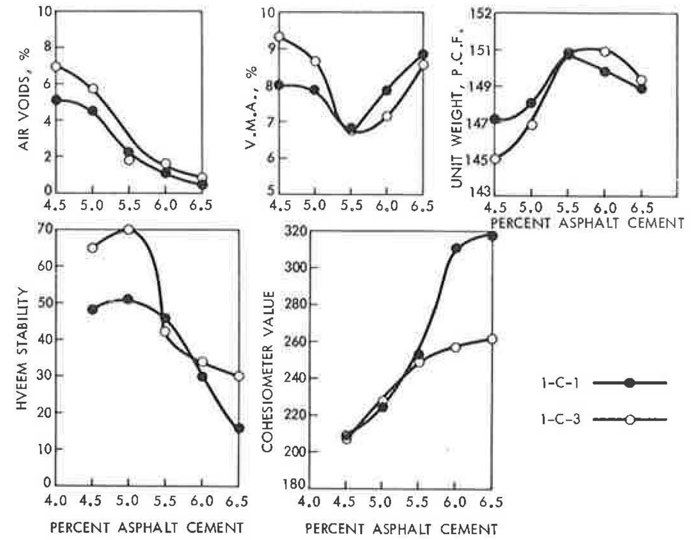

Figure 9. Mixture property curves by Hveem method, $^3/_8$-in. Menlo aggregates.

shall specimens can be made here; that is, mixtures can be made with gap-graded aggregates that are equivalent to or better than those made with Fuller or continuously graded aggregates, except that gap-graded mixtures: may tolerate more asphalt, can be compacted into equal or higher density (1-A-3 and 1-C-3), and may have higher cohesiometer values or tensile strengths and maximum stability values.

Marshall Specimens Made With Alden Aggregates

Figures 10, 11, and 12 show property curves for Marshall specimens made with Alden aggregates. The optimum asphalt contents, based on the criteria for medium

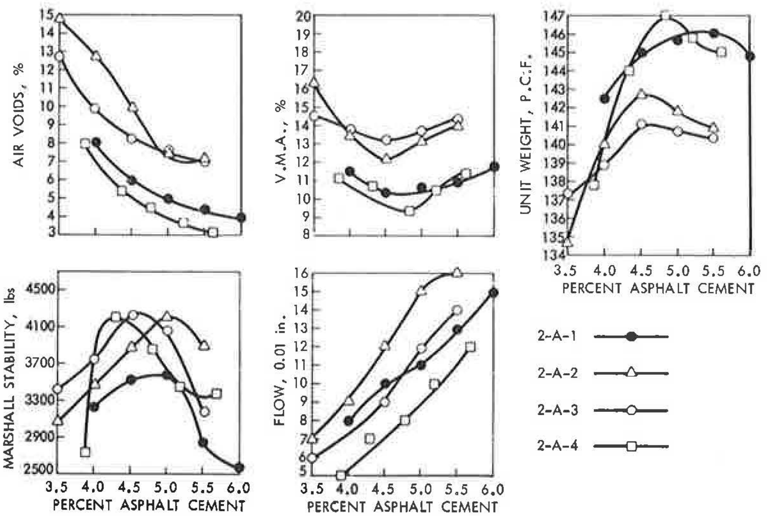

Figure 10. Mixture property curves by Marshall method, $^3/_4$-in. Alden aggregates.

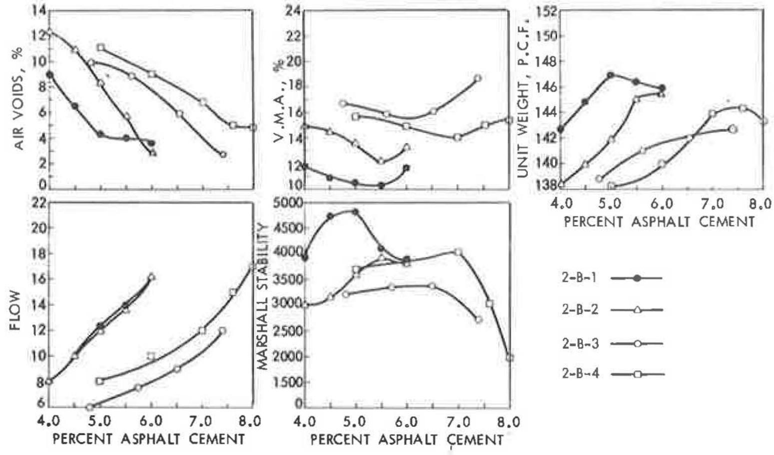

Figure 11. Mixture property curves by Marshall method, ½-in. Alden aggregates.

traffic, were estimated and are given in Table 3. It can be readily seen that all mixtures, continuously graded as well as gap-graded, at their optimum asphalt content satisfied stability and flow criteria and that, with no exception, the optimum asphalt contents for gap-graded mixtures were at higher values than those for the Fuller gradings.

It should be noted that the VMA and air void values were calculated from first principles (17). Because of the high absorption and low bulk specific gravity obtained for the aggregates, calculated volumes of aggregate in specimens could be larger than true volumes, resulting in smaller VMA values. When VMA was calculated as the sum of effective asphalt volume and air voids in a specimen, its value would be somewhat higher, depending on the accuracy of the asphalt absorption determination.

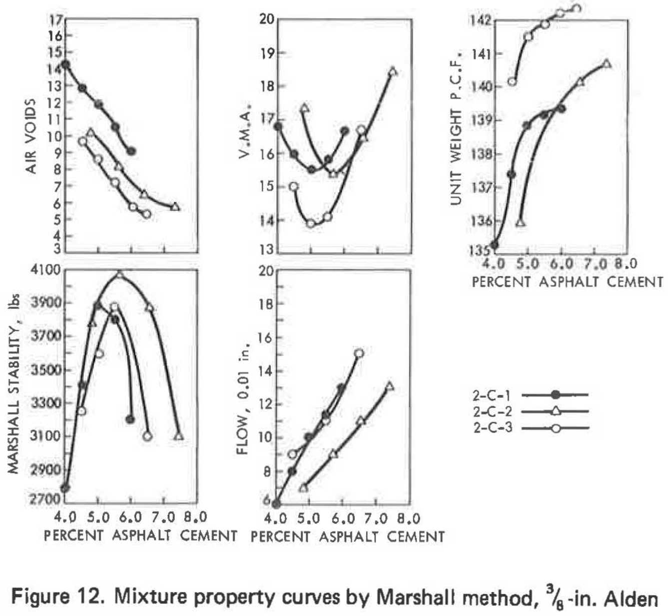

Figure 12. Mixture property curves by Marshall method, ⅜-in. Alden aggregates.

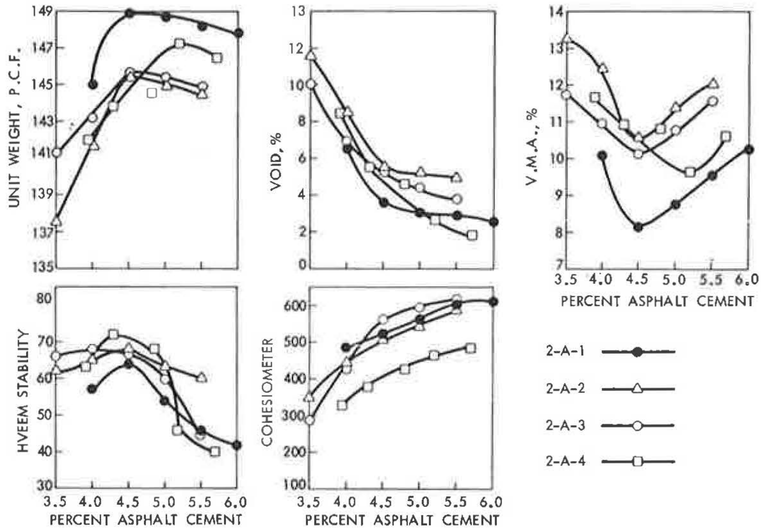

Figure 13. Mixture property curves by Hveem method, ¾-in. Alden aggregates.

Hveem Specimens Made With Alden Aggregates

Property curves for Hveem specimens made with Alden aggregates are shown in Figures 13, 14 and 15. The estimated optimum asphalt contents for medium traffic and property evaluation at optimum asphalt content are given in Table 3. It can be observed again that all mixtures, both continuously graded and gap-graded, meet design criteria for stability, cohesion, and air voids at optimum asphalt contents and that nearly all gap-graded mixtures could accommodate more asphalt than continuously graded aggregates.

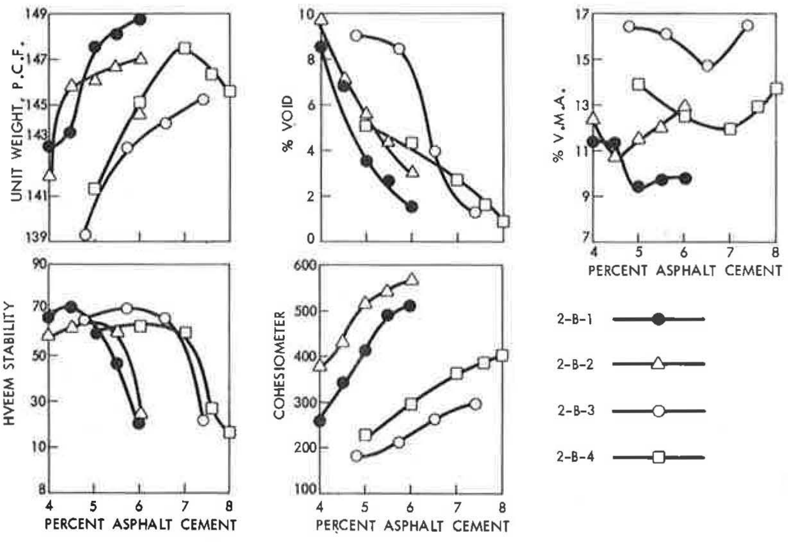

Figure 14. Mixture property curves by Hveem method, ½-in. Alden aggregates.

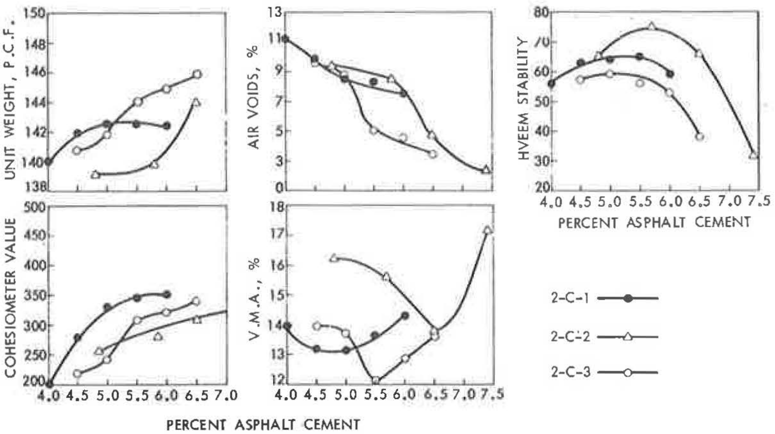

Figure 15. Mixture property curves by Hveem method, $^3/_8$-in. Alden aggregates.

When Hveem specimens of all aggregates, all gradings, and all asphalt contents are compared, the following can be noted: (a) Fuller gradings generally yielded the densest compacted mixtures with lowest voids, except that for $^3/_8$-in. mixtures, both Menlo and Alden aggregates, gradings with No. 8 to No. 30 (0.094 in. to 0.023 in.) gap resulted in highest densities; (b) gradings that produced mixtures of highest density did not necessarily produce mixtures of highest stability or cohesiometer values; and (c) for $^3/_4$-in. and $^1/_2$-in. mixtures, gradings with gap between No. 4 and No. 8 (0.187 in. and 0.094 in.) sieves seemed to give high maximum stability and cohesiometer values. The $^3/_8$-in. aggregates with gap in between No. 8 and No. 30 sieves seemed to give the best mixtures for stability and cohesion.

When all Marshall specimens are compared, the following can be noted: (a) Fuller's gradings gave the densest compacted mixture in most cases and (b) either Fuller's gradings or gradings with gap between No. 4 and No. 8 sieves could produce mixtures of highest maximum stability.

Although outside the scope of this investigation, it was noted with regard to the selection of optimum asphalt content that (a) for certain aggregate types and gradation, the optimum asphalt content determined by Hveem method may not necessarily be the same as determined by Marshall method and (b) with the same gradation and method of design, the optimum asphalt content can be quite different for different aggregate types.

CONCLUSIONS

Ninety asphalt concrete mixtures involving 8 gap gradings and 3 Fuller's maximum density gradings were evaluated in terms of the Marshall and Hveem design methods. The general conclusions that can be drawn within the premises of this investigation are as follows:

1. Mixtures can be designed by either the Marshall or the Hveem method for all aggregates, both continuously graded and gap-graded, to meet recommended design criteria for all relevant properties.
2. Although in most cases the Fuller grading yielded mixtures of highest density, gap-graded mixtures often resulted in better stability or cohesion.
3. With almost no exception, gap-graded mixtures had higher optimum asphalt content than equivalent Fuller graded mixtures.
4. At least for the aggregates studied, rigid requirements for the aggregate to meet Fuller's grading or stringent gradation tolerance control, especially involving additional cost of processing and transportation, may not be justified.

5. The results presented here, though they have shown some of the advantages of gap-graded over continuously graded asphalt concretes, are by no means exhaustive. Additional systematic studies of gap-graded asphalt mixtures are needed to include more aggregate types, asphalt grades, and mechanical and durability properties. Such investigation may result in wider use of gap grading in asphalt aggregates and more economical asphalt paving mixtures in certain areas.

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EXPERIMENTAL STUDIES ON VISCOSITY OF ASPHALT CEMENTS AT 77 F

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This paper provides the background for the direct application of capillary rheometry to asphalt cements. It is this background that has lead to the development of a practical rapid method for evaluating the apparent consistency in poises and the shear susceptibility of asphalt cements at 77 F and other ambient temperatures. The apparatus and studies employed in the original experimental work are presented. The theoretical complications in capillary measurements and their effect on the results for asphalt cements are discussed. Comments on the magnitude of the effects and measures to allow for them are considered. Experimental data on a number of asphalts from different sources are shown. It is concluded that the capillary type of apparatus is feasible and merits consideration by asphalt technologists as a first satisfactory answer to the problem of measuring asphalt viscosity at service temperatures.

•A MAJOR problem in asphalt technology is an evaluation of the apparent viscosity and shear susceptibility of asphalt cements at 25 C (77 F). These properties are important in highway construction and service of asphalts. The objective of this paper is to describe experiments with a rheological evaluation technique for asphalt cements at ambient temperatures. This technique is different from those previously proposed or currently in use.

Numerous procedures (1) are described in the literature, and the ASTM has proposed several methods (2) for measuring the consistency of asphalt cements in the range of high consistency. The cone and plate method and the sliding plate microviscometer are utilized to a considerable extent for research studies. The latter can be used for routine work and is useful for small samples and aging studies. However, this apparatus has some limitations (3).

Attention in this laboratory was directed to tubular rheometry application for asphalt that was investigated earlier by Traxler and Schweyer (4). Pezzin (5) has discussed capillary rheometry extensively where it is applied to elastomers and polymers at high temperatures to study their flow behavior.

The capillary rheometer is adaptable for asphalt consistency measurements over wide temperature and consistency ranges. It consists of a simple barrel and piston arrangement with a capillary through which the asphalt is extruded. The geometry may be varied to accommodate different consistencies, and suitable air temperature control equipment can be employed.

THEORY

The deformation of asphalts under stress may be considered to follow the classical power law concept for relating flow parameters.

$$\tau = \eta \dot{\gamma}^{C} \tag{1}$$

Sponsored by Committee on Characteristics of Bituminous Materials and presented at the 50th Annual Meeting.

where

τ = shearing stress at the wall, dynes/cm^2;
$\dot{\gamma}$ = average rate of shear, sec^{-1};
C = power function parameter;
η = correlation coefficient [when C has a value of unity (Newtonian fluid), η is the viscosity in poises].

