HIGHWAY RESEARCH RECORD

Number 150

Asphalt Pavements, Bases and Surface Treatments

4 Reports

Subject Classification

31 Bituminous Materials and Mixes
33 Construction
40 Maintenance, General

HIGHWAY RESEARCH BOARD

DIVISION OF ENGINEERING NATIONAL RESEARCH COUNCIL
NATIONAL ACADEMY OF SCIENCES—NATIONAL ACADEMY OF ENGINEERING

Washington, D. C., 1966 Publication 1425

Department of Materials and Construction

R. L. Peyton, Chairman
Assistant State Highway Engineer
State Highway Commission of Kansas, Topeka

HIGHWAY RESEARCH BOARD STAFF

R. E. Bollen, Engineer of Materials and Construction
W. G. Gunderman, Assistant Engineer of Materials and Construction

BITUMINOUS DIVISION

William H. Goetz, Chairman
Joint Highway Research Project
Purdue University, Lafayette, Indiana

COMMITTEE ON BITUMINOUS SURFACE TREATMENTS
(As of December 31, 1965)

Moreland Herrin, Chairman
Department of Civil Engineering, University of Illinois, Urbana

D. W. Anderson, Assistant Research Engineer, South Dakota Department of Highways, Pierre
E. W. Bauman, Managing Director, National Slag Association, Washington, D. C.
J. E. Bell, Division of Materials Development and Research, D. C. Department of Highways and Traffic, Washington, D. C.
C. W. Chaffin, Supervising Chemical Engineer, Materials and Tests Division, Texas Highway Department, Austin
Robert A. Crawford, Assistant Research Engineer, South Dakota Department of Highways, Pierre
Leslie B. Crowley, Senior Consultant, Directorate of Civil Engineering, Headquarters U. S. Air Force, Washington, D. C.
John B. Dunbar, District Engineer, The Asphalt Institute, Montgomery, Alabama
Bob M. Gallaway, Research Engineer, Civil Engineering Department, Texas Transportation Institute, Texas A and M University, College Station
B. C. Hartronft, Research and Development Engineer, Oklahoma Department of Highways, Oklahoma City
David C. Mahone, Highway Research Engineer, Virginia Council of Highway Investigation and Research, Charlottesville
Phillip L. Melville, Civil Engineering Branch, Engineering Division, Military Construction, Office, Chief of Engineers, Department of the Army, Washington, D. C.
William H. Mills, Consulting Engineer, Atlanta, Georgia
J. W. Reppel, Engineer of Maintenance, Ohio Department of Highways, Columbus
James M. Rice, Highway Research Engineer, Division of Physical Research, U. S. Bureau of Public Roads, Washington, D. C.
Ernest Zube, Supervising Materials and Research Engineer, California Division of Highways, Sacramento

COMMITTEE ON RELATION OF PHYSICAL CHARACTERISTICS OF BITUMINOUS MIXTURES TO PERFORMANCE OF BITUMINOUS PAVEMENTS

(As of December 31, 1965)

A. W. Eatman, Chairman
Materials and Tests Engineer
Materials and Tests Division, Texas Highway Department, Austin

Verdi Adam, Research and Development Engineer, Louisiana Department of Highways, Baton Rouge
R. W. Beaty, Chief Paving Engineer, Barber-Greene Company, Aurora, Illinois
Joseph E. Bell, Division of Materials Development and Research, D. C. Department of Highways and Traffic, Washington, D. C.
Charles R. Foster, Coordinator of Research, National Asphalt Pavement Association, Texas A and M University, College Station
Bob M. Gallaway, Research Engineer, Civil Engineering Department, Texas Transportation Institute, Texas A and M University, College Station
Bernard F. Kallas, The Asphalt Institute, University of Maryland, College Park
J. Hode Keyser, Chief Engineer, City of Montreal, Control and Research Laboratory, Montreal, Canada
C. T. Metcalf, Shell Development Company, New York, N. Y.
Carl L. Monismith, University of California, Berkeley
James M. Rice, Highway Research Engineer, Division of Physical Research, U. S. Bureau of Public Roads, Washington, D. C.
Harry A. Sandberg, Jr., Engineer of Laboratory Operations, Texas Highway Department, Austin
B. A. Vallerga, Director of Engineering, Materials Research and Development, Woodward-Clyde-Sherard and Associates, Oakland, California
Ernest Zube, Assistant Materials and Research Engineer—Pavement, California Division of Highways, Sacramento

COMMITTEE ON BITUMINOUS AGGREGATE BASES

(As of December 31, 1965)

Paul J. Serafin, Chairman
Bituminous Engineer
Michigan State Highway Department, Ann Arbor

K. O. Anderson, Department of Civil and Municipal Engineering, University of Alberta, Edmonton, Alberta, Canada
Harold E. Bessey, Bituminous Materials Engineer, Maine State Highway Commission, Augusta
Gail C. Blomquist, School of Civil Engineering, Michigan State University, East Lansing
J. T. Corkill, Senior Materials Engineer, Materials and Research Section, Department of Highways, Toronto, Ontario, Canada
Charles R. Foster, Coordinator of Research, National Asphalt Pavement Association, Texas A and M University, College Station
J. E. Gray, Engineering Director, National Crushed Stone Association, Washington, D. C.
James H. Havens, Director of Research, Kentucky Department of Highways, Lexington
Erling Henrikson, Construction Engineer, North Dakota State Highway Department, Bismarck
Edward T. Hignell, Asphalt Technical Advisor, British American Oil Company Ltd., Toronto, Ontario, Canada
Hyoungkey Hong, Assistant Professor of Civil Engineering, Marquette University, Milwaukee, Wisconsin

Rudolf A. Jimenez, Department of Civil Engineering, The University of Arizona, Tucson
James C. Johnson, Staff Engineer, The Asphalt Institute, University of Maryland, College Park
J. A. Lefebvre, Imperial Oil Enterprises Ltd., Research Department, Sarnia, Ontario, Canada
K. E. McConnaughay, President, K. E. McConnaughay, Inc., Lafayette, Indiana
T. E. McElherne, Materials Engineer, Iowa State Highway Commission, Ames
J. A. Murchie, Area Maintenance Engineer, Minnesota Department of Highways, Minneapolis
Charles F. Parker, Chief Engineer, Engineering Laboratory Service, Westbrook, Maine
Edward T. Perry, Administrative Vice President, The New Haven Trap Rock Company, New Haven, Connecticut
Robert M. Ripple, Technical Service Engineer, American Bitumuls and Asphalt Company, Cincinnati, Ohio
Stewart R. Spelman, Highway Research Engineer, Physical Research Division, U. S. Bureau of Public Roads, Washington, D. C.
J. E. Stephens, Professor of Civil Engineering, University of Connecticut, Storrs
Lansing Tuttle, Chief of Technical Service, Warren Brothers Company, Cambridge, Massachusetts
W. B. Warden, President, Miller-Warden Associates, Raleigh, North Carolina
George H. Zuehlke, Materials Tests Engineer, State Highway Commission of Wisconsin, Madison

Foreword

The four papers included in this RECORD, while seeming to cover a variety of subjects, have a unity which stems from their fulfillment of need for information of practical and immediate value to the bituminous engineer. The designer of asphalt pavements and those responsible for the maintenance and rehabilitation of them will also find the presentations of much interest.

The papers by Kallas and Manz each contribute basic temperature data that have long been lacking with respect to asphalt mix and flexible pavement design. This need stems from the fact that the mechanical properties of asphalt paving mixtures are temperature dependent. This has been recognized with respect to maximum surface temperatures where 140 F has been used for stability testing in the laboratory. The data presented support the use of this temperature for surface conditions, but indicate clearly that for the stability testing of mixtures placed lower in the pavement structure, lower temperatures are appropriate. With the use of thicker asphalt paving layers in today's heavy-duty highways this information is pertinent.

In these researches, variations in pavement temperatures with time are also presented and cold conditions as well as hot ones are considered. The insulating effects of asphalt pavements in relation to the frost problem are also reported. As more and more becomes known about the specific effects of temperature on the mechanical properties of bituminous mixtures, data such as those provided will become more and more valuable.

The paper by Johnson on the use of bituminous macadam as a thin overlay to control reflex cracking is pointed specifically to one of the most frustrating problems faced by pavement maintenance engineers in cold climates. The success indicated for the method employed will be of immediate interest to all concerned with such problems.

The study by Gallaway and Harper on the use of lightweight aggregates as coverstone for application treatments will be of interest to those in many areas where more conventional coverstone may not be available in sufficient supply or is expensive. Also, the information brought out with tests for relative windshield damage as compared to harder materials and other specific advantages will be of special interest to some. Students of relationships between aggregate properties and their performance in seal coat and surface treatment work will find much of interest in this paper.

Contents

Asphalt Pavement Temperatures

B. F. KALLAS, Research Engineer, The Asphalt Institute, College Park, Maryland

The use of thicker asphalt paving courses in heavy-duty highways has resulted in the need for more information on temperature variations in pavement structures. Temperature data are necessary in studies of pavement deflections, stresses and strains under moving wheel loads. Pavement temperature data are of interest in any study or test involving the temperature-dependent mechanical properties of paving mixtures or paving asphalts.

The pavement temperature studies reported here were conducted to provide information on temperature variations in thicker asphalt-concrete pavements that would be applicable in many areas of the United States. Pavement temperatures were measured at the surface and at depths of 2, 4, and 6 in. in a 6-in. thick asphalt-concrete pavement, and at depths of 2, 4, 6, 8, 10 and 12 in. in a 12-in. thick asphalt-concrete pavement. A temperature recorder was used to record air and pavement temperatures at the test site in College Park, Maryland, from June 1, 1964, to May 31, 1965.

The durations of various temperature levels, and maximum, minimum and average temperatures based on hourly temperatures at the various depths, are reported for each month and for the entire year. Test data and information on daily pavement temperature changes with depth, and rates of temperature change, are also reported.

•ASPHALT PAVING courses totaling 6 in. or more in thickness are now being constructed on many heavy-duty highways. Because of these recent developments in asphalt paving structural design there is a need for more information on temperature variations in thicker asphalt pavements. Studies in Louisiana of pavement temperatures at depths of ½ and 2 in. have been reported by Arena (1). A method for calculating maximum pavement surface temperatures from weather reports has been presented by Barber (2), and pavement temperatures were considered by Monismith et al. (3) in their investigations of thermal stresses and deformations in asphalt concrete. However, little temperature data are available for thicker asphalt pavements. Pavement temperature data are necessary in studies of deflections, stresses and strains in pavements subjected to moving wheel loads. Pavement temperature data are also of interest in laboratory studies or testing involving the mechanical behavior of asphalts or paving mixtures.

This paper presents pavement temperature data obtained from a year-long study by The Asphalt Institute. The studies were made from June 1, 1964, to May 31, 1965, on pavement test sections located at College Park, Maryland.

DESCRIPTION OF TEST SECTION

An asphalt concrete test section 12 ft wide and 24 ft long was used for the study (Fig. 1). It was constructed in a location not shielded from the sun and wind. A 10-ft

Paper sponsored by Committee on Relation of Physical Characteristics of Bituminous Mixtures to Performance of Bituminous Pavements and presented at the 45th Annual Meeting.

Figure 1. Asphalt-concrete pavement test section.

length of the section was constructed 6 in. thick. A transition section 2 ft long was used and the remaining 12-ft length of the section was constructed 12 in. thick. The test section was built adjacent to a parking lot which had a 4-in. thick asphalt-concrete surface. An asphalt curb was constructed over the joint between the test section and the parking lot. The section was built with conventional construction equipment. Asphalt concrete was used the full depth of the test section and was placed directly on the graded and compacted sandy-silt soil subgrade. The asphalt concrete was placed with a paver in 3-in. thick courses. Test results on the asphalt-concrete paving mixture and pavement core samples are given in Table 1.

INSTRUMENTATION

Standardized insulated iron and constantan thermocouple wires were installed with the temperature-measuring junctions at depths of 2, 4 and 6 in. in the 6-in. thick pavement, and at depths of 2, 4, 6, 8, 10 and 12 in. in the 12-in. pavement. The installations were made by cutting 4-in. diameter holes with a core drill to desired depths. The holes were dried, painted with hot asphalt, and the thermocouples were placed vertically with their temperature measuring junctions at the bottom of the holes. Paving mixture from the original construction, heated to 250 F, was placed in the holes and compacted with a Marshall Test Method compaction hammer. Hot asphalt cement was used to seal areas where the thermocouple wires emerged from the pavement surface. Periodic sealing of these areas and coating of exposed thermocouple wires with asphalt provided adequate thermocouple protection during the experiment. A surface temperature thermocouple wire was placed directly on the pavement surface with the temperature sensitive junction bent at a 90-deg angle and inserted about $\frac{1}{8}$ in. into the pavement. A small amount of asphalt used periodically to seal the thermocouple protected it during the experiment. The buried thermocouples were located in the middle of the two sections and were spaced 16 in. apart on centers. The surface temperature

TABLE 1

TESTS OF PAVING MIXTURES AND CORES

(a) Extraction Tests on Paving Mixture and Tests on Recovered Asphalt

Aggregate Analysis	Sieve Size	Percent Passing
	½ in.	100
	⅜ in.	78
	No. 4	55
	No. 8	45
	No. 16	37
	No. 30	30
	No. 50	12
	No. 100	7
	No. 200	5
Percent asphalt, total mix		4.9
Recovered asphalt penetration at 77 F (100 gm, 5 sec)		42
Recovered asphalt ductility at 77 F (5 cm/min)		142
Recovered asphalt softening point, deg F (ring and ball)		138

(b) Tests on Pavement Cores

Maximum specific gravity of paving mixture (ASTM test method D 2041)	2.509
Bulk specific gravity (avg for 8 cores)	2.281
Air voids, percent (avg for 8 cores)	9.1
Marshall stability at 140 F, lb (avg for 3 cores)	437
Marshall flow value, 0.01 in. (avg for 3 cores)	17
Hveem stability value at 140 F (avg for 2 cores)	23
Hveem cohesiometer value at 140 F (avg for 2 cores)	100
Unconfined compressive strength, psi (0.05 in./in. rate of loading):	
At 39 F	975
At 77 F	299
At 120 F	48
Tensile strength, psi (0.05 in./in. rate of loading):	
At 39 F	350
At 77 F	77
At 120 F	6

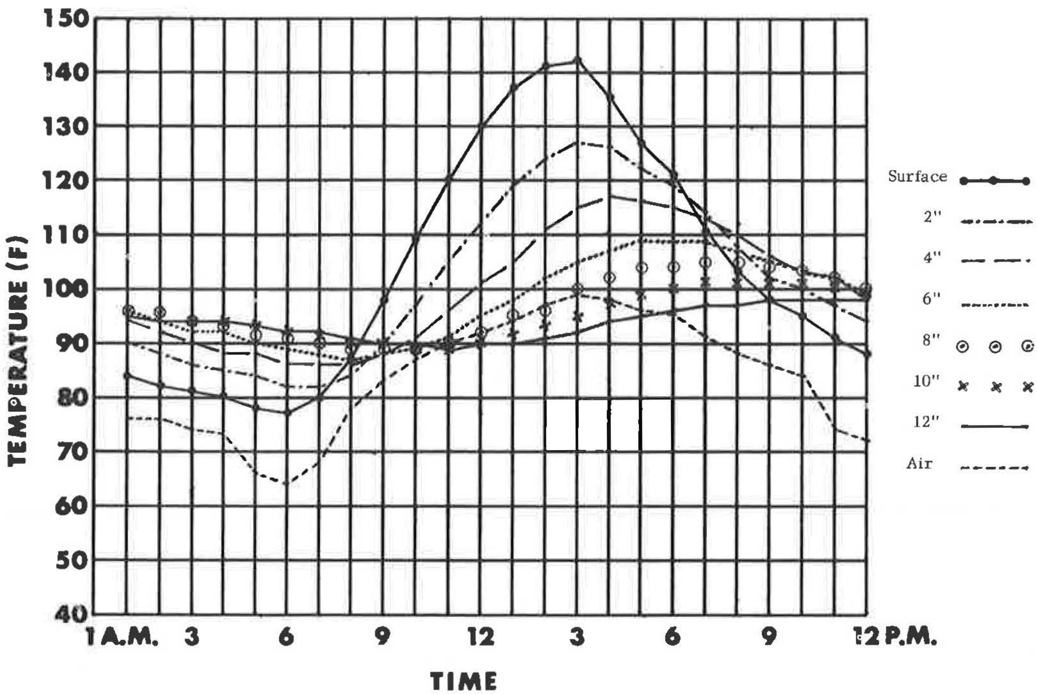


Figure 2. Asphalt-concrete pavement temperatures on June 30, 1964.

TABLE 2

HOURLY PAVEMENT TEMPERATURE (°F)

(June 30, 1964)

Time	Air	Surface	12 In. Section (Depths, in.)						6 In. Section (Depths, in.)		
			2	4	6	8	10	12	2	4	6
1 AM	76	84	90	94	96	96	96	95	90	94	96
2 AM	76	82	88	92	94	96	96	94	88	92	94
3 AM	74	81	86	90	92	94	94	94	86	90	92
4 AM	73	80	85	88	92	93	94	94	84	88	92
5 AM	66	78	84	88	90	92	93	93	84	87	90
6 AM	64	77	82	86	89	91	92	92	82	86	89
7 AM	68	80	82	86	88	90	91	92	82	86	88
8 AM	78	87	84	86	87	89	90	91	84	86	87
9 AM	83	98	90	88	88	89	90	90	90	88	88
10 AM	87	109	97	91	89	89	89	90	97	91	89
11 AM	90	120	105	96	91	90	89	89	105	96	91
12 Noon	92	130	112	101	95	92	90	90	112	101	95
1 PM	94	137	119	105	98	95	92	90	119	106	98
2 PM	97	141	124	111	102	96	93	91	125	111	101
3 PM	99	142	127	115	105	100	95	92	128	115	105
4 PM	98	135	126	117	107	102	97	94	127	117	106
5 PM	96	127	122	116	109	104	99	95	122	116	108
6 PM	95	121	119	115	109	104	100	96	120	115	108
7 PM	91	111	114	113	109	105	101	97	115	113	109
8 PM	88	103	108	110	107	105	101	97	108	110	107
9 PM	86	98	102	106	105	104	101	98	103	104	105
10 PM	84	95	100	103	103	103	101	98	100	103	103
11 PM	74	91	97	102	102	102	100	98	97	102	102
12 PM	72	88	94	98	99	100	99	98	94	98	99

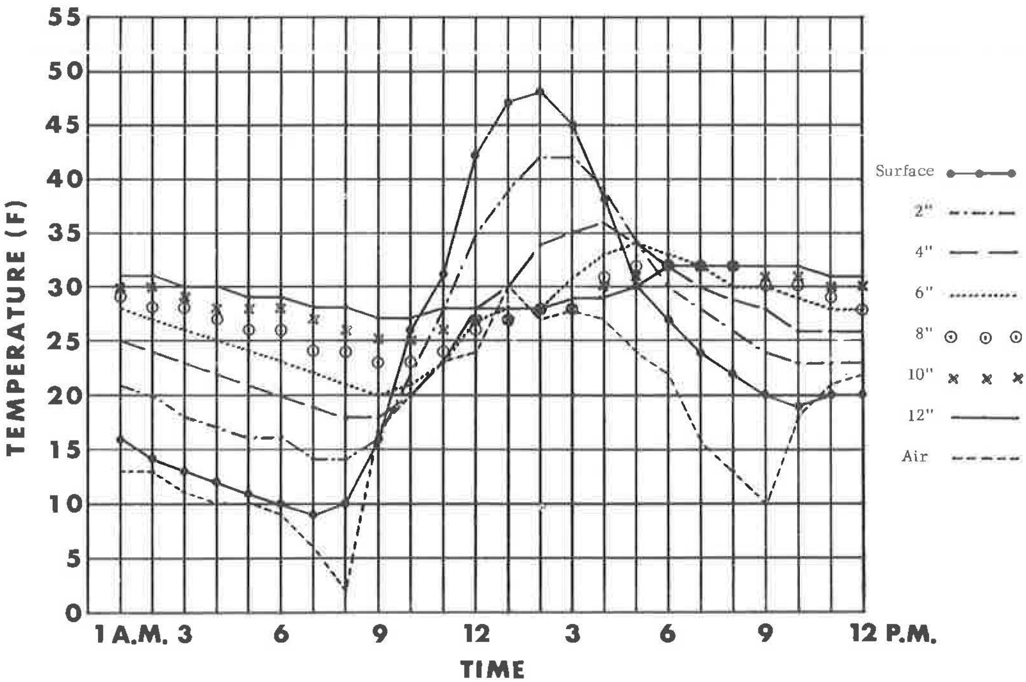


Figure 3. Asphalt-concrete pavement temperatures on January 19, 1965.

TABLE 3

HOURLY PAVEMENT TEMPERATURES (° F)
(January 19, 1965)

Time	Position Where Measurements Were Made							
	Air	Surface	12 In. Section (Depths, in.)					
			2	4	6	8	10	12
1 AM	13	16	21	25	28	29	30	31
2 AM	13	14	20	24	27	28	30	31
3 AM	11	13	18	23	26	28	29	30
4 AM	10	12	17	22	25	27	28	30
5 AM	10	11	16	21	24	26	28	29
6 AM	9	10	16	20	23	26	28	29
7 AM	6	9	14	19	22	24	27	28
8 AM	2	10	14	18	21	24	26	28
9 AM	17	16	16	18	20	23	25	27
10 AM	20	26	22	20	21	23	25	27
11 AM	23	31	28	23	23	24	26	28
12 Noon	24	42	35	28	27	26	27	28
1 PM	30	47	39	30	28	27	27	28
2 PM	27	48	42	34	28	28	28	28
3 PM	28	45	42	35	31	28	28	29
4 PM	27	38	39	36	33	31	30	29
5 PM	24	30	34	34	34	32	31	30
6 PM	22	27	30	32	33	32	32	32
7 PM	16	24	28	30	32	32	32	32
8 PM	13	22	26	29	30	32	32	32
9 PM	10	20	24	28	30	30	31	32
10 PM	18	19	23	26	29	30	31	32
11 PM	21	20	23	26	28	29	30	31
12 PM	22	20	23	26	28	28	30	31

TABLE 4

PAVEMENT TEMPERATURES (° F) DURING A RAIN STORM
(June 8, 1964)

Time (PM)	Position Where Measurements Were Made							
	Air	Surface	12 In. Section (Depths, in.)					
			2	4	6	8	10	12
1:00	84	124	106	93	87	82	80	79
2:00	85	120	108	97	90	85	82	80
2:05	84	117	108	98	90	85	82	80
2:10	77	110	108	98	91	85	82	80
2:15	68	102	107	98	91	85	82	80
2:20	64	93	105	98	91	86	82	80
2:25	63	86	103	98	92	86	83	81
2:30	65	83	99	98	92	86	83	81
2:35	66	83	97	98	92	86	83	81
2:40	66	83	95	97	92	87	83	81
2:45	66	83	93	96	92	87	84	81
2:50	66	83	92	96	92	87	84	81
2:55	66	82	91	95	92	87	84	81
3:00	66	80	91	94	92	87	84	81
3:05	66	80	90	94	92	87	84	81
3:10	66	80	89	93	91	87	84	81
3:15	66	81	88	93	91	88	84	82
3:20	67	82	88	92	91	88	84	82
3:25	68	83	88	92	91	88	84	82
3:30	68	83	87	91	90	87	84	82
4:00	74	93	89	89	89	87	84	82

TABLE 5

MONTHLY DURATION OF TEMPERATURE LEVELS, LOW, HIGH AND AVERAGE TEMPERATURES AT VARIOUS DEPTHS FOR 12-IN. ASPHALT CONCRETE PAVEMENT

Period During Which Measurements Were Made	Position Where Measurements Were Made	Percent of Month During Which The Temperature (° F) Was Between 0 & 9	10 & 19	20 & 29	30 & 39	40 & 49	50 & 59	60 & 69	70 & 79	80 & 89	90 & 99	100 & 109	110 & 119	120 & 129	130 & 139	140 & 149	Avg. Temp. (° F)	Avg. High Temp. (° F)	Avg. Low Temp. (° F)	High Temp. (° F)	Low Temp. (° F)
June 1964 (28 days)	Air	—	—	—	—	2	11	23	36	19	9	—	—	—	—	—	72	86	58	99	43
	Surface	—	—	—	—	—	1	10	23	22	12	10	8	8	5	1	93	124	69	142	55
	2-in. depth	—	—	—	—	—	—	5	21	29	17	15	9	4	—	—	90	112	73	127	61
	4-in. depth	—	—	—	—	—	—	2	17	35	26	15	5	—	—	—	89	104	77	119	65
	6-in. depth	—	—	—	—	—	—	1	16	41	29	13	—	—	—	—	88	98	79	109	68
	8-in. depth	—	—	—	—	—	—	—	16	44	32	8	—	—	—	—	88	95	81	105	70
	10-in. depth	—	—	—	—	—	—	—	17	49	31	3	—	—	—	—	87	92	82	101	71
	12-in. depth	—	—	—	—	—	—	—	16	58	26	—	—	—	—	—	86	89	82	98	72
July 1964 (25 days)	Air	—	—	—	—	—	4	29	35	26	6	—	—	—	—	—	75	86	63	98	53
	Surface	—	—	—	—	—	—	2	28	26	13	9	7	9	6	—	93	123	73	139	67
	2-in. depth	—	—	—	—	—	—	—	18	35	19	11	14	3	—	—	92	113	77	124	72
	4-in. depth	—	—	—	—	—	—	—	7	43	25	20	5	—	—	—	92	105	81	115	75
	6-in. depth	—	—	—	—	—	—	—	5	47	32	16	—	—	—	—	91	100	82	107	77
	8-in. depth	—	—	—	—	—	—	—	1	45	45	9	—	—	—	—	91	97	84	103	79
	10-in. depth	—	—	—	—	—	—	—	—	47	53	—	—	—	—	—	91	95	85	99	80
	12-in. depth	—	—	—	—	—	—	—	—	48	52	—	—	—	—	—	90	93	86	97	81
August 1964 (26 days)	Air	—	—	—	—	1	8	27	39	22	3	—	—	—	—	—	72	84	60	95	46
	Surface	—	—	—	—	—	—	11	31	21	12	12	8	5	—	—	88	116	69	133	60
	2-in. depth	—	—	—	—	—	—	4	29	31	17	13	6	—	—	—	87	106	73	120	65
	4-in. depth	—	—	—	—	—	—	—	21	42	25	12	—	—	—	—	87	99	77	108	70
	6-in. depth	—	—	—	—	—	—	—	16	53	28	3	—	—	—	—	86	94	78	103	70
	8-in. depth	—	—	—	—	—	—	—	7	62	31	—	—	—	—	—	86	92	81	98	74
	10-in. depth	—	—	—	—	—	—	—	4	71	25	—	—	—	—	—	86	90	82	96	76
	12-in. depth	—	—	—	—	—	—	—	1	82	17	—	—	—	—	—	86	89	83	93	78
September 1964 (30 days)	Air	—	—	—	—	3	27	28	24	14	4	—	—	—	—	—	67	79	53	94	43
	Surface	—	—	—	—	—	6	27	25	14	9	8	6	5	—	—	81	106	62	128	50
	2-in. depth	—	—	—	—	—	2	24	28	19	13	9	5	—	—	—	81	98	66	117	54
	4-in. depth	—	—	—	—	—	—	17	31	28	16	8	—	—	—	—	81	92	70	106	55
	6-in. depth	—	—	—	—	—	—	13	32	36	17	2	—	—	—	—	81	89	73	102	62
	8-in. depth	—	—	—	—	—	—	9	31	43	17	—	—	—	—	—	81	87	75	98	64
	10-in. depth	—	—	—	—	—	—	7	31	46	16	—	—	—	—	—	81	85	76	95	66
	12-in. depth	—	—	—	—	—	—	7	32	48	13	—	—	—	—	—	81	84	78	93	68
October 1964 (31 days)	Air	—	—	1	17	24	27	25	6	—	—	—	—	—	—	—	52	66	38	77	27
	Surface	—	—	—	1	21	29	23	13	9	4	—	—	—	—	—	61	84	46	105	36
	2-in. depth	—	—	—	—	12	34	33	14	7	—	—	—	—	—	—	61	78	50	95	40
	4-in. depth	—	—	—	—	3	33	46	16	2	—	—	—	—	—	—	62	72	53	87	44
	6-in. depth	—	—	—	—	2	32	56	10	—	—	—	—	—	—	—	62	69	55	82	47
	8-in. depth	—	—	—	—	—	25	67	8	—	—	—	—	—	—	—	63	68	58	79	50
	10-in. depth	—	—	—	—	—	20	75	5	—	—	—	—	—	—	—	63	67	59	76	52
	12-in. depth	—	—	—	—	—	12	85	3	—	—	—	—	—	—	—	64	66	61	75	54
November 1964	Air	—	1	6	23	24	25	18	3	—	—	—	—	—	—	—	48	62	33	76	16

