SURFACE COLOR AS A FACTOR IN ASPHALTIC CONCRETE AIRFIELD PAVEMENT DESIGN

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SURFACE COLOR AS A FACTOR IN ASPHALTIC CONCRETE AIRFIELD PAVEMENT DESIGN

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PREFACE

In the design of airfields there are many new and perplexing problems which have confronted the engineer in the period of developement of the modern jet transport aircraft. A few of the more important problems are: the requirement for greatly increased runway length; the requirement for much greater bearing capacity for pavements supporting aircraft operations; the requirement for increased blast and heat resistance in pavement surfaces; and the requirement for improved runway lighting.

The purpose of this study is, first, to investigate the influence of surface color in asphaltic concrete upon the runway length requirement for take-off of jet transports, and second, to determine the effect of surface color upon the stability of asphaltic concrete pavements under load. It is not the intention of the author to compare asphaltic concrete with portland cement concrete, but to obtain and present data which might enable the pavement designer to improve the properties of asphaltic concrete airfield pavement.

The author had the good fortune to be the Engineering Officer in the Base Civil Engineering Office, Mac Dill Air Force Base, Florida, when the first B-47 type aircraft were delivered to combat wings of the United States Air Force. The B-47 was the first military aircraft to pose problems of a magnitude similar to those which are inherent in the support of civil aircraft such as the Boeing 707 and the Douglas DC-8. The writing of this study has, of course, been strongly influenced by the author's experiences at Mac Dill Air Force Base and by subsequent experiences in the planning and maintenance of other air force bases.

Appreciation is expressed to: J. Rogers Martin, for his guidance and encouragement; Phillip G. Manke, for his advice and aid in accomplishment of laboratory work; Ivar L. Shogran, of Douglas Aircraft Company, for supplying data on the Douglas DC-8; and, R. L. Roark, of Boeing Airplane Company, for supplying data on the Boeing 707.

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CHAPTER I

BACKGROUND FOR THE STUDY

The Changing Airfield Concept

The jet aircraft of this age is fast becoming as commonplace as the trolley car of fifty years ago. This fact has thrust the necessity upon the engineer for rapid change in the concept of what an airfield must be. The first modern aircraft of importance, the DC-3, made an appearance in 1936. The DC-3 required less than 5,000 feet of runway for safe operation, weighed a maximum of 25,200 pounds, and imposed a load of 11,575 pounds upon the airfield pavement on a tire contact area of 237 square inches.[1] In contrast, the DC-8 requires a runway of 8,000 to 12,000 feet, weighs a maximum of 310,000 pounds, and imposes a load of 35,100 pounds upon the pavement on a tire contact area of only 200 square inches.[2]

The DC-3 could be operated from little more than a sod field. For the DC-8, based upon the design criteria of the Department of The Army, Corps of Engineers, the total required thickness of asphaltic pavement, base, and sub-base, varies from 22 inches to 56 inches depending on the quality of the available sub-grade soil.[3]

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[1] Air Transport Association of America, Airline Airport Design Recommendations, Part III (1950), p. 2.

[2] Douglas Aircraft Company, Airport Pavement Requirements For DC-8, Report No. SM-23255 (1958), p. 4.

[3] Department of The Army, Corps of Engineers, Engineering Manual For Military Construction, Part XII (1950), p. 14.

Extremely high wing loading factors, i.e., aircraft weight per unit of area of wing surface, make the jet aircraft very sensitive to control. Take-off speeds are such that there is little margin of safety for miscalculation of the required length of take-off run. The Federal Aviation Agency requires provision of a runway of full strength pavement, i.e., sufficient capacity to withstand repetitive loadings of maximum weight aircraft, for the full distance of the take-off run, plus light duty pavement for an additional distance equal to 15 per cent of the take-off run. Take-off run is defined as the distance required for the aircraft to attain an altitude of 35 feet above the runway surface.[4]

Calculation of Required Runway Length

Viewing runway length requirement for the present time, from the standpoint of "what must be designed", there are three general groups of factors which have a bearing on the problem:

1. Performance characteristics of the most critical aircraft to be accommodated. The critical aircraft is the one which requires the longest take-off run.
2. Aircraft performance requirements imposed upon aircraft manufacturers and airlines by the government.
3. The environment of the airfield site.

Performance characteristics for an aircraft are established by testing the aircraft under varying conditions of load, atmospheric pressure, and air temperature. Performance requirements established by the government cover such details as take-off under specific conditions of mechanical failure of the aircraft, for instance, an inoperative engine. An aircraft

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[4] Federal Aviation Agency, Aeronautics and Space, Reg. No. SR-422B (July 9, 1959), p. 4.

must be proven able to take off within certain limiting distances, and must be capable of maintaining specified rates of climb before it may be certified for commercial use. The aforementioned points are of interest to the civil engineer, but information on required take-off run, and required slope of the glide-path, which must be clear of obstructions, are the factors which must be known for runway design. The civil engineer is primarily concerned with factors in the environment which affect the ability of the critical aircraft to take off and land safely.