For non-Newtonian flow, the average rate of shear as normally computed must be changed by the Rabinowitsch correction to give the proper value, $\dot{\gamma}_W$, at the wall in capillary flow as noted by Pezzin (5). The apparent viscosity, η_a, is then computed as follows at a selected point:

$$\eta_a = \tau/\dot{\gamma}_W \text{ poises} \tag{2}$$

where all values are based on calculations at the wall.

The value of $\dot{\gamma}_W$ is computed from the average $\dot{\gamma}$ by use of the Rabinowitsch correction as stated by Pezzin (5) as well as others. The corrected value for $\dot{\gamma}_W$ is as follows:

$$\dot{\gamma}_W = \dot{\gamma}(0.75 + 0.25/C) \tag{3}$$

where C is the slope $(\partial \log \tau)/(\partial \log \tau)$ of a plot of

$$\log \tau = C \log \dot{\gamma} + \log \eta \tag{4}$$

Such a plot is shown in Figure 1 for a typical asphalt cement of 89 penetration. The evaluation of C, sometimes called the complex flow index, provides a method for evaluating the shear susceptibility of asphalt cements. If C is greater than one, the material is considered to be dilatant; whereas, if C is less than one, the material is pseudoplastic. If C is equal to one, the material is a Newtonian fluid with a line having a slope of 45 deg (Fig. 1). Thus, the greater the divergence of C from unity is, the greater is the shear susceptibility.

When η varies with the rate of shear, it cannot be used to evaluate the viscosity except at some selected rate of shear or shearing stress. It is proposed that viscosity be evaluated at a constant power input per unit volume of $\tau\dot{\gamma}$ = 100,000 ergs/(sec-cm^3). As shown in Figure 1, the line of constant power input falls close to the region of experimental data for asphalt cements used in paving construction. Traxler (6) suggested that measurements at a power input of 1,000 ergs/sec per unit volume be used, but the higher value appears more appropriate because it requires less extrapolation of the experimental data. The apparent viscosity is reported in poises according to Eq. 5 using the value for τ_p and $\dot{\gamma}_p$ read at the intersection of best straight line (by inspection or by numerical methods) for the experimental data and the constant power input line at 100,000 ergs/(sec-cm^3).

$$\eta_a = \tau_p/[\dot{\gamma}_p(0.75 + 0.25/C)] \text{ poises} \tag{5}$$

It will be noted that the correction for the value of C = 0.5 causes a 25 percent decrease in viscosity, whereas a value of C = 1.5 results in about an 8 percent increase compared to the uncorrected average value for $\dot{\gamma}$.

PROCEDURE AND APPARATUS

The capillary procedure is quite simple and consists of measuring the force required to extrude the sample at essentially a constant volume flow rate through a capillary by using different fixed piston speeds. From these machine data, the calculated shearing stress and rates of shear are plotted as shown in Figure 1. The apparent viscosity is calculated at a constant power input of 100,000 ergs/sec per unit volume, and the shear susceptibility is indicated by the value of C.

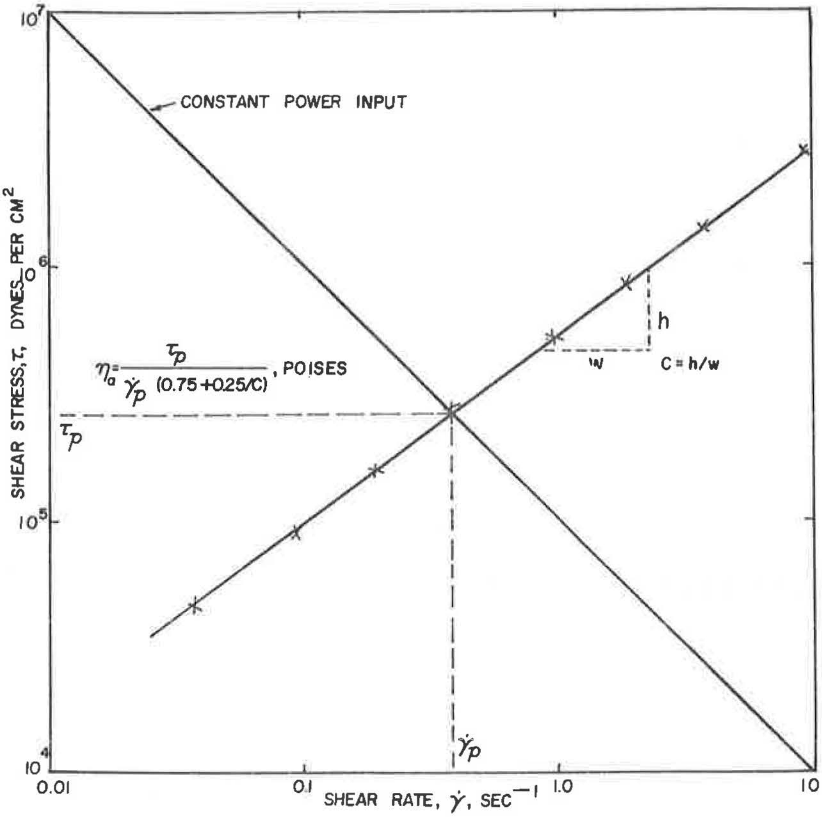

Figure 1. Rheogram for an asphalt cement.

A diagram of the barrel assembly is shown in Figure 2; the components are shown in Figure 3. The Instron machine used is shown in Figure 4, and Figure 5 shows a close-up of the environmental chamber that contains the cooling and heating coils and the ducts for circulating the air by means of a blower behind the chamber. Typical chart data obtained at different crosshead speeds are shown in Figure 6. The load scale multipliers (10X, 5X, and 2X) are shown for corresponding crosshead speeds with changes in the multiplier indicated for each change. (The multipliers are changed to increase the sensitivity at low developed loads.)

Typical results for several asphalt cements are given in Table 1, and typical plots are shown in Figure 7 for 3 samples. It is not the intent to discuss the significance of these data at this time. These preliminary data were sufficiently encouraging to warrant further study.

RHEOLOGICAL CONSIDERATIONS

The following factors affecting flow through a capillary from a reservoir must be considered in applying this technique to asphalt cements.

Sample Reservoir

Results of studies on the influence of a sample charge in the barrel that acts as a reservoir are given in Table 2 for several asphalts. These data demonstrate that the reservoir pressure drop must be considered. The effect decreases as the ratio of length to diameter (L/D) increases. Pezzin (5) indicated that the effect amounted to 0.5

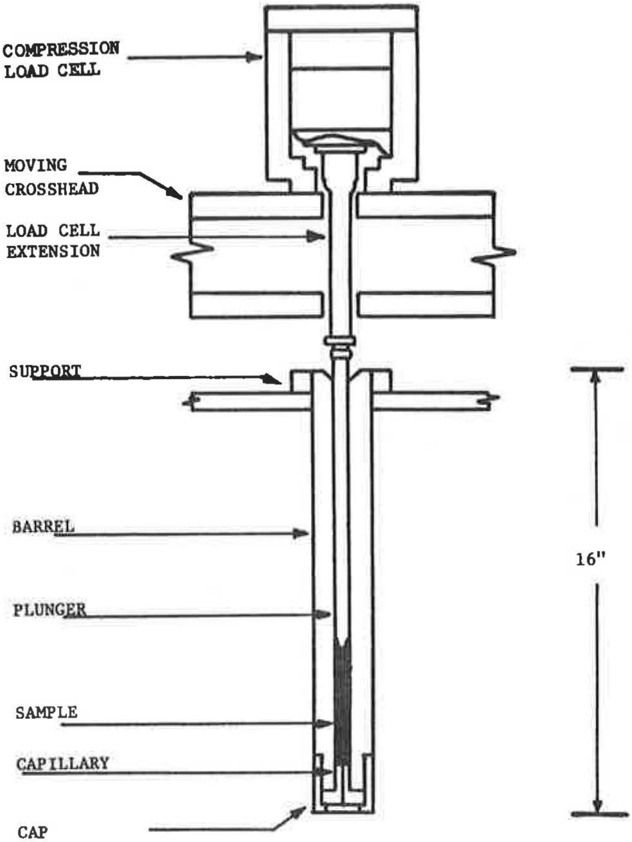

Figure 2. Barrel assembly for rheometer.

percent for polymers with an L/D ratio of the order 35. Further studies of this type are discussed later. The sample size may be a critical factor for certain materials.

Entrance Geometry

An entrance effect occurs (Couette's) where the flow accelerates from the reservoir rate to the capillary velocity. In general, a correction of equivalent length is used to account for the pressure drop due to this energy dissipation as discussed by Pezzin (5), Duvdevani and Klein (7), and Philippoff and Gaskins (8).

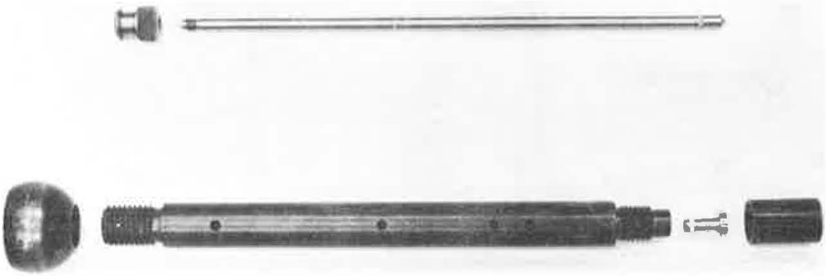

Figure 3. Rheometer components.

Figure 4. Instron machine.

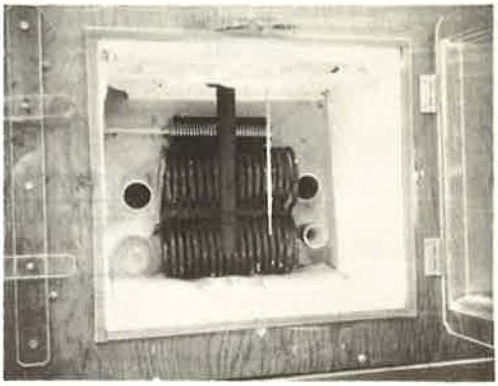

Figure 5. Environmental chamber.

A study on the practical effect of using a flat, 180-deg capillary entrance with a flat plunger was developed, and the results are given in Table 3. Variable results occur for conical or flat entry with the latter being considered as acceptable for asphalt cements at this time. Bagley (9) comments on flat entry as do Boles and Bogue (10). An extensive analysis with diagrams on this subject was given by Cook, Furno, and Eirich (11). Metzger and Knox (12) used 180- and 90-deg entrance geometry without appreciable differences.

The experimental results given in Table 3 indicate that the energy corrections required are small in comparison with flow energy loss through the capillaries for an L/D

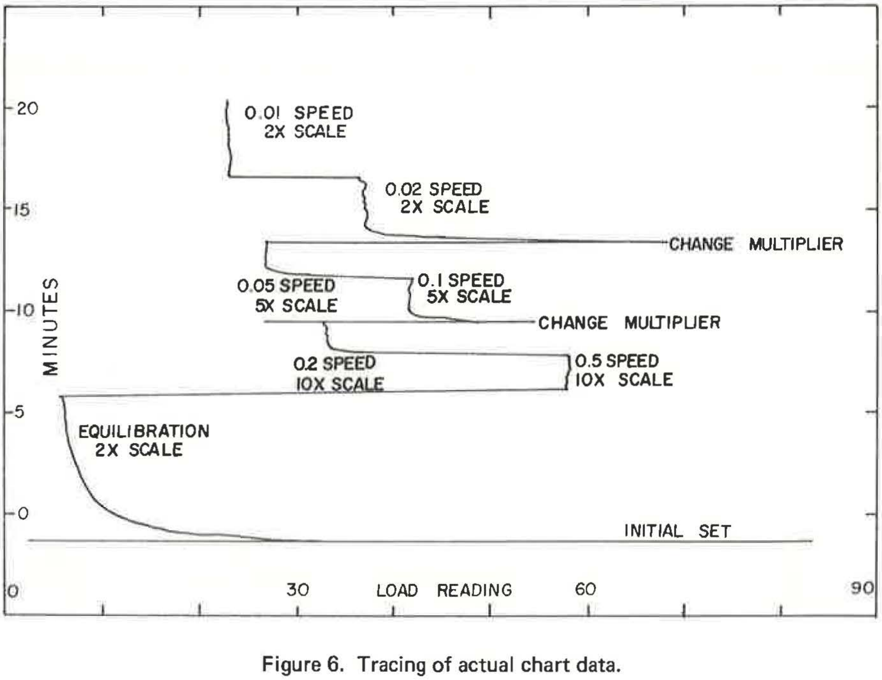

Figure 6. Tracing of actual chart data.

ratio above 15. Experiments indicate that asphalt S63-20 shows incipient dilatant flow behavior (C > 1) with anomalous flow developing above some critical pressure. Further study may elucidate this behavior.

Conditioning Time

Studies made showed that 30 minutes is necessary to cool the barrel and sample center in the regular barrel with forced convection fan cooling in an environment at 77 F. Other test temperatures might require different preparation times. The test procedure prescribes a preliminary stabilizing period at low flow rates that tends to ensure constant temperature before data are taken.

Incompressible Flow

The theoretical equations assume incompressibility for asphalt. This is not true, but in this paper such effects are considered negligible.

TABLE 1

RHEOLOGICAL DATA AT 77 F ON ASPHALT CEMENTS

Cement	Penetration at 77 F	Apparent Viscosity (mp)	Complex Flow Index C
S63-4 Smackover	89	1.48	0.84
S63-6 Florida AC-8	89	0.93	0.99
S63-13 Air Blown	89	0.62	0.64
S63-19 Panuco	90	0.98	0.85
S63-20 Los Angeles Basin	89	1.34	1.02
S63-21 Kern River	89	0.85	1.10
7171B Recovered AC-8	63	3.2	0.60
7192B Recovered AC-8	35	7.5	0.65
7442A Recovered AC-8	18	49.5	0.55
7136A Recovered AC-6	20	52.5	0.85
S62-2 Hawkins Residual	soft	0.0860	1.00
S63-2 Mid-Continent Flux	soft	0.00925	0.93

Radial Flow

Based on the literature (7), the effects of radial velocity are considered negligible because the asphalts are considered to follow the power law at the temperatures used.

Pressure Effect

The effect of pressure on viscosity is assumed to be negligible for the data reported here. This subject is discussed by Duvdevani and Klein (7). Any increased viscosity because of pressure offsets to some extent the decrease in viscosity caused by thermal effects.

TABLE 2

COMPARATIVE DATA ON 85 TO 100 PENETRATION ASPHALT CEMENTS WITH DIFFERENT RESERVOIR LENGTHS

Capillary[a]	Cement	8-in. Reservoir		3-in. Reservoir	
		Apparent Viscosity (mp)	Complex Flow Index C	Apparent Viscosity (mp)	Complex Flow Index C
No. 8	S63-13	1.09	0.73	0.72	0.71
Diameter = 0.147 in.	S63-19	2.31	0.81	2.21	0.79
L/D = 7.24	S63-20	2.35	1.19	2.31	1.03
No. 6	S63-13	–	–	0.78	0.60
Diameter = 0.097 in.	S63-19	–	–	1.20	0.83
L/D = 11.4	S63-20	–	–	1.20	0.99
No. 4 and No. 5	S63-13	0.71	0.66	0.62[b]	0.64[b]
Diameter = 0.060 in.	S63-19	1.10	0.91	0.98[b]	0.85[b]
L/D = 16.7	S63-20	1.14	0.89	1.34[b]	1.02[b]

Note: Data for 77 C with power input of 100,000 ergs/(sec·cm^3).

[a]L/D is ratio of length to diameter.

[b]From Table 5.