	6-in. depth	—	—	—	4	19	62	15	—	—	—	—	—	—	—	—	53	59	47	67	33
	8-in. depth	—	—	—	—	17	71	12	—	—	—	—	—	—	—	—	54	58	49	64	30
	10-in. depth	—	—	—	—	16	73	11	—	—	—	—	—	—	—	—	54	57	51	63	40
	12-in. depth	—	—	—	—	14	77	9	—	—	—	—	—	—	—	—	55	57	53	62	42
December 1964 (31 days)	Air	—	3	17	30	32	14	3	1	—	—	—	—	—	—	—	39	47	30	72	14
	Surface	—	1	11	32	35	18	3	0	—	—	—	—	—	—	—	41	52	32	70	17
	2-in. depth	—	—	11	35	43	11	—	—	—	—	—	—	—	—	—	40	47	33	58	20
	4-in. depth	—	—	2	36	47	15	—	—	—	—	—	—	—	—	—	42	47	38	57	26
	6-in. depth	—	—	1	34	56	9	—	—	—	—	—	—	—	—	—	41	45	37	52	27
	8-in. depth	—	—	—	29	60	11	—	—	—	—	—	—	—	—	—	43	45	40	52	30
	10-in. depth	—	—	—	24	69	7	—	—	—	—	—	—	—	—	—	43	45	40	50	33
	12-in. depth	—	—	—	16	77	7	—	—	—	—	—	—	—	—	—	44	45	42	51	35
January 1965 (27 days)	Air	1	14	27	31	21	5	1	—	—	—	—	—	—	—	—	32	43	20	62	2
	Surface	—	5	28	35	20	11	1	—	—	—	—	—	—	—	—	35	50	24	61	9
	2-in. depth	—	1	24	43	25	7	—	—	—	—	—	—	—	—	—	35	47	26	56	14
	4-in. depth	—	1	19	47	32	1	—	—	—	—	—	—	—	—	—	36	43	30	53	18
	6-in. depth	—	—	15	52	32	1	—	—	—	—	—	—	—	—	—	37	42	31	52	20
	8-in. depth	—	—	8	55	36	1	—	—	—	—	—	—	—	—	—	37	41	34	50	23
	10-in. depth	—	—	5	57	38	—	—	—	—	—	—	—	—	—	—	38	40	35	49	25
	12-in. depth	—	—	2	58	40	—	—	—	—	—	—	—	—	—	—	38	40	36	48	27
February 1965 (25 days)	Air	1	4	23	33	24	11	4	—	—	—	—	—	—	—	—	37	47	26	68	6
	Surface	—	3	12	24	31	19	7	3	1	—	—	—	—	—	—	43	63	30	84	12
	2-in. depth	—	1	9	26	39	18	6	1	—	—	—	—	—	—	—	43	57	32	72	16
	4-in. depth	—	—	5	27	48	19	1	—	—	—	—	—	—	—	—	43	51	35	64	20
	6-in. depth	—	—	3	26	58	13	—	—	—	—	—	—	—	—	—	42	48	36	56	23
	8-in. depth	—	—	3	22	67	8	—	—	—	—	—	—	—	—	—	43	47	38	54	26
	10-in. depth	—	—	2	20	74	4	—	—	—	—	—	—	—	—	—	43	46	39	52	28
	12-in. depth	—	—	—	18	81	1	—	—	—	—	—	—	—	—	—	42	45	40	50	30
March 1965 (31 days)	Air	—	—	12	38	39	9	2	—	—	—	—	—	—	—	—	39	49	29	72	11
	Surface	—	—	3	24	36	14	11	8	4	—	—	—	—	—	—	49	71	34	89	19
	2-in. depth	—	—	1	18	43	18	15	5	—	—	—	—	—	—	—	49	65	36	80	23
	4-in. depth	—	—	—	11	46	32	10	1	—	—	—	—	—	—	—	49	59	40	70	28
	6-in. depth	—	—	—	7	52	37	4	—	—	—	—	—	—	—	—	49	56	41	69	30
	8-in. depth	—	—	—	3	54	40	3	—	—	—	—	—	—	—	—	48	54	43	62	34
	10-in. depth	—	—	—	1	60	38	1	—	—	—	—	—	—	—	—	48	52	44	60	36
	12-in. depth	—	—	—	—	64	36	—	—	—	—	—	—	—	—	—	48	51	45	58	39
April 1965 (26 days)	Air	—	—	3	8	40	31	11	7	—	—	—	—	—	—	—	50	60	37	76	22
	Surface	—	—	—	4	26	33	14	9	6	5	3	—	—	—	—	60	84	42	110	29
	2-in. depth	—	—	—	3	20	37	19	12	6	3	—	—	—	—	—	59	76	45	98	32
	4-in. depth	—	—	—	1	14	45	25	11	4	—	—	—	—	—	—	59	69	48	88	36
	6-in. depth	—	—	—	—	12	49	30	9	—	—	—	—	—	—	—	58	65	50	80	39
	8-in. depth	—	—	—	—	9	54	31	6	—	—	—	—	—	—	—	58	63	51	76	42
	10-in. depth	—	—	—	—	7	59	31	3	—	—	—	—	—	—	—	57	62	52	74	43
	12-in. depth	—	—	—	—	6	69	25	—	—	—	—	—	—	—	—	57	60	53	70	45
May 1965 (27 days)	Air	—	—	—	—	8	32	28	19	12	1	—	—	—	—	—	64	77	49	91	40
	Surface	—	—	—	—	—	9	28	20	12	8	8	10	4	1	—	82	113	60	131	52
	2-in. depth	—	—	—	—	—	6	24	26	16	12	12	4	—	—	—	80	101	63	118	53
	4-in. depth	—	—	—	—	—	2	19	33	23	18	5	—	—	—	—	79	93	67	106	57
	6-in. depth	—	—	—	—	—	4	18	34	29	15	—	—	—	—	—	78	88	68	99	58
	8-in. depth	—	—	—	—	—	—	19	38	36	7	—	—	—	—	—	77	84	70	95	59
	10-in. depth	—	—	—	—	—	—	21	40	37	2	—	—	—	—	—	76	82	71	91	60
	12-in. depth	—	—	—	—	—	—	23	43	34	—	—	—	—	—	—	75	79	71	88	60

TABLE 6

DURATION OF TEMPERATURE LEVELS, LOW, HIGH AND AVERAGE TEMPERATURES AT VARIOUS DEPTHS FOR 12-IN. ASPHALT CONCRETE PAVEMENT FOR ONE YEAR

Position Where Measurements Were Made	Percent of Year (June 1, 1964-May 31, 1965) During Which the Temperature (° F) Was Between															Avg. Temp. (° F)	Avg. High Temp. (° F)	Avg. Low Temp. (° F)	High Temp. (° F)	Low Temp. (° F)
	0 & 9	10 & 19	20 & 29	30 & 39	40 & 49	50 & 59	60 & 69	70 & 79	80 & 89	90 & 99	100 & 109	110 & 119	120 & 129	130 & 139	140 & 149					
Air	—	2	7	15	19	17	17	14	7	2	—	—	—	—	—	54	65	41	99	2
Surface	—	1	5	11	17	15	13	14	9	5	4	3	2	1	—	64	87	48	142	9
2-in. depth	—	—	4	11	18	15	12	13	11	7	5	3	1	—	—	64	83	51	127	14
4-in. depth	—	—	2	11	18	17	12	11	14	9	5	1	—	—	—	64	74	55	119	18
6-in. depth	—	—	1	10	20	18	12	10	16	10	3	—	—	—	—	63	71	56	109	20
8-in. depth	—	—	1	9	21	18	12	9	18	11	1	—	—	—	—	64	69	58	105	23
10-in. depth	—	—	1	8	22	18	13	8	20	10	—	—	—	—	—	63	67	59	101	25
12-in. depth	—	—	—	7	24	17	13	8	22	9	—	—	—	—	—	64	66	60	98	27

TABLE 7

MONTHLY DURATION OF TEMPERATURE LEVELS, LOW, HIGH AND AVERAGE TEMPERATURES OF THE SOIL BELOW AND ADJACENT TO THE PAVEMENTS

Period During Which Measurements Were Made	Position Where Measurements Were Made	Percent of Month During Which the Temperature (° F) Was Between						Avg. Temp. (° F)	Avg. High Temp. (° F)	Avg. Low Temp. (° F)	High Temp. (° F)	Low Temp. (° F)
		20 & 29	30 & 39	40 & 49	50 & 59	60 & 69	70 & 79					
November 1964	6-in. below 6-in. pavement	—	—	14	81	5	—	55	56	53	61	43
	12-in. depth in soil	—	1	35	64	—	—	50	52	48	56	38
	6-in. below 12-in. pavement	—	—	10	84	6	—	56	57	55	61	47
	18-in. depth in soil	—	—	27	73	—	—	51	52	50	56	42
December 1964	6-in. below 6-in. pavement	—	18	78	4	—	—	43	45	42	50	36
	12-in. depth in soil	—	43	57	—	—	—	40	42	38	49	33
	6-in. below 12-in. pavement	—	5	92	3	—	—	45	46	44	50	38
	18-in. depth in soil	—	20	80	—	—	—	42	43	41	48	36
January 1965	6-in. below 6-in. pavement	—	58	42	—	—	—	39	40	37	48	31
	12-in. depth in soil	—	80	20	—	—	—	35	36	34	45	30
	6-in. below 12-in. pavement	—	53	47	—	—	—	39	40	39	48	31
	18-in. depth in soil	—	67	33	—	—	—	37	37	36	45	31
February 1965	6-in. below 6-in. pavement	—	20	80	—	—	—	42	44	40	48	30
	12-in. depth in soil	—	51	49	—	—	—	39	40	37	46	30
	6-in. below 12-in. pavement	—	16	84	—	—	—	43	44	41	48	33
	18-in. depth in soil	—	38	62	—	—	—	39	40	39	46	33
March 1965	6-in. below 6-in. pavement	—	1	87	12	—	—	48	50	45	56	39
	12-in. depth in soil	—	11	88	1	—	—	43	44	40	50	36
	6-in. below 12-in. pavement	—	—	82	18	—	—	48	49	46	54	42
	18-in. depth in soil	—	1	99	—	—	—	43	44	42	48	38
April 1965	6-in. below 6-in. pavement	—	—	6	74	20	—	56	59	53	67	45
	12-in. depth in soil	—	—	43	57	—	—	49	51	47	59	40
	12-in. pavement	—	—	3	78	19	—	56	58	54	64	48
	18-in. depth in soil	—	—	45	55	—	—	49	50	48	56	42

thermocouple was located near the center of the test section. A thermocouple for the air temperature was located at a height of 5 ft and at a distance of 3 ft from the test section. It was shaded from the sun but exposed to free air circulation. A plastic pipe was used to protect and carry the thermocouple wires about 100 ft to a building housing the temperature recorder.

In the fall of 1964, additional thermocouples were installed. They were placed at a depth 6 in. below the 6-in. thick pavement and 6 in. below the 12-in. thick pavement. Thermocouples were also placed at depths of 12 and 18 in. in the natural soil, 9 ft from the test section.

A Leeds and Northrup Speedomax G, Model S, 12-point temperature recorder was used for the study. It operated on a print cycle of 24 sec per point. Temperatures from 12 thermocouples could therefore be recorded in slightly less than 5 min. The thermocouple wires and the recorder were checked before installation and periodically during the year, using an ice bath. Weekly checks of the system were made using a manually-operated portable temperature potentiometer. A glass mercury-filled thermometer mounted beside the air temperature thermocouple was used for daily checks.

TEMPERATURE DATA

The highest pavement temperatures during the study were recorded on June 30, 1964. The surface reached a maximum temperature of 142 F at 3:00 p.m. A maximum temperature of 98 F was reached at 9:00 p.m. at the 12-in. depth. Hourly temperatures for that day are shown in Figure 2 for the 12-in. test section. These hourly temperatures show typical cycles of daily pavement temperature changes at different depths. Before sunrise, the lowest pavement temperature is at the surface and temperatures increase with increasing depth. After sunrise, surface temperatures increase rapidly until in the afternoon the highest temperature is at the surface, and temperatures decrease with increasing depth. The hourly temperatures on June 30, 1964, given in Table 2 for both the 12-in. and 6-in. test sections show no significant difference in temperatures or temperature changes between the 12-in. and 6-in. thick pavement sections at depths of 2, 4 and 6 in. Continuous recordings for both test sections through the summer and fall, and periodic checks on the 6-in. thick pavement section for the rest of the study, confirmed this behavior.

The lowest pavement temperatures were recorded on January 19, 1965. The surface reached a minimum temperature of 9 F at 7:00 a.m. A minimum temperature of 27 F was recorded at 9:00 a.m. at the 12-in. depth. Hourly temperatures for that day are shown in Figure 3 and are tabulated in Table 3. The data show that daily temperature changes in the pavement follow similar cycles in warm weather and in cold weather. However, the ranges of temperatures from maximum to minimum on January 19 were only slightly more than one-half the temperature ranges on June 30.

The greatest rate of pavement temperature change during the study occurred at the pavement surface during warm weather, when the surface temperature was near maximum and a sudden rain occurred. Typical pavement temperatures for these conditions on June 8, 1964, are shown in Table 4. The pavement surface temperature decreased 37 F during a period of 30 min after the rain began at 2:00 p.m. During the same time, the temperature decrease was only 9 F at a 2-in. depth, and at a depth of 6 in. the pavement temperature increased 2 F. Rapid rates of temperature change occur only at or near the surface of the pavement under these conditions. Relatively low rates of temperature change at pavement depths of 6 in. or more were recorded during the study. Daily differences between maximum and minimum temperatures were generally less than 20 F at the 6-in. depth and less than 10 F at the 12-in. depth. The typical hourly rate of temperature change at depths of 6 to 12 in. was 1 F per hour, often less, but sometimes as much as 3 F or 4 F.

Hourly temperature data obtained during the study were used to calculate the percent of each month and of the year during which the temperature was between 15 equal levels, each spanning 10 F, between 0 F and 149 F. These calculations were made for air and surface temperatures, and for pavement temperatures at the various depths in the 12-in. thick pavement. This method of expressing temperature durations is similar to the one described by Trott (4), except that durations of temperature levels were calculated from hourly readings rather than determined by temperature-classifying equipment. Determinations of temperature levels are very useful since it is of interest to know the duration of temperatures as well as maximum, minimum and average temperatures in studies of long-term pavement performance. The average temperatures, and average high and low temperatures, were calculated from the hourly temperature data for each month and for the entire year for air and pavement temperatures, as were temperatures at the different depths in the 12-in. thick pavement. The calculations were based on 8088 hourly temperature measurements during the study at each position where temperatures were measured.

Monthly durations of temperature levels, monthly average temperatures, average high and low temperatures, and monthly high and low temperatures for the 12-in. thick pavement are given in Table 5. The duration of temperature levels for the entire year, yearly average temperatures, yearly average high and low temperatures and the high and low temperature for the year for the 12-in. thick pavement are given in Table 6. Temperatures in the highest levels at or near the pavement surface occurred during relatively small fractions of the time throughout the year and throughout a month. Temperatures between 140 F and 149 F at the pavement surface occurred only 1 percent of the time during the month of June, and were not recorded during the rest of the year. At a depth of 6 in., temperatures did not exceed 109 F during the year and were between 100 F and 109 F for 16 percent of July, but for only 3 percent of the year. At a depth of 12 in., the pavement reached a maximum of 98 F during June and was between 90 F and 99 F for 26 percent of the time during June, but at that level only 9 percent of the time during the year.

The data in Tables 5 and 6 support the commonly used temperature of 140 F for paving mixture stability and for asphalt consistency testing, provided the mixtures are used near the pavement surface. It was approximately the maximum pavement surface temperature that occurred for a relatively short time at the test site. The data indicate that laboratory testing temperatures below 140 F should be considered for paving mixtures placed in the lower courses of the pavement. If it is desired to determine a temperature-dependent paving mixture property at the highest temperature expected at a 12-in. depth, the study indicates that a testing temperature of about 100 F would be appropriate. For temperature-dependent paving mixture properties at the highest temperature expected at a 6-in. depth, data from the study show that a testing temperature of 110 F should be used.

Hourly temperatures were used to calculate the duration of various temperature levels and average temperatures during the last 2 months of 1964 and the first 4 months of 1965 for thermocouples installed in the subgrade beneath the 2 pavement sections, and in the soil adjacent to the test sections. The data for temperatures 6 in. below the 6-in. and 12-in. thick pavements, and for temperatures in the soil adjacent to the test sections at depths of 12 and 18 in., are given in Table 7. Average monthly temperatures in the soil 6 in. below the 6-in. pavement ranged from 3 F to 7 F higher than average temperatures at a depth of 12 in. in the soil adjacent to the pavement. The average monthly temperatures in the soil 6 in. below the 12-in. pavement were from 2 F to 7 F higher than the average temperatures at a depth of 18 in. in the soil adjacent to the pavement. During January, the coldest month, temperatures 6 in. below the 6-in. pavement were between 30 F and 39 F for 58 percent of the time. Soil temperatures during the same time at a depth of 12 in. adjacent to the pavement were between 30 F and 39 F 80 percent of the time. Temperatures during the winter and spring beneath the pavements were appreciably higher, and were at higher temperature levels for longer periods of time than were soil temperatures at corresponding depths adjacent to the pavements. During the winter months no temperatures below 30 F occurred 6 in. below the 6-in. and 12-in. thick asphalt concrete pavements.

SUMMARY

1. The temperatures at depths of 2, 4 and 6 in. in a 6-in. thick asphalt concrete pavement were essentially the same as temperatures at the same depths in a 12-in. thick asphalt concrete pavement.

2. The maximum temperatures during a period of one year at the surface and at depths of 2, 4, 6, 8, 10 and 12 in., respectively, were 142 F, 127 F, 119 F, 109 F, 105 F, 101 F and 98 F.

3. The minimum temperatures during a period of one year at the surface and at depths of 2, 4, 6, 8, 10 and 12 in., respectively, were 9 F, 14 F, 18 F, 20 F, 23 F, 25 F and 27 F.

4. The average temperature at the various pavement depths varied slightly with depth and season, but during the period of a year the average temperature at all depths was either 63 F or 64 F, while the average air temperature was 54 F.

5. The pavement temperatures above about 120 F, experienced only at depths of 4 in. or less, occurred for relatively small fractions of the time, even during the warmest summer months.

6. Average temperatures during cold weather months in the subgrade soil 6 in. below the 6-in. and 12-in. thick asphalt concrete pavements were appreciable higher, and remained at higher levels for longer periods of time than average temperatures at corresponding depths of 12 and 18 in. in the soil adjacent to the pavement.

7. The test data support the commonly used temperature of 140 F for paving mixture stability and asphalt testing, if the paving mixture or the asphalt properties are desired at the highest expected temperatures reached at or near the pavement surface. At lower pavement depths, lower testing temperatures were indicated for determining the temperature-dependent properties of paving mixtures and asphalt at the highest expected pavement temperatures. Testing temperatures of about 110 F for a 6-in. pavement depth and 100 F for a 12-in. pavement depth were indicated.

REFERENCES

1. Arena, P. J. Service Temperature Study for Asphalt Concrete. Research Report No. 14, Research Project No. 61-3B, HPR 1(2), Louisiana Dept. of Highways (in cooperation with U. S. Bureau of Public Roads), Oct. 1964.
2. Barber, E. S. Calculation of Maximum Pavement Temperatures from Weather Reports. HRB Bull. 168, p. 1, 1957.
3. Monismith, C. L., Secor, G. A., and Secor, K. E. Temperature Induced Stresses and Deformations in Asphalt Concrete. Paper presented at AAPT Annual Meeting, Philadelphia, Feb. 1965.
4. Trott, J. J. An Apparatus for Recording the Duration of Various Temperatures in Roads. Roads and Road Construction, Vol. 41, No. 491, pp. 342-345, 1963.

Study of Temperature Variation in Hot-Mix Asphalt Base, Surface Course and Subgrade

GLENN P. MANZ, Assistant Executive Secretary, Michigan Asphalt Paving Association

•A STUDY of temperature variation in a 7-in. thick asphalt pavement and the underlying soil has been carried on in Michigan through the cooperative efforts of Leonard Refineries of Alma, Michigan, and the Michigan Asphalt Paving Association. The study was conducted from June 1963 through April 1965.

An experimental hot-mix base or black base pavement was constructed by The Hicks Company for the Gratiot County Road Commission. Thermocouples were placed at 4 levels in the pavement and at 3 levels in the subgrade. Temperatures were read each day to determine the maximum temperatures in the black base and the wearing course.

Because of the erratic results obtained from stability tests of the black base mixes, it was felt necessary to know the maximum temperatures which could be expected in the base courses. Laboratory mixes tested at the standard 140 deg temperature varied widely, and were of little value. Stability tests were run on the same mixes at 100 deg and 120 deg with results which were reproducible. The mix used had a Marshall stability at 120 deg, which was equal to surface course mixes at standard temperature. What we were attempting to determine then was that 120 deg would be greater than the highest expected temperature in the base mix.

After a few weeks of reading temperatures manually, it was decided to install a recording pyrometer which would record not only maximum temperatures but the variation between maximum and minimum on a 24-hr basis. Although the original purpose of the study was to determine maximum temperatures, it was soon apparent that there was considerable interest in knowing more about winter temperatures. Because of this, 3 more thermocouples were placed in the subgrade under a 7-in. gravel surface driveway, so that a comparison could be made between temperatures under the 7 in. asphalt pavement and under 7 in. of gravel during the cold winter months.

The following figures show the maximum temperatures during the hot summer weather and the minimum recorded during the coldest days of the winter.

Figure 1 shows the wide range of maximum temperatures which occurred during a 30-day period when the highest temperatures of the summer were recorded. This indicates a 13-deg drop in temperature in the top 2 in. of the pavement. In the next 3 in. of depth a further reduction of 5 deg was recorded.

The average drop in temperature in the 7-in. asphalt pavement was 25 deg, although the minimum differential was 9 deg and the maximum was 34 deg. The maximum surface temperature recorded was 130 deg. This peak was reached on the day after the highest atmospheric temperature of the summer was recorded.

The temperatures recorded at the pavement surface and at a depth of 2 in. are practically identical to those reported in a study made in 1924 and 1925 by W. J. Emmons and B. A. Anderton for the Bureau of Public Roads.

The single dash line in Figure 1 shows the temperature range in the top of the base course. The lowest temperature was 80 deg and the maximum was 115 deg, which occurred on 3 different days.

This maximum 115-deg temperature is important because it bears out our assumption that 120 deg would be the highest expected base temperature. The average temperature in the top of the base was 100.9 deg.

Paper sponsored by Committee on Bituminous Aggregate Bases and presented at the 45th Annual Meeting.

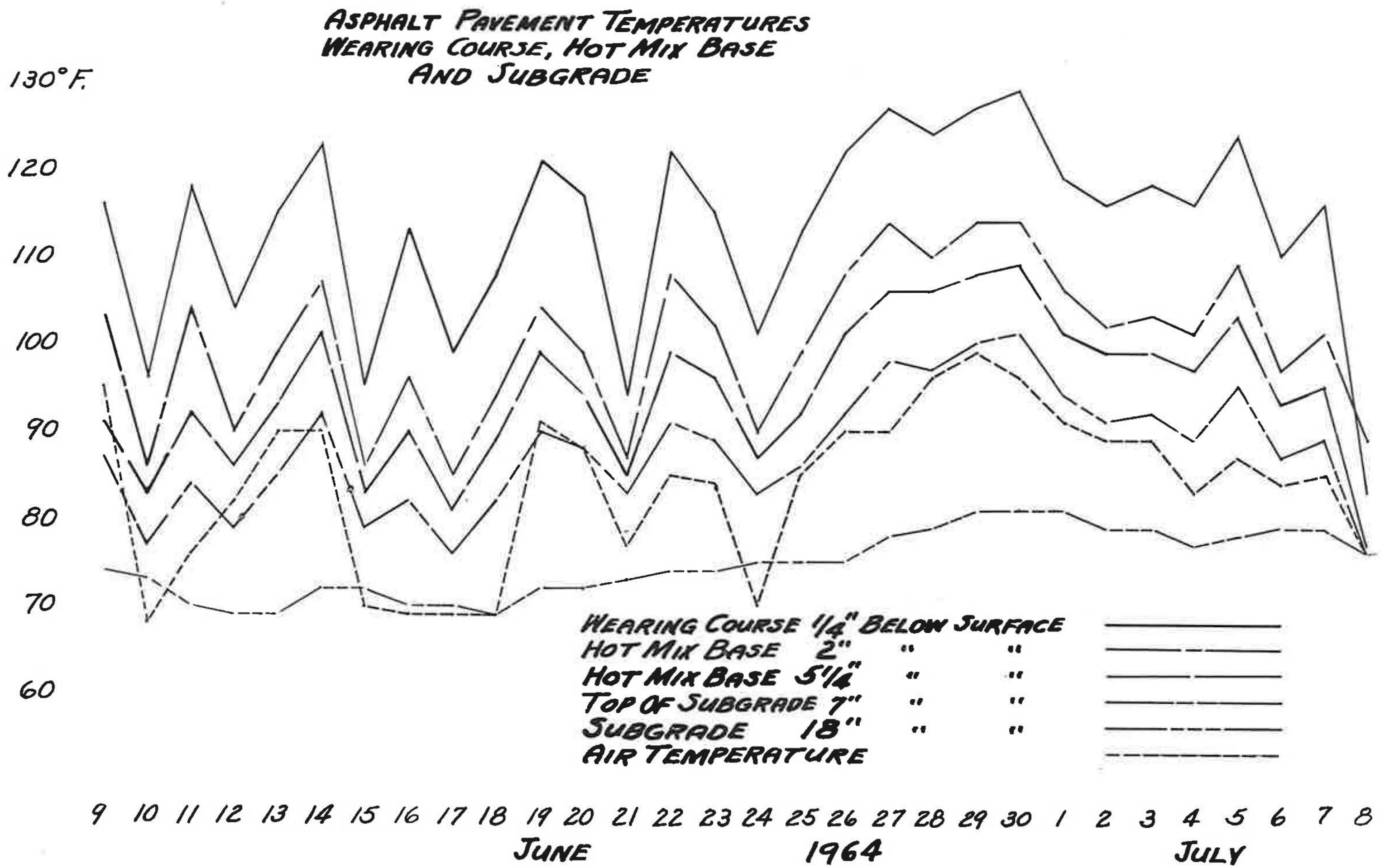

ASPHALT PAVEMENT TEMPERATURES
WEARING COURSE, HOT MIX BASE
AND SUBGRADE
130°F.
120
110
100
90
80
70
60
WEARING COURSE 1/4" BELOW SURFACE
HOT MIX BASE 2" " "
HOT MIX BASE 5 1/4" " "
TOP OF SUBGRADE 7" " "
SUBGRADE 18" " "
AIR TEMPERATURE
9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24 25 26 27 28 29 30 1 2 3 4 5 6 7 8
JUNE
1964
JULY

Figure 1.

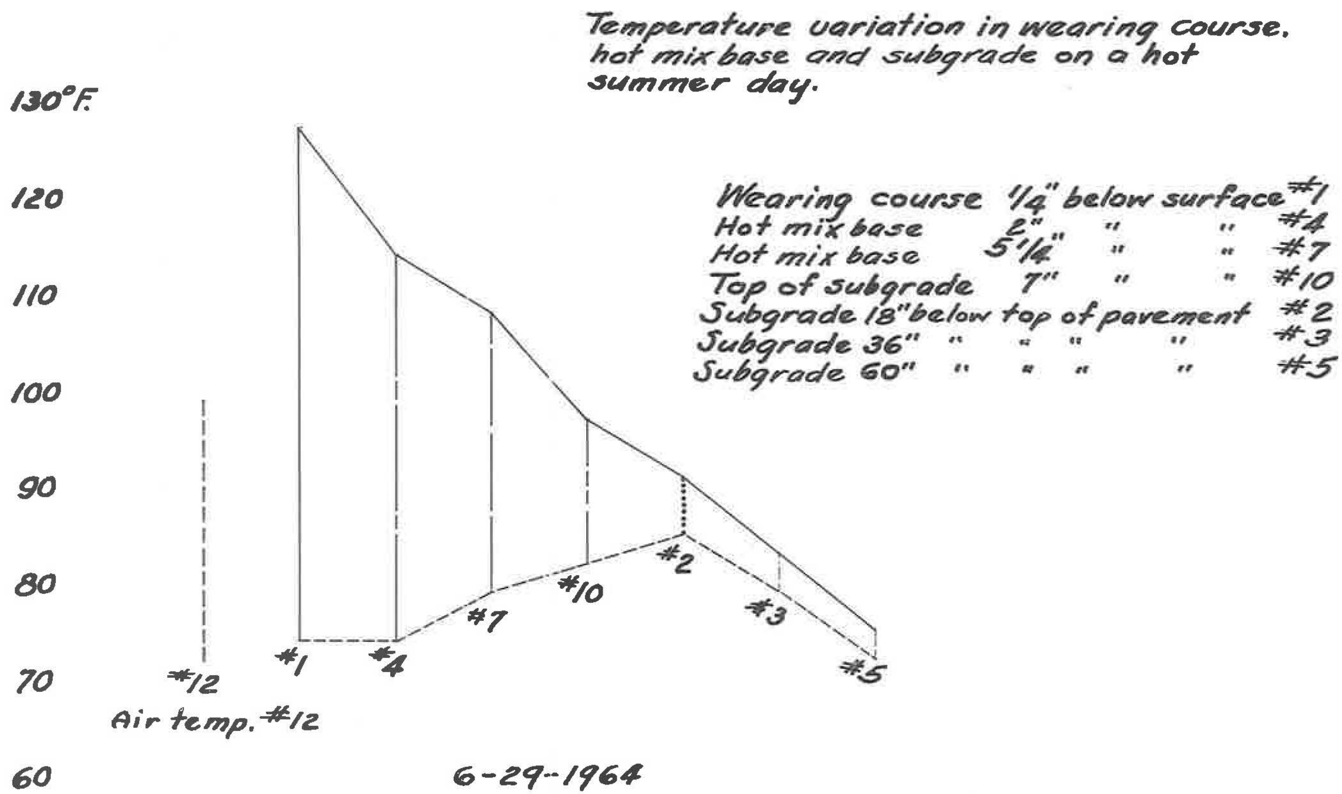

Temperature variation in wearing course, hot mix base and subgrade on a hot summer day.
Wearing course 1/4" below surface #1
Hot mix base 2" " " #4
Hot mix base 5 1/4" " " #7
Top of subgrade 7" " " #10
Subgrade 18" below top of pavement #2
Subgrade 36" " " " " #3
Subgrade 60" " " " " #5
130°F.
120
110
100
90
80
70
60
#12
Air temp. #12
#1
#4
#7
#10
#2
#3
#5
6-29-1964

Figure 2.