The take-off run for the critical aircraft as determined in a standard environment is modified for a specific airfield site by the following factors:

1. Wind direction and velocity at the time of take-off.
2. The density of the air at the elevation of the aircraft wing as the aircraft proceeds down the runway (barometric pressure and the air temperature at wing level are the primary factors which influence air density).
3. Longitudinal gradient of the runway.

The standard environment for computation of required take-off run for any aircraft is zero wind, sea level pressure, and zero gradient in the runway. Even the uninitiated airport engineer is quick to realize that wind is fickle, that there are wide variances in temperature at any site, and that barometric pressures vary with the altitude of the site. For purposes of economy and safety, runways are aligned in the direction of prevailing winds; however, the critical condition of zero wind must be considered when required runway length is determined. In consideration of variances in barometric pressure at a site, the term pressure-altitude is used. The pressure-altitude, or effective altitude above sea level, used in runway design is the average of the highest daily values for the hottest month of the year. The temperature used is also the average of the highest daily values for the hottest month of the year.[5] This last factor, the temperature, is one of the subjects of this study.

During hot summer weather, the author has often sensed a disparity between the free air temperature as recorded on or near the operations buildings on airfields, and the temperature on the aircraft parking apron. He also felt that the darker the surface color of the pavement, the hotter he was. He wondered if there could be a significant variation in the air temperature above pavements of varying color. He was concerned with the possibility that under certain conditions surface color of pavements might so affect air temperature at the wing level that the length of required take-off run would be increased.

Temperature and the Stability of Pavement

The author questioned whether or not there might be a significant correlation between surface color of pavements, the internal temperature of the pavement, the viscosity of the asphaltic binder, and finally, most important, the stability of the asphaltic concrete under wheel loads.

It was with the above thoughts that the author entered into his investigations.

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[5] Department of The Air Force, Installation Planning and Development, AFR No. 86-5 (1950), p. 2.

CHAPTER II

BASIC SCIENTIFIC FACTORS

Transmission of Heat

Heat is transferred from one body, or source, to another body by three processes, conduction, convection, and radiation. These processes have in common that temperature differences must exist between bodies, and that heat is always transferred in the direction of the lower temperature.

Conduction is a process whereby heat is transferred to one body from another by direct contact between the molecules of the two bodies. Substances differ widely in the ability to conduct heat from one point to another. Metals are relatively good conductors. Air is the poorest of conductors; any substance which has a high percentage of air-filled voids is a poor conductor.

Convection is a process of heat transfer by the flow of fluids (air is a fluid).[6] The process may be explained step by step by considering an air mass which is stationary above a relatively warm horizontal surface:

1. An infinitesimal layer of air immediately against the warm surface is heated by the conduction of heat from the surface to the air molecules.

2. When the air molecules against the surface are heated they are decreased in density and then they rise from the surface.

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[6] E. Hausmann and E. P. Slack, Physics (New York, 1944), p. 317.

3. As the warm air molecules rise, cool air molecules flow in laterally from the direction of the edges of the surface and take the place of the rising molecules.

4. The process is repeated over and over again and eventually a large mass of air has been warmed.

Radiation is a process of heat transfer which is a little more difficult to explain. A general understanding may be gained by considering the heat transfer from the sun to the earth. The thermal energy of the sun is first converted into an electro-magnetic wave motion called radiation. Radiant energy waves then travel from the sun to the earth at the same velocity as light waves, 186,000 miles per second. The radiant energy waves are absorbed by the earth and are reconverted to thermal energy at the time of absorption.

When radiant energy falls on a substance, part of the energy is absorbed, part is reflected, and depending upon the opaqueness of the substance, a part of the energy may be transmitted through the substance. A substance which would absorb 100 per cent of the impinging radiant energy is called a perfect black body; no such substance exists. Even the blackest of surfaces occurring in nature still have a reflectivity of about one per cent.[7] It follows logically that the temperature of a substance exposed to radiant energy waves is higher or lower depending upon the reflectivity of the surface; and further, that the degree to which the air above the substance is warmed by the process of convection, is dependent upon the percentage of energy which is absorbed by the substance.

A point of most importance to this study is that air does not become heated by the passage of radiant energy waves.[8]

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[7] Max Jakob, Elements of Heat Transfer and Insulation (New York, 1952), p. 169.

[8] Ibid., p. 168.

Viscosity

Viscosity is the property of a liquid which presents resistance to flow. Asphalt cements at normal temperatures may be in either a solid or a plastic state. The consistency of a specific grade of asphalt which is plastic or solid at normal temperatures varies with the viscosity of that asphalt. Viscosity decreases as temperature increases. However, it should be noted that the viscosity of all grades of asphalt cement is not affected equally by a unit increase in temperature.[9]

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[9] J. Rogers Martin and Hugh A. Wallace, *Design and Construction of Asphalt Pavements* (New York, 1958), p. 31.