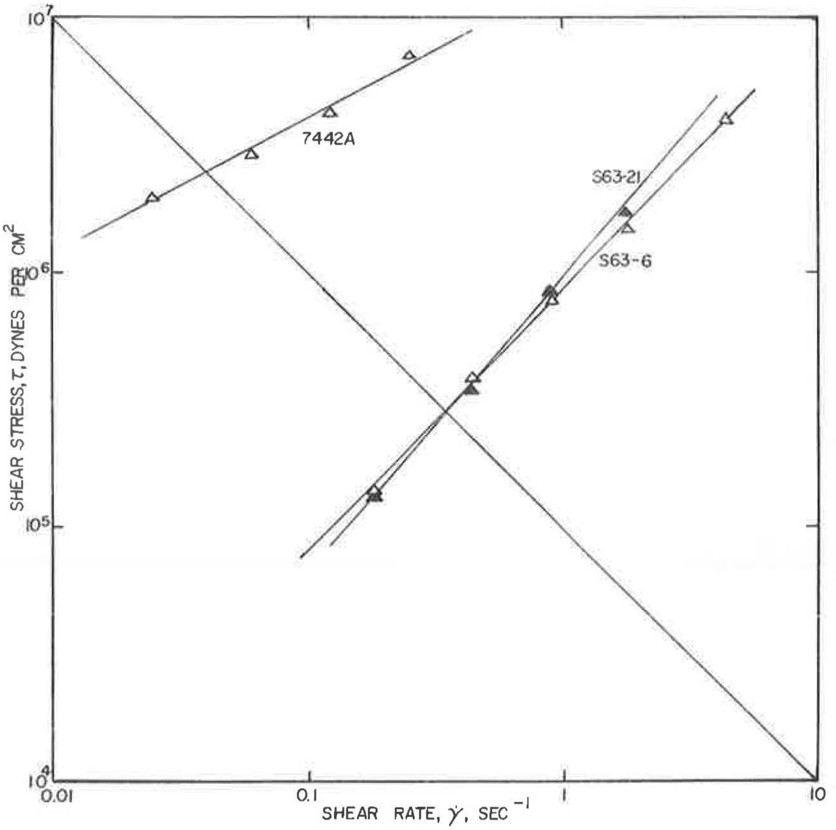

Figure 7. Example of rheograms for different asphalts.

Thermal Effects

There are two possible thermal effects in capillary viscometry. One is the rise in temperature resulting from the viscous energy dissipation as studied by Gerrard et al. (13), who showed that a 5 percent change in viscosity could be expected in so-called isothermal flow for apparent Newtonian flow at power inputs of about 1 billion ergs/(sec-cm^3). A second effect is the cooling resulting from the expansion of the fluid as the pressure decreases during the passage through the capillary (5). Both these compensating effects are not corrected for in this study because the power input of 10^7 ergs/(sec-cm^3) is 2 orders of magnitude less than the values of 10^9 noted earlier (13), and the expansion correction is considered unimportant.

TABLE 3

COMPARATIVE DATA ON 85 TO 100 PENETRATION ASPHALT CEMENTS WITH DIFFERENT ENTRANCE GEOMETRY

Cement	90-Deg Conical Entrance and Plunger[a]		180-Deg Flat Entrance and Plunger[b]	
	Apparent Viscosity (mp)	Complex Flow Index C	Apparent Viscosity (mp)	Complex Flow Index C
S63-4	1.48	0.84	1.53	0.76
S63-13	0.62	0.64	0.83	0.66
S63-19	0.98	0.85	0.88	0.87
S63-20	1.34	1.02	1.09	0.97

Note: 3-in. reservoir length for all samples.

[a]Capillaries No. 4 and No. 5.
[b]Capillary No. 7A.

Capillary Geometry

Study of the data given in Table 2 indicates that the geometry (L/D ratio) of the capillary affects the results. This was ex-

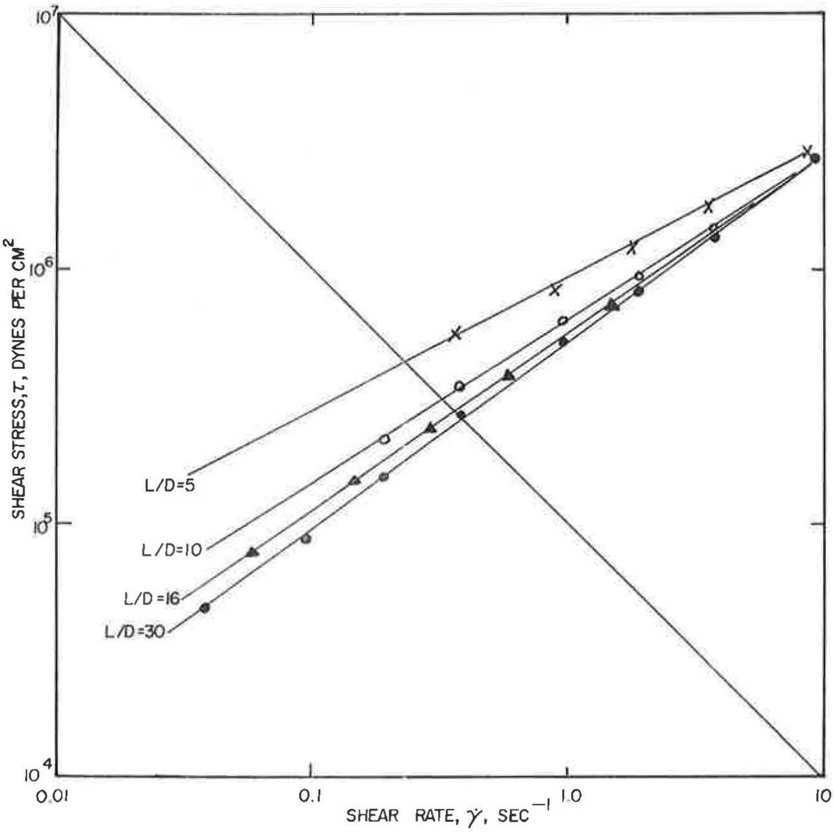

Figure 8. Rheograms for different geometry for asphalt S63-13.

TABLE 4

COMPARATIVE DATA ON 89 TO 90 PENETRATION ASPHALT CEMENTS WITH 180-DEG ENTRANCE GEOMETRY

Capillary	Cement	Apparent Viscosity (mp)	Complex Flow Index C
No. 15A	S63-13	1.56	0.53
Diameter = 0.10 in.	S63-19	3.09	0.73
L/D = 5	S63-20	1.01	0.95
No. 13A	S63-13	0.85	0.65
Diameter = 0.10 in.	S63-19	1.73	0.82
L/D = 10	S63-20	1.10	0.99
No. 12A	S63-13	0.64	0.75
Diameter = 0.10 in.	S63-19	1.39	0.95
L/D = 10	S63-20	1.00	0.89
No. 18A and No. 19A	S63-13	0.80	0.61
Diameter = 0.0625 in.	S63-19	0.78	0.81
L/D = 20	S63-20	1.21	1.09
No. 16A and No. 17A	S63-13	0.73	0.78
Diameter = 0.187[illegible] in.	S63-19	1.84	0.82
L/D = 16	S63-20	1.60	1.05
No. 16A and No. 17A	S63-13	0.66	0.78
Corrected by using	S63-19	1.69	0.82
Table 5 factors	S63-20	1.52	1.05
No. 4 and No. 5	S63-13	0.62	0.64
Diameter = 0.060 in.[a]	S63-19	0.98	0.85
L/D = 16.7	S63-20	1.34	1.02

Note: All samples 3 in.; at 77 F with power input of 10^5 ergs/(sec-cm^3).

[a]From Table 2 using conical entry.

plored further for different 180-deg entrance capillaries; the results are given in Table 4. Data for one asphalt are shown in Figure 8 and demonstrate for capillaries with decreasing L/D ratios that there is a trend of decreasing complex flow index with decreasing L/D ratio. It appears that, as the L/D ratio exceeds 15 to 20, the results become more consistent in accord with work of other investigators on polymers.

There is one observation that should be made regarding the data shown in Figure 8. Whenever for some reason or another the lines are not parallel indicating a variation in the complex flow index, then the relative values of the viscosities computed at different rates of shear will vary. In fact, it may be possible to select a value of $\dot{\gamma}$ at which all lines intersect at a common shear stress at the wall. However, the apparent viscosities would differ because plots with different C would have different correction factors according to Eq. 5.

Where certain geometries indicate, it is necessary to employ a modification of the basic Eq. 5. The correction is a function of the barrel geometry and the viscosity of the sample because the pressure drop is the sum of capillary pressure drop effects plus the reservoir pressure drop. Thus, for small capillaries, the pressure drop in the barrel, P_B, can be considered negligible compared to the pressure drop in the capillary, P_C, because

$$P_B/P_C = (L_B/L_C)\,(D_C/D_B)^{3C+1} \tag{6}$$

where L is length, D is diameter, and C is the complex flow index. The subscripts B and C refer to the barrel and capillary respectively. However, for other capillaries, the pressure drops in the capillary compared with those in the barrel are given in Table 5 and show a significant effect. When a fixed procedure is used with a given geometry, the capillary pressure drop, P_C, would be a corrected one from the total pressure read as $(P_C + P_B)$.

$$P_C = \frac{(P_C + P_B)}{1 + (L_B/L_C)\,(D_C/D_B)^{3C+1}} \tag{7}$$

As a first approximation, it is suggested that the apparent viscosity as calculated from Eq. 5 be corrected where indicated by dividing the apparent viscosity from Eq. 5 by 1 plus the appropriate factor given in Table 5. This is essentially the same as correcting the shear stress through Eq. 7.

Corrected values are given in Table 4. The agreement of all of the corrected values in capillaries 16A and 17A is not as good as hoped for, but additional study should result in improved techniques and better results.

Thixotropy

The effect of time on the stress-shear relation is considered negligible based on the procedure that requires reading an equilibrium stress value at different shear rates. In addition, limited data for different capillaries at L/D ratios above some critical

TABLE 5

CALCULATED PRESSURE DROP RATIOS IN CAPILLARY RHEOMETER WITH BARREL DIAMETER OF 0.375 IN. AND RESERVOIR LENGTH OF 3 IN.

Capillary						Pressure Drop Ratio, P_B/P_C			
No.	Length (in.)	Diameter (in.)	L_C/D_C	L_B/L_C	D_C/D_B	C = 0.5	C = 0.8	C = 1.0	C = 1.2
12A	3.000	0.100	30	1.0	0.267	0.037	0.011	0.005	0.002
13A	1.000	0.100	10	3.0	0.267	0.111	0.033	0.015	0.006
16A and 17A	3.004	0.1875	16	1.0	0.50	0.176	0.095	0.063	0.041
18A	1.255	0.073	20	2.4	0.195	0.040	0.009	0.003	0.001

value indicate similar values for the results. There are data available to show that in general the same results are obtained by programs for measurements at both decreasing and increasing rates of shear, but this may not be universally true.

Elastic Effects

The energy imparted for elastic deformation is carried through the capillary and dissipated on exit. This portion of the energy input is not used for viscous deformation. Pezzin (5) indicates this to be of the order of 0.05 percent of total energy and can be neglected.

Melt Fracture

A possible erratic behavior at high pressures occurs for asphalts having values of $C > 1$, but in general the rheological evaluation is made at stresses below which this phenomenon occurs. At this writing, the complicating effects (entrance compression, energy dissipation, melt fracture, and others) appear to be obviated by elimination of readings if the stress exceeds 2×10^6 dynes/cm^2.

REPRODUCIBILITY

Data given in Table 6 on different runs by different operators are shown to indicate reproducibility. An example of good data for S63-13 is shown in Figure 9 for conical entry that demonstrates the validity of Eq. 1 over the range of shear rates studied.

It is believed that the reproducibility can be improved up to 40 percent by experience, improved temperature control, and improved apparatus components.

RESULTS

For data at 77 F, it will be noted that, for different non-Newtonian asphalts, the value of the apparent viscosity will vary at different power inputs (or different rates of shear). Accordingly, in a comparison of asphalts, their relative viscosities may be different. For example, Figure 10 shows relative viscosities reversed for S63-4 and S63-20 when compared at 0.1 and 10 sec^{-1}.

Comparable data for apparent viscosity at 77 F by the proposed capillary method and by the cone and plate procedure (15) are given in Table 7 where the agreement is quite good at a shear rate of 0.05 sec^{-1}.

TABLE 6

ABBREVIATED STUDY OF VARIABILITY IN FLORIDA METHOD FOR 85 TO 100 PENETRATION ASPHALT CEMENTS

Cement	Replicate	Apparent Viscosity (mp)			Complex Flow Index C		
		Amount	Avg	Deviation From Mean (percent)	Amount	Avg	Deviation From Mean (percent)
S63-4	1	1.43	1.48	3.7	0.82	0.84	5.7
	2	1.42			0.92		
	3	1.49			0.85		
	4	1.58			0.76		
S63-13	1	0.77	0.62	12.9	0.66	0.64	3.1
	2	0.56			0.67		
	3	0.60			0.63		
	4	0.54			0.62		
S63-20	1	1.39	1.34	5.2	0.97	1.02	6.8
	2	1.19			0.94		
	3	1.38			1.11		
	4	1.38			1.07		

Note: Average deviations of 3 sets of data: apparent viscosities—7.3 percent of viscosity value; shear susceptibilities—5.2 percent of index value.

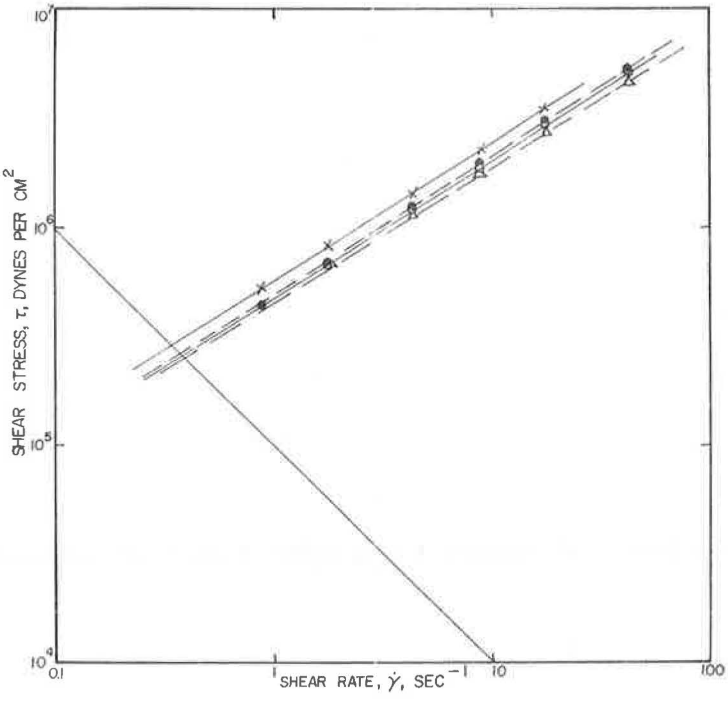

Figure 9. Reproducibility of data on asphalt S63-13.

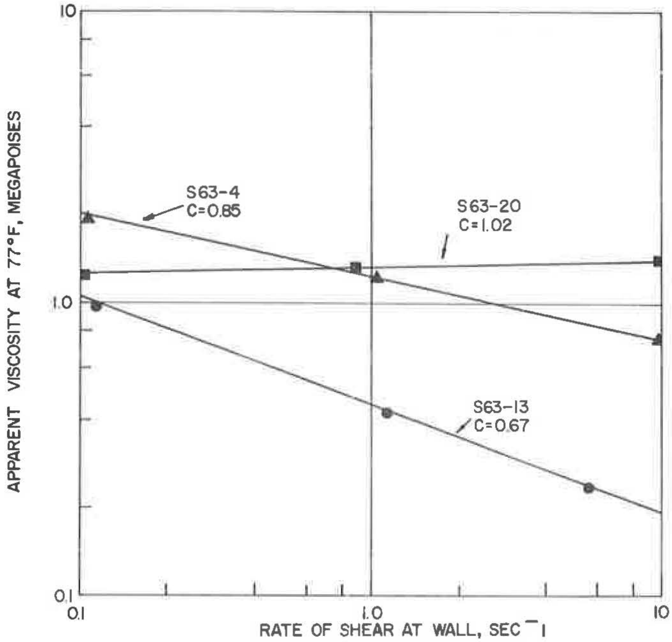

Figure 10. Variation of viscosity with shear rate.