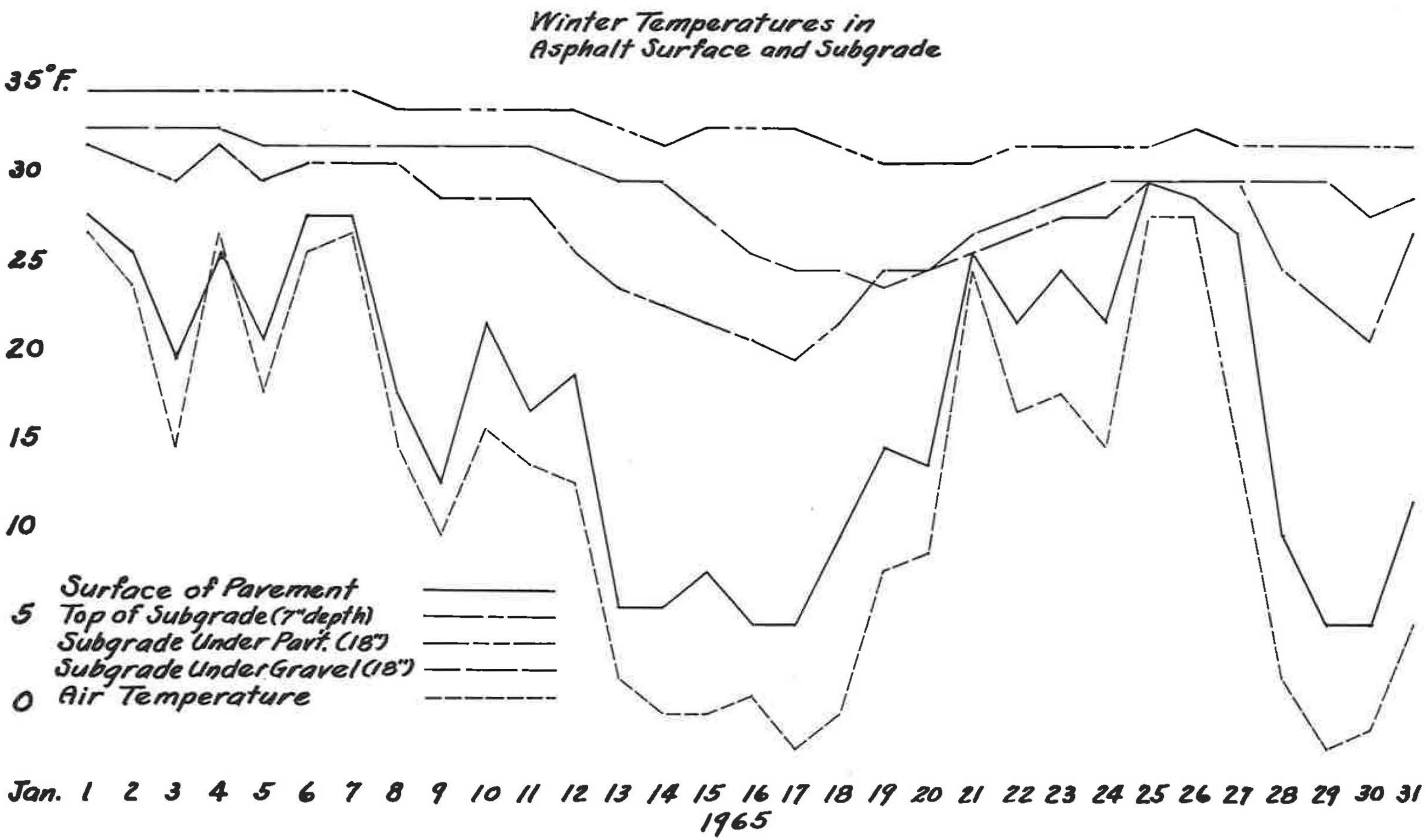
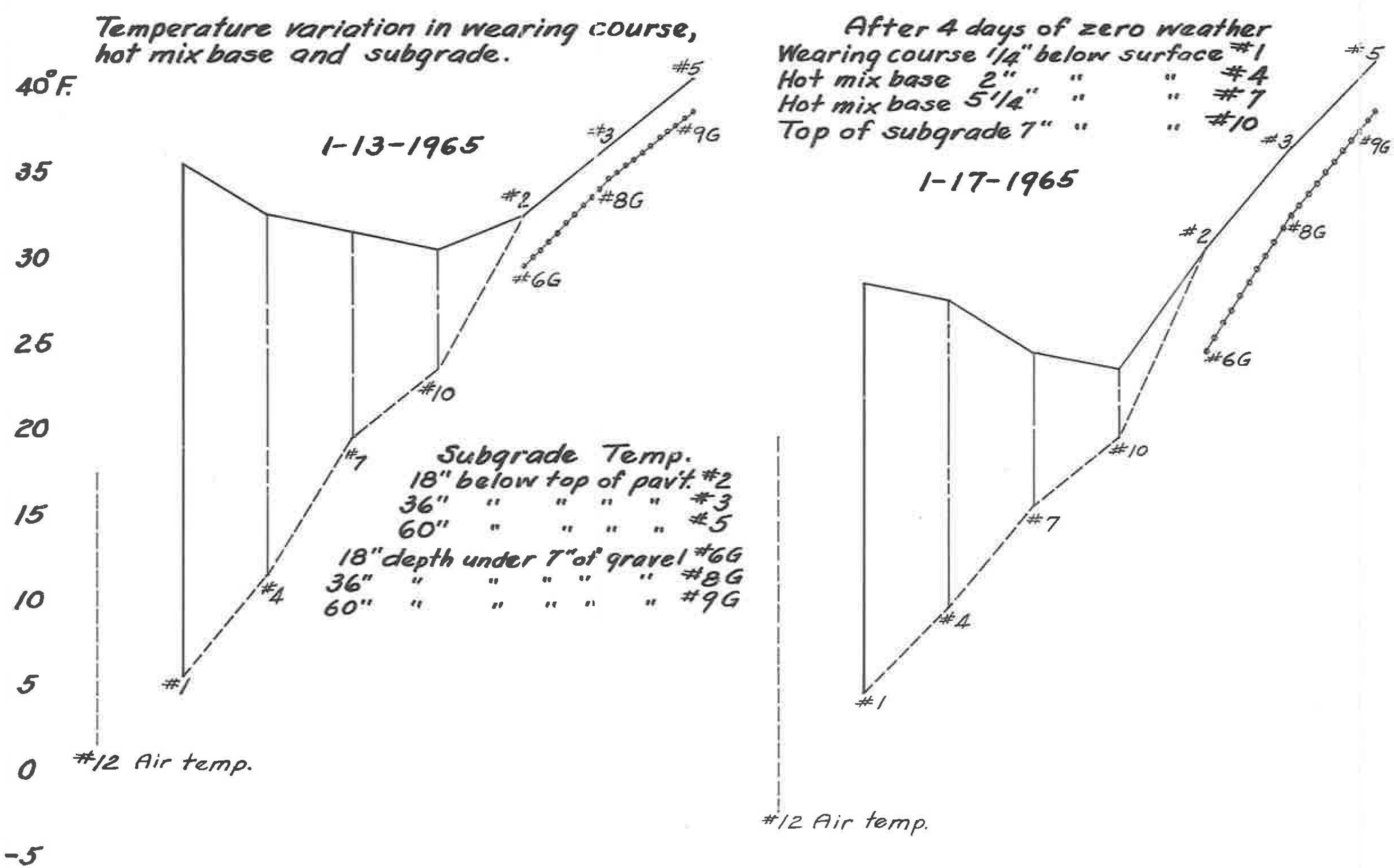

Winter Temperatures in
Asphalt Surface and Subgrade
35°F.
30
25
20
15
10
5
0
Surface of Pavement
Top of Subgrade (7" depth)
Subgrade Under Pavt. (18")
Subgrade Under Gravel (18")
Air Temperature
Jan. 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24 25 26 27 28 29 30 31
1965

Figure 3.



Temperature variation in wearing course, hot mix base and subgrade.
40°F.
35
30
25
20
15
10
5
0
-5
1-13-1965
#1
#4
#7
#10
#2
#3
#5
#6G
#8G
#9G
#12 Air temp.
Subgrade Temp.
18" below top of pav't. #2
36" " " " " #3
60" " " " " #5
18" depth under 7" of gravel #6G
36" " " " " " #8G
60" " " " " " #9G
After 4 days of zero weather
Wearing course 1/4" below surface #1
Hot mix base 2" " " #4
Hot mix base 5 1/4" " " #7
Top of subgrade 7" " " #10
1-17-1965
#12 Air temp.

Figure 4.

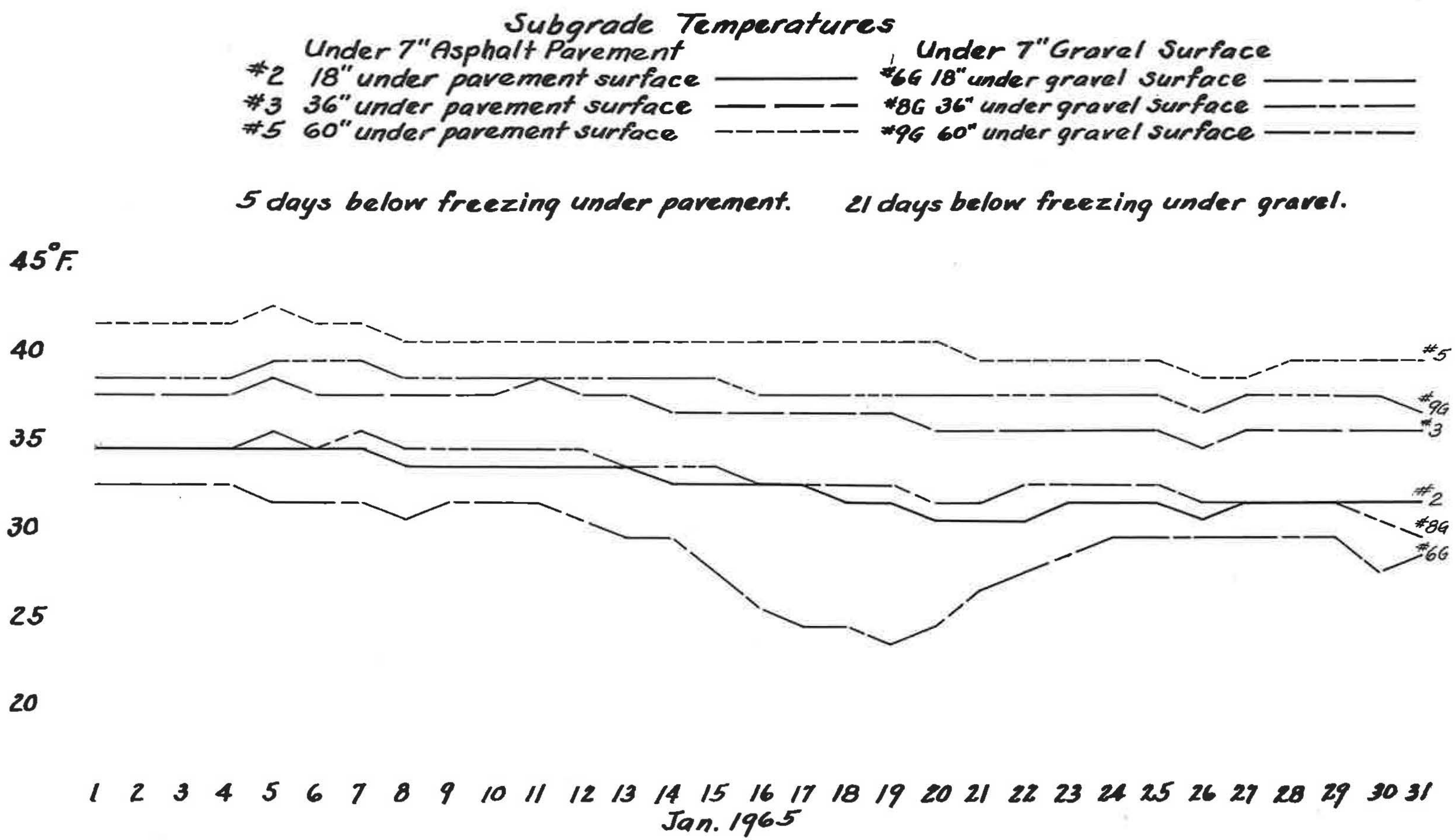

Subgrade Temperatures
Under 7" Asphalt Pavement
#2 18" under pavement surface
#3 36" under pavement surface
#5 60" under pavement surface
Under 7" Gravel Surface
#6G 18" under gravel surface
#8G 36" under gravel surface
#9G 60" under gravel surface
5 days below freezing under pavement.
21 days below freezing under gravel.
45°F.
40
35
30
25
20
#5
#9G
#3
#2
#8G
#6G
1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24 25 26 27 28 29 30 31
Jan. 1965

Figure 5.

Figure 2 shows the variation in temperature in the wearing course, black base, subgrade and the atmosphere. While the air temperature varied from 73 deg to 100 deg, the top of the pavement had a much greater variation, ranging from a low of 75 deg to a high of 128 deg. The black base variation was 75 deg to 115 deg, and the subgrade ranged from a high of 98 deg at the 7-in. level to a low of 63 deg at the 60-in. level.

Although a preliminary report was made on winter temperatures, no information was available showing the minimums reached during a period of below-normal temperatures for the central Michigan area.

During January 1965, the low temperatures hoped for occurred and Figure 3 indicates the minimums recorded during the month.

Since the temperatures in the black base are not particularly important at this time of the year, only the pavement surface, subgrade and air temperatures are shown.

As expected, an entirely different picture is presented during the winter; the pavement surface is now the coldest part of the pavement. There was a 6-day period when the air temperature was zero or within 2 deg of zero each day. The pavement surface followed along with low temperatures of 5 deg to 8 deg during this period.

Of prime interest in this part of the study are the minimum temperatures occurring in the subgrade. Temperatures below normal experienced during this period resulted in frost penetration to the 18-in. level under the pavement. A minimum temperature of 31 deg was recorded at this level and it was below the freezing point for 5 days, while at the same time and at the same level under the gravel surface, there was a low temperature of 24 deg on one day and below freezing temperatures during 21 days.

In the depth of winter when freezing temperatures are continuous, what happens to pavements and subgrades? Do temperatures fall to some unknown low point and remain stationary until a warming trend occurs, or is there still further fluctuation? What actually happens is shown in Figure 4, which indicates the range of temperature variation on 2 midwinter days. On January 13, the air temperature reached a low of 2 deg and a maximum of 18 deg, while the pavement surface was rising from a low of 6 deg to a high of 36 deg.

The range of temperature variation diminishes rapidly with depth. At the top of the subgrade, 7 in. below the pavement surface, a variation of 7 deg occurs—from a low of 24 deg to a high of 31 deg.

There is little variation at lower levels in the subgrade during one 24-hr period, but several days of zero temperatures do have a definite effect at all levels down to the 36-in. level in the subgrade. This is illustrated graphically in Figure 5. On January 17 the air temperature fell to a low of 2 deg below zero and reached a high of 20 deg. After 4 days of very low temperatures, temperatures fell at every level except at the 60-in. depth, which changed very gradually over a period of several days. The range of temperatures also narrowed—surface 5 deg to 29 deg and top of subgrade 20 deg to 24 deg.

A most interesting change occurred in the subgrade at the 18-in. level—the temperature under the pavement dropped from 33 deg to 31 deg, while at the same level under the gravel surface the temperature fell from 30 deg to 25 deg. This indicates a deeper frost penetration under the gravel surface, and very possibly points up some insulating value in the 7-in. layer of asphalt base and surface.

Figure 5 shows only subgrade temperatures recorded during the month of January 1965. Notice that the temperatures are fairly uniform except during the very coldest part of the month, when a very definite drop occurred at the 18-in. level under the gravel surface.

Under the pavement the temperature fell 2 deg while under the gravel surface a 5-deg drop occurred, and this may have been reduced by a covering of ice over the gravel for a few days.

Some heaving did occur during the coldest weather of the winter, with a maximum of 1.5 in. being recorded. Any movement which occurred must have been quite uniform, since no cracking has developed up to this time.

CONCLUSIONS

1. The data included in this paper indicate that the maximum temperature in the black base will not exceed 120 deg.
2. The insulating value and heat-absorbing ability of the thick asphalt pavement is quite evident in the figures showing winter temperature variation. A definite reduction in frost penetration into the subgrade is indicated.

A great amount of information was accumulated during the months this study was being made. As time permits, much additional information may be derived from a further study of the recorded data.

ACKNOWLEDGMENTS

Credit must be given to Berl Fleury, engineer for Leonard Refineries; Arthur Taylor, Superintendent of the Gratiot County Road Commission; The Hicks Company; and the Michigan Asphalt Paving Association for their assistance and cooperation in making this study possible.

The assistance of Leonard Refineries and their furnishing of the data recording equipment is also appreciated.

Thin Overlay Bituminous Macadam for the Control of Reflex Cracking

ROBERT D. JOHNSON, Maine Aeronautics Commission

•REFLEX CRACKING in pavement overlays has been an extensive problem, often with an unsatisfactory solution. This problem is brought about by attempts to correct another far-reaching problem—old pavement in need of rehabilitation. Both problems now exist in practically all pavements.

Determination of a satisfactory solution to these problems has been one of the most pressing demands on airport owners and the Aeronautics Commission in Maine for the past 10 years.

Pavements in concern, for the most part, have been over 20-years-old, have retained their general shape, but are severely cracked. The cracks commonly prevail in two directions—longitudinally with a reasonably consistent pattern and straight alignment, and transversely with a very irregular pattern and irregular alignment. The width of the cracks is most often from ½ to ¾ in.

PREVIOUS METHODS

One method used extensively to rectify badly cracked pavements was to clean the cracks with a router, or by some other means, and then fill them by successive applications of asphalt and sand. This often was followed by a seal coat or bituminous concrete overlay 1 to 2 in. in thickness.

Another method used to some extent was, after routing, to fill the cracks with a rubber asphalt or synthetic and then apply a seal coat or bituminous concrete overlay.

A third method was to fill the cracks with either asphalt and sand, rubber asphalt or synthetics and then apply a bituminous concrete overlay, 1 to 3 in. in thickness, underlain with wire mats.

For each of these methods results were unsatisfactory. It was common to have a great many of the original cracks reappear the following year as reflex cracks.

NEW APPROACH

Because of the dissatisfaction with previous methods, in 1959 the decision was made to use a 3-in. course of bituminous macadam pavement to rehabilitate an extreme case of old, badly cracked bituminous concrete runway pavement. It was predicted that the large 2-in. base stone of the macadam would bridge the existing cracks and therefore would resist or actually eliminate reflex cracking. It was further theorized that because of this expected bridging action, the existing pavement cracks would not have to be pretreated.

In the summer of 1959, a typical 3-in. bituminous macadam overlay course was placed over 1 of the 2 runways at the Eastport Airport, Eastport, Maine. Inspections during the next 4 years revealed no evidence of reflex cracking. During the fifth year one crack occurred, but it had actually healed itself. Obviously the 2½ gal./yd^2 of flexible asphalt used in this macadam course added greatly to its ability to resist cracking; when cracking did take place, the asphalt was a great asset in rehealing it.

The actual material description, amounts and order of application used on the Eastport project, is described in Table 1. Pavement conditions before and after this project are shown in Figures 1 and 2.

Paper sponsored by Committee on Bituminous Aggregate Bases and presented at the 45th Annual Meeting.

TABLE 1

EASTPORT AIRPORT, MAINE, BITUMINOUS MACADAM OVERLAY

Order of Application	Amount of Material per Square Yard		Material Description
1	225	lb	Stone—$2\frac{1}{2}$ in. minus
2	1.60	gal.	Asphalt cement
3	35	lb	Stone—$\frac{3}{4}$ in. minus
4	0.60	gal.	Asphalt cement
5	25	lb	Stone—$\frac{1}{2}$ in. minus
6	0.15	gal.	Asphalt cement
7	15	lb	Stone—$\frac{1}{4}$ in. minus

Because the first bituminous macadam project produced such satisfactory results, macadam was used again when, in 1964, another badly cracked runway pavement was scheduled for rehabilitation. A major adjustment adopted, however, primarily for economy, was to reduce the pavement thickness from 3 in. to $1\frac{1}{2}$ in. The specifications that were used generally followed the Maine State Highway specifications for bituminous macadam surface course, except for the adjustments described in Table 2.

Figure 1. Eastport Airport before bituminous macadam overlay.

Figure 2. Eastport Airport after a 3-in. bituminous macadam overlay.

The construction of the $1\frac{1}{2}$-in. macadam course generally followed typical macadam construction, with minor exceptions. Equipment that was used was an Adnun paver, a modern bituminous distributor truck, chip spreader, 6-ton roller, and a broom drag.

Two conditions were encountered that had not been anticipated. It was found that a 12-ton, 3-wheel tandem roller that had originally been brought to the job was too heavy and tended to push the stone up in a wave in front of the rollers. This condition was corrected by replacing this larger roller with a smaller 7-ton, 2-wheel roller. Some difficulty was encountered in obtaining the desired uniform covering of Keystone and Chip Stone. This problem was attributed to an extensively used and worn tail gate type chip spreader. This spreader was condemned on this project and, on another similar job, a more modern spreader was used with very good success.

A $1\frac{1}{2}$-in. macadam overlay pavement was placed on runways at 2 different airports. One was at Caribou, Maine, and the other at Dexter, Maine. Pavement conditions before and after the Caribou project are shown in Figures 3 and 4.

TABLE 2

CARIBOU AIRPORT, MAINE, BITUMINOUS MACADAM OVERLAY

Order of Application	Amount of Material per Square Yard	Material Description
1	100 lb	Stone—$1\frac{1}{4}$ in. minus
2	0.75 gal.	Asphalt cement
3	15 lb	Stone—$\frac{1}{2}$ in. minus
4	0.50 gal.	Asphalt cement
5	10 lb	Stone—$\frac{1}{4}$ in. minus
6	0.25 gal.	Asphalt cement
7	15 lb	Stone—$\frac{1}{4}$ in. minus

An unfortunate condition that prevailed during the Dexter construction was unusually cold weather. This proved somewhat of a deterent in gaining the desired adherence of Keystone and Chip Stone to the asphalt.

However, even though it was our first experience with a $1\frac{1}{2}$-in. thin overlay bituminous macadam, we found its placement progressed very satisfactorily, with few unexpected or unusual problems. Actually, its construction is very much like that of the more common 3-in. bituminous macadam surface course.

Observations in May 1965 of both the Caribou and the Dexter projects revealed

Figure 3. Caribou runway before bituminous macadam overlay.

Figure 4. Caribou runway after a 1½-in. macadam overlay.

that they were holding up very well, and most importantly, that reflex cracking was practically nonexistent. Therefore, we now believe that thin overlay bituminous macadam is the most economical and practical approach to the control of pavement reflex cracking that we have tried, observed, or researched to date.

Opinions offered by some paving specialists state that macadam does not provide a satisfactory wearing course for some purposes. They suggest that, in these cases, the macadam could and should be used first to provide an insulation course to prevent reflex cracking, but then the macadam should be overlaid with a thin course of bituminous concrete. We can only say that this sounds as if it may have merit and that we would be greatly interested to hear of results from anyone that may experiment with it.

Discussion

CHARLES F. PARKER, Chairman, Subcommittee on Penetration Macadam, Committee on Bituminous Aggregate Bases—There are several items that I would like to discuss regarding this method, which has been so ably presented by Robert D. Johnson. The resident engineer employed on much of this work is a retired engineer who devoted the greater part of his career to the construction of macadam pavements. I worked many years with this man and have a great deal of respect for his ability. His name is David A. Smith.

In an interview Dave Smith stated, "Macadam is a good type of pavement but lacks research . . . . If one-half of the research had gone into macadam as for other types, it would be a popular type of construction today."

With this type of thin overlay using penetration grade asphalt, the penetration does not go through the stone. For that reason there is positively no bond between the overlay and the old pavement. Furthermore, there is a thick coating of asphalt on the stone and this adds greatly to the flexibility and aging characteristics of the overlay pavement.

Emulsified asphalt or cutback types would probably not be as successful, as there would be greater penetration and bonding of the overlay to the old pavement and less flexibility due to a thinner coating.

Relative to the compaction, it is my understanding that on one project, with greater than normal grades, they experienced some difficulties in rolling and changed to a lighter type roller. Probably larger diameter rolls or pneumatic-tire rolling would have been helpful in this case.

Weather could be quite a controlling factor. I was interested in one case about which Johnson may give us more details. It appears that on one of these projects there was a severe shower shortly after the application of the penetration, which prevented the application of the stone chips. As the field was being used, it was necessary to spread the chips before the pavement dried. Later, when the chips did dry, they rolled the pavement again and obtained a good bond. It would, of course, be necessary to have clean, well-graded stone chips.

The resident engineer attributed the smooth riding qualities to the long-wheelbase stone spreader used; they received many complimentary reports from visiting aircraft. With this type of construction, it is my understanding that they took out irregularities ranging from 1 to $1\frac{1}{2}$ in. The thickness of this overlay, as reported by Johnson, seems adequate for the normal traffic on these airfields. The author might give his opinion as to the use of increased thicknesses for heavier traffic.

Proof of aging characteristics of macadam pavement using penetration grade asphalt has been observed many times. There are many cases in which macadam pavement over 30 years old, when excavated, showed the asphalt in thick layers still bright and sticky with a minimum amount of oxidation.

There is a considerable amount of skill required in the construction of macadam pavements. This is undoubtedly a contributing factor to its apparent decrease in popularity.

ROBERT D. JOHNSON, Closure—Based on knowledge gained primarily from actual experience, in order to develop a satisfactory macadam pavement, some of the important conditions that should be met are:

1. Provide for appropriate equipment;
2. Provide for competent supervision;
3. Make sure that stone is sufficiently hard, is not susceptible to excessive stripping by bituminous binders, does not consist of an excessive amount of elongated particles, is satisfactorily clean, and does not contain an excessive amount of moisture;
4. Call for a bituminous binder that will adequately fulfill its intended function; and
5. Actually construct the pavement only in satisfactory weather.

Another important point is that macadam has some outstanding favorable characteristics, one of which is flexibility. Because of these characteristics, macadam, in many instances would be the pavement that best fulfills the requirements established. Unfortunately, macadam has been substantially neglected by research and suffers from a tremendous amount of careless, unproven and unwarranted criticism. A modern research program, specifically oriented to clear up many of macadam's uncertain influencing factors, is very much in order, and actually long overdue.

Laboratory and Field Evaluation of Lightweight Aggregates as Coverstone for Seal Coats and Surface Treatments

BOB M. GALLAWAY and WILLIAM J. HARPER
Respectively, Research Engineer and Assistant Research Engineer, Texas Transportation Institute, College Station

•THE RECENT introduction of lightweight aggregate as a coverstone for seal coats and surface treatments was prompted by predicted improved construction and service characteristics of the material. The Texas Highway Department during 1963 and 1964 accepted synthetic (lightweight) aggregate as an alternate to precoated crushed limestone as a cover material for seals and surface treatments placed in Districts 2, 8, 18, 23 and 25. In 1965 it was accepted as optional.

This report includes the laboratory evaluation of lightweight aggregates from 7 sources, 6 in Texas and 1 in Louisiana. Two of these materials have been field tested on an extensive scale. This study covers the field evaluation of material A only.

The use of seal coats with and without cover aggregate dates back many years in the maintenance programs for highways and city streets in the United States and many other countries. Construction procedures vary widely with the different groups responsible for the use of this maintenance tool. Some procedures are rather simple while others are quite detailed; however, as a general statement it has not been possible to eliminate the need for experience and good judgment in the successful design and construction of this type of surface.

Because of widespread use and the many variables that exist, it is not surprising that errors are made, although these errors are not readily apparent at the time of construction. In certain instances it is not practical to eliminate potential errors in limited segments of a given road. For example, many farm-to-market roads have numerous sharp curves for which it is not practical to made adjustments in asphalt application rates. Upgrades present similar problems. There are also natural variations in the precise nature of the surface being repaired. Many roads, when they finally receive a seal coat, have been patched and in some cases sections have been completely rebuilt (Fig. 1), resulting in wide variations in the demanded rate of asphalt and/or coverstone application. Normal construction procedures do not take these variations into account.

It is, however, usually wise to design a seal coat—and all seal coats should be designed and not simply constructed as an expedient—to meet certain needs, such as:

1. To seal the bituminous mat against the entrance of air and water;
2. To absorb the wear of traffic action;
3. To increase the skid resistance of the wearing surface;
4. To reduce the brittleness of the underlying layer of bituminous material; and
5. To increase the night visibility or luminosity of the surface.

In these respects, using lightweight aggregates as coverstone is no different from using precoated material or regular aggregates. The fact that precoated aggregate and lightweight aggregate have been and are now being included in specifications as alternates implies that the materials are equal, at least to the desired end points of construction and service.

Paper sponsored by Committee on Bituminous Surface Treatments and presented at the 45th Annual Meeting.

Figure 1. Old surface has variable demand for asphalt from point to point.

OBJECTIVES

This study is concerned with laboratory and field evaluations of expanded clay and shale for use as a coverstone for highway seal coats. The primary objectives of this study were to determine whether or not lightweight (synthetic) aggregates are acceptable as equal to precoated limestone available in the same general market area. To compare the physical characteristics of the lightweight aggregate with the accepted serviceability of precoated stone, it was necessary to design and carry out an extensive laboratory study on the lightweight material. The evaluation measurements then became a part of the primary objectives.

Because such a study is incomplete without actual field trials, a large number of seal coat and surface treatment jobs built under regular Texas Highway Department specifications were included in the program. Field evaluations on both lightweight aggregate and precoated crushed limestone seals were included for comparison.

RESEARCH PLAN

Research began in 1963 in both the laboratory and the field, with the first designated test section being built in Foard County on FM 267 on June 7, 1963. Other jobs were already completed, under construction or planned in other districts in northwestern Texas. In areas where it was expedient to work with the contractor and/or highway department personnel, additional designated test sections were set up. Because of the delay in getting the research program under way a major part of the 1963 seal program was already completed or under way before arrangements could be made for setting up test sections. Therefore, it was necessary to take road samples from completed projects where no changes in materials, application rates or construction procedures were effected. However, due to differences in these factors from job to job and district to district, this was not considered a particularly important disadvantage.

Outlined below are the specific items of research planned in the program.

Basic Characteristics of the Aggregate

1. Source and type clay or shale used;
2. Bloating agent used, if any;
3. Method of presizing and necessary crushing and sizing after manufacture;
4. Burning time and exit temperature of kiln; and
5. Nature of storage, handling and shipping.

Figure 2. Open pit mining of shale.

Laboratory Evaluations

1. Coverstone retention as affected by application rate of asphalt;
2. Windshield damage;
3. Freeze-thaw effects on soundness;
4. Grading variations;
5. Asphalt absorption; and
6. Resilience.

Construction and Service Evaluations

1. Windshield damage in the field;
2. Design cover rates and asphalt content;
3. Handling methods;
4. Coverstone retention and bond tenacity as affected by road layout;
5. Aggregate degradation due to construction and traffic;
6. Effects of weather; and
7. Acceptance evaluations.