CHAPTER III

PREFACE TO RESEARCH

In order to obtain data and knowledge concerning the questions which were posed in Chapter I, the author decided upon the use of the following tests:

1. The measurement of temperatures above various airfield pavements under field conditions.
2. The measurement of internal temperatures in specimens of asphaltic concrete having various surface reflectivity characteristics under laboratory conditions.
3. Determination of the effect of temperature on the stability of paving grade asphaltic concrete specimens under laboratory conditions, using the Marshall test with slight modification.
4. Determination of the effect of temperature on the stability of paving grade asphaltic concrete specimens of varying asphalt content using a plate bearing test under laboratory conditions.

CHAPTER IV

AIRFIELD TEMPERATURE TEST

Discussion and Procedure

The author decided that the most accurate method for obtaining data on the temperature variations above airfield pavements of varying color was to use temperature recordings in the field. The calibration of two Fahrenheit thermometers was checked by taking simultaneous readings in an area which was shaded from the sun. It was decided for convenience that simultaneous temperature readings should be taken at an elevation of five feet above the pavement surfaces being compared; a second person was employed by the author to permit the use of this procedure.

After preliminary tests on street surfaces in the city of Stillwater, and later at the Stillwater Municipal Airport, it was found that the difficult factor which had to be overcome in this test was wind disturbances. It was soon apparent that a wind of only five miles per hour was sufficient to eliminate any differences in air temperatures anywhere on the airfield area regardless of the type of surface. It was decided that conditions of dead calm, or near calm, were essential to the success of the test. It was also decided that greater expanses of pavement than those available in Stillwater would aid in providing data of significant character. The fact that the wind is blowing in Oklahoma most of the time was soon evident. Periods of calm were sometimes noted in the early morning, but as the sun rose the velocity of the wind increased.

Authority was obtained from the Base Civil Engineer, Tinker Air Force Base, Oklahoma, to make temperature observations on the airfield pavements at that base. After that it was a matter of waiting for correct weather conditions. The author was situated seventy miles from Tinker Air Force Base, therefore weather conditions had to be very promising before a trip to the air base was warranted. Coupled with the necessity for a dead calm, was the necessity for higher than normal temperature, since both conditions are required to create a critical environment for take-offs.

Attempts to obtain significant data were made without success on July 23, 1959, and July 30, 1959. On August 30, 1959, conditions were ideal, and the data in Table I was obtained.

When the idea of measuring air temperatures above pavements, and attempting the correlation of temperatures with the surface color of the pavements, was conceived a much greater volume of data was visualized. The author felt the need for a means of quickly measuring the reflectivity of pavement surfaces. As a result of this need the apparatus in Fig. 1, a "Reflectometer", was constructed. Essentially this apparatus consisted of two light meters of the type commonly used in photography. One meter was permanently directed at a smooth surfaced, eight inch square of plaster of Paris, and the other meter was directed, on the same angle, at the airfield pavement. Prior to use, the "Reflectometer" was calibrated by placing a second plaster of Paris square beneath the light meter normally directed at the pavement. Table II was constructed for use with the "Reflectometer" to obtain the reflectivity of a pavement surface in comparison with the reflectivity of plaster of Paris. It should be noted here that the reflectivity of plaster of Paris is only 91 per cent.[10] However, plas-

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[10] V. M. Ehlers and E. W. Steel, Municipal and Rural Sanitation (New York, 1958), p. 411.

ter of Paris is the most easily obtained standard substance for comparison of reflective characteristics of various surfaces.

The areas in which the data of Table I were obtained are described as follows: a. Light colored surface, 350 feet by 800 feet; reflectivity, 66.7 per cent of plaster of Paris. b. Dark colored surface, 400 feet by 1000 feet; reflectivity, 33.3 per cent of plaster of Paris. Simultaneous temperature readings were made in the two areas at two to five minute intervals. All of the two areas were exposed to the sun. About 30 minutes after the first recordings, the differential between the two areas began to diminish, and soon afterward a slight breeze was perceptible.

Effect of Findings Upon Runway Design

A variation of six degrees Fahrenheit was noted between the air temperatures in the two areas. The effect of this temperature differential on runway design is illustrated by the graphs in Fig. 2 and Fig. 3. These graphs were prepared using performance data for the DC-8 aircraft which was supplied by Douglas Aircraft Company. The runway length requirement is plotted for various pressure-altitudes and for the maximum temperatures recorded in Table I for the two types of pavement. The graph in Fig. 2 is based on a flap setting of ten degrees, and the graph in Fig. 3 is based on a flap setting of 25 degrees. The plotted runway length requirement is 1.15 times the required take-off run. The use of a greater flap angle reduces the required length of take-off run. However, some factor of flying safety is sacrificed, during the most critical stage of flight, for every additional degree of flaps used, due to reduction in flying stability of the aircraft as flap angles are increased.

TABLE I

AIR TEMPERATURES ABOVE AIRFIELD PAVEMENTS OF VARYING SURFACE REFLECTIVITY

Time of Recording	Temperature Above Light Colored Pavement (Degrees F.)	Temperature Above Dark Colored Pavement (Degrees F.)
1545	101	107
1550	100	106
1555	101	106
1600	100	106
1605	101	107½
1607	101	107
1609	101	107½
1612	100	106
1614	100	106
1616	99	104
1620	98	* 102

* Slightly perceptible wind noted.