TABLE 7

COMPARABLE VISCOSITY DATA AT 77 F

Asphalt	Penetration at 77 F	Florida Method: Complex Flow Index C	Florida Method: Viscosity (mp) 10^5 ergs/(sec-cm^3)	Florida Method: Viscosity (mp) 0.05 sec^{-1}	Asphalt Institute Method, Viscosity (mp), 0.05 sec^{-1}
8533	60	0.80	2.24	3.06	2.78
8621	60	0.64	2.31	3.72	4.00
8581	59	0.92	2.68	2.94	2.60
8534	90	0.73	0.97	1.50	1.19
8692	89	0.92	1.00	1.16	0.90

TABLE 8

RHEOLOGICAL DATA AT 60 F ON 89 TO 90 PENETRATION ASPHALT CEMENTS

Cement	Observed: Apparent Viscosity (mp) 10^5 ergs/(sec-cm^3)	Observed: Apparent Viscosity (mp) 0.05 sec^{-1}	Observed: Complex Flow Index C	Computed[a]: Penetration at 60 F, 100g/5 sec	Computed[a]: Viscosity (mp)	Computed[a]: Range[b]
S63-13	4.4	7.3	0.72	40	7.5	5.5 - 14
S63-19	17.1	20.0	0.90	32	11.0	8 - 18
S63-20	49.5	49.6	0.96	24	17.9	13 - 35

Note: Capillaries No. 16A and No. 17A, 0.187 × 3.0 in.; data corrected for reservoir pressure effect.

[a]Calculated from measured penetration at 60 F by using the following equation for viscosity at 60 F at a rate of shear equal to 0.05 reciprocal seconds (14): $\log \eta = 3.6 - 1.7 \log \text{pen}$; $\eta = 3{,}980\ \text{pen}^{-1.7}$ mp.

[b]Allowable range shown by Welborn and Griffith (14).

For data at 60 F, the general applicability of the procedure was demonstrated by measurements using capillaries with diameters of 0.187 in. and a length of 3 in.; results are given in Table 8. The influence of temperature is shown by comparing the results with those given in Table 1.

The results at 60 F are reported both at a power input of 10^5 ergs/(sec-cm^3) and at a rate of shear equal to 0.05 sec^{-1} for comparison with the values computed by the formula of Welborn and Griffith (14). The measured data are of the same order of magnitude as the range for computed values.

Data on 3 asphalt cements at 140 F by the method in this paper compared to results by the Cannon-Manning efflux method (also a capillary method) are given in Table 9. It is shown that low viscosity does not ensure Newtonian flow at either 140 or 77 F (Table 1). Each asphalt at any temperature is a separate entity, and its composition will determine its flow characteristics.

DEVELOPMENT OF A TEST METHOD

The philosophy observed in considering the applicability of capillary rheometry to asphalt cements as a standard test method has been to develop a satisfactory procedure even though it may have limitations. There is a great need for apparent viscosity measurements at ambient temperatures; the proposed method (or modifications of it) could be an acceptable one. Development work on modifications for a rapid measurement is now in progress, and it is expected that a final recommended procedure will soon be available. It now takes about 40 minutes to make preparations, equilibrate the temperature, and run a sample. This total elapsed time could be reduced with multiple samples and fewer points for control testing.

TABLE 9

RHEOLOGICAL DATA AT 140 F ON 89 TO 90 PENETRATION ASPHALT CEMENTS

Cement	Capillary: Apparent Viscosity (poise)	Capillary: Complex Flow Index C	Cannon-Manning Viscosity (poise)
S63-13	1,670[a]	0.94[a]	1,726
S63-19	4,680	0.93	3,468
S63-20	1,370	1.11	1,109

[a]Average of three.

There remain certain other details such as establishment of optimal capillary dimensions and tolerances and development of an inexpensive constant rate piston machine. These are being investigated.

ACKNOWLEDGMENT

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EFFECTS OF FIELD AGING ON FUNDAMENTAL PROPERTIES OF PAVING ASPHALTS

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An extensive 30-month field and laboratory study has been completed on paving-grade asphalts subjected to 11 to 13 years of service in 53 highway pavements located throughout the United States. During the construction seasons of 1954-1956, the Bureau of Public Roads accumulated more than 300 asphalt retain samples from more than 285 identifiable construction projects in 37 states. These original asphalts were considered representative of the production of 130 asphalts of known source from 100 asphalt refiners. During the years, these original retain samples have been tested extensively. In 1967, the present study was initiated with a field survey to rate the performance of asphalts from a preselected number of survivor and nonsurvivor projects and to take statistically selected samples of the pavements for laboratory recovery and analyses of the field-aged asphalts. The immediate objectives were to determine the overall changes that had occurred in the asphalts as a result of construction and more than 10 years of service in pavements and to relate these changes to laboratory-measured fundamental properties of the original and recovered asphalts. The 13 studies undertaken are described and reported in this paper, significant relationships and correlations are developed, and findings and conclusions are presented.

•DURING the past 40 years, research studies on the nature and properties of highway asphalts have increased in both scope and complexity as evidenced by the published literature. Nevertheless, the state of the art in asphalt technology is still such that universal agreement does not exist among asphalt producers and consumers as to the requirements needed in specifications for highway asphalts in order to ensure a completely satisfactory product.

This situation is not surprising, for an analysis of the data and information available reveals that most of the reported research has been on asphalts that were (a) not well defined in terms of fundamental chemical and physical properties; (b) insufficiently characterized by simulated laboratory performance tests under accelerated aging conditions; and (c) rarely documented as to their actual performance characteristics under various construction, traffic, and environmental conditions.

Although carefully designed and controlled laboratory experiments can produce numerical values of fundamental physical and chemical properties of asphalts comparable to those found in field asphalts, the significance of the test results, in terms of the serviceability of the material, must at some stage be related to usage conditions and pavement life. Thus, laboratory studies must be conducted with well-defined asphalts of known serviceability characteristics under a range of usage conditions.

Although it is true that some well-designed field experiments have been conducted, they have generally been limited to regional asphalts exposed to one particular set of construction and environmental conditions. Such studies, although indicative of regional experience, do not meet the need for the development of relationships having general applicability.

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It was the lack of such field serviceability data that prompted the Federal Highway Administration (then the Bureau of Public Roads) to initiate the accumulation of over 300 asphalt samples from over 285 identifiable construction projects in 37 states during the 1954-1956 construction seasons. These asphalts are representative of the production of about 130 asphalt cements of known source from about 100 refineries. It was planned to use this "bank" of asphalt specimens for relating laboratory-determined physical and chemical properties with the observed serviceability of the asphalts.

The testing of the original asphalts was carried out by the Materials Division of the Office of Research of FHWA, either in its own laboratories or in cooperation with others. The results of this work have been reported in a series of 4 technical papers (1, 2, 3, 4).

In 1967, the FHWA initiated a field survey of the pavements that had been built with these asphalts. The primary purpose of the field survey was to determine the overall changes in the physical and chemical properties of the asphalt that had occurred as a result of construction and more than 10 years of service in pavements. A contract was awarded to Materials Research and Development, Inc., on May 1, 1967, to assist in planning and executing the field survey to include the following:

1. Contact all state highway departments for information on general conditions of all candidate asphalt pavements (i.e., those pavement sections from which asphalt samples were retained) and categorize them into survivors or nonsurvivors;
2. Design an experiment to provide a basis for representative sampling of projects, taking into account the factors of asphalt type, climate, and loading conditions;
3. Perform a subjective field examination with a team of experienced asphalt technologists to rate the condition or serviceability of the pavements, with particular emphasis on the role of the asphalt;
4. Collect samples from all pavements rated, employing statistical techniques and procedures for determining number and location of samples;
5. Conduct a program of laboratory testing to determine the physical properties of asphalt-aggregate mixtures and the physical and chemical properties of recovered asphalts; and
6. Analyze the data obtained, together with data previously collected on the original asphalts, and search for any appropriate relationships or correlations between laboratory-measured properties of original and recovered asphalts of known serviceability characteristics.

The terms of the contract called for Materials Research and Development to (a) carry out the field rating of the asphalts and the asphalt pavements with the assistance of the staff of the Materials Division of FHWA and representatives of state highway departments, (b) perform the laboratory testing of the recovered asphalts for chemical properties, and (c) conduct the overall analysis of the data. The Materials Division was to (a) determine the physical properties of the asphalt-aggregate pavement samples, (b) extract the asphalt for further testing, and (c) measure the physical properties of the recovered asphalts and aggregates.

The responsibility for sampling of the asphalt pavements, at the points marked by the rating team, was assumed by the state highway department having jurisdiction over the highway. The states cooperating in this study, listed alphabetically, and the number of projects in each state were as follows:

State	Projects	State	Projects
Arkansas	4	Minnesota	2
California	3	New Hampshire	2
District of Columbia	2	New Jersey	3
Florida	3	New Mexico	1
Georgia	5	North Carolina	1
Iowa	2	Oklahoma	6
Louisiana	2	Rhode Island	2
Maine	2	Tennessee	1
Maryland	4	Texas	6
Massachusetts	2	Total	53

Excellent cooperation and workmanship on the part of the state sampling crews resulted in delivery of all samples in good condition to the FHWA laboratory in Washington, D. C.

SPECIFIC OBJECTIVES

The specific objectives of this research study were as follows:

1. To measure changes that have occurred in the fundamental physical and chemical properties of asphalt cements, after 11 to 13 years in service in pavements, from a sufficient variety of sources to be representative of national asphalt production;
2. To relate these changes to the properties of the original asphalts, rationalizing the effects of mixture properties and their variability; and
3. To correlate the measured changes in field-aged asphalts with the corresponding changes in laboratory-aged asphalts from the same sources.

The ultimate application of the results of the study is the long-term goal of writing asphalt specifications that will define asphalts of high durability and, consequently, provide the basis for more durable asphalt pavements.

OPERATIONAL APPROACH

In 1966 a search by FHWA personnel of state highway department records determined that, of the original 210 projects in 26 states constructed in 1954-1956 with 85 to 100 penetration grade asphalts, 130 pavements were still in service with no resurfacing. The results of this search and the available data on the original asphalts were used to select projects for evaluation and sampling to provide a representative coverage of the various asphalts differing in original physical and chemical characteristics. An operational approach in 3 phases for carrying out the study was then devised.

Phase 1—Selection of Projects and Conduction of Field Survey by Rating Team

On the basis of information and data available, 53 projects in 19 states were selected for study. A 2-man rating team, consisting of one member each from Materials Research and Development and the Materials Division, visually examined and rated each pavement location on the basis of surface characteristics such as cracking, raveling, and instability. The rating team also decided the specific location of sampling points. Samples were taken by state highway department personnel and shipped to the FHWA laboratory in Washington, D. C., where physical tests were performed on the asphalt mixtures and on the recovered asphalts.

Phase 2—Chemical Analysis of Recovered Asphalts

Samples of the recovered asphalts were supplied to Materials Research and Development for chemical analysis by the Rostler precipitation method and for determination of glass transition temperature.

Phase 3—Analysis of Data and Preparation of Interpretive Report

Upon completion of Phases 1 and 2, the following 13 studies were carried out by using the data obtained:

1. Determine means, standard deviations, and ranges in properties of the 85 to 100 penetration grade asphalts included in the study and determine within-laboratory variability of test data on physical and chemical properties of pavement samples;
2. Establish within-project and between-project variability of data developed in pavement study;
3. Make decision on probability of recovered asphalts being the same as original records indicate;
4. Relate mixture and environmental factors to change in asphalt properties;
5. Calculate variability of recovered asphalt properties transversely across pavement and relate to traffic intensities;

6. Establish correlations between measured changes in properties of asphalts caused by accelerated laboratory weathering test and aging for 11 to 13 years of service in field pavements;

7. Correlate observed pavement condition with mixture properties, original and recovered asphalt properties, and environment;

8. Relate original chemical composition of asphalt to properties of recovered asphalts (i.e., determine if predictive);

9. Relate original chemical composition of asphalt to observed pavement condition factors;

10. Determine which parameters of pavement performance are related to traffic loading experienced and pavement structure as built;

11. Segregate all of these correlations by geographical regions and determine variations, if any;

12. Compare changes in asphalt properties between in-service and out-of-service (i.e., with overlay) pavements to evaluate effect of overlay on aging, if any; and

13. Evaluate original properties of currently available 85 to 100 penetration grade asphalts to establish if current production is representative of 85 to 100 asphalts available in 1954-1955.

An interpretive final report (5) of the findings was prepared that summarizes the results of the field survey, test data, and various analyses and presents conclusions and recommendations.

SELECTION OF PROJECTS

Approximately 300 paving projects were identified with the asphalt samples collected in the 1954-1956 period. Of these, 210 projects were constructed with 85 to 100 penetration grade asphalt. Information collected by FHWA during 1965-1966 on the status of the pavements revealed that, of the 210 projects, 130 were still in service as constructed and 80 either had been overlaid or had received surface treatment or had been reconstructed. Fifty-three projects in 19 states, constructed with asphalt from 29 sources, were selected for study. Forty-nine were constructed with 85 to 100 penetration grade asphalt and 4 with 60 to 70 penetration grade. At the time the information was collected in 1965-1966, 38 of the projects were in service as constructed and 15 had been resurfaced. When the pavement condition survey and sampling program were carried out in 1967, 19 projects had been resurfaced and 34 were still in service as constructed. Evaluation of the data for geographic and climatic distribution, crude source and refining method, and chemical composition of the original asphalts indicated that the selected projects were representative of the available projects. Identification of the selected projects is given in Table 1.

FIELD SURVEY PROCEDURE

The field survey was made during the period from July to December 1967 by rating teams composed of 2 permanent members and 2 local members; one of the latter represented the state involved and the other the Federal Highway Administration at the division or regional level. A description of the field survey is contained in another paper (6). The purpose of the field survey was threefold: to obtain information pertaining to construction and traffic, to designate sampling locations, and to make subjective evaluations of the condition of the pavements constructed with the asphalts under study.

The method of selection of projects, as described previously, resulted in a heterogeneous sampling of projects ranging from heavily trafficked urban freeways to residential streets and included conventional construction of flexible pavements and composite pavements (i.e., asphalt concrete overlay of portland cement concrete pavements). The condition survey phase of the investigation was tied to "where the asphalts were," and condition per se was not a significant criterion in the selection of projects.