AGGREGATE HISTORY AND PRODUCTION METHODS

Lightweight inorganic aggregates fall into 2 categories: man-made and natural lightweight materials. Man-made aggregates may be subdivided into 2 more groups. According to ASTM Designation C 331-59T (1), the general types of lightweight inorganic aggregates are as follows:

> Aggregates prepared by expanding, calcining or sintering products such as blast furnace slag, clay, diatomite, fly ash, perlite, shale, slate or vermiculite.

Figure 3. Shale storage and conveyor feed to kilns.

Aggregates prepared by processing natural materials such as pumice, scoria or tuff.

Aggregates consisting of cinders derived from the combustion of coal (lignite) or coke.

The specifications further state that the aggregate shall be predominately composed of lightweight cellular and granular inorganic particles with a maximum unit weight of 55 pcf for the coarse fraction.

Lightweight aggregates of various types have long been used in a variety of services from high-strength structural concrete units to acoustical plaster and insulating materials.

During World War I, a number of cargo ships were constructed of structural-grade reinforced lightweight aggregate concrete. One vessel, the Selma, lies awash in Galveston Bay. Recent cores taken from the hull of this ship tested more than 10,000 psi in compression, revealing the fine durability of concrete made from lightweight aggregates. The Haydite patent covering the production of expanded shales and clays was granted to Hayde in 1918 and numerous other patents in this field were granted in the 1920's.

Introduction of lightweight aggregates in the highway field of bituminous pavements has occurred within the last 7 or 8 years, but in the Texas highway system only within the last 3 years. Personnel of the Texas Transportation Institute have worked with industry on the design and application of lightweight aggregate in bituminous pavements for more than 6 years. Test sections were placed in Louisiana on city streets and parish roads and one section, a hot-mix asphaltic surface course, was placed on a state highway and has given excellent service for more than 6 years under heavy traffic.

The use of lightweight aggregate as a coverstone for seal coats in Texas began in the Abilene District in 1962. A section about 1000 ft in length was placed on the inbound lanes of I-20 near the west city limits of Abilene. A double surface treatment was constructed in the Brownwood District in 1961 (2), and another double surface treatment was built in this same district in 1962. In 1963, several hundred miles of secondary roads were sealed with hot asphalt-cement and lightweight aggregate was used as a coverstone. Again in 1964, lightweight aggregate was used as a coverstone for seal coats constructed in 5 or more districts. To date, lightweight

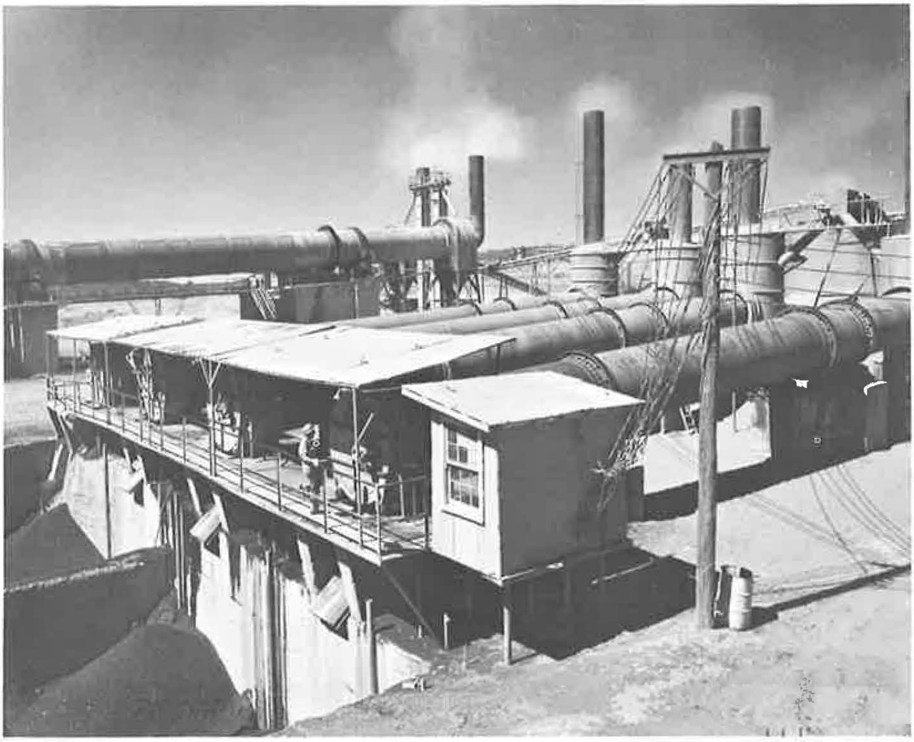


Figure 4. Battery of rotary kilns used to burn shale.

aggregate has been used as cover material on approximately 700 miles of secondary roads on force account and contract maintenance.

The production of this material should be of interest to the reader. The following description applies only to the production of aggregate A, which was used in the first phase of the overall program. There are several plants that produce synthetic or man-made aggregates in Texas and elswhere in the United States (3). However, little has been done and even less published on the use of synthetic aggregates in bituminous pavements.

The production of the lightweight aggregate (4) used in this study consists of pit operations, burning or calcining, crushing and grading, and testing and shipping.

The raw shale, which geologically speaking is a part of the Pennsylvania system, is mined from open pits after removal of the overburden (Fig. 2). The shale is mined from a vertical bank with a power shovel. This method is used to insure a uniform material of reasonably constant moisture content.

The raw shale is trucked from the open pit to a roll crusher where it is crushed, sized and conveyed to covered storage. At this point the moisture content of the shale ranges from 10 to 12 percent. The material is taken from storage by an underground conveyor system and fed into the kilns (Fig. 3). The feed consists of shale sized from $\frac{5}{16}$-in. to 3-in. particles. Presizing of the feed makes possible a more nearly uniform final product of consistently high quality.

The raw shale or clay fines may be formed into pellets and processed in much the same manner as that produced by normal operating procedures. The presized or pelleted material is then burned or calcined for approximately one hour at temperatures in excess of 2000 F in large rotary kilns (Fig. 4). After the clay or shale is heat-processed, it is gravity fed into large rotary coolers and then conveyed to a screening system for removal of certain specified aggregate sizes (Fig. 5). The oversize is sent by conveyors to crushers. The crushed aggregate is then passed through an additional screening system where it is separated into the proper sizes for the market.

Figure 5. General view of commercial lightweight aggregate plant: crushing and sieving is in right foreground.

The processed and sized aggregate is then tested for compliance with buyer specifications. The Texas Highway Department uses Special Specification Item 1269, Aggregate for Surface Treatments—Lightweight (see Appendix).

Of general interest to the reader is some additional background on the subject of burning or calcining shales and clays (5). The early work of Bauer (6) covers most of the basic concepts involved in the bloating and heat stabilization of shales and clays. According to Bauer the raw material requirements are:

1. The material must develop sufficient glassy phase under heat to entrap evolving gases.
2. The material must contain gas-forming ingredients of sufficient quantity to bloat the glass so formed.
3. The gas-forming constituents must release a sufficient amount of their volatile constituents at an optimum rate, and at a temperature and time which coincides with the optimum pyro-plastic conditions of the clay.
4. At these optimum time-temperature glass-forming conditions, the glass must be of a viscosity which will allow formation of suitably-sized blebs or vesicules (for lowest density), and have bleb wall thicknesses that reflect in maximum glass strength.
5. The material should bloat into a vesicular structure at the lowest temperatures for reasons of process economics. On the other hand, such low temperatures must not be the results of alkali or salt flux action causing soluble salts to break down in the final concrete body.

Figure 6 shows the principal stages of gas-forming reactions and in turn relates these to the glass-forming reactions along the temperature scale (6). It is evident that most of the reaction periods overlap on the time-temperature scale, and as the chemistry of the clay or shales changes from one deposit to another many and varied shades

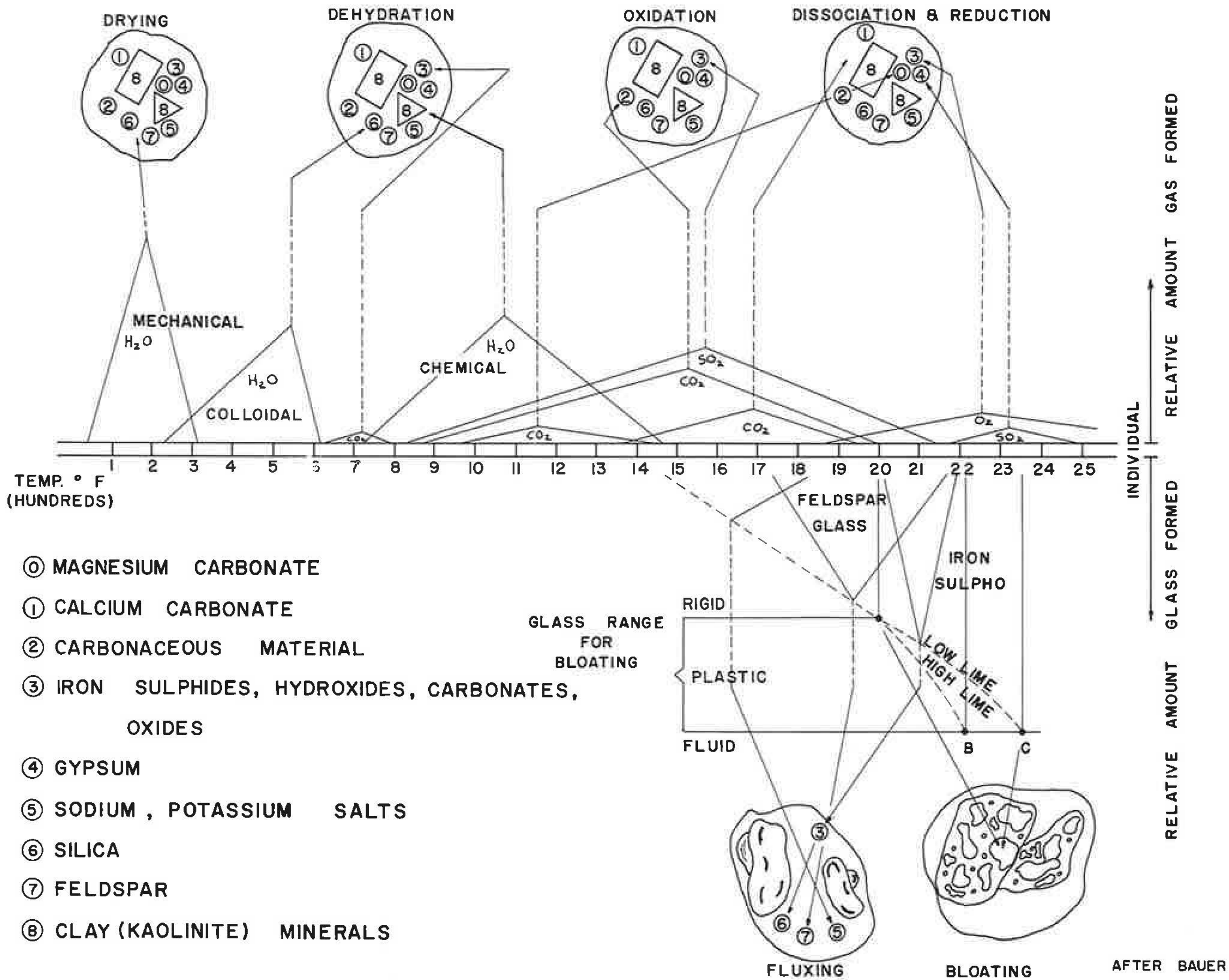


Figure 6. Flow diagram of gas-glass forming vs temperature (6).

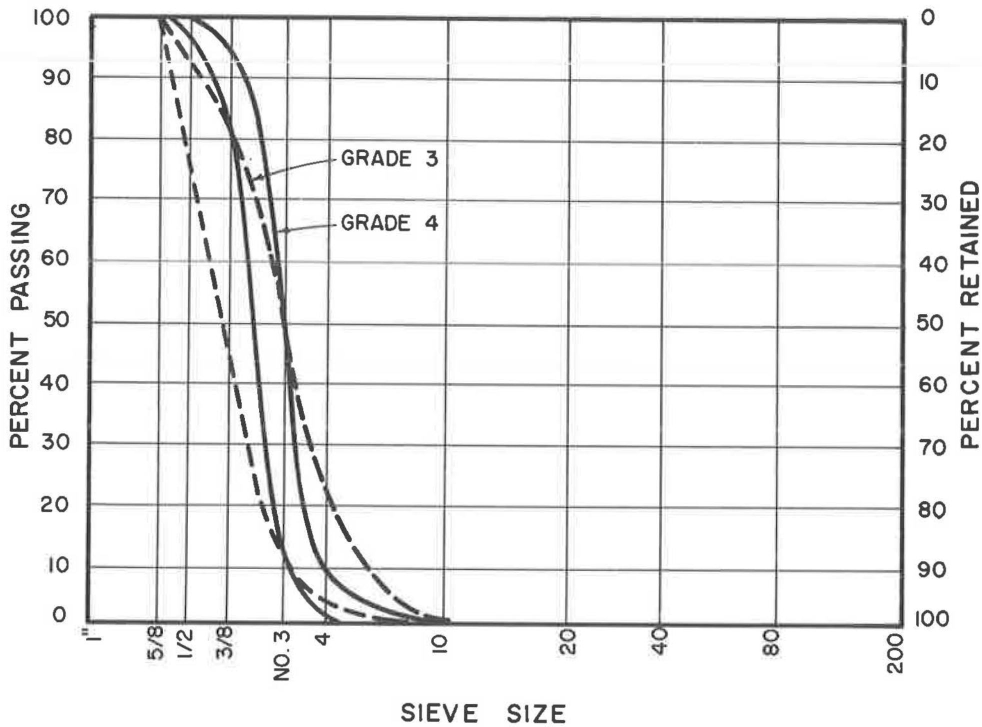


Figure 7. Grading of stockpile and production run type F Grades 3 & 4 material.

of differences may occur. The possible variations multiply as a chemically complex material passes through a rotary kiln where it is subjected to changing conditions from entry to exit.

The glass-forming phase is particularly important to lightweight aggregate producers interested in changing the absorption characteristics of a given material. Bauer (6) points out the importance of the fluxing action of minerals melting at the lower temperatures. Feldspar, for example, has a fluxing action that extends over a considerable temperature range. Such fluxing will lead to the melting of such refractory minerals as lime, magnesium, zinc and various oxides at temperatures lower than they would melt alone.

It is further pointed out that the melting of clays and shales is strongly influenced by: (a) composition (5); (b) density; (c) grain size; (d) dispersion; (e) heating rate; and (f) heating atmosphere.

When the clay or shale reaches that temperature corresponding to the principal gaseous state in commercial kilns, time, temperature and the partial pressure of the combustion gas, excess air and gases from the burned material have their effects on the final results. To form the desired amount and quality of bubbles or blebs in the finished material, expansion of the heated clay or shale is necessary. This expansion is controlled primarily by gas density and glass viscosity properly timed in the reactions involved.

For uniform quality, appreciable changes in kiln temperature must be avoided if the proper glass viscosity is to be maintained during bleb formation. Theoretically, very little gas-forming material is required; less than 0.1 percent sulfur, for example, is sufficient.

Although it is beyond the scope of this report to go into the details of the lightweight aggregate production business, it should be pointed out that for a business of this type, requiring large investments in equipment, radical changes in processes will not be made rapidly. One may hope that the best use be made of equipment and the most

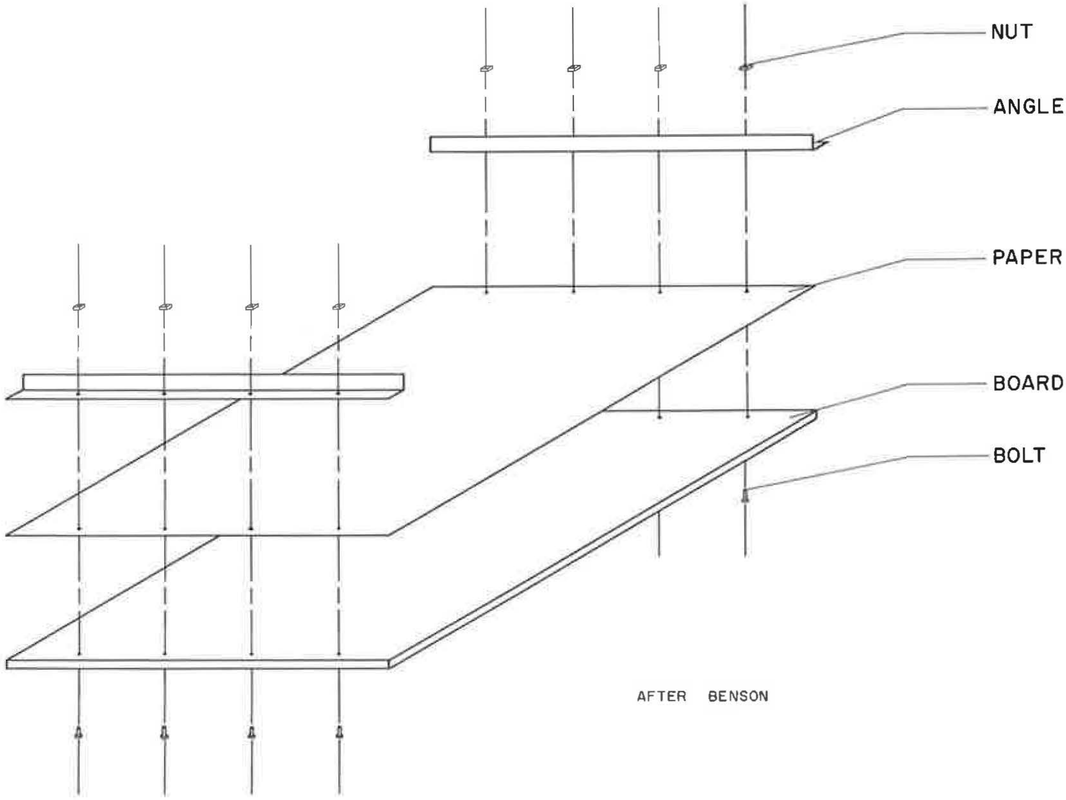


Figure 8. Exploded assembly of board, paper and angles.

Figure 9. Laboratory surface treatment board being covered with paper.

Figure 10. Boards in line for asphalt shot.

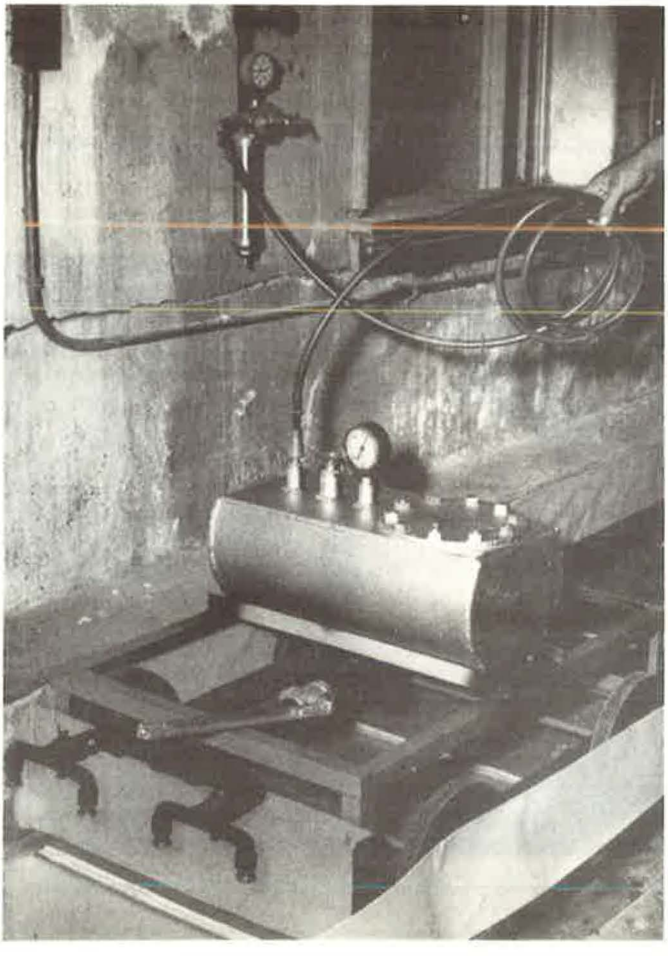


Figure 11. Asphalt distributor used for laboratory retention studies.

Figure 12. Hot asphalt cement being sprayed from small distributor.

Figure 13. Asphalt coated board being weighed to measure application rate.

Figure 14. Aggregate being spread by hand on asphalt coated board.

Figure 15. Pneumatic roller used to seat stone on laboratory surface treatments.

Figure 16. Completed surface is tilted 75 deg and brushed to remove loose stone.

efficient techniques be utilized to maintain a uniformly high quality product to lessen the problems of the user.

Coated aggregates, i.e., aggregates with fused outer shells, are possible today and should do much to solve the absorption problem. At the same time, this coating should increase the effective strength of products incorporating the aggregate.

LABORATORY EVALUATIONS

Although limited use was made of other gradations, the materials used in this study were primarily Grades 3 and 4 lightweight aggregate and precoated crushed limestone (7). Gradation requirements for the various size ranges are listed in the Appendix. Grading curves obtained from producer and field stockpile samplings are shown in Figure 7. The unit weight of Grades 3 and 4 lightweight aggregate was in the range of 38 to 50 pcf, with the higher values generally associated with Grade 3 material. The generally higher unit weight value for Grade 3 material may be explained by a greater variation in the range of particle size. Grade 4 material was consistently more uniform in grading and therefore had a higher void content.

Retention studies indicated a general need for preparatory design work in the laboratory to determine asphalt and coverstone application rates. Laboratory and field tests rule out the practical use of a steel flat-wheel roller for seating lightweight aggregates. The pneumatic roller is highly effective.

The experimental work of damage to windshields from "flying stones" proved that the likelihood of breakage is rather remote for the lightweight materials under study. The work further showed that crushed limestone would cause severe damage at the impact energies included in the experimental work. The frequency of damage was high for the plus ½-in. size material.

The need for altering the test procedure (present modification) to determine laboratory abrasion of lightweight aggregate is based on the fact that the volume of lightweight aggregate may be as much as 3 times that of an equal weight of conventional aggregate. Any appreciable changes in the volume of the sample might be expected to affect the crushing and abrasion characteristics caused by the testing equipment for given testing procedures. However, the data indicate that test method ASTM Designation C 131-55 (1)

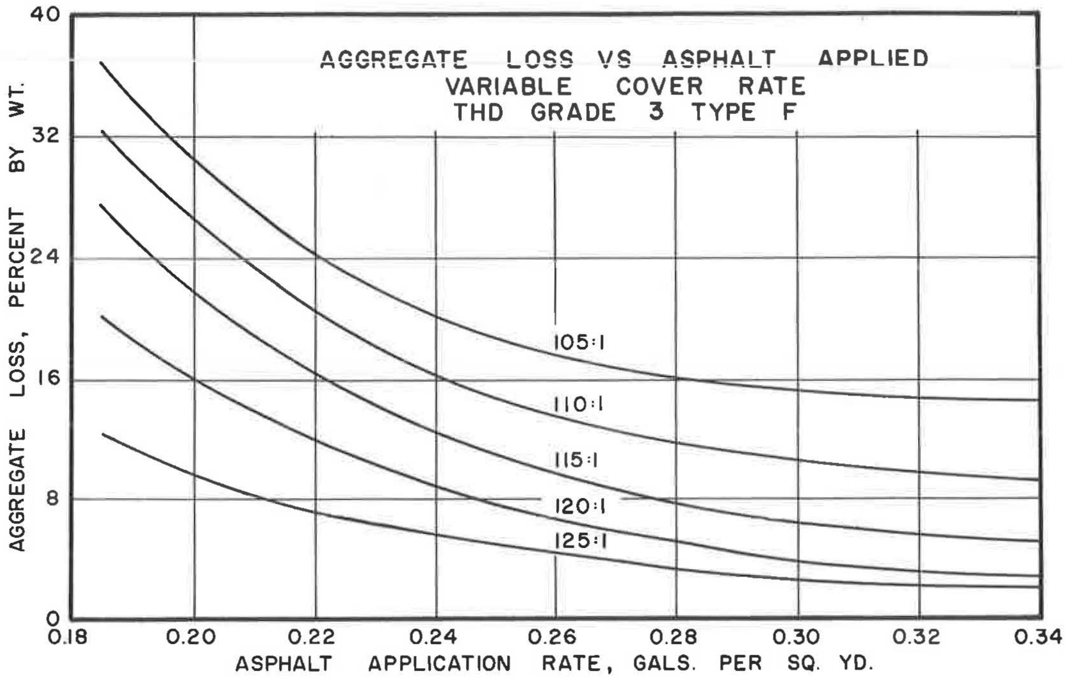


Figure 17.

was the most severe test, and the crushing and abrasion characteristics were not altered appreciably. Modifications may be required as service data are collected and evaluated, but this may require several years. Hence, it is quite possible that the maximum wear of 35 percent as set by Special Specification Item 1269 and determined by Test Method Tex-410-A (Part II) is not a restrictive requirement. Earlier work by Woolf (8) indicates that a wear value of 40 percent maximum for surface treatments would be satisfactory. This assumes natural aggregates and ASTM standard evaluations.

It was anticipated by some that the freeze-thaw damage to lightweight aggregate might be severe; however, for the materials and conditions of the test this was not generally true. But 50 cycles of rapid freezing to 0 F or lower caused appreciable degradation to aggregates B and F. The same finding resulted from the magnesium sulfate soundness for aggregate A.

LABORATORY RETENTION STUDIES

Laboratory design of seal coats and surface treatments was based on previous work done by Kearby (9), Benson and Gallaway (10) and Hank and Brown (11). According to them, the optimum quantity of coverstone required is the amount necessary to cover the area in question one stone deep. The proper amount of asphalt-cement is a function of the average mat thickness and the embedment depth. Careful laboratory measurements revealed that the cover rate for the Grade 3 stone should be in the range of 115 to 125. Under average laboratory conditions it was not possible to retain these optimum amounts of stone even though the asphalt application was changed over a considerable range. It was also found that for rates lower than these amounts it was not possible to adhere all of the stone applied. Other types of stone react in the same manner. There appears to be some double-decking of stone even at very low application rates.

It is felt that the reader will have a better appreciation of the data to follow if the procedures used to obtain them are described.

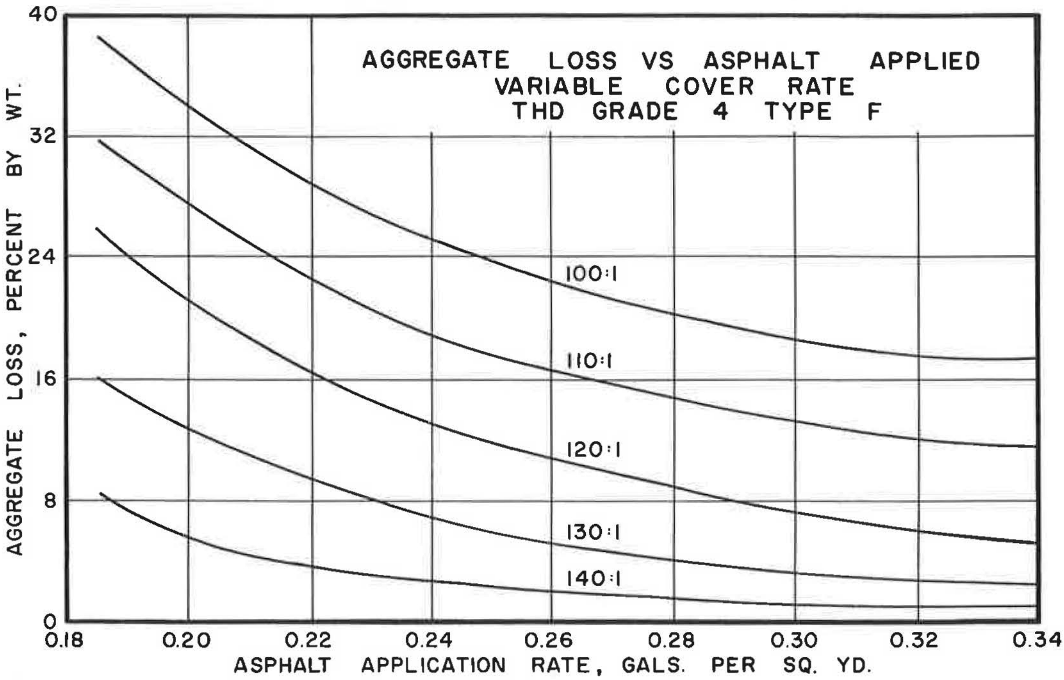


Figure 18.

Figure 8 shows an exploded drawing of the board, paper, angles and bolts. In Figure 9 the actual assembly of these items is taking place. These boards are ½ sq yd in surface area and are covered with heavy brown wrapping paper. After a shot is made and all data obtained, the paper-asphalt-stone composite is easily removed and discarded. The remainder of the assembly, after minor cleaning, is then ready for re-use. After the boards are covered with paper, the exposed upper surface of the angle is covered with masking tape and the covered boards are placed in the "run" (Fig. 10). The boards are centered in the run which is also covered with paper. Side boards about one foot high prevent splatter during the application of the hot asphalt-cement. Masking tape is used to cover the abutting ends of the boards (Fig. 10); its removal after the asphalt is applied exposes a clean surface which simplifies removal of the boards from the run.

The laboratory distributor is shown in Figure 11. The unit is designed to contain about 5 gal of asphaltic material and can be operated at pressures up to 100 psi and temperatures up to 400 F. Pressure is supplied by compressed air through a regulator and filter. Care should be exercised never to allow water to enter with the air. The asphaltic material is heated with gas burners and distributed under pressure through standard Etnyre nozzles at a temperature that produces a Saybolt Furol viscosity of 50 sec ± 10. For the 120-150 penetration asphalt-cement used, this required a temperature of about 310 F. Application of the asphalt is shown in Figure 12.

The exact amount of cement on each board was then determined by weighing the assembly (Fig. 13), after which a weighed quantity of stone was applied (Fig. 14). The aggregate was usually applied beginning 5 min after the asphalt was sprayed on the board, and this operation was completed in 5 min or less.