Place of Recordings: Tinker Air Force Base, Oklahoma

Date of Recordings: August 30, 1959

Wind Condition: Dead calm

Official Airfield Temperature: 96 degrees F.

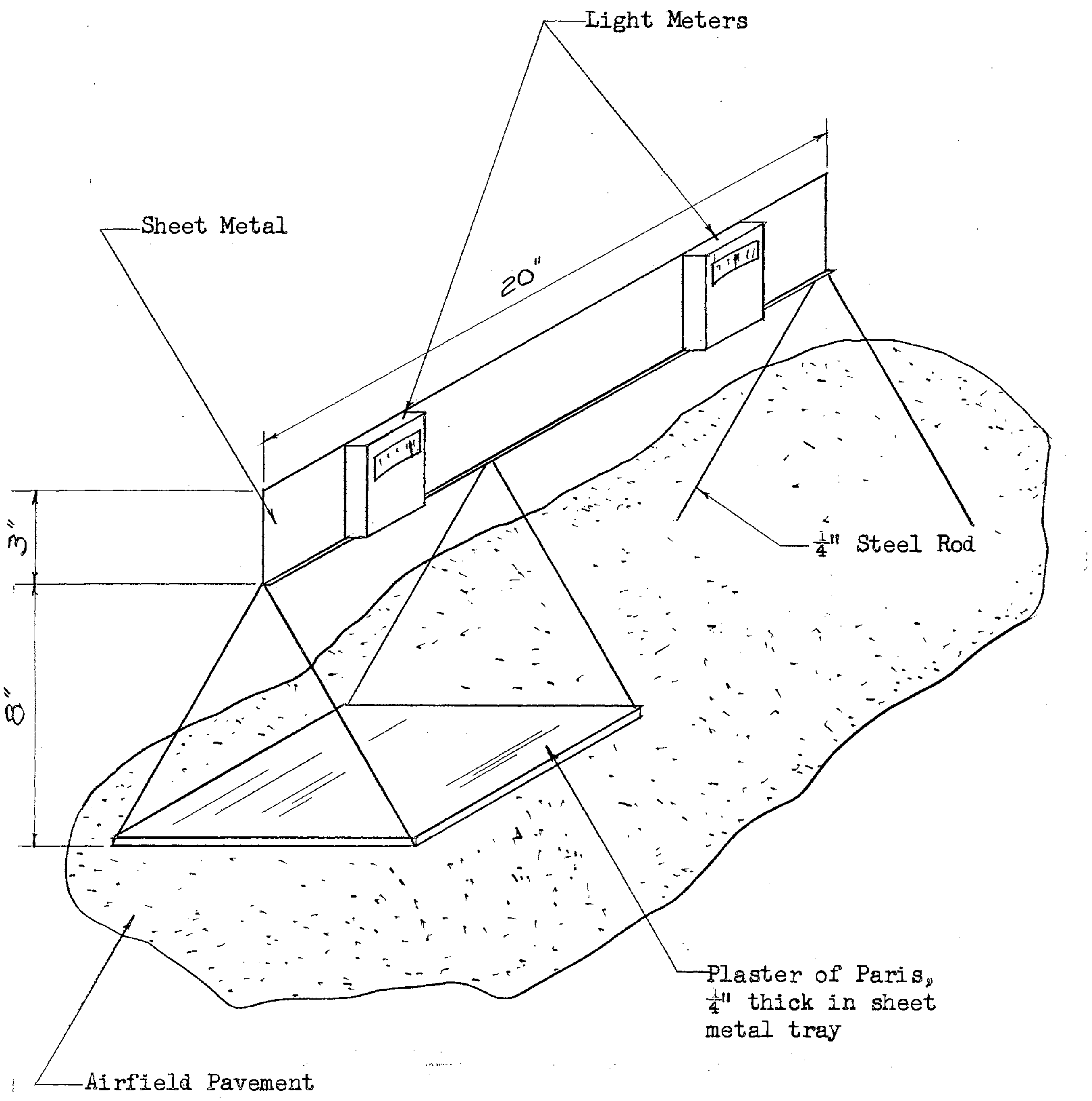


Fig. 1. "Reflectometer" Sketch.