The factor of survival of the original projects played a major role in the final experiment design. It was reasoned that pavements that had been overlaid or to which a

TABLE 1

PROJECTS SELECTED FOR STUDY

Project[a]	Asphalt[b]	Service Status	State	Contract	Location
2	29	In	Georgia	F-026-1(4)	Ga-1, access road to Fort Benning from US-27, Muscogee County
3	30	In	Georgia	F-007-2(2)	Ga-38 in Thomasville, Thomas County, South Broad Street to Smith Avenue
4	32	In	Georgia	F-075-1(3)	US-441 from Florida state line to 6 miles north
6	73	In	Louisiana	SN70-06-09 SN70-05-06	La-2 and La-143 near Sterlington, between junctions of La-2 and La-143
7	67	In	Texas	C177-11-10	US-377, Tarrant County, south city limits of Keller to Broadway Avenue
9	74	In	Texas	C508-1-12	Harris County, Houston, Nance Street (South Frontage Road for I-10) between Jensen Drive and Gregg
10	72	In	Oklahoma	SAP-1047(2)	Old Okla-51, Tulsa County, from US-64 to 5.5 miles east
11	25	In	Georgia	F-004-3(4)	Ga-12 in Thomson, McDuffie County, Johnson Avenue to 0.4 mile west of west city limit
12	25	Out	Tennessee	F-014-2(12)	Tenn-14, north of Memphis, from Raleigh Place to Yale Road
13	24	In	Florida	P-001-1(1)	Fla-95, Goulding to Cantonment, Escambia County
14	24	Out	Florida	P-006-2(10)	US-90, Okaloosa County line to DeFuniak Springs
15	70	In	Oklahoma	U-102(4)	US-81, Stephens County, from Chestnut Street in Duncan to 1.9 miles north
16	71	In	Oklahoma	F-198(7)	Okla-51, Payne County, East Brush Creek to 1 mile east of Stillwater
17	7	In	Massachusetts	U-229(5)	Mass-128, Needham to Wellesley, Mass-9 to 3.8 miles south
18	7	In	Massachusetts	U-229(6)	Mass-128, Dedham to Westwood to Needham, from Great Plain Avenue to 3.1 miles south
19	1	In	New Jersey	MA, Mi-2A-55	Whittier Avenue, Dunellen Borough, Middlesex County, New Market Avenue to Center Street
20	1	In	New Jersey	FAS-S-142(2)	Terrill Road, Union County, South Avenue to Raritan Road
21	11	In	Rhode Island	S-0231(1)	Main Avenue, Warwick, Kent County
22	19	In	Maryland	F-537-17 FI-464(10)	US-40, Frederick County, Monocacy River to Howard County line
24	57	In	Iowa	FN-540	US-61, Fort Madison, 35th Street to 10th Street
25	61	In	Minnesota	SP-6934-25	Minn-73, Hibbing, from intersection with US-169 approx. 0.7 mile north
26	7	In	Maine	F-046-1(3)	US-1, Washington County, between Topsfield and Princeton, Indian Township
27	7	In	New Hampshire	F-018-2(1)	NH-101 and NH-108, in Stratham from Exeter town limit to 2.15 miles east
28	91	Out	California	55-6 VC 11	Calif-142 Section A Kern County, from Beardsley Avenue to 0.6 mile north of China Grade Loop
29	92	In	California	54-7 VC 68	Calif-61, Section VII-LA-61-Gndl, in Glendale between Canada Boulevard and La Crescenta Avenue
31	95	In	California	54-7 VC 37	Carson Street, Long Beach, between Lakewood and San Gabriel River
32	68	Out	Texas	C-1-2-17	US-80, El Paso, from Broadway Avenue to 1.1 miles north
33	69	Out	Texas	C-33-5-17	US-83, Jones County, from 5.8 miles north of Taylor County line to FM-605 junction
34	32	Out	Georgia	F-002-1(5)	US-1, between Folkston and Waycross, from Ware County line to 11 miles north
35	67	Out	Texas	FI-826(41)	I-30 from Texas-205 to FM-548 at Royse City, Rockwall County
36	74	Out	Texas	F-471(19)	Texas-6 in Waller County, from Grimes County line to US-290 junction
37	73	Out	Louisiana	SN 8-02-13	US-190, Morganza Floodway Bridge west of Lottie, east to Lobdell
38	19	In	District of Columbia	CN 17901	Macomb Street between Idaho Avenue and Massachusetts Avenue, and Van Buren Street between 14th Street and Georgia Avenue, Washington
39	19	In	District of Columbia	CN 17891	MacArthur Boulevard between Macomb Street and Little Falls Road, Washington
40	7	In	New Hampshire	C 3104	Bypass NH-1, Portsmouth, from Landers and Griffin Office south to US-1
41	7	Out	Maine	S-0226(1)	Maine-137, Kennebec County, Winslow to China
42	70	Out	Oklahoma	U-60(4)	US-66, Oklahoma City, from Drexel Boulevard to 0.57 mile west
43	70	Out	Oklahoma	F-186(8)	US-69, Pittsburg County, from US-270 junction to 4.7 miles north

TABLE 1 (continued)

Project[a]	Asphalt[b]	Service Status	State	Contract	Location
44 45	1	Out	New Jersey	FAS-S-196(1)	Flemington-Voorhees Corner Road, from US-202 junction to 1.4 miles southeast
45	15	Out	Maryland	M-528-3-320	Md-355 (formerly Md-27) from US-240 junction to Cedar Grove
46	14	Out	Maryland	CL 337-1-720	Md-27 from 1.0 mile south of Warfieldsburg to Taylorsville
47	14	Out	Maryland	CL 337-2-720	Md-27 from Taylorsville to Mt. Airy
48	103	Out	New Mexico	FI-001-5(4)	Between Santa Fe and La Bojada
50	11	In	Rhode Island	FAS S-0235(1)	Davisville Road, East Greenwich Township
52	57	In	Iowa	FN 845	US-61 and US-218 (Main Street), Keokuk, from 7th Street to 20th Street
53	30	In	Florida	SP-4609-109	Fla-79, Bay County, from Fla-388 junction to 11 miles north
54	28	In	North Carolina	F-393(3)	US-15, Durham Bypass to Chapel Hill Bypass
55	61	In	Minnesota	SP-6934-24	US-169 in Chisholm
56	71	In	Oklahoma	SAP-70(2 and 3)	Okla-51, Payne County, west end of Project 16 to 2.58 miles west
57	166	In	Arkansas	6502	Ark-161, from First Street, Jacksonville, to 1.5 miles northwest
58	158	In	Arkansas	10476	US-67, from Current River Bridge to 6.2 miles southwest
59	160	Out	Arkansas	5412	US-62, Fulton County, from Ark-9 junction to 15.5 miles east
60	158	Out	Arkansas	5383	Ark-11, Sharp County, from Evening Shade to Ash Flat

[a]Projects 1, 5, 8, 23, 30, 49, 51 were dropped from the study because records were inadequate.
[b]Asphalt numbers are the same as those used in the literature (1, 2, 3, 4, 5, 6, 7).

surface treatment had been applied (i.e., nonsurvivors) might be representative of poor quality asphalts. Hence, a proportion of the total number of projects was selected from nonsurvivors. The projects could obviously not be included in the visual condition survey because they were no longer observable. However, where these nonsurviving projects had been only partly overlaid, observations were recorded relative to the condition of the exposed surfacing.

The purpose of the condition survey was to obtain specific quantitative information relative to the types and the severity of distress observable. It was recognized from the start that evaluation of the relationship of the properties of the asphalt to the performance of the surfacing would be extremely difficult because many factors (e.g., mix design, structural design, and construction) mask evaluation of the role of the asphalt. However, employing as a working hypothesis that the condition of an asphalt surface is related to the properties of the asphalt (with the limitation that it might prove applicable for only certain types of distress, e.g., surface wear but not rutting), it was reasoned that certain correlations might be found among various types and levels of distress and certain asphalt properties. Specifically, it was decided that properties with the highest probability for valid correlation were (a) chemical composition of original asphalt, (b) physical properties of recovered asphalt, and (c) physical properties of asphalts after accelerated aging tests.

TESTING PROGRAM

The pavement samples, cut from the pavement at each of the 313 test sites, were sent by the various state highway departments to the FHWA laboratory. After identification and measurement of thickness of the various layers, the layer of interest was separated, its density determined, and the asphalt binder extracted. Extensive physical tests were run on the recovered asphalt and the aggregate, and an aliquot portion of each recovered asphalt was sent to the Materials Research and Development laboratory for chemical analysis. Complete details of the testing schedule are included in the detailed final report of the study (5).

Test results for the original properties of the asphalts, needed for determining the changes during service, were taken from previous reports where available. Additional data, when needed, were determined in the FHWA laboratory by using standard procedures.

ANALYSIS OF DATA

The accumulated data were stored on computer cards. These included data on physical and chemical properties of the original asphalts, laboratory-aged asphalts, and recovered asphalts; construction information obtained from the state highway departments; climatological information for the pavement locations; data on condition of the pavements; measurements made on the pavement samples; gradations and petrographic determinations of the recovered aggregates; and values, factors, or parameters calculated from the measurements. The number of data columns needed required the use of 19 differently coded cards for each asphalt. A separate card was needed to record the appropriate codings for each sample. For recording the information on pavement condition, description of the cores, and properties of the recovered asphalts and aggregates, a separate card was needed for each sampling location. The types of data stored on each card are indicated by the titles given in Table 2.

Analyses were performed on the 13 studies in Phase 3 of the project, utilizing all of the data on field construction, pavement performance, and laboratory tests. The results of the data analyses are presented in detail in the final report (5). The correlations and their limitations developed from the foregoing data analyses are given in Tables 3 and 4, which are designed to be self-explanatory.

FINDINGS

The major thrust of this project has been oriented to the development and evaluation of information on the changes that have occurred in highway asphalts contained in pavements for 11 to 13 years during the range of environmental conditions encountered in the continental United States. Information of this type is essential to the understanding of the behavior characteristics of highway asphalts, particularly with respect to durability as measured by changes in their chemical composition and engineering properties. Although limited to determining the effects of long-term aging on asphalts used in asphalt concrete pavements on state highway systems carrying conventional highway traffic, the results of the present study are generally applicable wherever asphalts are used to bind aggregates in asphalt concrete mixtures serving as pavement surfaces.

The scope and magnitude of this study are such that continuing analysis of the complete data will be necessary in order to obtain full benefit from it. It is hoped that the extensive quantitative documentation of changes that occurred in the wide range of asphalt used in this study will permit future use of the data for verification of hypotheses that may result from additional research. The more significant preliminary findings are summarized in this section.

TABLE 2

TYPES OF DATA STORED ON COMPUTER CARDS

Card	Title
1	Properties of original asphalts, published data, 1
2	Properties of original asphalts, published data, 2
3	Properties of asphalts after thin-film oven test
4	Properties of recovered asphalts, FHWA laboratory
5	Properties of recovered asphalts, MR and D laboratory
6	Properties of Ottawa sand mixes, published data
7	Pavement condition survey
8	Climatological data for pavement locations
9	Construction data, 1
0	Construction data, 2, thickness measurements and petrographic data
A	Properties after California weathering oven exposure, 1
B	Properties after California weathering oven exposure, 2
C	Present properties of stored original asphalts
D	Properties of asphalts after modified thin-film oven tests
E	Rheological properties, American Oil Company laboratory, 1
F	Rheological properties, American Oil Company laboratory, 2
H	Properties of asphalt concrete and recovered aggregates
J	Penetration indexes and stiffness moduli of original asphalts
K	Penetration indexes and stiffness moduli of asphalts after thin-film oven test
L	Penetration indexes and stiffness moduli of recovered asphalts

1. The pavement condition evaluation plan developed for use in the field survey conducted by the rating team proved to be reliable where quantitative data on the various types of distress could be obtained. Quantitative data were obtained for raveling, loss of matrix, cracking (transverse, longitudinal, shrinkage, polygon, and reflection), spalling, and distortion (rutting and corrugation).

TABLE 3

SUMMARY OF CORRELATIONS

Study	Property	Correlation	Limits of Validity
4	Recovered penetration at 60 F Recovered penetration at 77 F Recovered ductility at 60 F Recovered ductility at 77 F Recovered viscosity at 60 F Recovered viscosity at 77 F	More hardening with higher air voids content	Within projects
4	Recovered penetration at 77 F	Decreased with percentage air voids up to about 4 percent voids; unaffected by increase of >4 percent voids	Between projects
4	Recovered viscosity at 77 F Recovered viscosity at 140 F	Increased with percentage air voids, rapidly up to 4 percent voids and slowly over 4 percent voids	Between projects, in service; single asphalt
4	Recovered ductility at 77 F	Remained high or moderate up to certain level of air voids; very low at higher voids content (critical point differs for different asphalts)	Within projects; single asphalt
4	Recovered ductility at 60 F	Variable at air voids <4 percent and very low at voids >4 percent	Between projects, all in service
4	Increase in A Decrease in A_1 and A_2	More change with increasing air voids content	Between projects, all in service
8	Recovered penetration at 77 F Recovered viscosity at 77 F	Less hardening at $(N + A_1)/(P + A_2)$ to about 1.4; hardening increases slowly with decreasing $(N + A_1)/(P + A_2)$ below 1.4; and hardening increases sharply with increasing $(N + A_1)/$ $P + A_2)$ above 1.4, especially at high voids contents	At constant air voids contents above about 3 percent voids; at voids contents less than 3 percent, the increased hardening with decrease of $(N + A_1)/(P + A_2)$ below 1.4 not established
5	Recovered penetration Recovered ductility Recovered viscosity Recovered softening point Change (increase) in A Change (decrease) in $A_2 + P$	More hardening in traffic lane than in passing lane	Within projects of transverse variability study (22 and 54)
5	Longitudinal (load) cracking	More severe in traffic lane than in passing lane	Within project
5	Raveling	More severe in passing lane than in traffic lane	Within project
5	Transverse cracking	Some in passing lane, none in traffic lane	Within project
7	Rutting	More severe at lowest voids content	Within 1 project; not found in other projects
7	Rutting	Less severe at higher voids and lower bitumen index	Within 1 project; not found in other projects
7	Transverse cracking Polygon cracking Spalling	More severe at higher voids and lower bitumen index	Within 1 project; not found in other projects
7	Raveling, in cold climates	Severe where recovered penetration at 60 F $\leq$ 10 and ductility at 60 F $\leq$ 3 (bitumen index was <15 for 2 of 3 worst)	Only in projects with >5,000 deg-days/year <65 F
7	Raveling Longitudinal cracking, low traffic	More severe with increasing parameter $(N + A_1)/$ $(P + A_2)$ from 0.59 to 1.10	Only in projects with <2,000 vehicles/day
7	Loss of matrix, high traffic	More severe with higher recovered viscosity at 140 F, but not more severe with higher traffic at constant viscosity, tends to be less	Only in projects with >3,000 vehicles/day
7	Longitudinal cracking, high traffic	More severe with lower voids content and lower recovered viscosity at 140 F	Only in 4 projects with >25,000 vehicles/day
7	Loss of matrix	More severe with harder asphalt as shown by recovered penetration and ductility at 60 and 77 F, viscosity at 140 F	Only in 4 projects on portland cement concrete base

TABLE 4

SUMMARY OF COMPARISONS BETWEEN LABORATORY AGING AND FIELD AGING, STUDY 6

Property	Result of Comparison
Change in viscosity at 77 F	TFOT = field aging[a] at air voids content about 1.5 to 2 percent 7-hr MTFOT = field aging at air voids content about 3 to 4 percent 9-hr MTFOT = field aging at air voids content about 4 percent 200-hr CWO = field aging at air voids content about 2 percent 1,000-hr CWO = field aging at air voids content about 4 to 5 percent
Change in viscosity at 275 F	7-hr MTFOT = field aging at air voids content about 4 to 5 percent 9-hr MTFOT = field aging at air voids content about 6 percent
Change in penetration at 60 F, 77 F, 95 F Change in ductility at 60 F, 77 F Change in viscosity at 60 F, 77 F, 275 F	TFOT = field aging at air voids content about 1.5 to 2 percent
Increase in A	7 days MR and D oven >field aging at air voids content <4 percent 7 days MR and D oven <field aging at air voids content >4 percent
Increase in N	Little or no increase in laboratory (MR and D oven) aging; considerable increase in field aging, not related to voids content
Decrease in A_1	7 days MR and D oven = field aging at air voids content ≤4 percent 7 days MR and D oven < field aging at air voids content >4 percent
Decrease in (A_2 + P)	Small decrease in laboratory (MR and D oven) aging; considerable decrease in field aging, especially at voids content >4 percent

[a]Field aging includes hardening effect of both pug-mill mixing and subsequent weathering.

2. Where quantitative evaluations were not feasible under field conditions, subjective ratings were made that, although not as dependable as quantitative evaluations, were still quite indicative of pavement conditions. An analysis of the results of the field survey revealed that the items of little consequence in the analysis were surface texture, visual estimation of amount of asphalt, subjective evaluation of asphalt performance, disintegration of aggregate particles, ladder cracking (in wheel tracks), and drainage conditions.

3. Wide within-project variation in mixture properties and properties of recovered asphalts emphasizes the importance of multiple samples, and complete characterization of the samples, for obtaining reliable data on durability of asphalts and pavements.

4. None of the measured properties on the original asphalts was by itself a determining factor as to whether the pavements survived in service. Differences in the means of values indicative of durability [e.g., penetrations and viscosities after thin-film oven test; Rostler parameter, $(N + A_1)/(P + A_2)$; and pellet abrasion test] were found to be in the predicted direction, but the differences are not considered governing. It is noteworthy in this connection that most of the asphalts originally rated by composition parameter to be of low durability could not be included in this study because of lack of survivors in this group. Of the pavements constructed with asphalts of Rostler durability groups 4 and 5, predicted to be of fair and inferior durability, only 3 in-service pavements were available.