The aggregate-covered boards were then placed on heavy paper for the rolling operation (Fig. 15). During the application of the asphaltic material, the boards were arranged with metal angles abutting each other; whereas, when these same boards are arranged for the rolling operation, the boards are rotated so the angle is at the outside (Fig. 15). This prevents the angle from being damaged by the roller.

The stone was seated by the pneumatic roller (Fig. 15). The pneumatic tires were inflated to 30 psi and 12 coverages of the roller were used. After rolling, the boards were tilted at an angle of 75 deg with the horizontal and brushed lightly (Fig. 16). Loosely attached and unstuck stone was dislodged and this material was collected and weighed. Data collection and analysis completed a given test.

A complete series of tests was run for both Grades 3 and 4 material in which coverstone and asphalt application rates were varied. Results of these tests are shown in Figures 17 and 18. In the analysis and use of the data, several items should be taken into consideration. Specifications (Appendix) allow Grades 3 and 4 to be very nearly the same in particle size distribution if one takes, say, the fine side of the specification on Grade 3 and the coarse side of the specification on Grade 4. The producer is also allowed to vary the unit weight of the material supplied, but this does not appear to be a disadvantage in this area of application, at least in the field. It does, however, present a problem in a laboratory study where the materials come from several different lots of production. Actual measurements showed that for some shipments Grade 3 had a lower unit weight than Grade 4 from other shipments; other samples, the reverse. Normally it is expected that Grade 4 would have a lower (12) unit weight, if both materials were equally uniform in grading.

Due to the somewhat greater average particle size for Grade 3 material, more stone (by weight) is required to cover a given unit of surface area. Figure 17 shows that the rate of application of the coverstone is given on the individual curves as a ratio. This ratio is the number of square yards of surface covered by 1 cu yd aggregate. For example, the uppermost curve in Figure 17 is labeled 105:1, which means that 1 cu yd was applied at a rate to cover 105 sq yd of surface. If it is assumed that this material weights 43 pcf, then the cover rate would be 11 lb/sq yd. Taking this analysis a bit further, one might assume that asphalt-cement is applied at the rate of 0.30 gal/sq yd and find from this curve that about 15 percent by weight of the stone would not be retained under the conditions of the test. On the other hand, if the stone is applied at the rate of 120:1, then the loss would be 4 percent by weight. For the average Grade 3 material tested and for asphalt-cement application rates in the range 0.28 to 0.32, the coverstone should be applied at the ratio of 120:1. Even then not all the stone is retained in the laboratory experiments, although separate tests on the stone alone indicated that this amount would be retained.

For the segments of the curves to the left of asphalt application rates of about 0.23, the data were quite erratic. However, it is felt that the curves are logically located, and it is not likely that rates this low and lower would be used in seal coat work for this size and grading of aggregate. None of the curves has been extended beyond the 0.34 asphalt application rate. It should not be assumed that rates above this might not be warranted; however, it should be observed that for rates above about 0.28 to 0.30 all the curves become rather flat. This is an indication that under the conditions of the test it was difficult to increase the coverstone retention rate by increasing the amount of asphalt applied. There is some small gain in the amount of stone retained as the stone application rate is increased, but the excess that is applied is, for all practical purposes, wasted.

The foregoing remarks are relegated to single shots of asphalt and single applications of coverstone with operations done under laboratory-controlled conditions. It has been found that with good equipment properly operated under adequate supervision similar results can be obtained in the field. Normally, seal coat work does not require more than a single application of asphalt and coverstone. Should it be considered necessary to place a double application, changes in the design are necessary. Further, the trends indicated in the curves of Figures 17 and 18 do not apply to doubles in all their details.

WINDSHIELD DAMAGE STUDIES

For many years newly constructed seal coats and surface treatments using cover aggregate of any appreciable size have caused some damage to the glass and finish of vehicles using the roads. Even at relatively low vehicular speeds some stone will be

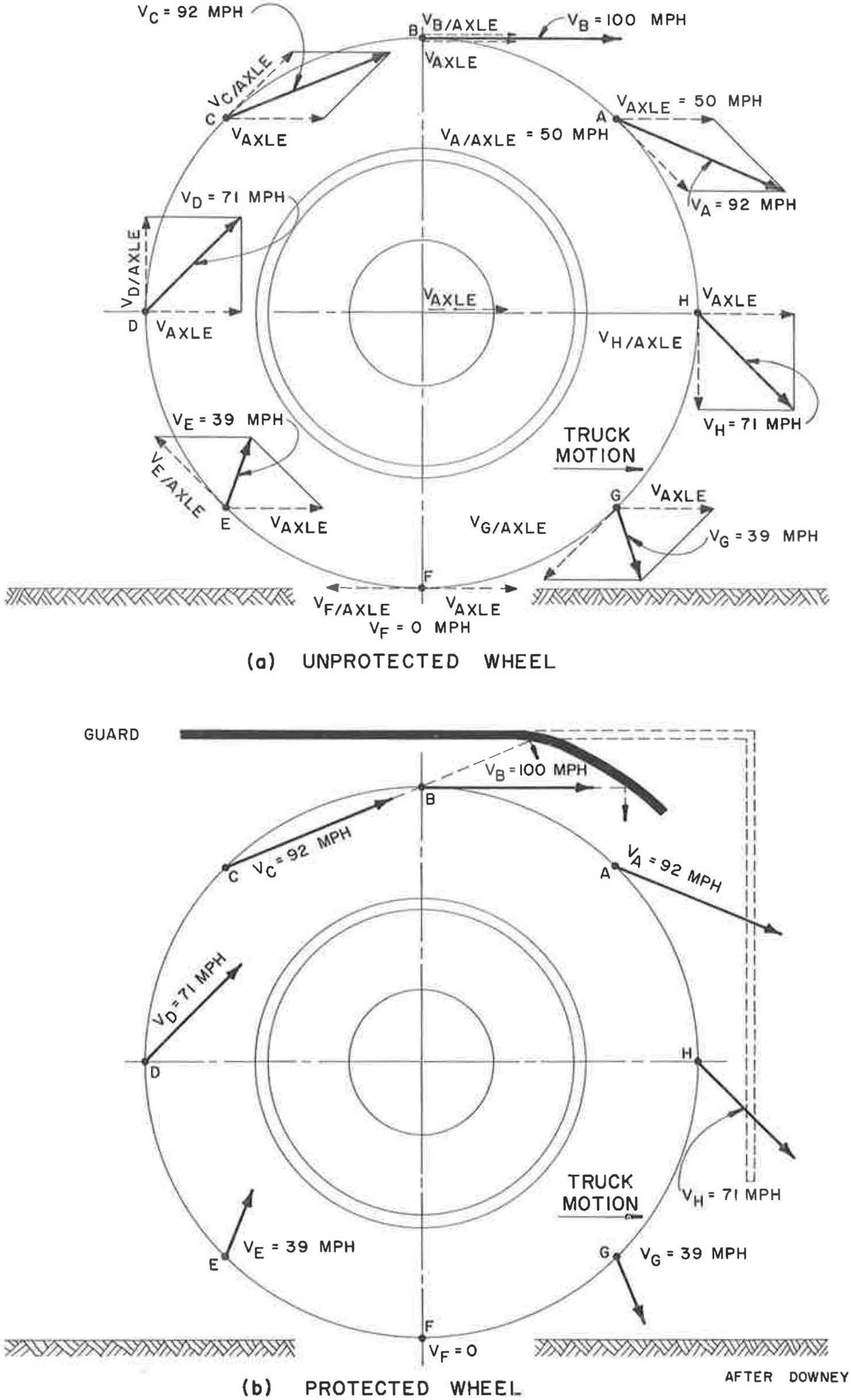


Figure 19. Stone motion as it leaves a truck wheel.

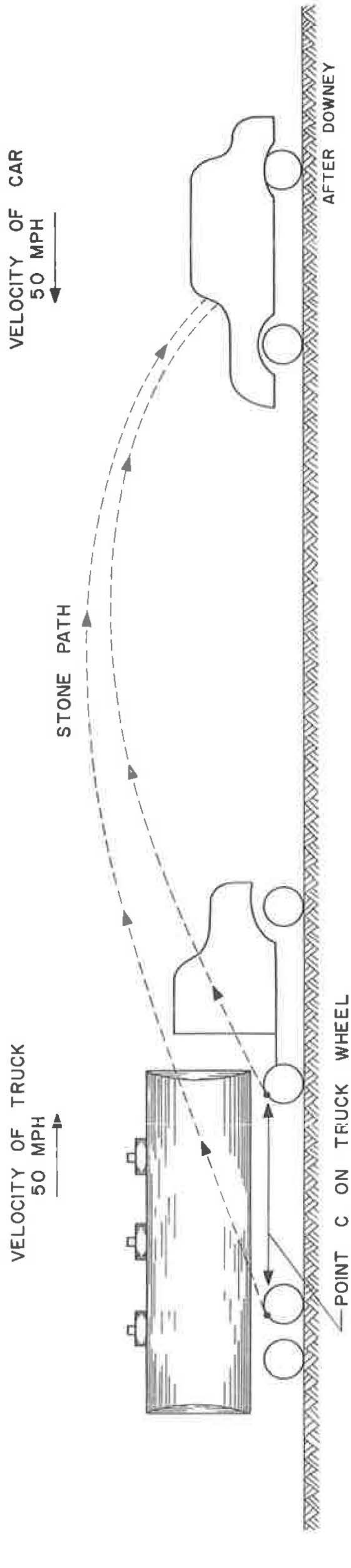


Figure 20. Relationship of car and stones thrown from open truck wheel.

plucked from the road surface and thrown into the path of another vehicle. "Loose Gravel" caution signs are not uncommon during the asphalt construction season and may appear throughout the year on various maintenance jobs.

In 1957, Downey (13) reported a study of the mechanics of stone damage to windshields. This report showed that 57 percent of the windshields examined revealed damage of some type, presumably from flying objects. A questionnaire indicated that almost half of the 415 cases of damage reported were caused while meeting another vehicle. The article also reported that approximately 80 percent of the damage occurring on the open paved road was caused by trucks.

An analysis of the mechanics of the motion of a stone as it leaves a truck wheel is shown in Figure 19. For an axle motion of 50 mph and no slip between the tire and the road surface, a stone leaving the tire tread at point C will have a theoretical velocity of 92 mph. If no loss is suffered from wind resistance this stone would strike an oncoming vehicle (traveling in the opposite direction at 50 mph) at a relative speed of 142 mph. Naturally, there is a reduction in the velocity of the stone as it moves through the air from the truck tire to the windshield to the oncoming vehicle and the velocity at impact will be less than 142 mph, but still it would never be less than 50 mph.

Figure 20 shows the path a stone might take from an open truck's wheels to an oncoming vehicle (13). A simple truck wheel guard suggested by Downey is shown in Figure 19b (13). The dotted lines represent a flap which should be completely effective in stopping any flying stone.

In an effort to reduce or eliminate the damage caused by flying stone, producers of gravel and crushed stone began the production of what is referred to as precoated aggregates for seals and surface treatments. This practice began about 10 years ago. The Texas Highway Department revised its grading requirements for all the materials used in this type construction. The net result of these changes was to reduce materially the "fly stone" hazard.

Lightweight aggregates producers suggested that if their product were used as coverstone, no windshield damage would be caused. This, they reasoned, could be explained on the basis of the much lower weight compared to standard precoated limestone.

An air gun for shooting the stone was fabricated in the local shop (Fig. 21). The gun is based on a design furnished by Monsanto Company (14). The unit is composed primarily of an air pressure regulator, an air storage tank made from a piece of 4-in. steel pipe, a solenoid valve and the gun barrel which is a 15-in. segment of 1-in. steel pipe (Fig. 22). The gun is operated by inserting 2 felt or sponge rubber wads into the barrel and brought in contact with the wadding. The air pressure is set on the air regulator and the air storage tank is pressurized by opening the gate valve between the regulator and the gage. A shot is then fired by action of the solenoid valve with the aid of an electrical switch.

Damaged windshields were obtained from local auto glass repair shops (Fig. 23). Those used were carefully selected and positioned in the assembly so damaged areas would not be in the impact area. The gun muzzle-to-glass distance was set at 10 ft and a fly screen enclosure (Fig. 24) was built to catch broken stone and glass, serve as a safety precaution and make it possible to recover and examine broken stone. The gun operator was required to wear a face guard or fire the gun while facing away from the windshield. Both safety laminated sheet and safety (tempered) plate glass were tested. The safety plate glass used is sold under the trade name of Herculite (15).

The lightweight aggregate fell in the size range $^5/_8$-in. to No. 4 sieve size, so the stone was divided into 3 sizes by sieving. These 3 groups were composed of $^5/_8$-in. to $^1/_2$-in., $^1/_2$-in. to $^3/_8$-in., and $^3/_8$-in. to No. 4 material. Each size range was analyzed by taking representative samples and weighing each stone on a semiautomatic analytical balance. From these data, histograms were prepared and the stones were selected from each of the families of stone making up the histogram. Table 1 gives the average weight and standard deviation computed from the histogram data for each of the materials. The number of stones selected and shot from a given family was a function of the frequency of occurrence of size and weight. Typical histograms are shown in Figures 25 and 26; a family of stones is shown in Figure 27.

As previously mentioned, the muzzle-to-glass distance was 10 feet. This distance was selected for 2 reasons: (a) the directional accuracy of the gun was more dependable at this or a shorter distance, and (b) the lightweight stones lost elevation quite rapidly at shooting pressures below 30 psi, particularly those of the least weight in a given family. For the arrangement used, a distance much less than 10 feet was considered hazardous.

Representative stones were initially shot at 40-, 50- and 60-psi air pressure from 9 families of stone for the lightweight and 9 families of stone for the precoated material. After the initial portion of the study was completed, an analysis of the data indicated that it would not be necessary to shoot at all 3 pressures to determine the relative damage to the windshields. Therefore, the remainder of the lightweight aggregates were shot at pressures of 40 and 60 psi. More than 5,000 stones were prepared and approximately 1,200 of these were shot, 900 lightweight and 300 precoated stones.

As a general rule little glass damage was caused by the lightweight material. It was, however, found possible to break a laminated windshield with the lightweight stones. One such break, the most severe caused by any of the lightweight aggregates, is shown in Figure 28. The diagonal crack to the right of the "star" crack existed in the glass before test. Other noticeable breaks were caused by the lightweight materials, but they were comparatively minor. The crack (Fig. 28) was caused by a stone from aggregate A which weighed 3.78 grams. This stone is one of the heaviest lightweight particles shot, and it is approximately the same weight as the average PB stone. It was hot at a 50-psi pressure and was estimated to be traveling in excess of 100 mph on impact.

The most common result observed was similar to that shown in Figure 29. Here the stone has been "powdered" on impact and some of the shattered material would usually remain on the glass. The scale and sheet of white paper are behind the glass. Also included are numerous invisible points of stone impact. The glass was cleaned by scraping it with a razor blade and washing it with a glass cleaning liquid after each shot. Usually no visible evidence of the shot remained after the cleaning operation.

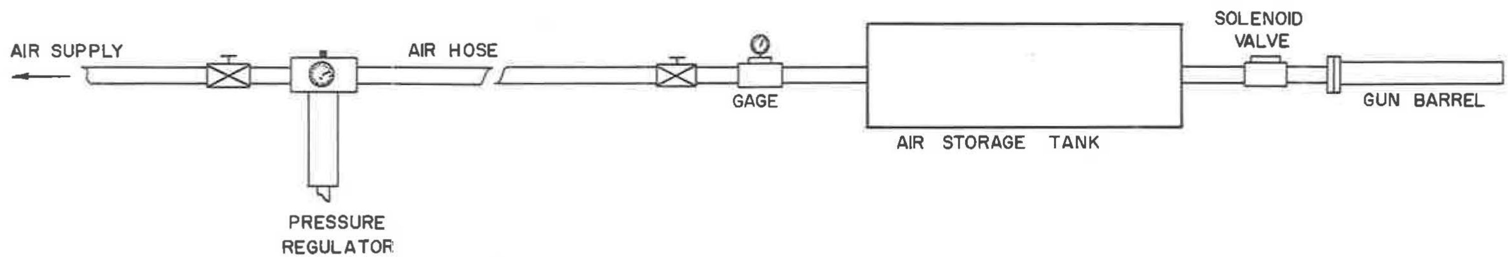


Figure 21. Air gun for shooting stones.

Figure 22. Air-powered gun for shooting stones.

Figure 23. Windshield being placed into position for "flying stone" study.

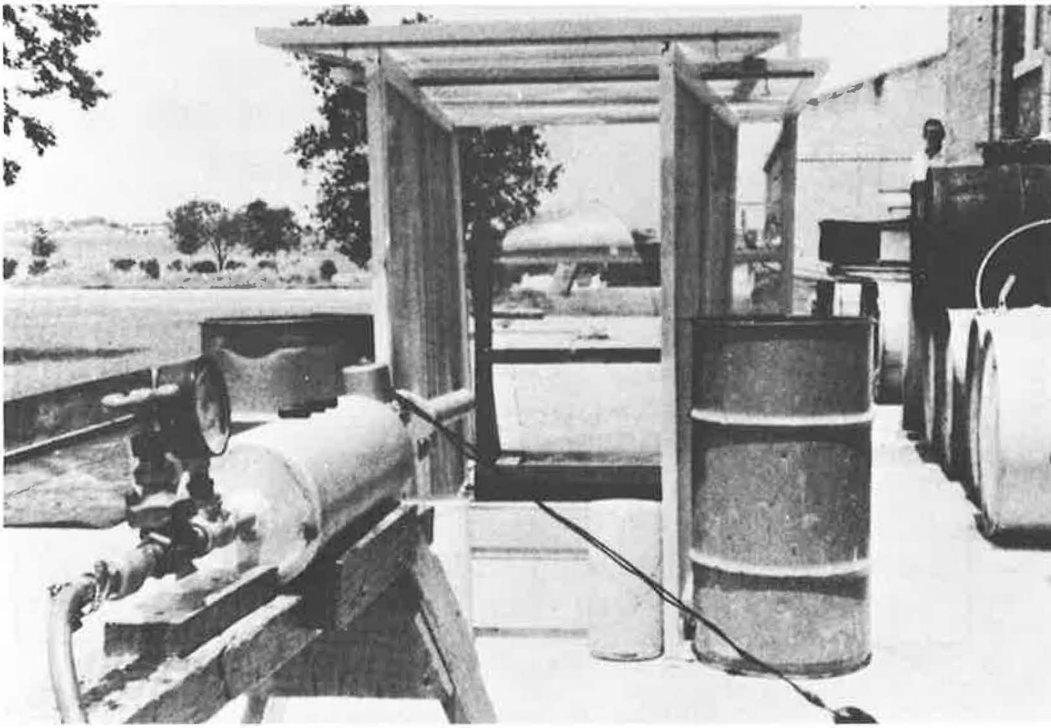


Figure 24. Windshield (target) in screened tunnel from gunner's view.

TABLE 1

AVERAGE WEIGHT AND STANDARD DEVIATION OF STONES

Material Source	Aggregate Size: 5/8 in. to 1/2 in.		1/2 in. to 3/8 in.		3/8 in. to No. 4	
	Avg wt (gm)	σ (gm)	Avg wt (gm)	σ (gm)	Avg wt (gm)	σ (gm)
A	1.576	0.556	0.852	0.346	0.415	0.197
B	—	—	0.850	0.274	0.329	0.144
C	1.629	0.516	0.895	0.345	0.270	0.165
D	2.478	0.882	1.039	0.341	0.568	0.237
E	2.452	1.128	1.025	0.350	0.325	0.102
F	2.772	1.005	1.220	0.437	0.394	0.105
G	2.034	0.771	1.107	0.433	0.473	0.207
H	3.790	0.780	1.889	0.325	0.778	0.372

The shooting of the precoated limestone was scheduled to follow because it was anticipated that the damage would be more severe with the heavier stones. To conserve the supply of windshields, the work plan included shooting stones from the families with the smallest stones first. Naturally the early shots were made beginning with the lowest pressure.

The 3/8-in. to No. 4 precoated limestone shot at 40 psi caused only minor damage to the laminated windshield used; however, as the gun pressure was increased to 60 psi, small cracks were formed and some chips of glass were broken from the impact side.

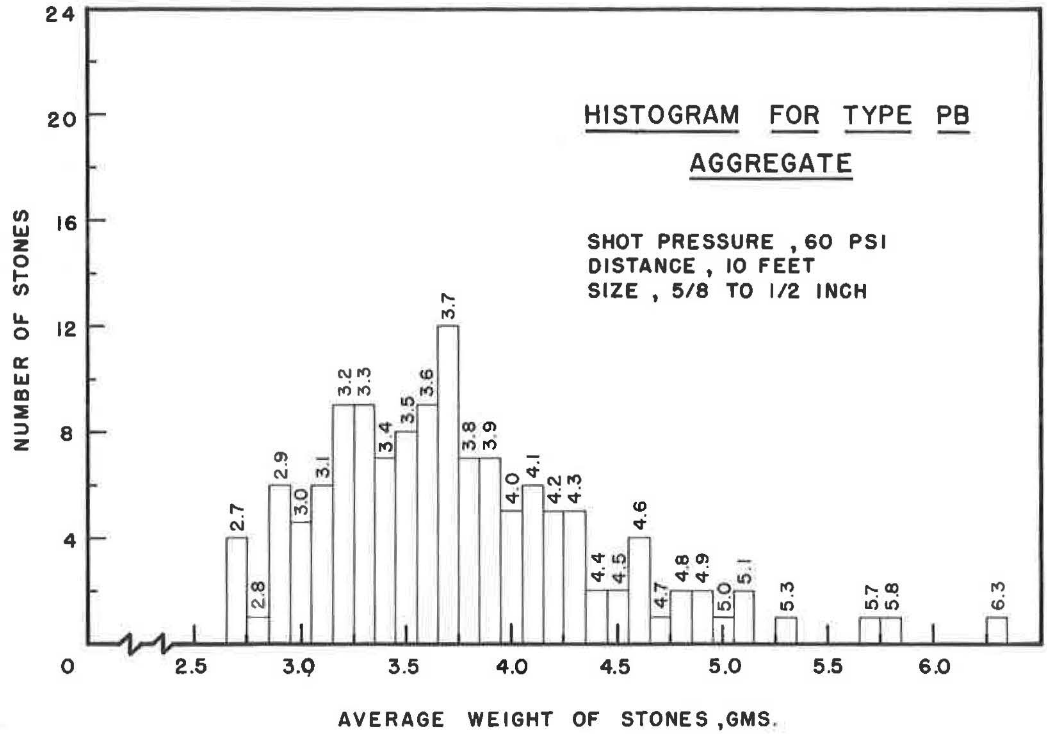


Figure 25.

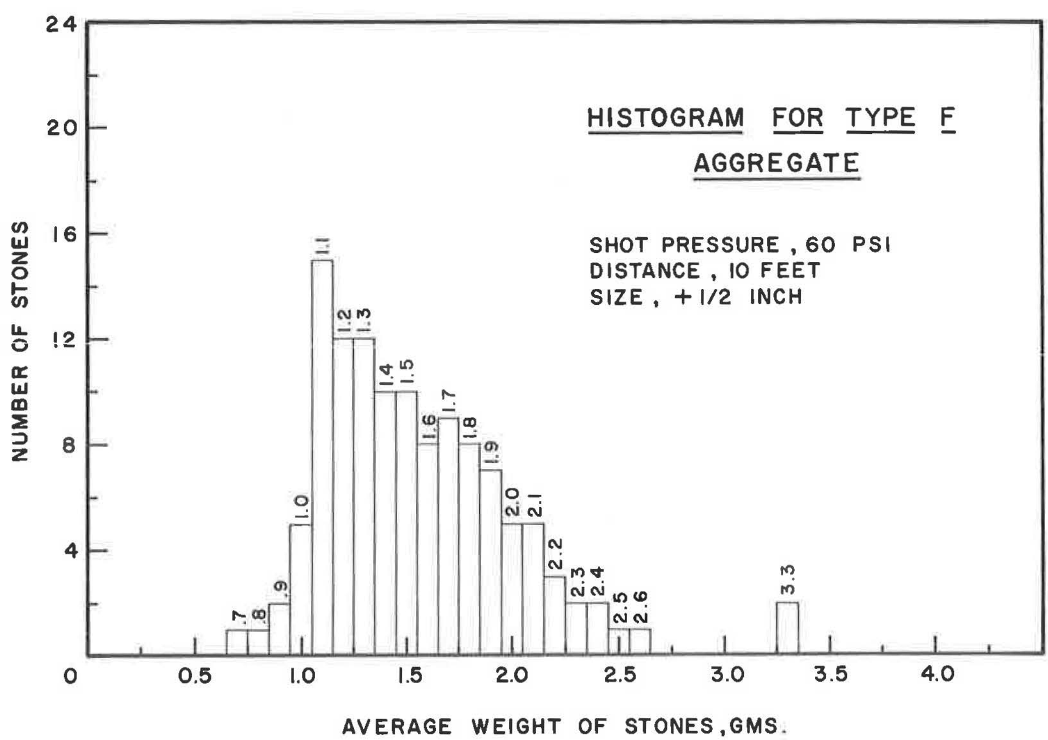


Figure 26.

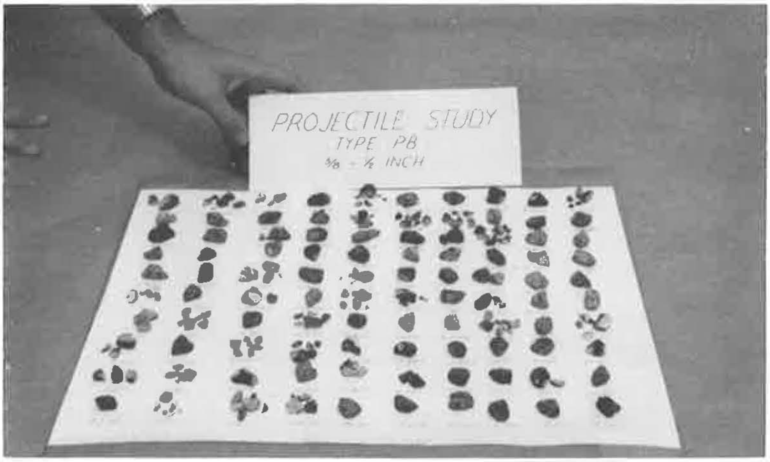


Figure 27. Type PB family of $\frac{5}{8}$ to $\frac{1}{2}$ in. stones after shooting.

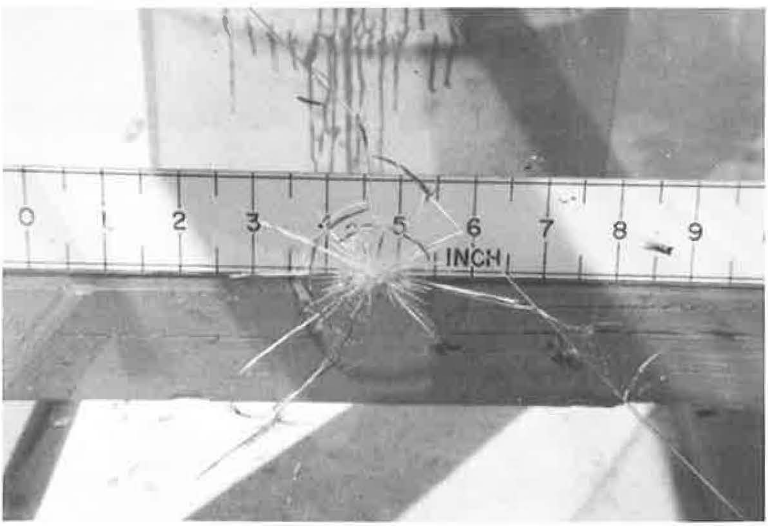


Figure 28. Most severe windshield damage caused by type F material.

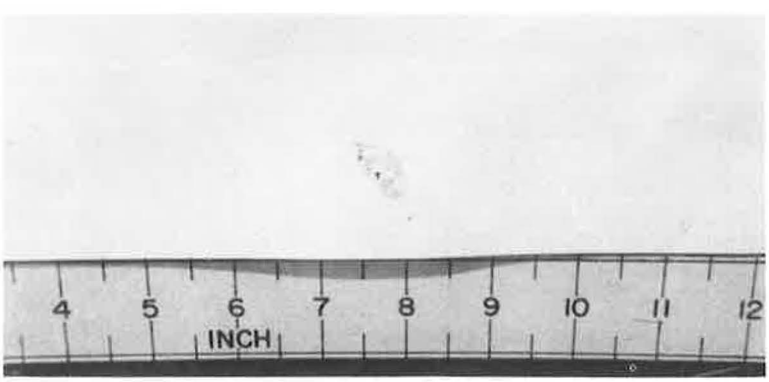


Figure 29. Typical "powder burn" of lightweight aggregate on impact into laminated glass windshield.

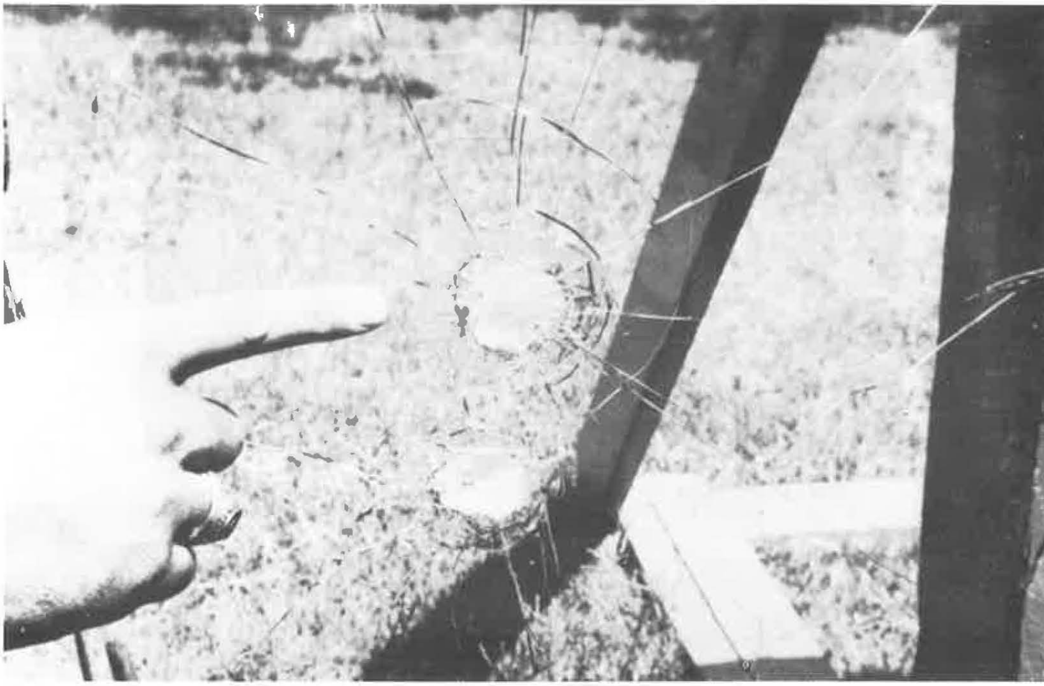


Figure 30. Actual in-service windshield damage caused by a flying object.