TABLE II

SUGGESTED TABLE FOR USE IN INSPECTION FOR COMPLIANCE WITH SPECIFICATIONS FOR PAVEMENT SURFACE REFLECTIVITY

Pavement Surface Reading

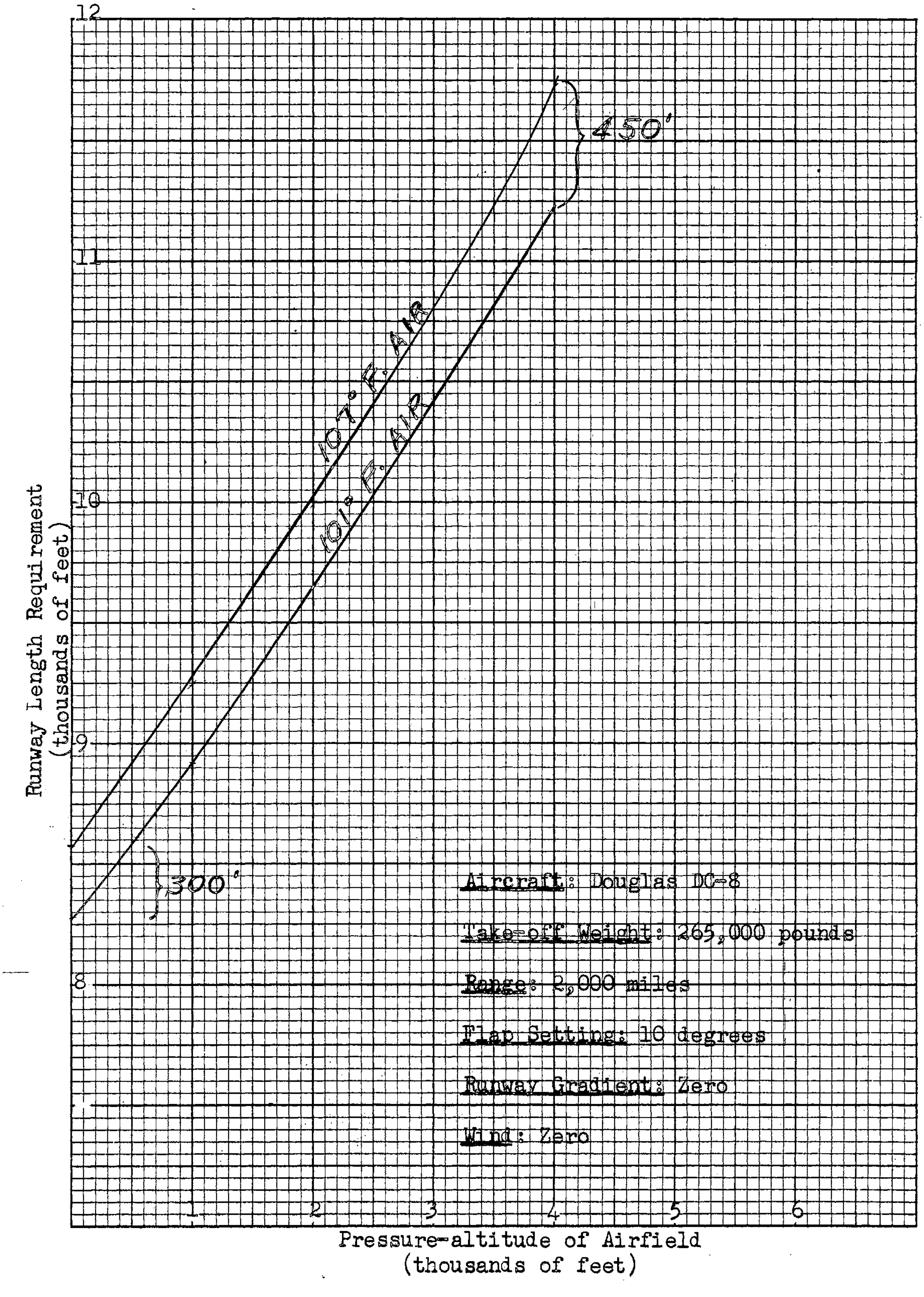
Plaster of Paris Surface Reading

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
2	100													
3	66.7	100												
4	50.0	75.0	100											
5	40.0	60.0	80.0	100										
6	33.3	50.0	66.7	83.3	100									
7	28.6	42.8	57.1	71.4	85.8	100								
8	25.0	37.5	50.0	62.5	75.0	87.4	100							
9	22.2	33.3	44.5	55.5	66.7	77.7	88.9	100						
10	20.0	30.0	40.0	50.0	60.0	70.0	80.0	90.0	100					
11	18.2	27.3	35.4	45.5	54.6	63.6	72.7	81.8	91.0	100				
12	16.7	25.0	33.3	41.6	50.0	58.3	66.7	75.0	83.3	91.7	100			
13	15.4	23.1	30.8	38.4	46.1	53.8	61.6	69.3	77.0	84.7	92.3	100		
14	14.3	21.4	28.5	35.6	42.8	50.0	57.1	64.4	71.5	78.6	85.7	92.9	100	
15	13.3	20.0	26.7	33.3	40.0	46.7	53.4	60.0	66.7	73.3	80.0	86.7	93.3	100
	2	3	4	5	6	7	8	9	10	11	12	13	14	15

Percentage of Reflectivity as Compared With Plaster of Paris

Pavement Surface Reading

Note: Plaster of Paris has reflectivity factor of 90% to 92% of impinging light rays.