5. No significant differences were found in any of the properties of the asphalt concretes or of the aggregates between the surviving and nonsurviving pavements. However, some differences in properties of recovered asphalts were found. In particular, differences were found among the means of ductility measurements; this supports the hypothesis that those pavements with asphalts of higher ductility are more likely to remain in service.

6. Because of the possibility of errors in records, it was necessary to give consideration to the possibility that the asphalts actually used in the pavements were not the same as recorded. On the basis of comparison of chemical composition of the recovered asphalts with that of the original asphalt, it was determined that 74.8 percent of the 313 recovered asphalts were "representative" of the original asphalt, 16.3 percent were "questionable," 3.5 percent were "improbable," and 5.4 percent were "nonrepresentative." The 28 samples rated as improbable and nonrepresentative were excluded from further consideration in most of the data analyses.

7. Relationships sought between mixture and environmental factors and changes in asphalt properties due to field aging were found to be greatly overshadowed by the voids

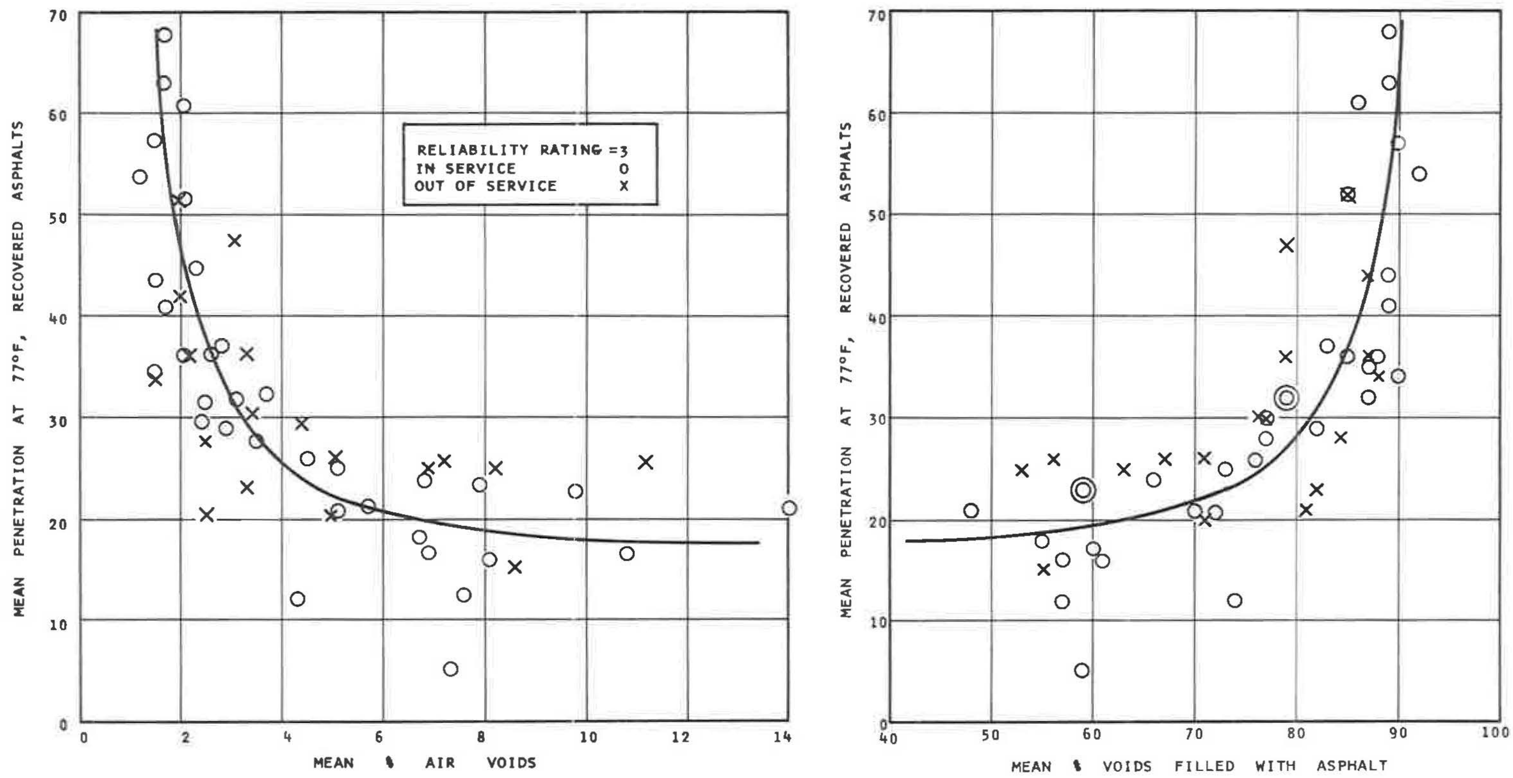

Figure 1. Relation of retained penetration after field aging to air voids and to percentage of voids filled with asphalt.

content of the asphalt concrete mixture. The effect of voids on hardening, as revealed in this study, was discussed by Welborn (7). In his report, Welborn discussed in some detail the data shown in Figures 1 through 5, and that discussion will not be repeated here. However, it should be noted that Welborn presented the data on the basis of the relationship of log penetration versus voids, whereas the relation shown in Figure 1 is on an arithmetical basis. Welborn's analysis indicated that the nonsensitivity of hardening to voids content higher than around 3 percent results, to a great extent, from the fact that "equivalent" hardening for penetration is more nearly related to the change in the logarithm rather than the change in the actual penetration. This is particularly true at low values of penetration for which apparent viscosity changes are quite large for small changes in the penetration value.

Welborn's analysis also showed the mirror relationship between the percentage of voids and the percentage of voids filled with asphalt versus the log of consistency of the recovered asphalt measured by penetration at 77 F or viscosity at 140 F. Similar relationships plotted on an arithmetic basis are shown in Figures 1 and 3. The relatively small changes in asphalt consistency for percentages of voids filled greater than 80 percent are apparent.

Figures 6, 7, and 8 show ductility relationships to voids content after field aging. Ductility values at 60 and 77 F of recovered asphalts tend to remain high at low voids

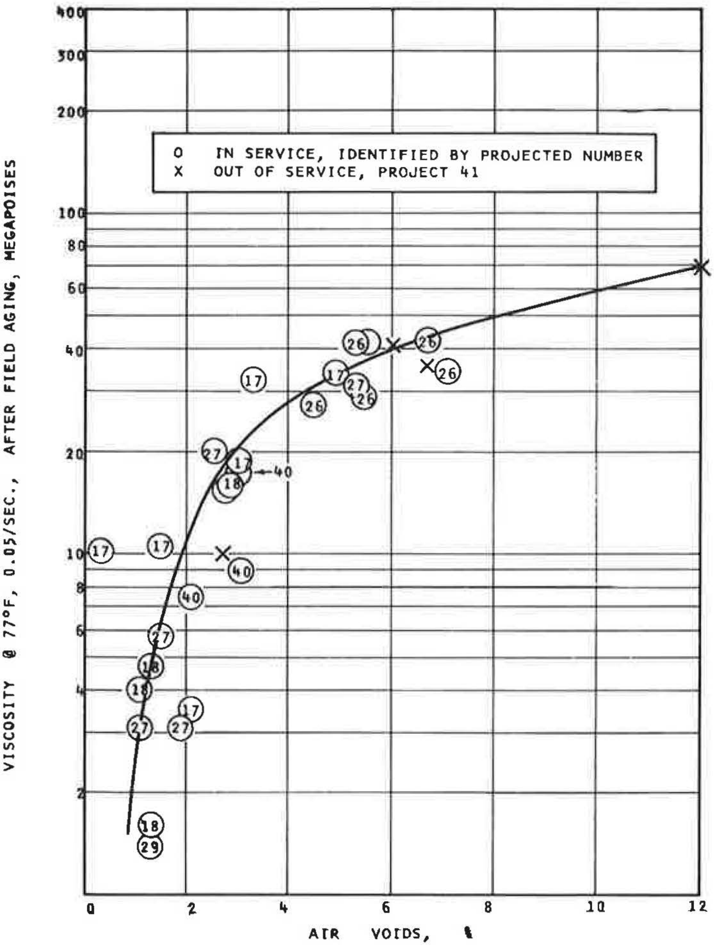

Figure 2. Relation of viscosity at 77 F after field aging to air voids, asphalt 7.

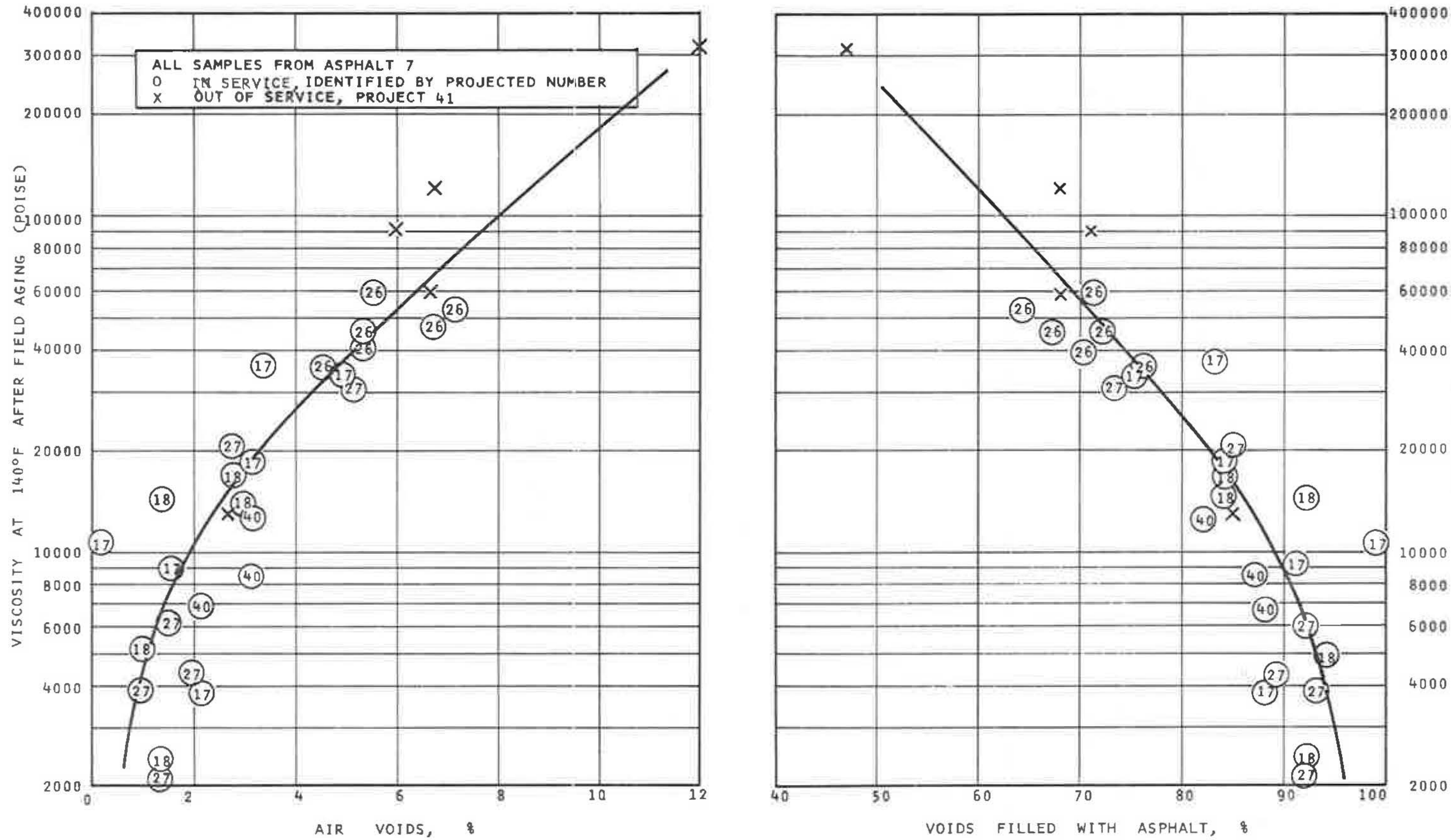

Figure 3. Relation of viscosity at 140 F after field aging to air voids and percentage of voids filled with asphalt.

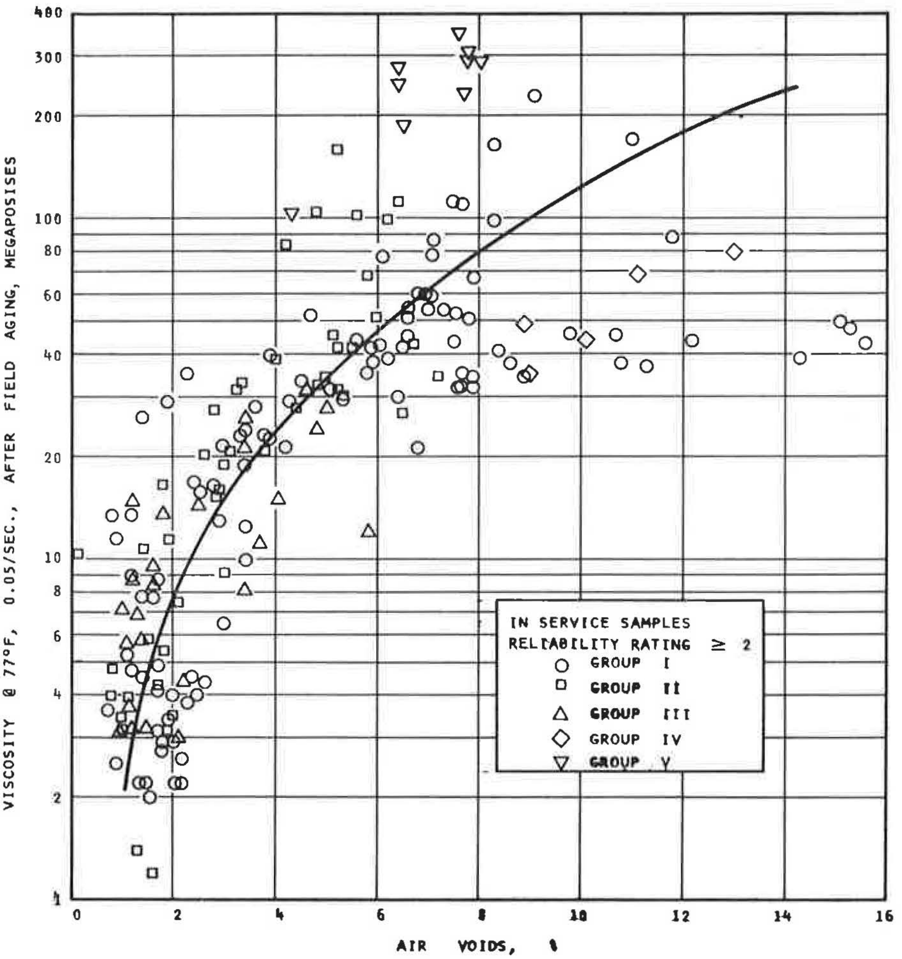

Figure 4. Relation of viscosity at 77 F after field aging to air voids, groups 1 through 5.

contents (little hardening). Ductility dropped to very low values in 11 to 13 years of field aging at high voids contents (high hardening). Further analysis of the data is required to determine whether this relationship is solely dependent on the degree of hardening or whether the ductility-consistency relationship has changed significantly.

Again, changes in chemical composition (as determined by the Rostler method) of asphalts were closely associated with the hardening of asphalt cement. Thus, the increase in asphaltenes and decrease in acidaffins were also found to be generally related to voids content of the mixture. However, considerable scatter of the data is evident. Figures 9, 10, and 11 show these relationships.