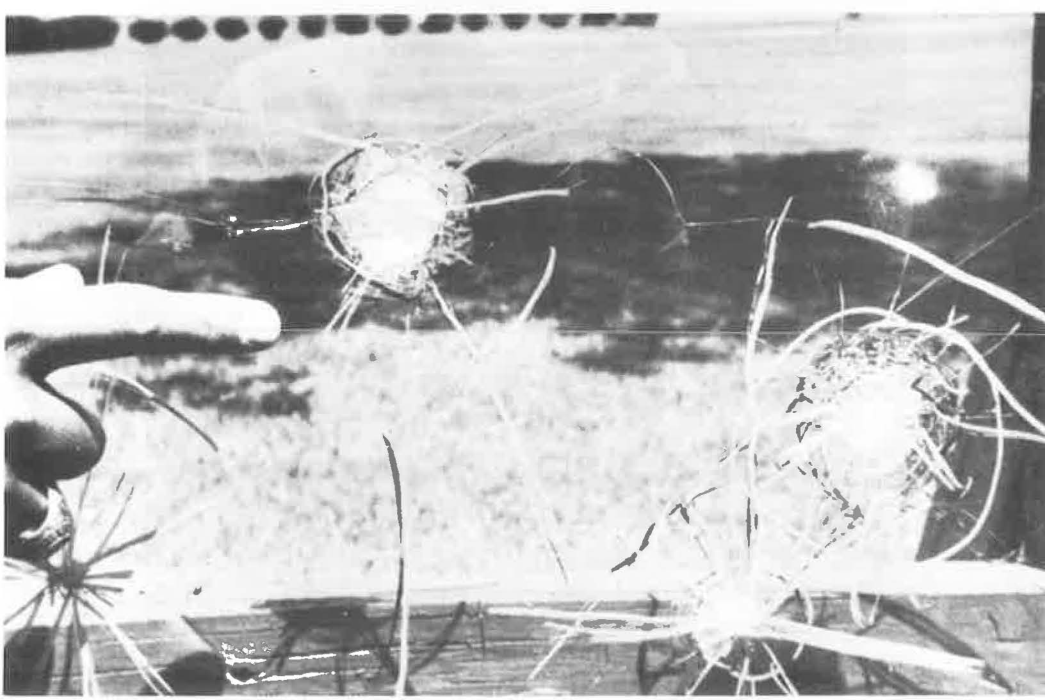


Figure 31. Laboratory windshield damage caused by type PB material. (Same windshield as Fig. 30.)

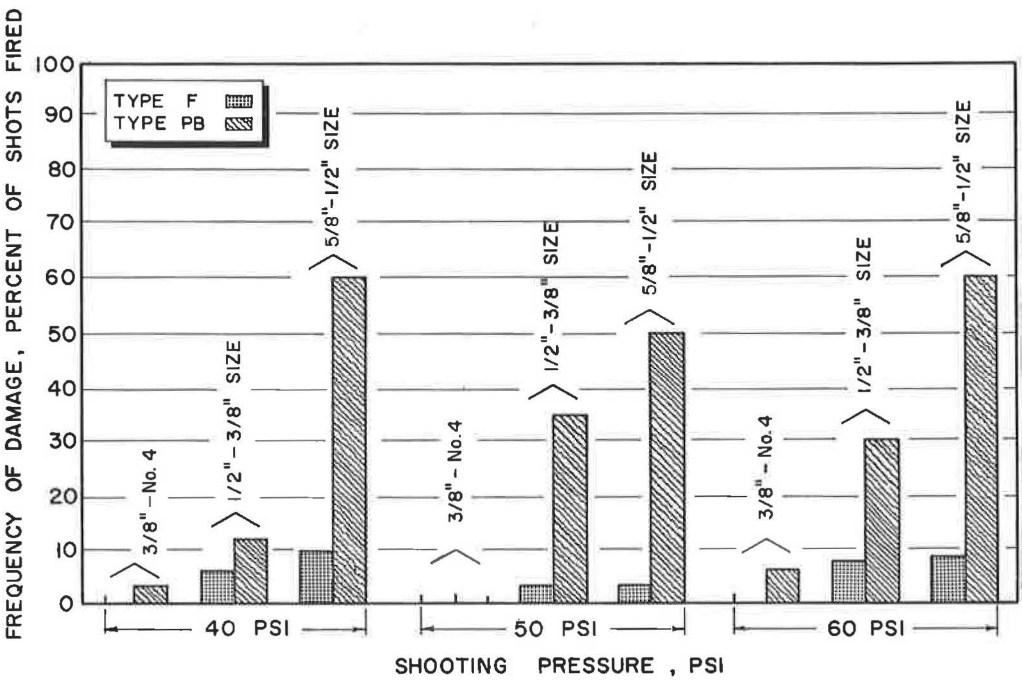


Figure 32. Comparative damage to windshields for type F and type PB aggregate shot at different pressures.

When the largest stones were shot, severe damage to the windshield was a frequent occurrence. One family of stones made up from the $^5/_8$ to $^1/_2$-in. precoated limestone is shown in Figure 27. The stones shot out of this family were selected from Figure 25. The number of shots was in keeping with the frequency of occurrence of the different weights in the family. Of the 100 stones in the family about 30 were shot and the pieces were gathered up and returned to the display board (Fig. 27).

One of the used windshields obtained for a target was already damaged by flying stone (Fig. 30). An area of this windshield with laboratory induced damage is shown in Figure 31. Comparing the damage in the upper center of Figure 31 with damaged areas of Figure 30, there is striking similarity in the actual and the experimental. Other breaks on other windshields included similar damage done in the experiments. It seems reasonable to conclude that the damage in both cases was caused by impacts of similar magnitude, although the stones causing the original damage might have been smaller or larger than those in the study.

A summary of the results from the shooting of lightweight aggregate A and a standard weight material is shown in Figure 32. The frequency of damage to the glass is shown as a percent of the shots fired, and this percentage includes only those shots that caused actual cracks of such size as to be visible to the naked eye. Not included are numerous very small scratches, many of which were discernible only by softly passing one's fingernail over the imperfection on the glass.

The lightweight material described in the previous figure was aggregate A, which appeared to cause a little more damage than the other lightweight aggregates. A comparison of the breakage created by the larger particles of the lightweight material is shown in Figure 33. The smaller sizes, $^1/_2$-in. to $^3/_8$-in. and $^3/_8$-in. to No. 4, cause no appreciable damage. Only aggregates A and F caused damage in the middle size range, whereas aggregate A was the only one causing breakage in the small size range.

Most of the damage was caused by stones whose weight was greater than a unit standard deviation from the mean. For instance, aggregate A had 10.3 percent breaks

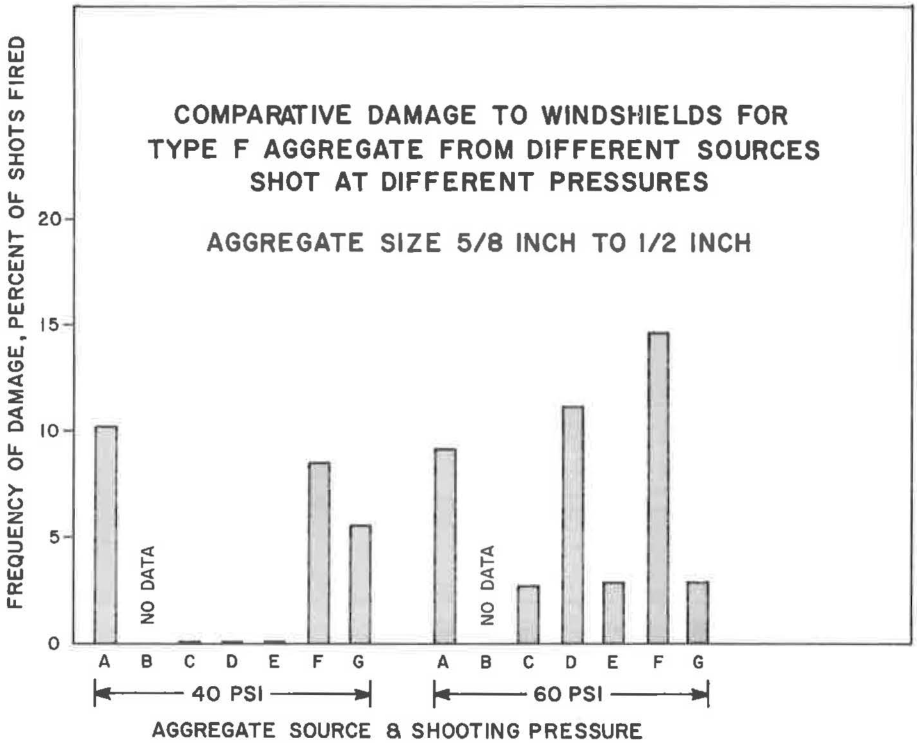


Figure 33.

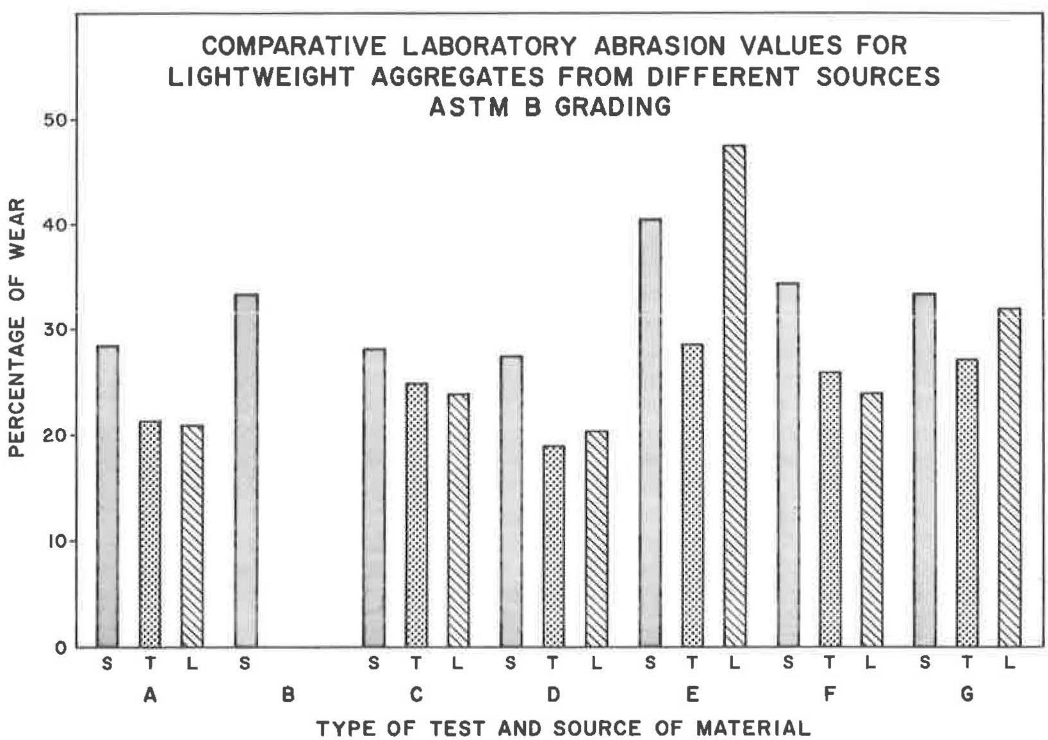


Figure 34.

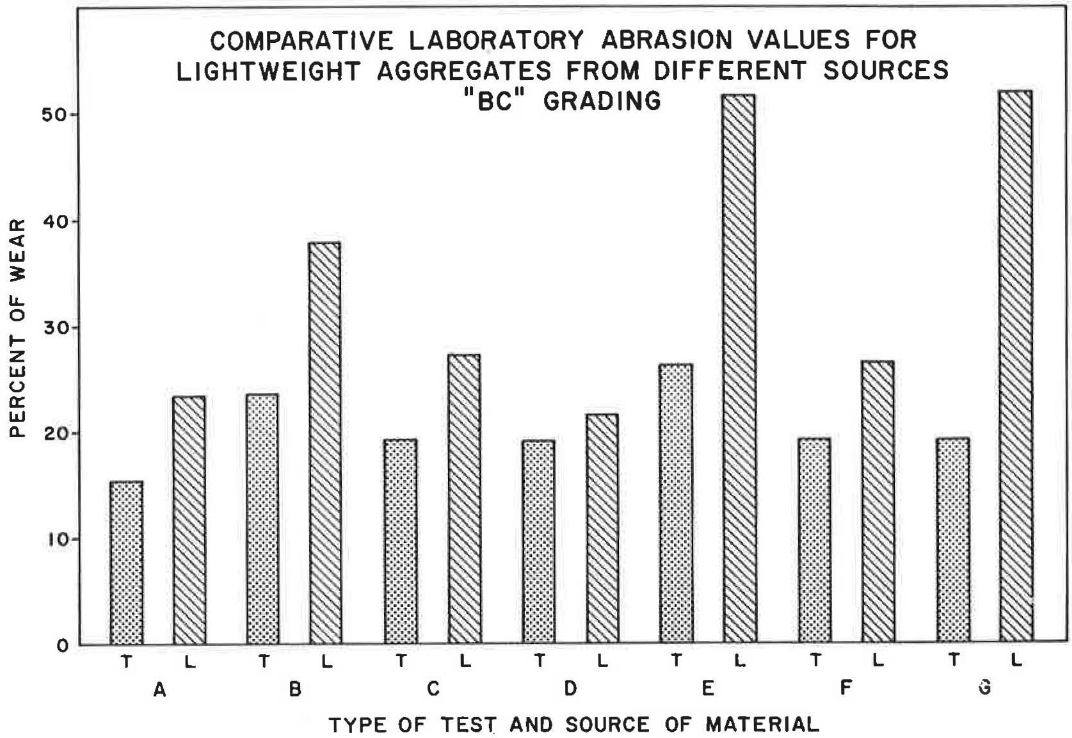


Figure 35.

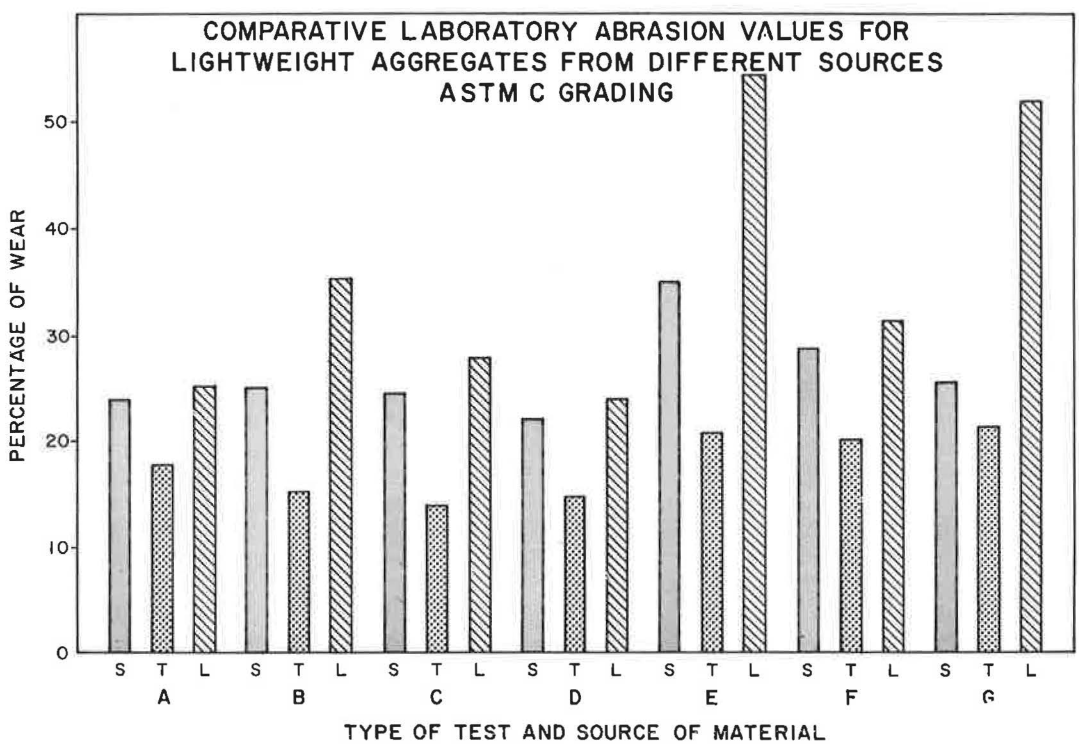


Figure 36.

Figure 37. Type F materials after testing by Texas and Louisiana methods for Los Angeles abrasion.

at 40 psi for the ⅝-in. to ½-in. stone. Of this 10.3 percent, 3.4 percent was caused by stone weights within a standard deviation of the mean and 6.9 percent was caused by stone weights greater than a standard deviation. In keeping with the frequency dictated by the histogram, only 5.6 percent of the stones actually shot within the standard deviation group caused damage, whereas 33.3 percent of the stones shot in the greater than this unit standard deviation group caused damage. If the precoated stone is considered, the total damage was 66.6 percent with 45.5 percent in the unit standard deviation group, and 21.1 percent fell in the group of stones whose weight was greater than the unit standard deviation from the mean. However, 68.2 percent of the shots fired in the unit standard deviation group caused damage, and 87.5 percent of the shots fired in the group whose weight was greater than a unit standard deviation caused damage.

Admittedly the damage picture is not as awesome as it may appear; to create damage the stone must first be thrown and then it must be made to travel in the right direction and have sufficient energy on impact to damage the target. The results serve to show what happens under controlled laboratory conditions when the variables involved are the weight and velocity of the stones that were shot.

MODIFIED LOS ANGELES ABRASION TESTS

An abrasion study of lightweight aggregates was carried out by Rushing (16) at the Louisiana Department of Highways in cooperation with the Bureau of Public Roads. The author concluded that the Deval abrasion test (AASHO Designation T4-35), with certain modifications, would give better results than the Los Angeles abrasion test (17). However, due to the extensive time (about 5 hr) required for the Deval test, Rushing suggested the use of the Los Angeles test modified as follows: (a) a No. 4 sieve should be used for the determination of the loss; (b) 100 revolutions be used in lieu of 500; and (c) the dry aggregate sample be determined by using the same volume of lightweight aggregate as is used for gravel and stone.

The Texas Highway Department (18), made a somewhat similar modification of ASTM's Designation C 131-55. Test Method Tex-410-A Part II is given in the Appendix. The modification calls for reducing the weight of the lightweight aggregate test sample so it will have the same volume as the regular stone or gravel sample, the unit weight of which is assumed to be 97 pcf. The abrasive charge is reduced in the same ratio as the sample weights. No change is made in the number of revolutions of the drum nor is the method of analysis of the loss changed.

TABLE 2

LOS ANGELES ABRASION TEST ON LIGHTWEIGHT AGGREGATE A
(Three Methods)

Size Range	Sample Weight (gm)	Abrasive Charge (gm)	Test Method	Percent Loss
3/4-3/8 in.	5000	4574	ASTM	28.2
3/4-3/8 in.	5000	4569	ASTM	28.2
3/4-3/8 in.	5000	4574	ASTM	29.6
3/8 in.-No. 4	5000	3335	ASTM	24.1
3/8 in.-No. 4	5000	3344	ASTM	23.5
3/8 in.-No. 4	5000	3344	ASTM	23.8
3/4-3/8 in.	2280	2083	Texas	22.6
3/4-3/8 in.	2280	2090	Texas	21.8
3/4-3/8 in.	2280	2090	Texas	20.5
1/2-1/4 in.	2090	1666	Texas	16.3
1/2-1/4 in.	2090	1676	Texas	15.2
1/2-1/4 in.	2090	1676	Texas	15.1
3/8 in.-No. 4	2510	1665	Texas	17.9
3/8 in.-No. 4	2510	1665	Texas	18.2
3/8 in.-No. 4	2510	1673	Texas	17.3
3/4-3/8 in.	2214	4579	Louisiana	21.0
3/4-3/8 in.	2214	4574	Louisiana	20.3
3/4-3/8 in.	2214	4579	Louisiana	21.8
1/2-1/2 in.	2214	4160	Louisiana	22.0
1/2-1/4 in.	2214	4162	Louisiana	20.8
1/2-1/4 in.	2214	4160	Louisiana	26.8
3/8 in.-No. 4	2214	3329	Louisiana	29.3
3/8 in.-No. 4	2214	3346	Louisiana	21.1
3/8 in.-No. 4	2214	3330	Louisiana	26.9
3/8 in.-No. 4	2214	3346	Louisiana	23.3

TABLE 3

AVERAGE VALUES FOR PHYSICAL PROPERTIES OF ROCK[a]

Rock	Toughness		Hardness		Loss by Abrasion			
					Deval Test		Los Angeles Test	
	No. of Tests	Avg	No. of Tests	Avg	No. of Tests	Avg, Percent	No. of Tests	Avg, Percent
Amphibolite	70	14	56	16	87	3.9	30	35
Basalt	203	19	192	17	203	3.1	24	14
Chert	29	12	29	19	78	8.5	6	26
Diabase	285	20	253	18	340	2.6	63	18
Diorite	48	15	45	18	60	3.1	—	—
Dolomite	612	9	586	14	708	5.5	134	25
Felsite[b]	127	17	118	18	150	3.8	9	18
Gabbro	42	14	38	18	45	3.0	4	18
Gneiss	386	9	365	18	602	5.9	293	45
Granite[c]	703	9	589	18	718	4.3	174	38
Limestone	1315	8	1209	14	1677	5.7	350	26
Marble	188	6	162	13	175	6.3	41	47
Quartzite	161	16	146	19	233	3.3	119	28
Sandstone	681	11	613	15	699	7.0	95	38
Schist	212	12	180	17	314	5.5	136	38
Syenite	32	14	26	18	31	4.1	14	24

[a]After Woolf, "Results of Physical Tests on Road-Building Aggregate."
[b]Including andecite, dacite, rhyolite, and trachyte.
[c]Including granodiorite, pegmatite, and unakite.

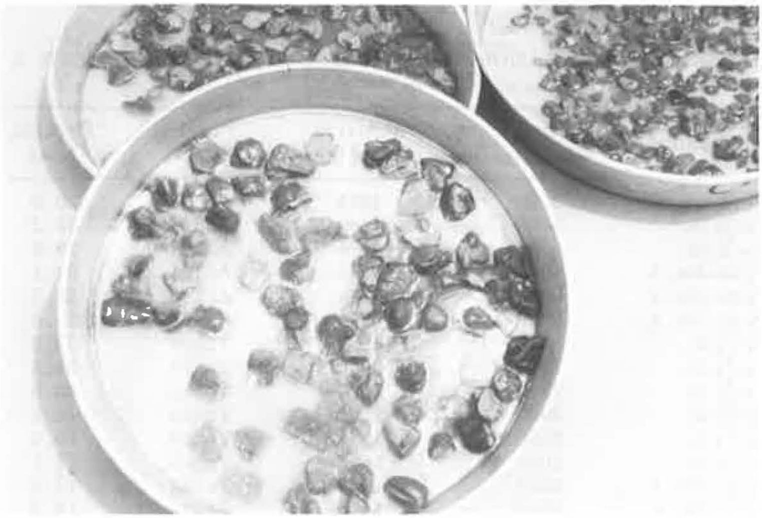


Figure 38. Type F material after freezing in water.

Three methods for evaluating the wear characteristics of the lightweight aggregate were used and the average results are compared in Figures 34 through 36. The materials after test are shown in Figure 37.

Since the aggregates were primarily in the size range $^5/_8$-in. to No. 4 sieve, it was considered advisable to select samples fitting both the B and C gradings of ASTM C 131. Actually the materials were made up of sizes that straddled B and C grading; therefore, another group of abrasion tests was run using samples designated as BC grading.

Table 2 gives an example of comparative rates of wear for lightweight aggregate A tested by the 3 different methods listed. Details on the grading and weights of the samples, as well as the weight of the abrasive charge are given. It appears that the regular ASTM test is more restrictive than either of the other 2 modifications for the coarser B grading, but the Louisiana method is the more severe test for the finer C grading. This is attributed to the difference in method of evaluation. The C grading contains 50 percent of a $^3/_8$-in. to No. 4 materials, and if the evaluation is to be made on the No. 4 sieve, it would not require much breakage to accumulate high percentages of loss. However, the Louisiana method is quicker and easier to run, since it requires only 100 revolutions of the drum. Also, the variability of the individual tests suggests that in the interest of saving laboratory testing time washing and drying of the retained material could be an optional requirement.

Table 3 from Woolf gives the average values for the physical properties of rock. Comparing results obtained from testing the lightweight aggregate of this project with the values in Table 3, it is apparent that none of the test procedures used gives a true picture of the impact and abrasion resistance of the lightweight material. Many of the lightweight particles may be individually crushed by foot pressure; yet, the service record on the material is good. Still, some means of specifying and evaluating a material preparatory to its use is needed, and acceptance based on such tests could be made conditional until sufficient proof from the field is available.

FREEZE-THAW TESTS

Since lightweight aggregate was introduced as a coverstone, some doubt has arisen concerning the resistance of such materials to freezing and thawing in the presence of water. It is reasonable to suspect that a material with a relatively high absorption capacity might be damaged appreciably if saturated and cooled to low temperatures of 0 F or colder. Neither present standard nor special specifications of the Texas

TABLE 4

RAPID FREEZE-THAW OF LIGHTWEIGHT AGGREGATE

Sample	Size Range	No. of Particles	Wt Before Test (gm)	Wt After 50 Cycles (gm)	Wt After 100 Cycles (gm)
A	5/8-1/2 in.	100	163.1	140.0	125.8
A'	5/8-1/2 in.	100	171.1	135.9	111.9
B	1/2-3/8 in.	200	189.8	174.8	171.6
B'	1/2-3/8 in.	200	199.1	181.9	178.8
C	3/8 in.-No. 4	300	95.9	94.3	94.2
C'	3/8 in.-No. 4	300	104.1	103.1	102.8

TABLE 5

ACTUAL PERCENT LOSS DUE TO FREEZING AND THAWING OF LIGHTWEIGHT AGGREGATES

Material Source	50 Cycles			100 Cycles		
	5/8-1/2	1/2-3/8	3/8-No. 4	5/8-1/2	1/2-3/8	3/8-No. 4
A	17.4	8.3	1.3	28.8	9.9	1.5
B	42.8	45.2	16.0	71.5	68.7	30.7
C	6.0	14.8	2.6	17.0	22.4	6.2
D	5.2	9.3	2.2	10.8	15.2	2.5
E	3.3	8.8	4.0	11.5	18.0	7.5
F	12.5	14.5	5.3	26.8	27.7	14.3
G	29.0	28.6	3.9	39.1	35.8	7.3

TABLE 6

CORRECTED PERCENTAGE LOSS AFTER 100 CYCLES OF FREEZING AND THAWING

Sieve Size	Grading of Original Sample	Actual Loss (%)	Weighted, Loss (%)
	Type F Grade 4		
5/8-1/2 in.	1	28.8	.28
1/2-3/8 in.	14	9.9	1.39
3/8 in.-No. 4	76	1.5	1.40
Total loss			3.07
	Type F Grade 3		
5/8-1/2 in.	12	28.8	3.45
1/2-3/8 in.	22	9.9	2.18
3/8 in.-No. 4	55	1.5	.83
Total loss			6.46

TABLE 7

FREEZING AND THAWING CORRECTED PERCENTAGE LOSS

Material Source	Grade 3		Grade 4	
	50 Cycles	100 Cycles	50 Cycles	100 Cycles
A	4.7	6.5	2.3	3.1
B	24.8	41.9	20.9	36.6
C	5.7	10.3	3.0	6.5
D	5.4	8.2	2.6	3.4
E	6.6	13.4	3.7	6.9
F	8.5	18.8	5.0	12.8
G	12.5	17.8	5.8	9.0

Highway Department requires that cover aggregate be subjected to a freeze-thaw test. Neither ASTM nor AASHO lists a test procedure specifically designed for testing aggregates of this type. It was therefore necessary to design a freeze-thaw test for the lightweight aggregate used in this study to approximate the nature of the exposure experienced in the field.

A chest-type freezer was used as the freezing chamber and the prepared samples were exposed to a freezing atmosphere in shallow metal pans. Before the first cycle began, distilled water was added to the pans to bring the level up to a point about half the depth of the stone (Fig. 38). As the test progressed from cycle to cycle, distilled water was added when necessary to maintain this level. A freeze-thaw cycle consisted of about 2 hr and 15 min of quick freezing and about 30 min of thawing at 75 ± 3 F. The freezing chamber temperature was in the range -14 to +4 F through 100 cycles of freezing.

Of the 7 sources of lightweight aggregate, most of the material would pass a ⅝-in. sieve and be retained on a No. 4 sieve. For test purposes the material was therefore divided into 3 fractions in accordance with the data given in Table 4. The number of stones selected for test in each size range was the approximate number required to cover the pan one stone deep.

After the first 50 cycles, the samples were dried and weighed and any particles passing the sieve on which they were retained before the test began were removed. The remaining stones, those retained on the sieve, were then subjected to an additional 50 cycles of rapid freezing and thawing. The loss was again checked by sieving. Any particles passing the sieve on which they were retained before the test were reported as loss (Tables 4, 5). The greater losses occur in the 2 larger aggregate sizes. There are 2 reasons for this behavior. First, the loss was created by a spalling action or breaking of the corners of the rocks. In the larger sizes a broken corner may have caused sufficient size reduction to allow the aggregate particle to pass the sieve, but in the smaller size (⅜-in. to No. 4) a particle just passing the ⅜-in. sieve could be broken in half by the freeze-thaw action and still be retained on the No. 4 sieve, thus showing no loss. Second, the small particles may actually be stronger. As previously mentioned, some of these lightweight aggregate particles are produced from shale which presents planes of weakness parallel to the bedding plane. In the crushing operation of the burned shale, the smaller particles were often created by fracture along these or similar planes of weakness, thus making these small pieces comparatively stronger.