Aircraft: Douglas DC-8

Take-off Weight: 265,000 pounds

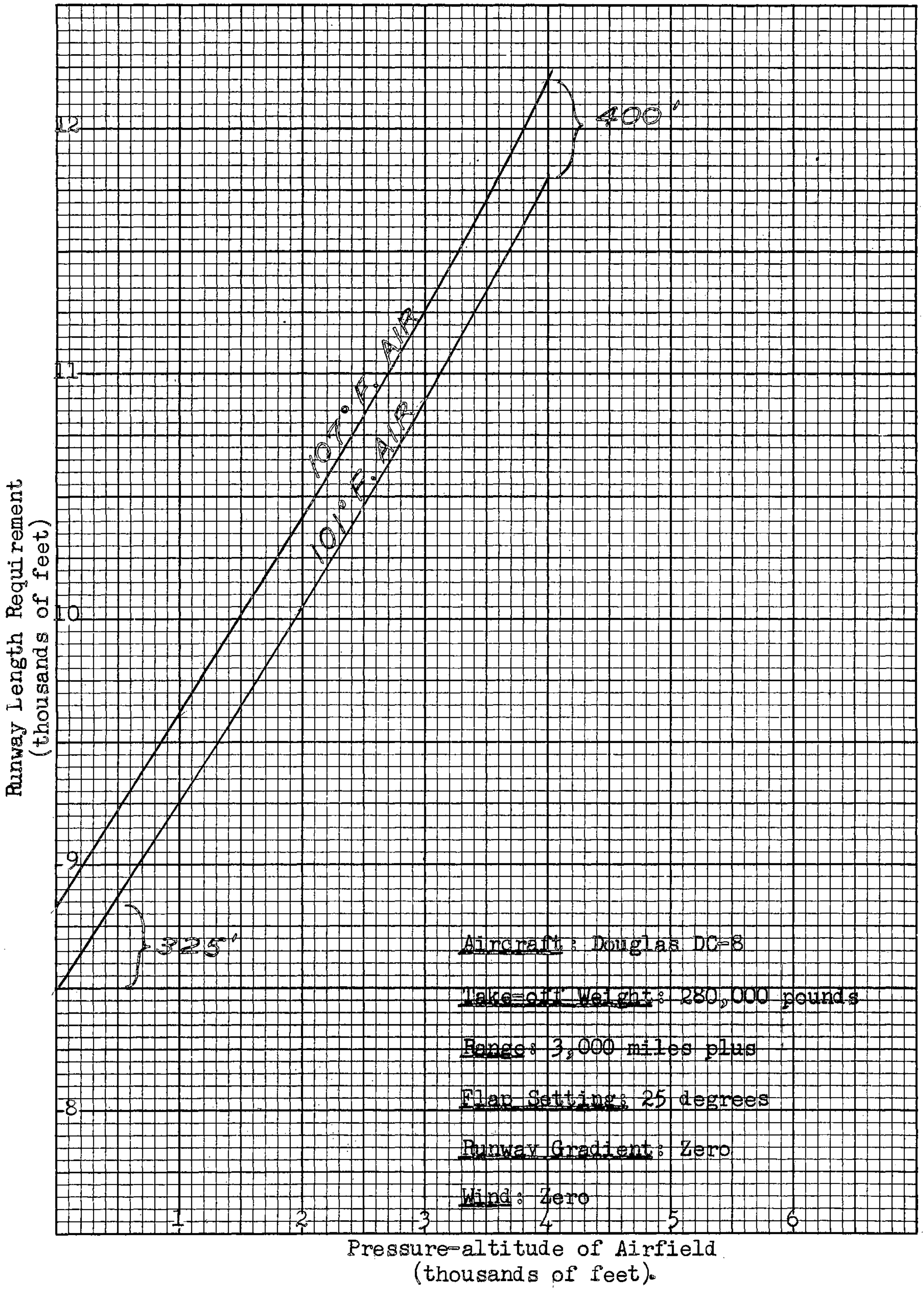
Range: 2,000 miles

Flap Setting: 10 degrees

Runway Gradient: Zero

Wind: Zero

Fig. 2. Variation in Runway Length Requirement.



Aircraft: Douglas DC-8

Take-off Weight: 280,000 pounds

Range: 3,000 miles plus

Flap Setting: 25 degrees

Runway Gradient: Zero

Wind: Zero

Fig. 3. Variation in Runway Length Requirement.

CHAPTER V

INTERNAL PAVEMENT TEMPERATURE TEST

Discussion and Procedure

As a basis for use in subsequent determination of the correlation between surface reflectivity and stability of asphaltic concrete pavement, it was necessary to devise a means of determining a correlation between surface reflectivity and internal pavement temperature.

It was decided that four different types of aggregate should be used for surfaces in the test: plant mix asphaltic concrete, without seal coat; grey limestone chip seal coat; Joplin chat seal coat; white limestone chip seal coat. The size of the aggregate used was a maximum of 3/8 inch. The limestone was obtained from the east quarry of the Standard Industry Company, Tulsa; the Joplin chat was obtained from a stockpile of the Turnpike Authority on the Turner Turnpike near Chandler, Oklahoma.

Consideration was given to the use of existing pavement and the natural radiation of the sun for the test. However, when the desirability of using several different surfaces was considered, the task under field conditions appeared too difficult. The intensity of radiation had to be the same for each type of surface. It was felt that the soil type under the pavement, and variations of moisture content of soil and the pavement, might be a source of great error. It was finally decided that the test should be made in the laboratory using an artificial source of radiant energy of fixed position and intensity for all types of surfaces.