8. An analysis of the data to relate chemical (Rostler) composition of the original asphalts to properties of the recovered asphalts was conducted within groups of similar voids content. The scatter of the data and the limited number of asphalts having a relatively high Rostler parameter make it impossible to draw well-substantiated conclusions. However, Figures 12 (penetration) and 13 (viscosity) show the following trends: (a) For pavements at air voids content less than 2 percent, hardening was indicated to be directly related to the parameter $(N + A_1)/(P + A_2)$; and (b) for pavements at higher air voids contents, i.e., exceeding 2 percent, the relationship of penetration and viscosity to Rostler's parameter is not directly proportional. An increase in parameter value above about 1.4 resulted in a sharp increase in hardening. However, it was also

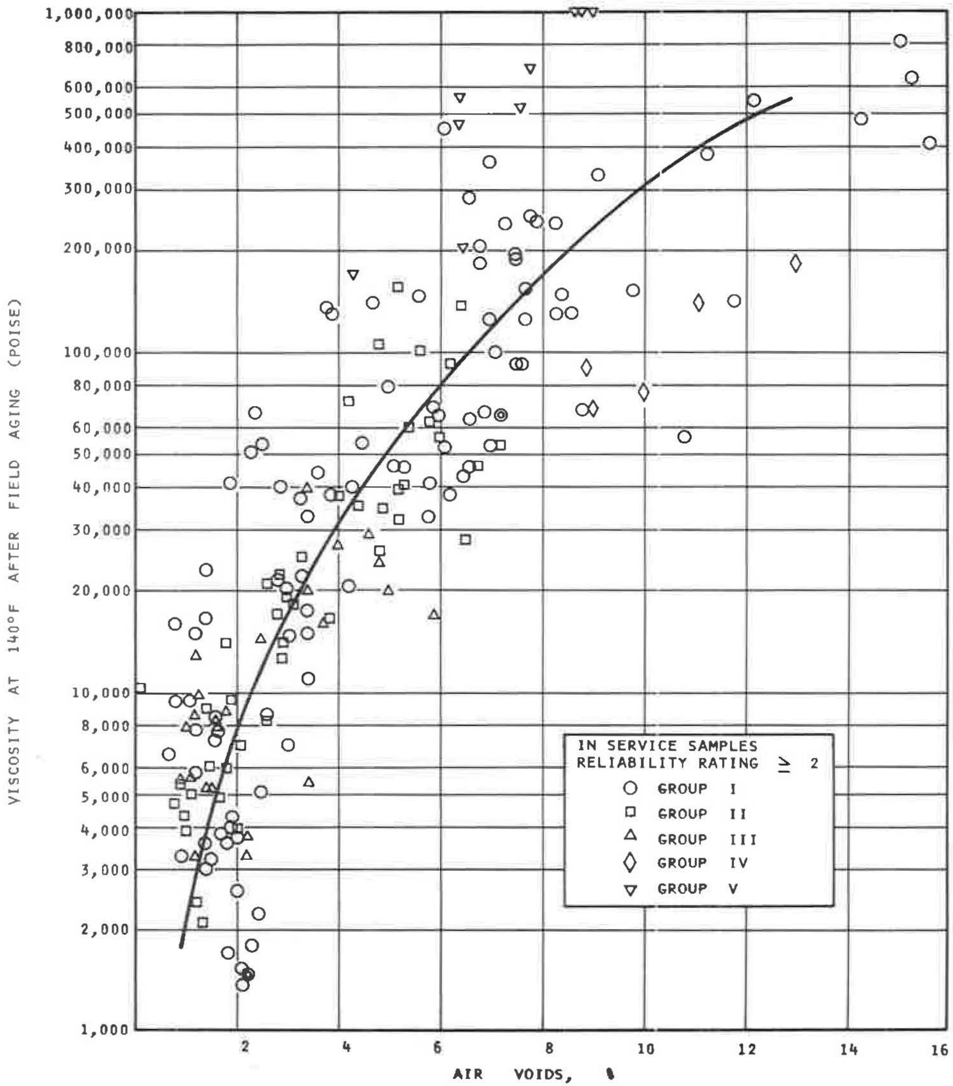

Figure 5. Relation of viscosity at 140 F after field aging to air voids, groups 1 through 5.

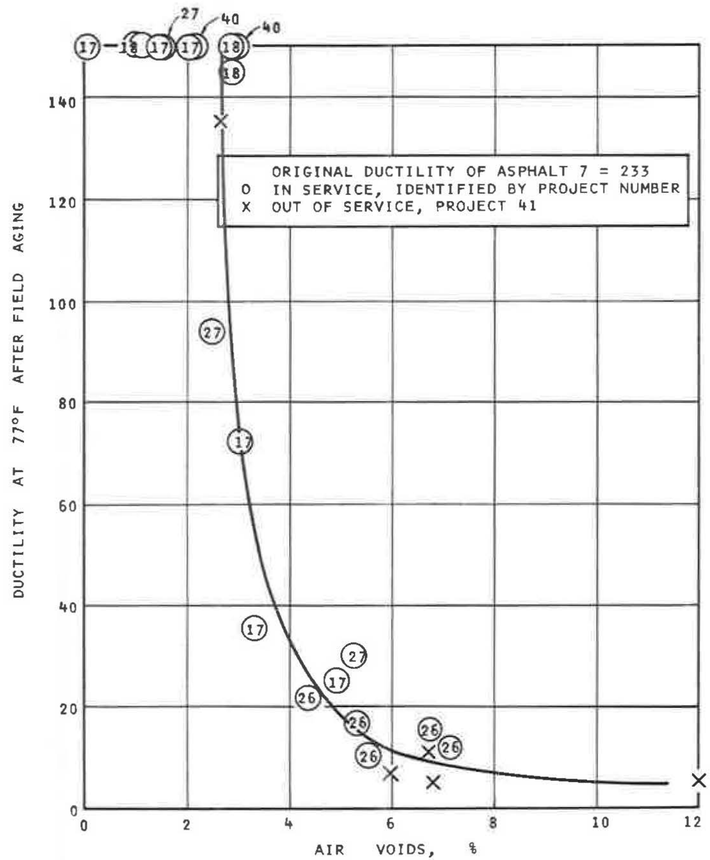

Figure 6. Relation of ductility at 77 F after field aging to air voids, asphalt 7.

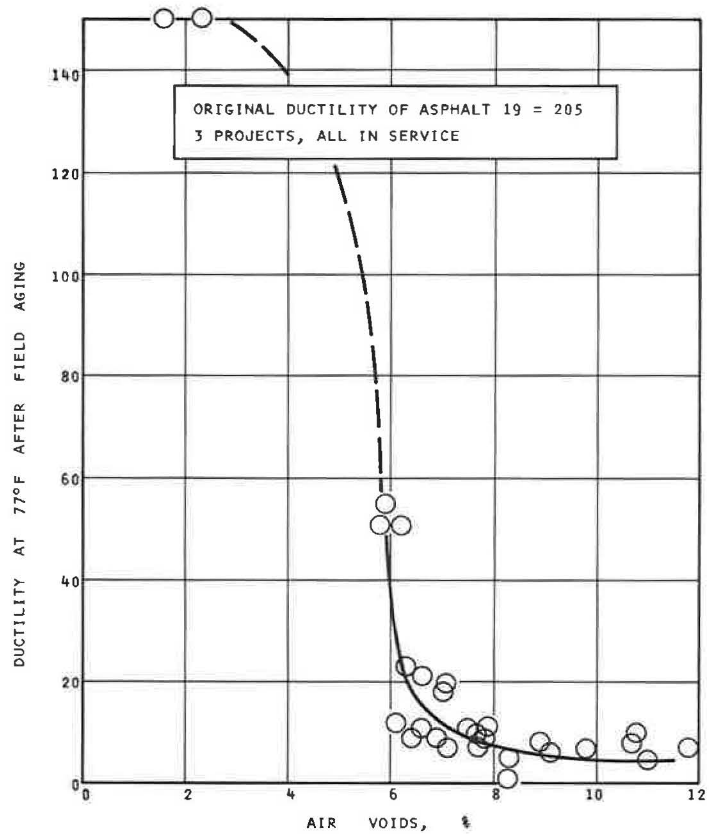

Figure 7. Relation of ductility at 77 F after field aging to air voids, asphalt 19.

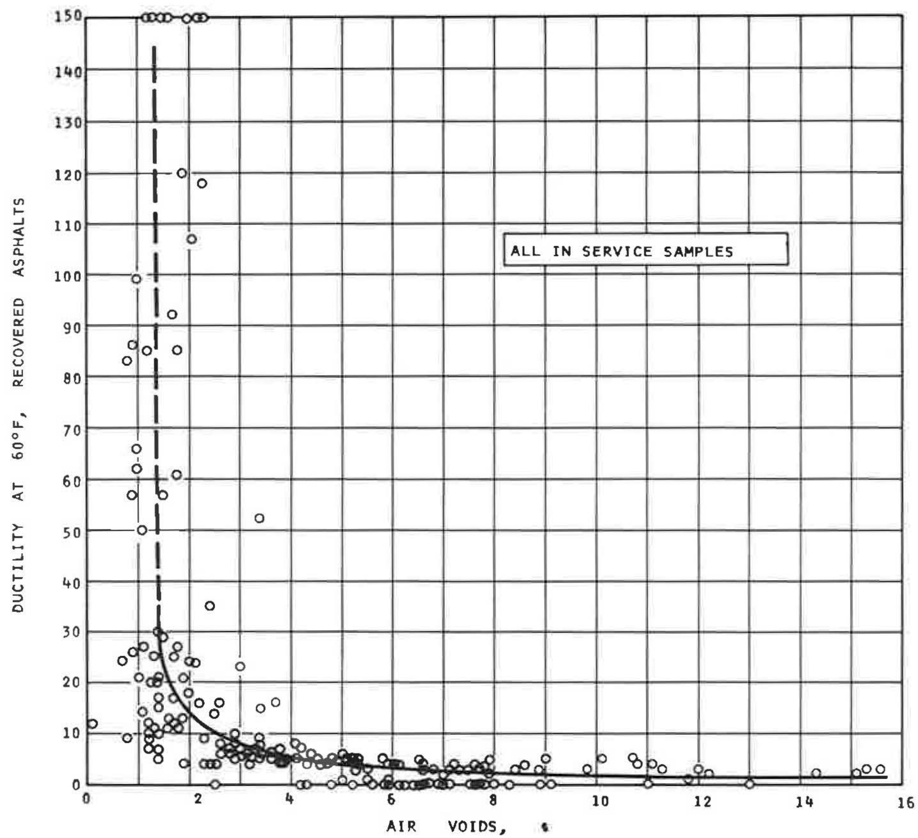

Figure 8. Relation of ductility at 60 F after field aging to air voids.

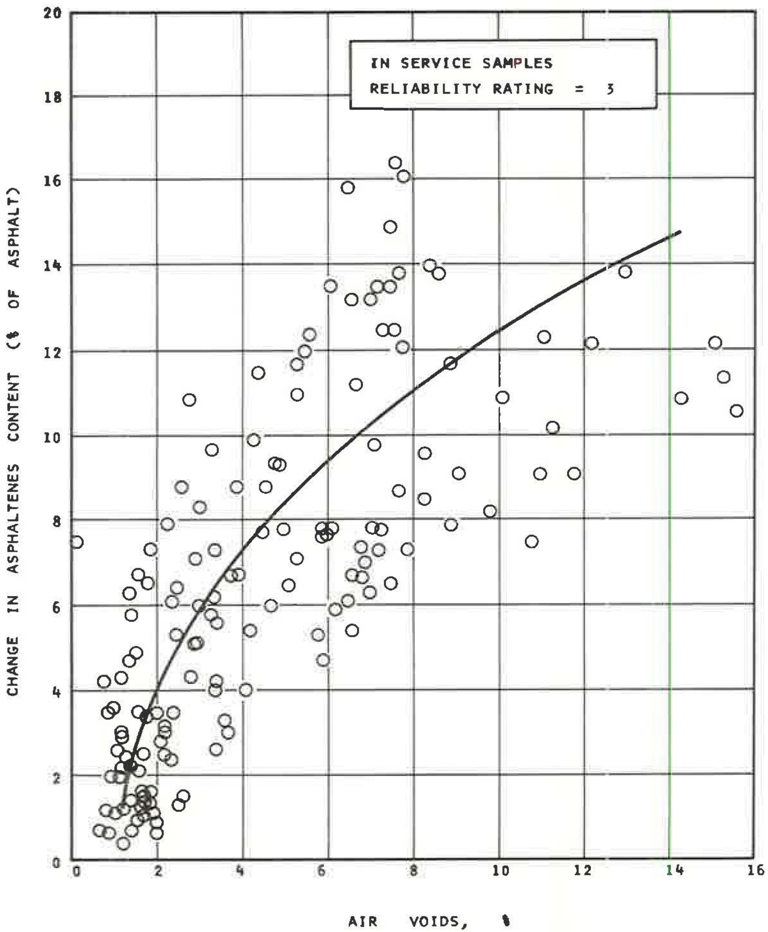

Figure 9. Relation of change in asphaltenes content during field aging to air voids.

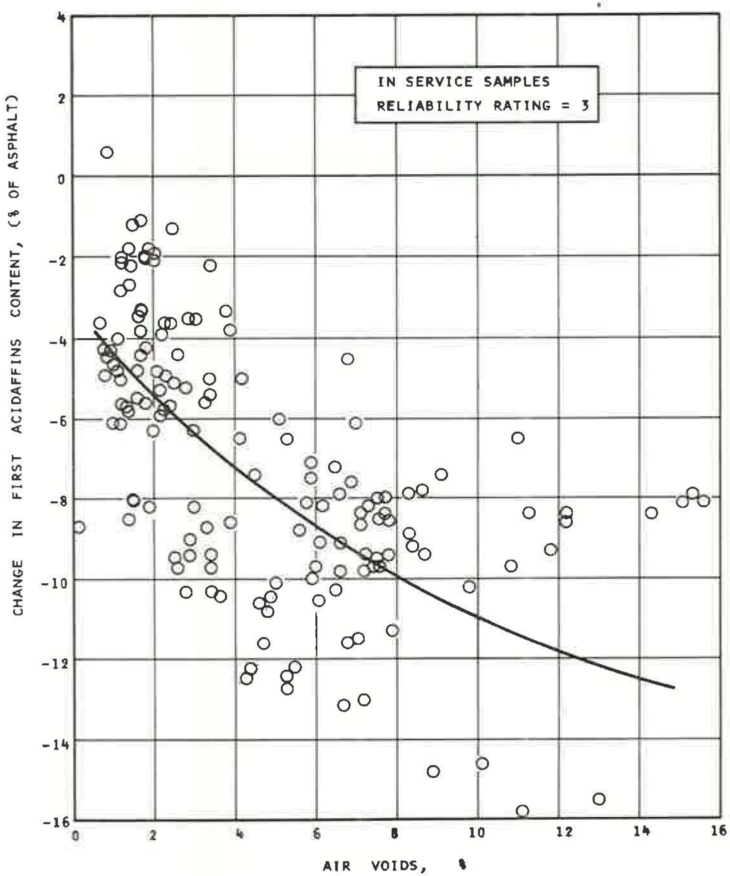

Figure 10. Relation of change in first acidaffins content to air voids.

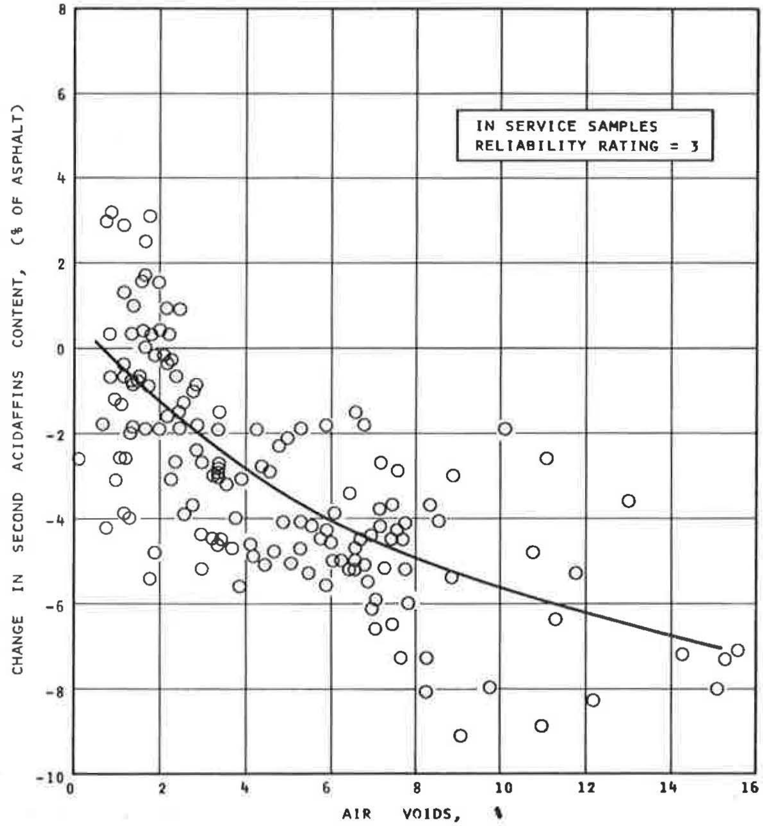

Figure 11. Relation of change in second acidaffins content to air voids.