Due to the difference in the amounts of the different sizes in the 2 grades of aggregate, it was considered advisable to correct the actual measured losses in accordance with the original sieve analysis of the 2 grades of lightweight aggregate (Table 6). The Grade 4 stone showed a corrected loss of 3.07 percent compared to 6.46 percent for the Grade 3 material for 100 cycles of exposure. The variation in the amounts of each size material in the original samples caused this difference. The corrected percent loss for all of the lightweight aggregates is given in Table 7.

Normally a seal coat would be expected to last about 4 years, although some jobs may have a much longer life. In the colder areas, it is possible that a road would be subjected to 10 or more cycles of zero weather, but it is considered unlikely that any part of the state would be subject to more than 25 cycles of zero weather in one winter. Nevertheless, in setting up the test conditions, 100 cycles were chosen for evaluating this material. Further study and more field data may indicate the need for a change in the test procedure.

A tentative recommendation would be to restrict the weighted total loss to 8 percent after 50 cycles of rapid freezing and thawing in the presence of distilled water. The freezing should be done at 0 ± 5 F.

SOUNDNESS TESTS

Lightweight aggregate A was subjected to 5 cycles of the soundness test, ASTM Designation C 88-61T, using magnesium sulfate solution.

Results of these tests are given in Tables 8 and 9. The aggregate sizes differ in the fractions making up the sample when compared to ASTM requirements. Modifications

TABLE 8

SOUNDNESS TEST NO. 1

Sieve Size	Grading Orig. Sample	Actual Loss (%)	Weighted Loss (%)
	(a) Sample A, Type F Grade 4		
5/8-1/2 in.	1	0.70	0.01
1/2-3/8 in.	14	1.92	0.27
3/8 in.-No. 4	76	1.50	1.14
No. 4-No. 8	5	2.70	0.14
Total loss			1.56
	(b) Sample A, Type F Grade 3		
5/8-1/2 in.	12	0.70	0.08
1/2-3/8 in.	22	1.92	0.42
3/8 in.-No. 4	55	1.50	0.83
No. 4-No. 8	5	2.70	0.14
Total loss			1.47
	(c) Sample B, Type F Grade 4		
5/8-1/2 in.	1	0.60	0.01
1/2-3/8 in.	14	0.99	0.14
3/8 in.-No. 4	76	1.37	1.04
No. 4-No. 8	5	4.40	0.22
Total loss			1.41
	(d) Sample B, Type F Grade 3		
5/8-1/2 in.	12	0.60	0.07
1/2-3/8 in.	22	0.99	0.22
3/8 in.-No. 4	55	1.37	0.75
No. 4-No. 8	5	4.40	0.22
Total loss			1.26

made in the samples tested were considered necessary due to the original grading of the materials. It is evident that the losses are rather low, but it should be pointed out that the difference in loss of similar fractions was high in certain instances. This, no doubt, may be explained by differences in the original samples from which these fractions were selected. But not to be neglected is the difference in actual particle size within a given range before test. Because the losses were small, any difference is revealed as a large change in the actual loss where these losses are reported as percentages.

If the weighted average loss caused by 5 cycles of the magnesium sulfate soundness test is compared to the loss caused by 50 cycles of the freeze-thaw test, it is evident that the freeze-thaw test is much more severe at least for the number of cycles involved in this study. It may be concluded that 50 cycles and 8 percent loss of rapid freeze-thaw in water may be unduly severe as a requirement for an aggregate of this type. In Figure 39 the results of the 50 and 100 cycle freeze-thaw tests are plotted and extrapolated to zero loss, then superimposed on this graph are the soundness test losses on corresponding grades of lightweight material. It appears that to get approximately equal losses for this particular material, Grade 3 should be subjected to about 10 freeze-thaw cycles and Grade 4 about 25.

These problems were encountered in the soundness test. The coarse lightweight aggregate absorbed a large quantity of the salt solution and this in turn made it

TABLE 9

SOUNDNESS TEST NO. 2

Sieve Size	Grading Orig. Sample	Actual Loss (%)	Weighted Loss (%)
	(a) Sample A, Type F Grade 4		
5/8-1/2 in.	1	1.00	0.01
1/2-3/8 in.	14	1.20	0.17
3/8 in.-No. 4	76	1.63	1.24
No. 4-No. 8	5	2.50	0.13
Total loss			1.55
	(b) Sample A, Type F Grade 3		
5/8-1/2 in.	12	1.00	0.12
1/2-3/8 in.	22	1.20	0.26
3/8 in.-No. 4	55	1.63	0.90
No. 4-No. 8	5	2.50	0.13
Total loss			1.41
	(c) Sample B, Type F Grade 4		
5/8-1/2 in.	1	0.50	0.01
1/2-3/8 in.	14	0.90	0.13
3/8 in.-No. 4	76	1.17	0.89
No. 4-No. 8	5	4.50	0.23
Total loss			1.26
	(d) Sample B, Type F Grade 3		
5/8-1/2 in.	12	0.50	0.06
1/2-3/8 in.	22	0.90	0.20
3/8 in.-No. 4	55	1.17	0.64
No. 4-No. 8	5	4.50	0.23
Total loss			1.13

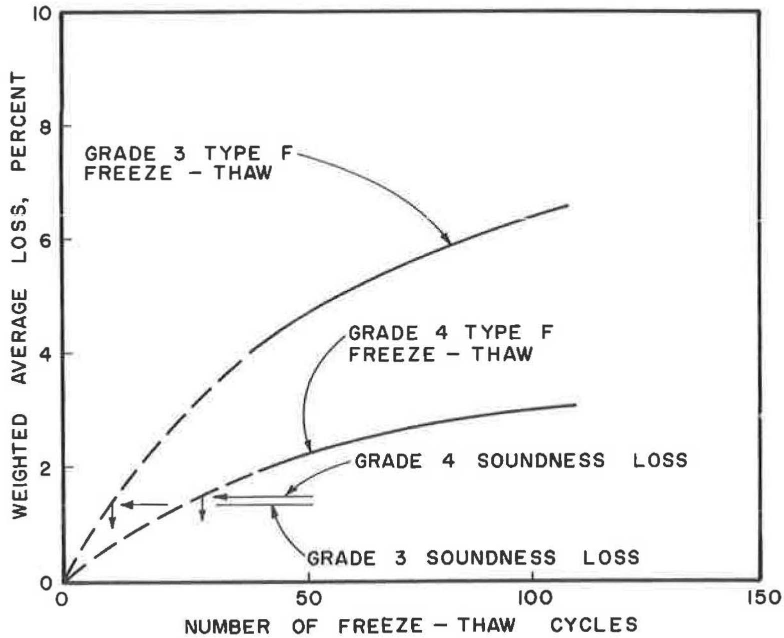


Figure 39. Comparative freeze-thaw and soundness losses for aggregate A.

TABLE 10

DESIGNATED PRECOAT AND LIGHTWEIGHT AGGREGATE FIELD TEST SECTIONS

Highway (FM)	County, District	Length of Sec (ft)	Coverstone Type, Grade	Rolling (hr/mi)	Asphalt (gal/sq yd)	Coverstone (cu yd/sq yd)	Traffic (vpd)
267	Foard-25	2250	F-4	Steel-15	0.30	1-105	150
267	Foard-25	2250	F-4	Pnu-12	0.30	1-105	150
267	Foard-25	1640	F-4	Pnu-12	0.36	1-105	150
1192	Johnson-2	1680	F-3	Pnu-4	0.29	1-105	100
1192	Johnson-2	1560	F-3	Steel & Pnu-5	0.30	1-130	100
1192	Johnson-2	2100	F-3	Steel & Pnu-5	0.28	1-105	100
1192	Johnson-2	1680	F-3	Steel & Pnu-5	0.29	1-105	100
1192	Johnson-2	1920	F-3	Pnu-5	0.28	1-120	100
1715	Erath-2	1980	F-4	Steel-1 Pnu-4	0.29	1-110	150
1715	Erath-2	1980	F-4	Pnu-4	0.28	1-110	150
1715	Erath-2	1980	F-4	Steel-1 Pnu-4	0.27	1-110	150
1715	Erath-2	1980	F-4	Pnu-4	0.28	1-110	150
1715	Erath-2	1980	F-4	Steel-1 Pnu-4	0.32	1-110	150
1715	Erath-2	1980	F-4	Pnu-4	0.32	1-110	150
1884	Parker-2	1980	PB-4	Steel-1 Pnu-4	0.29	1-110	100
1884	Parker-2	1980	PB-4	Steel-1 Pnu-4	0.27	1-110	100
1884	Parker-2	1980	PB-4	Pnu-4	0.25	1-110	100
1884	Parker-2	1220	PB-4	Pnu-4	0.25	1-110N 1-125S	100

necessary to extend the drying period and consequently the overall time of the test. After each cycle it was necessary to reestablish the correct specific gravity of the sulfate solution by heating, stirring and cooling it. After the last cycle was completed it was difficult to wash the aggregate free of salt. Some 36 to 48 hours of continuous washing was required. These problems extended the overall test time to 8 or 9 days for any given sample. Normally, it was possible to effect 5 freeze-thaw cycles in one day and this would mean about 6 days total for a 25-cycle test, or about 3 days for the 10-cycle test. More work must be done on both tests before firm recommendations can be made.

FIELD PERFORMANCE OF SEAL COATS

Although there are many control problems associated with the study of actual field samples, few studies of highway materials are considered complete without going to the field and observing performances of the material in service. In this study of the comparative merits of lightweight and precoated dense-rock aggregates a rather comprehensive field evaluation program was carried out.

Data collected from the field were obtained through the district engineers of those districts in which lightweight aggregates were used as coverstone for seal coats constructed in 1963 and 1964. One exception to this occurred in the Abilene District where an experimental section using lightweight was constructed in 1962. All other seal coat jobs from which field samples were taken were constructed by contractors who followed the normal procedure of bidding from a set of plans and specifications.

Where it was practical, arrangements were made with the contractor through highway department supervising personnel for incorporating selected design and construction variables in limited sections of several different jobs. However, for most of the sections sampled and tested, no changes were made in the plans or construction procedures. The roads were simply sampled at selected spots and field observations and records were made.

Because several different districts were involved and because of the wide variations in original road condition and level of service, variables in design, construction and service were naturally incorporated into the study.

Field Variables

An idealized simplification of all the problems associated with seal coat design and specifications would be the availability of a single adhesive and a single companion

TABLE 11

UNDESIGNATED PRECOAT AND LIGHTWEIGHT AGGREGATE FIELD TEST SECTIONS

Highway	County, District	Date of Construction	Coverstone Type, Grade	Rolling (hr/mi)	Asphalt (gal/sq yd)	Coverstone (cu yd/sq yd)	Traffic (vpd)
SH 352	Dallas-18	6-4-63	F-4	Pnu 3.9	0.28	1-127	1390
FM 55	Ellis-18	5-29-63	F-4	Pnu 3.1	0.27	1-125	370
FM 987	Kaufman-18	6-3-63	F-4	Pnu 4.6	0.27	1-125	870
FM 1390	Kaufman-18	5-31-63	F-4	Pnu 3.7	0.27	1-125	160
FM 1603	Navarro-18	5-29-63	F-4	Pnu 4.7	0.28	1-125	310
FM 1838	Navarro-18	5-27-63	F-4	Pnu 4.1 Steel 0.4	0.28	1-130	280
FM 740	Rockwall-18	6-4-63	F-4	Pnu 3.4	0.28	1-127	370
FM 548	Rockwall-18	6-7-63	F-4	Pnu 4.0	0.27	1-127	420
SH 78	Collin-18	7-23-63	PB-4	Pnu 3.8 Steel 1.6	0.23	1-124	790
FM 540	Collin-18	7-24-63	PB-4	Pnu 3.2 Steel 1.4	0.23	1-125	1230
FM 2478	Collin-18	7-31-63	PB-4	Pnu 3.9 Steel 1.4	0.23	1-123	430
FM 156	Denton-18	8-12-63	PB-4	Pnu 4.2 Steel 1.4	0.23	1-123	1210
FM 1830	Denton-18	8-2-63	PB-4	Pnu 3.9 Steel 1.6	0.23	1-122	850
US 83	Taylor-8	5-19-64	F-4M	Pnu 5	0.35	1-104	1580
SH 16	Palo Pinto-2	8-9-63	F-4	Steel 1.5 Pnu 4.5	0.31	1-110	160
FM 218	Mills-23	8-2-63	F-4	Steel 1.7 Pnu 5.0	0.29	1-100	260
FM 2731	Eastland-23	9-3-63	F-4	Steel 2.5 Pnu 2.5	0.35	1-90	100
FM 570	Eastland-23	7-27-63	F-4	Steel 1.7 Pnu 5.0	0.27	1-100	710
SH 6	Eastland-23	7-29-63	F-4	Steel 1.7 Pnu 5.0	0.27	1-100	710
FM 2214	Eastland-23	5-10-61	F-3 F-4	Steel 4.2 Steel 4.2	0.32 0.25	1-100 1-120	500
FM 2689	Eastland-23	4-17-62	F-3 F-5	Steel 4.6 Steel 4.6	0.31 0.37	1-90 1-120	200
IH 20	Taylor-8	—62	F-3	Pnu 5.0	0.30	1-100	7700
FM 572	Mills-23	8-7-63	F-4	Steel 1.7 Pnu 5.0	0.30	1-100	250

coverstone that could be universally and successfully used in fixed amounts on any and all road surfaces. No such materials are economically available today; therefore, in the design and construction of seal coats the engineer is faced with a number of variables, and he should take into account as many of these as is economically practical. The more important variables include the following:

1. Existing condition of the road,
2. The amount of traffic handled,
3. Construction procedures and controls,
4. Whether the road is urban or rural,
5. Horizontal and vertical alignment,
6. Weather conditions during construction and immediately thereafter, and
7. Climate of the area.

Some of these factors will be considered in a limited way as they affect this study. Such variables are encountered in all seal coat work regardless of the type of cover aggregate used; however, the magnitude of their effect may change somewhat for different combinations of materials.

Field Test Sections

For any selected test section, it would be possible by prior agreement with the contractor and the Texas Highway Department to vary, within reasonable limits, the application rates of the asphalt-cement and/or coverstone and the type and amount of rolling. The first 3 sections for study were selected in District 25 in Foard County on FM 267. Construction was completed in late July 1963. Details on this road and many others are given in Tables 10 and 11.

Figure 40. Cutting a field sample with a portable saw.

Figure 41. Cutting a field sample with an ax.

For those roads in Table 10, some material application rate or construction procedure variation was included in each of the different sections. These sections varied in length from 1, 220 to 2, 250 ft, these lengths being set by construction procedures and not by design.

The roads in Table 11 do not incorporate any variables other than those normally produced by construction procedures. They were selected at random for field sampling, observation and analysis.

Field samples were taken from a point beginning 30 in. from the outside edge of the pavement and included a section 2 ft square. As a general rule this meant that the sample came from an area falling in the outside wheelpath of a 2-lane pavement.

Two different methods were used in taking these samples (Figs. 40, 41). Sawing the sample is the preferred method; however, equipment of this type is not always available. In taking road samples of this type with an ax and grubbing hoe (Fig. 41) care must be exercised to prevent damage to the coverstone within the bounds of the area to be analyzed.

After the samples were taken from the roadway surface, they were transported in bags to the laboratory for evaluation. The precoated surfaces were treated in a

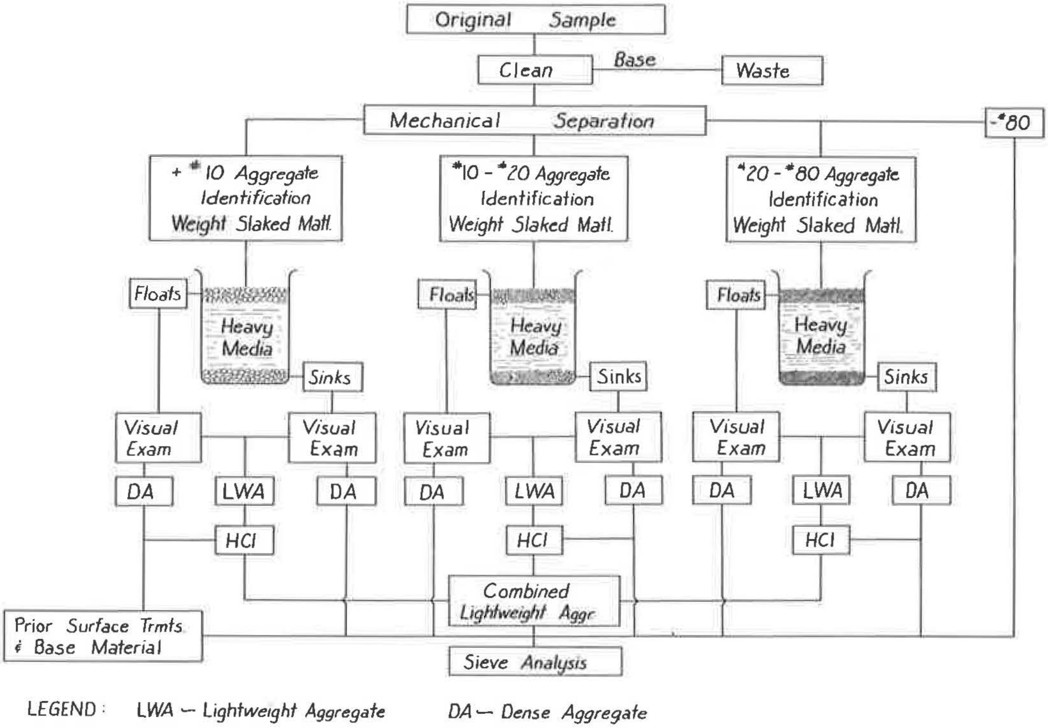


Figure 42. Flow diagram for heavy media separation of lightweight aggregates from field samples.

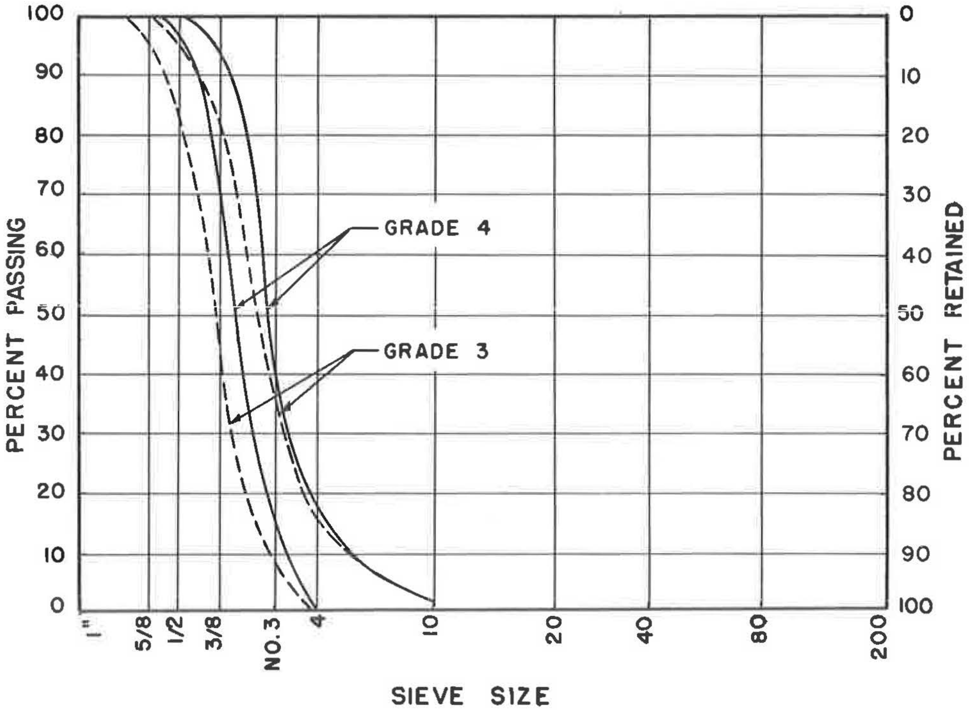


Figure 43. Gradation requirements of THD Item 302 coverstone, Grades 3 & 4.

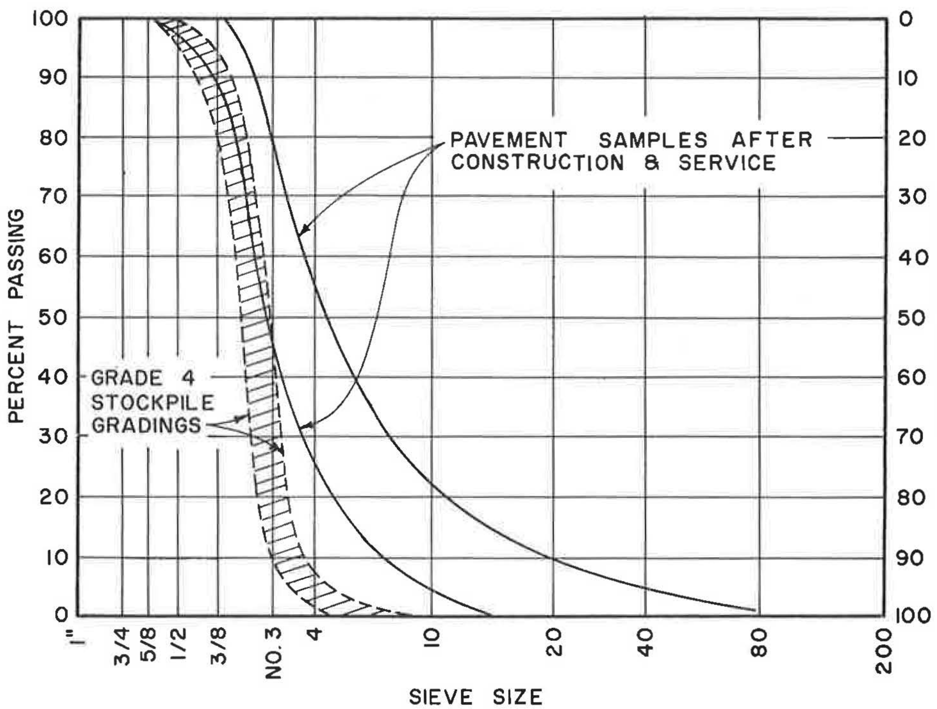


Figure 44. Comparative degradation of lightweight aggregate due to construction and service.

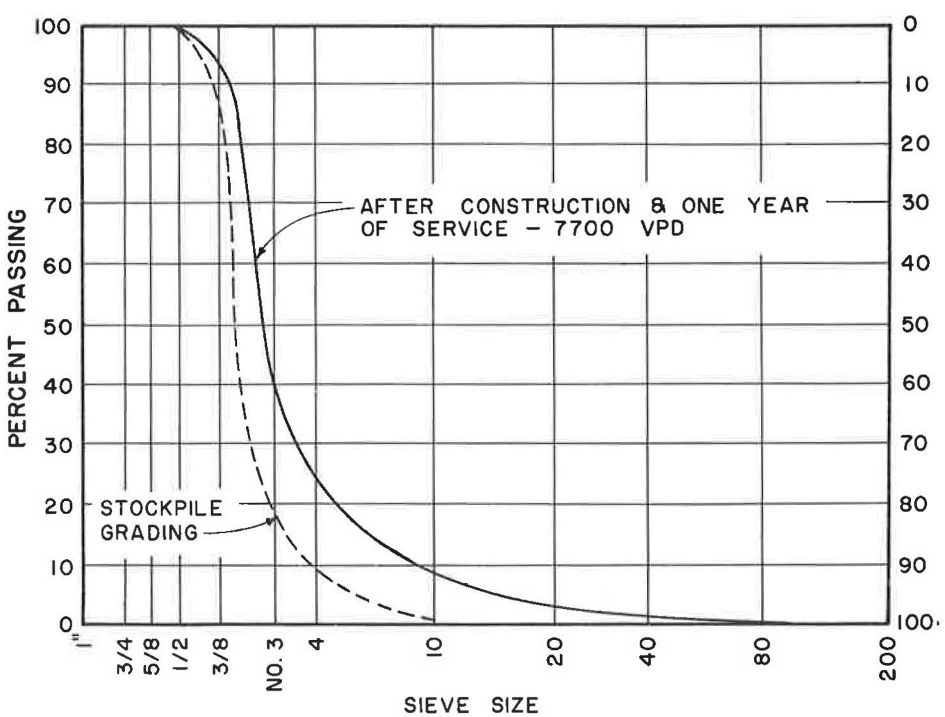


Figure 45. Time and level of service a minor factor in degradation of type F coverstone.

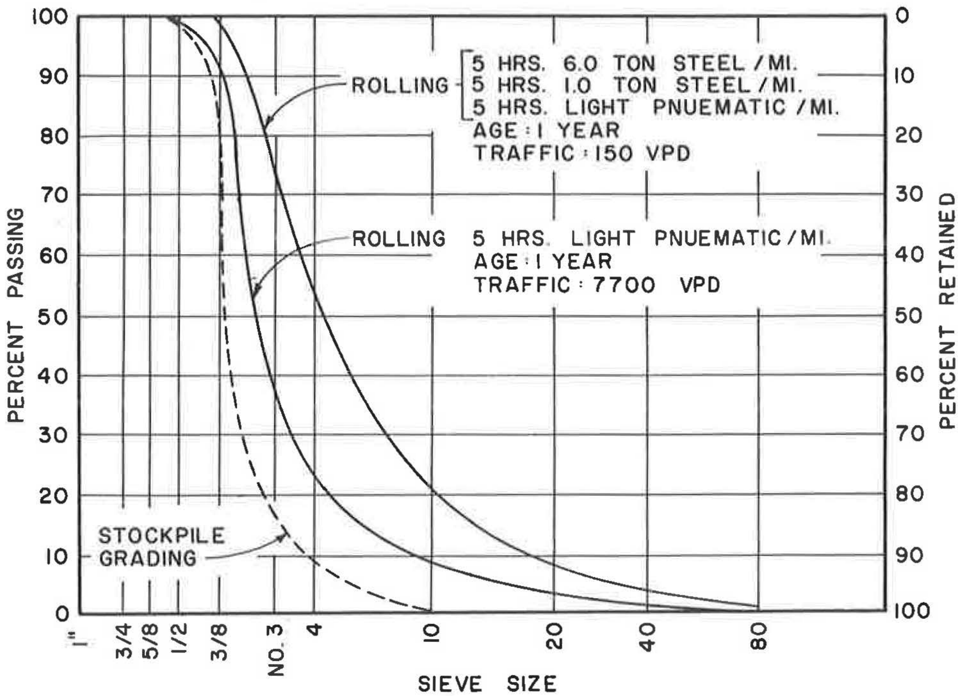


Figure 46. Degradation of type F aggregate due to construction and service.

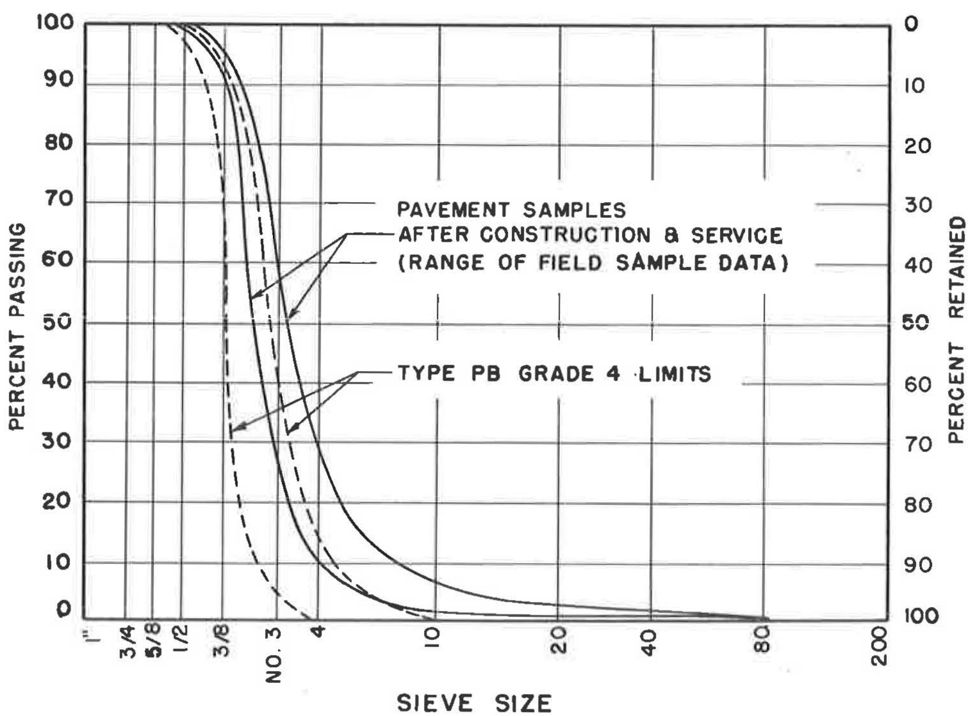


Figure 47. Comparative degradation of precoated limestone due to construction and service.

different manner from the lightweight aggregate samples. The precoated coverstone was removed stone by stone from the sampled area with the aid of heat and tongs. That is, the surface was heated to soften the asphalt and then the stone was plucked from the surface and placed in a pan of solvent for cleaning and further analysis. The lightweight material, on the other hand, was subjected to an entirely different recovery procedure.

Solvent was used to slake the lightweight coverstone from the field samples; however, it was found that in this slaking process some of the previously placed material, an old seal, surface treatment or hot-mix, would be removed with the lightweight aggregate. It was necessary to use heavy media (20) separation as a means of separating the lightweight aggregate, since sieve analysis was used to analyze for changes in grading caused by construction and/or traffic.