For convenience, eight specimens were prepared using the same procedure as for Hveem stability test specimens, with the exception that a hole three inches deep and 3/8 inch in diameter was molded in the mid-layer of each specimen; a steel rod wrapped in waxed paper was used in forming the holes. The finished specimens were four inches in diameter and approximately 2 1/8 inches high. An asphalt content of six per cent by weight was used to obtain maximum stability asphaltic concrete.[11] Details of the mix design and compaction of the specimens are described in Chapter VII of this study.

After the specimens were molded, they were allowed to cure for several days. A double layer of aluminum foil insulation was then wrapped over all except the top surface and was taped in place. A coating of hot asphalt cement was brushed onto the top surface, and seal coat aggregate was sprinkled on and pressed into place by hand pressure. After several additional days of curing, the excess aggregate was brushed from the surfaces.

A 275 watt commercial sunlamp (General Electric) in an aluminum reflector was set up on a ringstand at a fixed distance above a ring which was positioned to support the specimens. The distance between the sunlamp and the specimen was set by adjustment until a thermometer laid on the surface of one of the grey limestone specimens indicated a constant 105 degrees F. This temperature was determined appropriate by recordings made under parallel circumstances during the tests which were made at Tinker Air Force Base.

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[11] Moreland Herrin and J. Rogers Martin, Principles of Asphaltic Concrete Pavement Design, Control and Construction, Publication No. 96, Oklahoma Engineering Experiment Station (November, 1958), p. 31.

To reduce the time required for heating specimens under the sunlamp, pre-heating in a thermostatically controlled oven was used. The first specimen tested was pre-heated to 125 degrees F.; other specimens were pre-heated to as high as 140 degrees F.

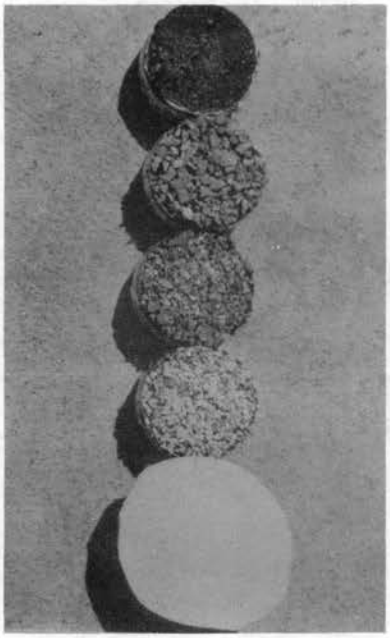
Upon removal of the test specimen from the pre-heat oven, a thermometer was inserted into the specimen, a small amount of Kleenex was packed around the thermometer at the mouth of the hole, and the specimen was put under the sunlamp.

The windows of the laboratory were kept closed to reduce air movement to a minimum. To further insure that a standard environment was maintained for all specimens, the temperature was checked at a fixed position on the table top, one foot from the center of the lamp, at the same time that a maximum temperature was attained in the specimen. The table top temperatures were within a two degree span, whereas, the room air temperature varied over an eight degree span. From this it was concluded that the variations of room air temperature had no significant influence on the resulting maximum temperatures attained in the specimens.

The reflectivities for the specimens and the method used for obtaining them are shown in Table III and in Fig. 6.

The results of the tests are shown in Tables IV through VII; review of these reveals close agreement between the two specimens of each aggregate type.

A plot of the maximum temperature attained, versus reflectivity, is shown in Fig. 7. It will be noted that the specimens with the white limestone seal coat were sixteen degrees cooler than the black specimens, and over seven degrees cooler than the specimens which were surfaced with the commonly used grey limestone and Joplin chat.



Black (without seal coat)

Grey limestone seal coat

Joplin chat seal coat

White limestone seal coat

Plaster of Paris

Fig. 4. Specimens in Natural Sunlight.

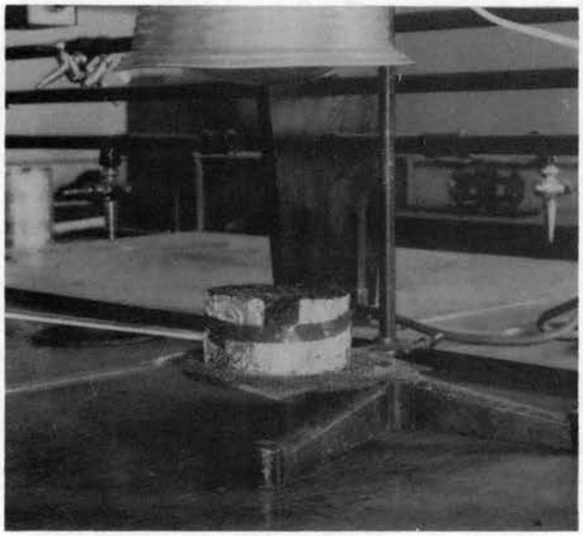


Fig. 5. Insulated Specimen on Ring-stand (note thermometer).