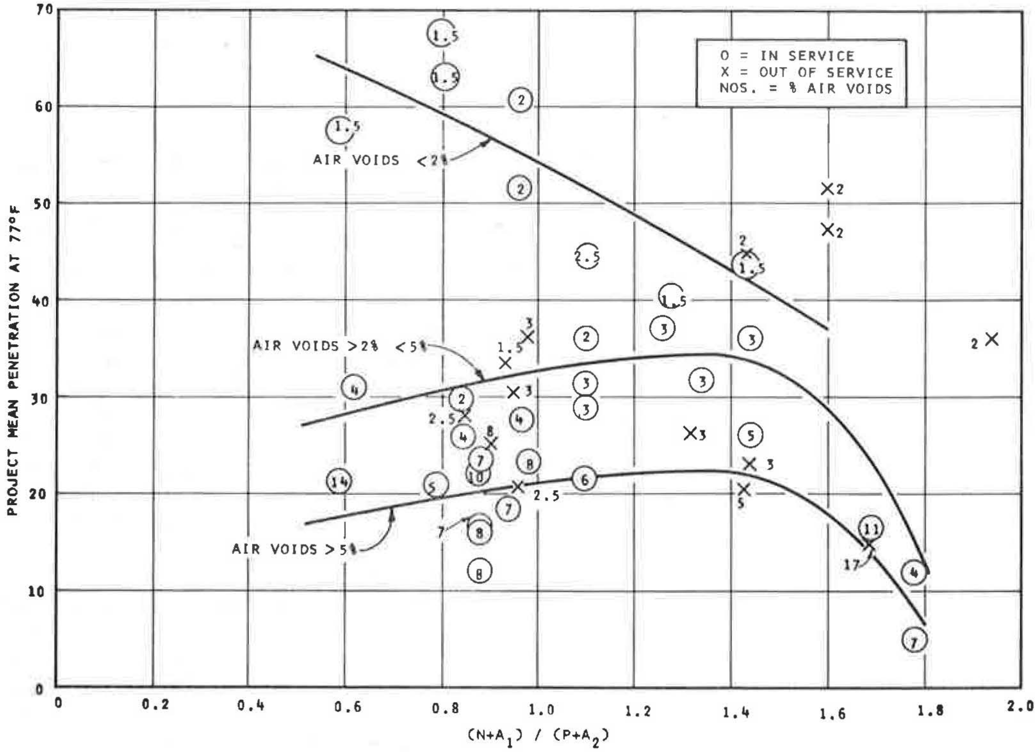

Figure 12. Relation of retained penetration after field aging to Rostler parameter of original asphalt.

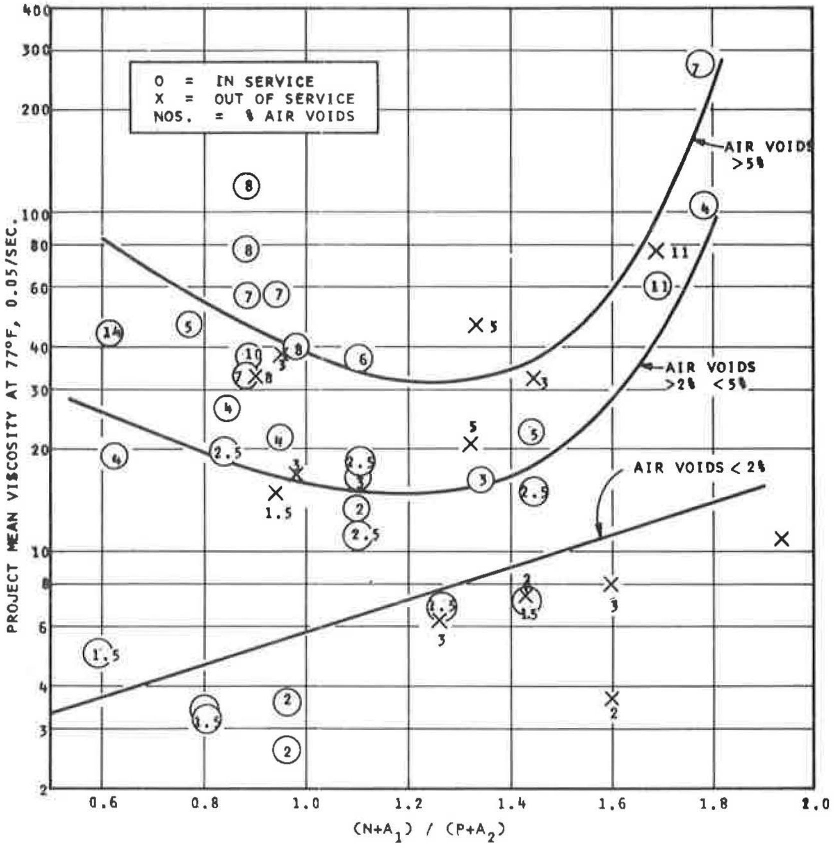

Figure 13. Relation of viscosity at 77 F after field aging to Rostler parameter.

indicated that increase in hardening may occur with low parameter values. The lowest amount of hardening appears to occur in a Rostler parameter range of about 1.0 to 1.5.

9. In the two construction projects where transverse variability was investigated, variability of recovered asphalt properties transversely across the pavement was found to be quite high in accord with the following relationships: (a) The recovered asphalts were generally harder in the wheelpaths of the traffic lane than between the wheelpaths or in the adjacent passing lane, i.e., lower penetration, lower ductility, higher softening point, and higher viscosity; and (b) the recovered asphalts from the traffic-carrying pavement areas were generally lower in paraffins and second acidaffins and higher in asphaltenes fractions. These relationships are consistent with the hardening trends indicated in (a). At most of the sample locations in the two transverse variability experiments, the air voids were found to be higher in the wheelpaths where the greater changes in asphalt properties occurred. This observation may be the result of a coincidence of occurrences and not a generally valid rule because it is contrary to what would normally be expected. It is generally assumed that the kneading action of traffic would compact the mixture in the wheelpath to a lower voids content.

10. The relative severity of laboratory and field aging of the asphalt was found to be relatable to the voids content of the asphalt concrete mixture in the field pavements. For various laboratory aging procedures, the following relationships were developed:

Modified thin-film oven test—The 7-hour test is approximately equivalent to 11 to 13 years of field aging at a field air voids content of 3 to 5 percent. The 9-hour test is approximately equivalent to 11 to 13 years of field aging at a voids content of 4 to 6 percent. Field aging was less severe than laboratory aging at lower voids contents and more severe at higher voids contents. These relationships were found to hold regardless of other mixture or environmental factors.

California weathering oven—A 200-hour period of weathering in the California weathering oven is more severe than 11 to 13 years of field weathering of asphalts in asphalt concrete pavements having less than 2 percent air voids. A 1,000-hour period of oven weathering is less severe than 13 years of field aging of asphalts in mixtures with more than 5 percent air voids and approximately equivalent for mixtures with 4 to 5 percent air voids. Intermediate periods of oven weathering (200 to 1,000 hours) are representative of 11 to 13 years of field aging of asphalts in asphalt concrete pavements with air voids contents of 2 to 4 percent.

Thin-film oven test—As expected, weathering of asphalts in the thin-film oven test proved to be less severe than either the modified thin-film oven test or the California weathering oven. In all cases, except where the air voids in the asphalt concrete were less than 2 percent, the thin-film oven test weathering was less severe than aging of asphalts for 11 to 13 years in asphalt pavements in the field. If the postulate is accepted that the thin-film oven test is essentially comparable to aging of asphalt in the pug mill, the data suggest that, in asphalt concrete pavements with less than 2 percent air voids, field aging of asphalt is negligible in service.

MR and D infrared oven—A 7-day weathering period in the infrared oven produces changes in chemical composition of original asphalts similar to those produced in 11 to 13 years of field aging in asphalt concrete pavements but with significant differences in the changes occurring in some individual components. Relative severity of aging between laboratory and field, however, is again dependent on the air voids content. The following differences between rate of change in field aging and in MR and D oven were found in the individual fractional components: asphaltenes, less increase at less than 4 percent and more at more than 4 percent air voids in the field pavements; nitrogen bases, little change during laboratory aging and consistent increase in field aging relatively independent of voids content in asphalt concrete pavement; first acidaffins, consistent decrease during both laboratory and field aging, of same order at air voids less than 4 percent but decrease always greater in field aging at air voids above 4 percent; and paraffins and second acidaffins, consistently greater loss of these fractions in field aging for voids greater than 4 percent, but data do not give a clear trend at air voids less than 4 percent because changes appear to be of different types. Although the end result of all changes in chemical composition are in the direction of increased hardness of the asphalt, it appears that different types of chemical changes are taking place in the short-term, high-temperature laboratory aging as compared to long-term, low-temperature field aging. These differences are most likely attributable to rain, which causes water-soluble, intermediate reaction products to be removed from the asphalt during various stages of weathering.

11. All of the pavement evaluation ratings obtained from the field survey were compared with the following: properties of original asphalts; properties of recovered asphalts; properties of the pavement samples; climatological factors including deg-days/year below 65 F, deg-days/year above 89 F, and normal total precipitation; and average daily traffic. No overall correlations were found in the comparisons. However, some trends were noted during the preliminary analysis that were used as guides in carrying out specific analyses that resulted in the following findings: (a) A significant correlation was found between the penetration and ductility at 60 F and the rating for raveling in projects subjected to more than 5,000 deg-days/year below 65 F, and severe raveling was found where the penetration of the recovered asphalt was 10 or less and the ductility 3 or less; (b) a similar relation was found for low ductility at 60 F and severe spalling in colder climates, but the correlation was less pronounced than for raveling; (c) a relation between bitumen index and raveling was found, with severe raveling occurring at values less than 15 and minimal raveling occurring at values greater than 18; (d) no numerical correlation could be developed between voids content and any of the pavement evaluation ratings, even though air voids were found to have considerable bearing on asphalt hardening; (e) no correlations were found between any types of pavement distress and exposure to warm climates (high deg-days/year above 89 F); (f) no significant

correlations were found between pavement distress and normal total precipitation, except in the 4 projects of lowest rainfall where raveling appeared to be associated with low asphalt viscosity and high deg-days/year lower than 65 F; and (g) for asphalt pavements over portland cement concrete base, greater loss of matrix appeared to be associated with increased hardness of asphalt expressed as lower penetration and ductility and higher viscosity.

12. A comparison of the properties of asphalt with the condition of the pavements separated into groups according to traffic load yielded some interesting correlations. Only data from cores and condition ratings in the outer wheelpath of the outer lane were considered. The correlations found were as follows: (a) Both raveling and longitudinal load cracking increased with higher Rostler parameter of the original asphalt for traffic less than 2,000 vehicles/day; (b) a correlation was found between the viscosity of the recovered asphalt at 140 F and the rating for loss of matrix (in pavements with more than 3,000 vehicles/day), and loss of matrix was also found to be higher at air voids above 4 percent, but the correlation is not as good as with high viscosity at 140 F; and (c) a potential correlation was discerned between severity of longitudinal cracking and lower viscosity of recovered asphalt at 140 F and lower air voids content in pavements with traffic exceeding 25,000 vehicles/day, but the data were insufficient to verify this correlation because only 4 pavements were available for observation in this traffic grouping.

13. An analysis of the data to find overall relationships between pavement condition ratings and either traffic loading or pavement structure as built did not prove fruitful. The only relationship shown by the data is that loss of matrix may be related to traffic intensities, being less severe in higher trafficked areas. This was found in a comparison of projects having asphalts of similar viscosities at 140 F. The same relationship was evident in interior passing lanes as compared to exterior traffic lanes. Correlations between pavement distress and asphalt properties at low or high traffic intensities have been discussed earlier.

14. Attempts to segregate the data by geographical regions to determine any variation found in cause-and-effect phenomena attributable to geographical region proved to be nonproductive. However, this did not invalidate the findings as to specific climatic effects in other analyses because geographical regions comprise a great variety of climatic conditions.

15. The generally held view that a slower rate of aging of asphalt cements occurs in those pavements that have been overlaid was confirmed in 2 specific projects. However, the remainder of the data analyzed did not show significant changes in rate of aging attributable to overlaying. Because the actual length of service life was unknown for over half of the out-of-service pavements, this analysis could not be deeply explored.

16. A comparison of properties of the most recent collection of asphalts representative of current asphalt production revealed that they were within the range of properties of the 1954-1955 series. Only a few asphalts were significantly different, and these changes can be traced to changes in refinery processes and, therefore, do not invalidate applicability of the findings of the present research study.

CONCLUSIONS

The major purpose of this investigation was to conduct a field survey of pavements to determine the overall changes in the physical and chemical properties of asphalts occurring as a result of construction and more than 10 years of service in pavements. The data collected and the analyses conducted to date permit the following overall conclusions:

1. The most important factor in hardening of the binder in a pavement is voids content of the pavement. Although this factor has been previously noted by other research reports, the quantitative evaluations obtained in this study for essentially the complete range of asphalts produced in the United States are significant. In pavements of below 2 percent voids, field aging during 11 to 13 years of service subsequent to hardening in pug-mill mixing and laydown operations appeared to have been negligible. Above this level, hardening increased with higher air voids.

2. For pavements with low voids contents (2 percent or less), a significant trend for increased hardening for higher values of the Rostler parameter $(N + A_1)/(P + A_2)$ was noted; for higher voids content, the effects of asphalt composition as indicated by the parameter were overshadowed by other effects. However, the optimal value for minimum hardening appears to be between 1.00 and 1.40. This latter observation is believed significant. Because the great majority of asphalts in this study had parameter values within the optimum range, the conclusion drawn must be that present asphalt manufacturing processes normally result in satisfactory chemical compositions and not that chemical composition is an unimportant durability parameter.

3. Comparison of field-aging with laboratory-aging tests shows that both the California weathering oven aging (1,000 hours) and the FHWA modified thin-film oven aging (7 hours) resulted in hardening of the asphalt to a degree comparable to 11 to 13 years of field aging in a pavement with about 4 percent air voids. Hardening during field aging had been less severe at lower air voids content and much more severe at higher voids content than that occurring during laboratory aging.

4. The type of chemical change taking place in an asphalt in long-term field aging differs somewhat from that taking place in short-term, laboratory aging. Although formation of asphaltenes occurs in both cases, the disappearance of the second acidaffins fraction and the increase in the nitrogen bases fraction appear to occur to a much greater extent in field aging than in accelerated laboratory aging.

5. No direct relationship appears to exist between voids content and pavement distress, although some limited correlations were found between some types of distress and properties of the recovered asphalt that had been found to depend on voids content. This conclusion must also be considered from the viewpoint that the ranges of voids and the pavements studied were within those obtained by "good" construction practice. Poor mix design or inadequate construction procedures most likely would have given different results.

6. Attempts to correlate observed pavement distress with construction variables, environment, traffic history, or original or present asphalt properties were thwarted by the large number of variables between projects. Although a few likely correlations were obtained when some variables were held constant, the resulting number of projects available for comparison was always too small to permit conclusions of general validity. Variations of pavement evaluations within projects, where variables were fewer, were in most cases too small to be significant. A much better controlled experiment, with variables held to a minimum, would be required to establish generally valid rules pertaining to causes of pavement distress.

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The opinions, findings, and conclusions expressed in this report are those of the authors and not necessarily those of the Federal Highway Administration.

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