A flow diagram of the heavy media separation procedure is shown in Figure 42. The samples, after slaking, were cleaned essentially free of asphalt and then air-dried preparatory to heavy media separation. These samples were then placed in a large beaker containing a mixture of carbon tetrachloride and acetylene tetrabromide. By trial-and-error adjustment of the specific gravity of this mixture, satisfactory separation of the lightweight material could be effected. The materials were separated into different sizes (Fig. 42). This was necessary because the specific gravity of the lightweight material increased somewhat with decreasing particle size. An acid wash of the fine material was used to remove some of the very fine particles of limestone material. All fractions were visually examined after separation to assure that the recovered material was all lightweight aggregate. In some cases it was necessary to visually inspect and hand separate foreign material from the lightweight stone. The entire procedure was much more tedious and time consuming than was anticipated. Nevertheless, it was possible to make a satisfactory separation of the lightweight material from the contaminated composite.

Following the cleaning and separating procedures the individual fractions were recombined and analyzed for grading to determine the extent of degradation.

Specification requirements for grading of the various sizes of coverstone under THD Item 302 are given in the Appendix. Specification grading curves for Grades 3 and 4 are shown in Figure 43. The range in the grading of typical field stockpile samples of Grade 4 stone is shown in Figure 44. It is evident that the grading does not vary appreciably from sample to sample and that most of the material passes the ⅜-in. sieve and is retained on the No. 4 sieve. Similar analyses on the Grade 3 aggregates showed that it was predominantly ½-in. to No. 4 material.

The extent of degradation caused by construction is shown in Figure 44. Field samples of these same materials were taken from the road surface and recovered according to the flow diagram (Fig. 42). The grading of these pavement samples fell within the bounds indicated in Figure 44.

Time in service was not a significant factor in changing the grading of the cover material. It was also observed that there was no major difference in the after-construction grading of Grade 3 and Grade 4 type F material. The majority of the field samples giving dependable data had not been in service more than 4 to 6 months when the samples were taken. Two of the test areas under study included double surface treatments, one of which was 2 years old, but the nature of the base and type of construction of these jobs differ to such an extent that data from these samples are of questionable value. However, the road sample from I-20 at the western city limits of Abilene tells a clear story. The before and after gradings of the lightweight material that went into this surface are shown in Figure 45. Comparing the after curve with the range of after gradings shown in Figure 44, it is evident that heavy traffic (7700 vpd) had a very minor effect on the material. Furthermore, only pneumatic rollers were used for rolling the Abilene sample during construction.

Figure 46 clearly indicates that the type and amount of construction rolling has a decided effect on the degradation of the coverstone. Admittedly this is no new finding but proper rolling of lightweight (type F) aggregates is quite important, and it is evident from these data that if additional fines were desired, these fines can be produced on the road surface during construction. It should be pointed out that it seems foolish

Figure 48. Type F seal two years old with 7700 vpd.

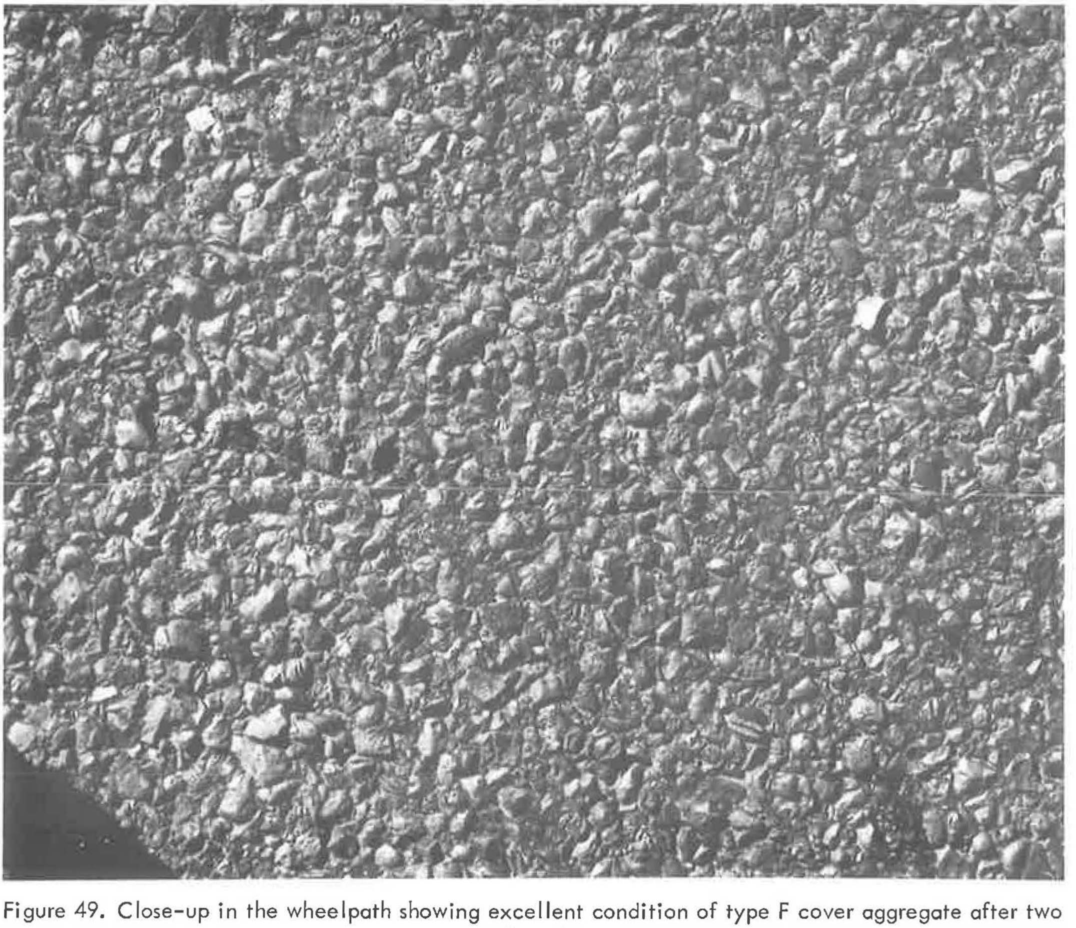


Figure 49. Close-up in the wheelpath showing excellent condition of type F cover aggregate after two years' service.

Figure 50. Stockpile and loading operation of type F material.

Figure 51. Distributor operator placing wind guard on spray bar.

Figure 52. Patches create variations in asphalt demand of surface.

Figure 53. Experimental section proves that the use of steel flat wheel roller is not advisable.

Figure 54. Type F material after one year of light traffic.

Figure 55. Type F material presents a contrast for center stripe.

Figure 56. No center stripe concentrates traffic in center third of FM 1192.

Figure 57. Close-up of center third of above road. Surface not flushed.

Figure 58. Typical farm-to-market road surfaced with type F material.

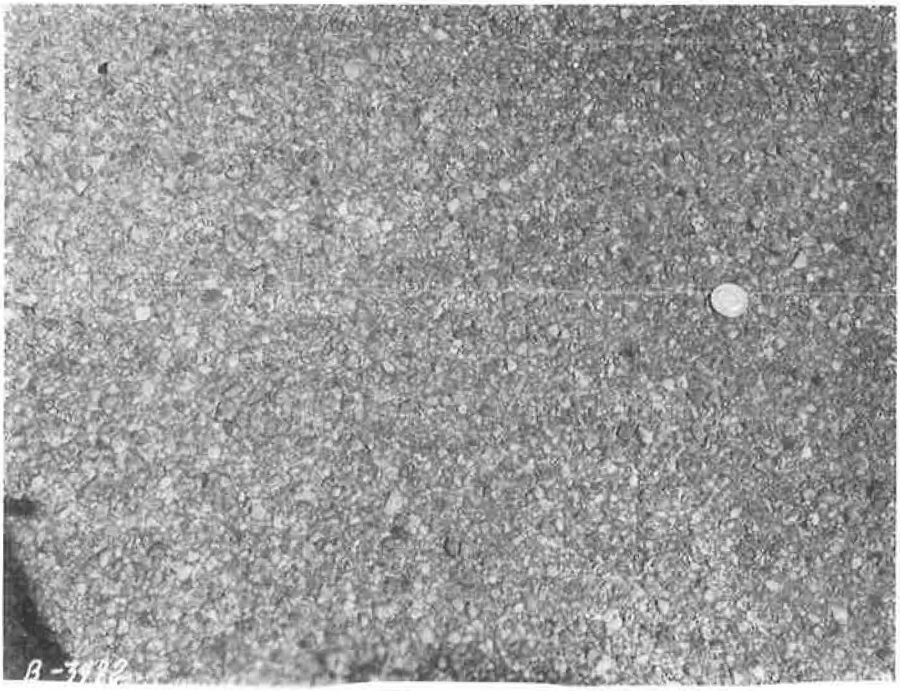


Figure 59. Type F Grade 4 coverstone 3 months after construction FM 744.

Figure 60. Type F cover aggregate in service three months, 1500 vpd, US 190.

Figure 61. Blade broom successfully used on type F coverstone.

Figure 62. Adhesion of asphalt to type F material is very good.

Figure 63. Type F Grade 3 after brooming. Spread rate—130 sq yd/cu yd.

to specify a uniform graded material provided at extra cost and then unnecessarily degrade this same material at additional cost to a net disadvantage in both service and cost.

The data show that the type F cover aggregate is highly suitable for seal coat and surface treatment work when the job is properly designed and constructed. Based on the service records to date, traffic density appears to have a minor effect on this material as measured by degradation of aggregate recovered from the road surface.

Comparative data on the construction degradation of precoated limestone are shown in Figure 47. There was some crushing of the cover material during construction, but it is not quite as severe as that for the type F material subjected to similar rolling equipment. None of the precoated material was subjected to the severe steel flat wheel

Figure 64. Hot-mix with burned clay aggregate placed on SH 6.

rolling used on some of the test sections involving type F material so precise comparisons are not made. The range of values for the two materials (Figs. 43 and 44) overlap but this, of course, incorporates a number of variables that have individual effects on the grading of a given material.

DATA ON CONSTRUCTION AND FINISHED PAVEMENTS

For a better picture of the construction operations and service performance of the materials, a pictorial review of selected projects is presented.

One of the first experimental lightweight aggregate seal coat jobs in the state is shown in Figures 48 and 49. This surface is 2 years old and carried 7,700 vpd. It is evident that heavy traffic has caused no noticeable wear on the surface aggregate.

A series of pictures was made on FM 267 in Foard County covering a 1-yr time interval. Figure 50 shows the roadside stockpile of type F material and the contractor's loading operation. Figure 51 shows the use of a windguard on the spray bar of the distributor and the use of paper at the construction joint which minimizes overlap. A self-propelled aggregate spreader is shown ready to apply the coverstone immediately behind the asphalt distributor.

Figure 52, where the asphalt cement has been applied and half of the road has been covered with type F material, shows the newly patched area at the left edge of the pavement and the striations in the asphalted surface. Striations are caused by poor distribution of asphalt, and probably in this case it occurred in a previous application creating a difference in the asphalt absorption demand across the surface. Reasonable proof of this is demonstrated by the dull appearance of the patched area. Here, due to lack of densification, the asphalt demand was high and unsatisfied. As previously mentioned this is a variable difficult to take into practical consideration. Figure 53 shows a steel roller being used to "seat the stone." Laboratory and field data strongly indicate that the steel flat wheel roller should not be used on type F aggregate. Crushing of the aggregate was excessive in this experimental strip. One year after FM 267 was constructed the photographs appearing as Figures 54 and 55 were taken. This was not in a designated experimental section but represents regular construction control. The excellent appearance of the surface is evident.

FM 1192 in Johnson County was constructed with type F cover aggregate and presents somewhat similar conditions (Figs. 56 and 57). This pavement, however, will not give any trouble, mainly due to a low traffic volume of about 100 vpd. In Figure 56, the center third of the road is darkened by asphalt near the surface. The road has no center stripe and the traffic tends to ride near the center of the road. Horizontal curves accentuate this tendency; therefore, if a surface bleeds, this bleeding will often start at or be more severe in the curves. Not to be neglected as an added factor is the kneading action of the vehicular compaction on curves and the possible difference in distributor performance.

Figure 58 shows FM 1603, a type F coverstone job in Navarro County, 6 weeks after construction. Figure 59, FM 744 in Navarro County, shows the excellent uniform surface made with type F Grade 4 material. Figure 60 shows US 190 in Polk County. The type F material had been in service for 3 months, carrying 1500 vpd.

Figure 61 shows a blade broom during construction of FM 1192 in Johnson County where type F Grade 3 material was used. This sample surface is shown in Figure 62. Also on FM 1192, one experimental section used Grade 3 stone at the rate of 130 sq yd/cu yd (Fig. 63). The coverage is adequate, and inspection of this section revealed no loose stones. Some asphalt can be seen through the voids in the stone but this is only evidence of the proper distribution rate for the stone.

Figure 64 shows an experimental section (SH 6 in Ft. Bend County constructed in August 1963) of hot-mix asphaltic-concrete made with burned clay and field sand as the aggregates. The hot-mix was placed on a flexible base made with burned clay and a sandy clay binder. Limited laboratory tests on the hot-mix from this section indicated that the surface course mix has limited fatigue life. The compacted mix was high in voids and had low flexural strength. A short life is predicted for the surface.

SUMMARY AND RECOMMENDATIONS

The use of one producer's lightweight aggregate (expanded shale) as a coverstone for seal coats and surface treatments was introduced experimentally on Texas highways in 1961 and 1962. During 1963 and 1964 more than 10 million square yards of this material were placed as an alternate to precoated limestone in 5 northwest districts of the Texas Highway Department. Considerable additional material from the same source was placed in 1965 on an optional basis. Lightweight aggregate has been used primarily in the secondary road system; however, limited but successful use of the material as a surfacing in the primary system is a proven fact.

Laboratory tests and field evaluations were effected to determine whether or not lightweight aggregate should be accepted as equal to precoated standard weight material for seal coat coverstone. For the materials under study the data suggest the following conclusions and recommendations.

1. The loose unit weight of the lightweight materials under study was in the range 38 to 50 pcf. For seal coats and surface treatments a minimum as well as a maximum unit weight is recommended.

2. Laboratory design and evaluation of seal coats, preparatory to construction, result in improved overall economy.

3. Laboratory studies and field observations showed that the lightweight material had a strong affinity for all the asphalt-cements used in the project. This was a qualitative observation.

4. Crushing of coverstone is minimized when the pneumatic roller alone is used to seat the cover material, and it is therefore recommended that only pneumatic rolling of lightweight aggregate be practiced.

5. The steel flat wheel roller caused degradation of both types of coverstone, particularly in areas of irregular cross-section.

6. Laboratory induced windshield damage was severe for the crushed limestone and practically insignificant for the lightweight materials.

7. The Texas and Louisiana modifications of the Los Angeles abrasion test were found to be less severe than the ASTM standard test when used to measure the abrasion resistance of the lightweight materials under study.

8. One hundred cycles of rapid freeze-thaw caused a significant loss for some Grade 3 and Grade 4 lightweight materials.

9. Lightweight aggregate A showed a maximum weighted average loss of 1.56 percent when subjected to 5 cycles of the magnesium sulfate soundness test. This compared to 3.07 percent loss for the same material after 100 cycles of rapid freeze-thaw.

10. Under a variety of construction and service conditions, lightweight aggregate A has, after 1 to 4 years of service, proved to be a highly successful cover aggregate for seals and surface treatments.

11. Volume of vehicular service appears to have no measurable effect on the degradation of lightweight aggregate A.

12. The lightweight aggregate was favorably accepted by contractors and Texas Highway Department personnel throughout the area in which it was used.

13. Lightweight aggregate A is considered equal to precoated limestone for seal coat and surface treatment work. Laboratory results indicate that several of the other lightweight aggregates under study are also acceptable for this and similar service.

Based on the laboratory and field evaluation work performed during the past 26 months, and considering only those materials involved in these studies, the following recommendations are submitted:

1. Consideration should be given to setting a minimum as well as a maximum unit weight for lightweight aggregate used in seals and surface treatments. This minimum could be a set figure or it could be provisionally based on service records and/or laboratory data from an abrasion test and rapid freeze-thaw results.

2. The definite advantages of clean uniform graded materials were emphasized in the study. Improved construction control and extended service would result from further restrictions of range of particle size presently permitted. Grades 1 through 5 permit 2 percent of the material to pass the No. 10 sieve. Of this minus No. 10 material not more than one half of one percent (based on the total aggregate) should pass the No. 80 sieve. There appears to be no practical need for more than 4 grades (size-wise) of lightweight aggregate.

3. Only pneumatic rolling of lightweight aggregate coverstone is recommended.

4. It is suggested that consideration be given to adopting the Louisiana modification of the L. A. abrasion test with washing of the plus No. 5 material after test being provisional. (Analysis for wear should be made by use of the No. 5 sieve rather than the No. 4.)

5. Considering availability of equipment, rapid freeze-thaw test might be substituted for or made optional to a sulfate soundness test. Fifty cycles and 8 percent maximum loss are tentatively suggested.

6. New lightweight materials or lightweight materials produced from unproven sources of raw materials should be subjected to and pass acceptable field service trials before final acceptance and general use.

7. The use of synthetic aggregates in paving systems of all types should be encouraged where these materials meet service requirements. No maximum unit weight restriction should be imposed on materials of this general type unless some definite purpose is served by the restriction, for example, the minimizing of windshield damage in seal coat and surface treatment work.

8. To establish realistic quality boundaries on the many lightweight aggregates that might be used for seal coats and surface treatments, it would be advisable to evaluate these materials in the laboratory before controlled field serviceability tests are made.

9. Finally, general specifications should be prepared which would place the various synthetic aggregates in use categories. Three or 4 categories would be required.

REFERENCES

1. ASTM. 1961 Book of ASTM Standards Including Tentatives, Part 4. American Society for Testing and Materials, Philadelphia, 1961.

2. Haun, R. P. Tar and Feather Highways. Texas Highways, Vol. 10, No. 3, p. 7, April 1963.
3. Mueller, J. I., Riley, R. P., and Shapero, H. L. Lightweight Aggregate From Pacific Northwest Clays and Shales. The Trend in Engineering, Univ. of Washington, Vol. 5, No. 2, pp. 5-9, April 1953.
4. Marslender, W. H. Featherlite Corp., Ranger, Texas (private communication).
5. Riley, C. M. Relation of Chemical Properties of the Bloating of Clays. Jour. Amer. Ceram. Soc., Vol. 34, No. 4, pp. 121-128, April 1951.
6. Bauer, W. G. Mechanics, Techniques and Economics of Expanded Clay-Shale Aggregate Production. Pit and Quarry, Vol, 41, No. 1, pp. 71-73, July 1948; No. 3, pp. 93-96, Sept. 1948; No. 6, pp. 91-95, Dec. 1948; No. 12, pp. 87-89, June 1949.
7. Standard Specifications for Road and Bridge Construction. State Highway Dept. of Texas, Items 302 and 304, pp. 207-216, Jan. 2, 1962.
8. Woolf, D. O. The Relation Between Los Angeles Abrasion Test Results and the Service Records of Coarse Aggregate. HRB Proc., Vol. 17, pp. 350-359, 1937.
9. Kearby, J. P. Thoughts and Theories on Penetration Surfaces. Texas Highway Dept. Construction and Maintenance Bull. No. 11, pp. 43-65, 1952.
10. Benson, F. J., and Gallaway, B. M. Retention of Coverstone by Asphalt Surface Treatment. Texas Eng. Expt. Sta. Bull. 133, Sept. 1953.
11. Hank, R. J., and Brown, M. Aggregate Retention Studies of Seal Coats. Proc. AAPT, Vol. 18, pp. 261-272, 1949.
12. Richart, F. E., and Jenses, V. P. Tests of Plain and Reinforced Concrete Made With Haydite Aggregates. Univ. of Illinois Eng. Expt. Sta. Bull. No. 237, Oct. 1931.
13. Downey, G. L. The Mechanics of Stone Damage to Automobile Windshields. Highway Research Abstracts, Vol. 27, No. 4, pp. 29-32, April 1957.
14. Buckley, F. T., and Nelson, J. S. Monsanto Co., Research Dept., Plastics Div., Springfield, Mass. (private communication, Sept. 1963).
15. Bowen, G., and Hamblet, C. H. Du Pont Research on the Safety Performance of Tempered Glass. (private communication). Oct. 1957.
16. Rushing, H. B. Lightweight Aggregate Abrasion Study. Research Project 61-7C, HPS 1 (18) U. S. Bureau of Public Roads and Louisiana Dept. of Highways, Feb. 1963.
17. Standard Specifications for Highway Materials and Methods of Sampling and Testing, Part II. American Association of State Highway Officials, Eighth Edition, 1961.
18. Manual of Testing Procedures. Texas Highway Dept., Vols. 1 and 2, Revised Feb. 1963.
19. Woolf, D. O. Toughness, Hardness, Abrasion, Strength and Elastic Properties. ASTM Spec. Tech. Publ. No. 169, pp. 314-324, 1956.
20. Higgenson, E. C., and Wallace, G. B. Control Testing for Separation of Lightweight Material from Aggregate. ASTM Bull. (TP15), No. 243, pp. 60-68, Jan. 1960.

Appendix

SPECIAL SPECIFICATION

Item 1269

AGGREGATE FOR SURFACE TREATMENTS

(Lightweight)

1. DESCRIPTION. This item establishes the requirements for lightweight aggregates to be used in the construction of surface treatments.

2. MATERIALS. Aggregates shall be composed predominately of lightweight cellular and granular inorganic material prepared by expanding, calcining, or sintering products such as clay or shale.

The aggregate shall contain not more than 1 percent of organic matter, impurities or objectionable matter when tested in accordance with Test Method Tex-217-F.

The dry loose unit weight of course lightweight aggregates shall not be less than 40 and shall not exceed 60 pounds per cubic foot. If the unit weight of any shipment of lightweight aggregate differs by more than 6 percent from that of the sample submitted for acceptance tests, the aggregates in the shipment may be rejected. Tests shall be in accordance with Test Method Tex-404-A, except that the aggregate shall be tested in an oven-dry condition. The percent of wear, as determined by Test Method Tex-410-A (Part II), shall not exceed 35 percent.

The aggregate, when tested in accordance with Test Method Tex-411-A, shall show a loss of not more than 12 percent after five cycles of the sodium sulfate soundness test or 18 percent after five cycles of the magnesium sulfate soundness test.

3. GRADES. When tested by Test Method Tex-200-F, the gradation requirements for the several grades of aggregate shall be as follows:

		Percent by Weight
Grade 1:	Retained on 1" sieve	0
	Retained on $^7/_8$" sieve	0-2
	Retained on $^5/_8$" sieve	15-45
	Retained on $^3/_8$" sieve	85-100
	Retained on No. 4 sieve	95-100
	Retained on No. 10 sieve	98-100
Grade 2:	Retained on $^7/_8$" sieve	0
	Retained on $^3/_4$" sieve	0-2
	Retained on $^1/_2$" sieve	20-35
	Retained on No. 4 sieve	85-100
	Retained on No. 10 sieve	98-100
Grade 3:	Retained on $^3/_4$" sieve	0
	Retained on $^5/_8$" sieve	0-2
	Retained on $^1/_2$" sieve	5-20
	Retained on No. 4 sieve	85-100
	Retained on No. 10 sieve	98-100

1269.000
11-64

Grade 4:	Retained on 5/8" sieve	0
	Retained on 1/2" sieve	0-2
	Retained on 3/8" sieve	5-25
	Retained on No. 4 sieve	85-100
	Retained on No. 10 sieve	98-100
Grade 5:	Retained on 1/2" sieve	0
	Retained on 3/8" sieve	0-2
	Retained on No. 4 sieve	40-85
	Retained on No. 10 sieve	98-100
Grade 6:	Retained on 1/2" sieve	0
	Retained on 3/8" sieve	0-2
	Retained on No. 4 sieve	5-40
	Retained on No. 10 sieve	70-100
	Retained on No. 20 sieve	99-100
Grade 7:	Retained on 1/4" sieve	0
	Retained on No. 4 sieve	0-10
	Retained on No. 20 sieve	25-55
Grade 8:	Retained on No. 4 sieve	0
	Retained on No. 10 sieve	0-10
	Retained on No. 20 sieve	10-55

4. MEASUREMENT AND PAYMENT. Aggregates will be measured and paid for in accordance with the governing specifications for the items of construction in which these materials are used.

ABRASION OF CONVENTIONAL AND LIGHTWEIGHT COARSE AGGREGATE BY THE USE OF THE LOS ANGELES MACHINE

(Test Method Tex-410-A
Rev: November 1963)

Scope

This Test Method covers the procedure for testing conventional and lightweight coarse aggregate for resistance to abrasion in the Los Angeles testing machine with an abrasive charge. The apparatus and procedure used in this test are identical with ASTM Designation: C 131 with the exceptions noted under Part II of this method.

PART I

ABRASION OF CONVENTIONAL COARSE AGGREGATE

Procedure

Use the apparatus specified to prepare and test the required gradings of aggregate in accordance with the procedure described in ASTM Designation: C 131.

PART II

ABRASION OF LIGHTWEIGHT COARSE AGGREGATE

Procedure

To avoid the excessive volume of material in the testing machine which will occur when the lightweight aggregate sample is prepared according to ASTM Designation C 131, it is necessary to reduce the weight proportionately to obtain an equal volume of lightweight aggregate comparable to that normally obtained with a conventional aggregate sample.

The abrasive charge must also be reduced in a similar manner.

1. Determine the unit weight (U_L) of the lightweight aggregate by Test Method Tex-404-A.
2. Assume an average unit weight of conventional aggregate to be 97.0 lbs. per cu. ft.
3. Reduce the lightweight aggregate sample.

$$\frac{U_L}{97.0} = \frac{X}{C}$$

$$X = \frac{(C)\,(U_L)}{97.0}$$

Where:

U_L = Unit weight of lightweight aggregate sample (lbs. per cu. ft.)
C = Weight of conventional aggregate required for grading in ASTM 131
X = Reduced lightweight aggregate sample charge.

4. Reduce the abrasive charge:

$$\frac{U_L}{97.0} = \frac{X_L}{C_L}$$

$$X_1 = \frac{(C_L)\,(U_L)}{97.0}$$

Where:

U_L = Unit weight of lightweight aggregate (lbs. per cu. ft.)
C_L = Weight of abrasive charge required for grading in ASTM 131
X_1 = Reduced abrasive charge for lightweight aggregate

5. Remainder of procedure as set forth in ASTM 131.

NOTE:

It is sometimes impossible to obtain the exact abrasive charge with the steel balls available. In this case, obtain the closest abrasive charge possible to the reduced value and then adjust the weight of the sample in proportion to the new abrasive charge.

Reporting Test Results

Report the test data and type grading and the wear to the nearest 0.1 percent on Form No. 272.

COMMENTS ON THE HANDLING, CONSTRUCTION AND SERVICE OF LIGHTWEIGHT AGGREGATE COMPARED TO PRECOAT

The following comments represent a cross-section of those received in interviews with THD personnel and contractors who used these materials in Districts 2, 8, 23 and 25.

I. State and District Personnel

A. Within its area of competitive haul, the Type F expanded shale aggregate is an important alternate to other materials because of reduction in windshield breakage alone. The material is dark in color which reduces glare and it appears to have a natural affinity for asphalt. The material is not degraded appreciably under normal surface rolling.

B. The hard freezes during the winter of 1963 did not damage the lightweight. It performs as well as precoat and has less flying particles immediately after construction. Lightweight dusts a little but the grading is good and it is a valuable material for seal coat and surface treatment work.
C. After two years of service we are still pleased with the performance of Type F aggregate. The color contrast produced by lightweight is maintained throughout the life of the surface whereas precoat fades out in a few months.
D. Of all the stone available for seal coat and surface treatment I prefer the overall characteristics of precoated rock asphalt with lightweight running a close second. The contractor's men prefer the handling ease afforded by lightweight aggregate and it bonds well to the asphalt.
E. We had one job, a double surface treatment, (Lightweight) that bled severely but this was in the early trial stages and was caused by a fault in design. We have had some trouble with variation in amount of oil used on our precoated material. However, both materials do a good job when properly designed and constructed.
F. High speed traffic on new surfaces of lightweight do not create a flying stone hazard. Loose stone is thrown but is carried only a short way from the vehicle wheel. It is not necessary to sweep loose stone back on a new surface made with lightweight. Initial adhesion is good with both precoat and lightweight.
G. Where lightweight is used the reduced gross loads of equipment during construction minimize damage to shoulders on low traffic roads.
H. Retention of lightweight aggregate is as good as that of precoated aggregate when placed under identical conditions. Lightweight aggregate is naturally dust free and has an inherent affinity for asphalt. This material has produced excellent results on high-traffic roads when placed under favorable weather conditions.

II. Resident Engineer and Contractor Personnel

A. Some dusting was experienced on one surface one to four days after construction. (This lightweight aggregate seal was rolled with steel and pneumatic rollers.) At speeds up to 60 mph some stone was thrown by traffic. Stones were airborne for a distance of 20 to 40 feet. No windshield damage was observed or reported on this lightweight aggregate section.
B. Lightweight aggregate adheres well to the asphalt. The grading is uniform and the material is clean when delivered. Due to its lightweight and good bond, it can be broomed effectively with a blade broom.
C. In-place crushing (of lightweight) helps key in the coverstone. A nonglare surface is produced.
D. The material (lightweight) is easy to handle and easy on equipment. Job progress is more rapid and laborers handling the hand touch-up work find their job easier.
E. Without special modification of hauling equipment, overloading is eliminated and this extends equipment life.

Summarizing these observations on Type F and Type PB aggregate we find:

A. Retention is comparable for like designs and service conditions.
B. Bleeding, where observed, was about the same and could not, for either material, be definitely attributed to any characteristic of the materials involved.
C. Serious raveling was encountered on one precoat job and this was attributed to improper design. Minor raveling was observed on several other sections but there was no great difference in degree of raveling for the two materials. As a general rule where minor raveling occurred this took place between the wheel paths, possibly, indicating the need for a slight increase in asphalt application rate.
D. Degradation during construction rolling was comparable except where the Type F material was rolled excessively with steel flat wheel rollers.
E. General appearance of the two types of material is good. Type PB material used for contrast purposes often fades or loses color within a few months.

F. Contractors prefer the lightweight material due to ease of handling and increased production rate of finished road surface. Wear and tear on equipment is reduced materially.
G. No broken windshields attributable to either material were reported from any of the sections under observation.
H. Some Engineers and Maintenance Personnel indicated a preference for the Type F material. No one contacted objected to its use and all were satisfied with its performance.