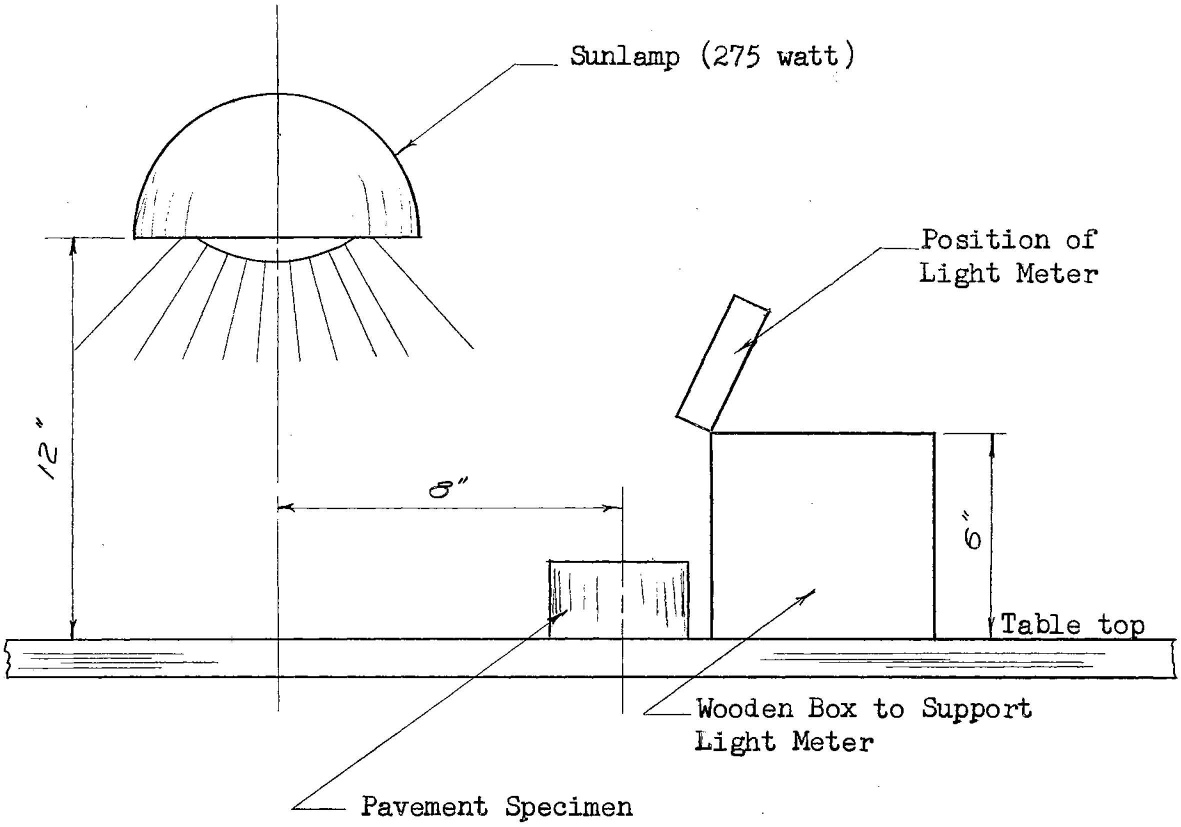


Fig. 6. Equipment set-up for obtaining reflectivities of laboratory specimens.

TABLE III

REFLECTIVITY OF PAVEMENT SPECIMENS USED IN LABORATORY TESTS

Type of Surface	Meter Reading	*Reflectivity
Plaster of Paris	9.75	100
Plant Mix (black)	3.20	33
Joplin Chat	4.50	46
Grey Limestone	4.50	46
White Limestone	6.40	65

* All reflectivities are expressed as a percentage of the reflectivity of plaster of Paris.

TABLE IV

INTERNAL PAVEMENT TEMPERATURES

Type of Surface: Black (without seal coat)

Specimen No.	Time	Temperature (degrees F.)	Remarks
1	0945	125	Specimen pre-heated to 125 degrees and placed under sunlamp at 0940
	1100	148	
	1130	152½	
	1200	154½	
	1215	155½	
	1230	156¼	
	1245	157	
	1300	157 3/4	
	1315	158¼	
	1330	158½	Table top temperature, 118 deg. Room air temperature, 84 deg.
	1340	158½	
2	1445	135	Specimen pre-heated to 135 degrees and placed under sunlamp at 1438
	1600	153	
	1630	155	
	1645	156	
	1700	156 3/4	
	1715	157½	
	1730	158¼	
	1745	158 3/4	
	1800	159	
	1810	159	Table top temperature, 118 deg. Room air temperature, 88 deg.

TABLE V

INTERNAL PAVEMENT TEMPERATURES
Type of Surface: Joplin Chat

Specimen No.	Time	Temperature (degrees F.)	Remarks
1	1230	130	Specimen pre-heated to 130 degrees and placed under sunlamp at 1225
	1300	138	
	1340	144½	
	1400	146½	
	1410	147½	
	1420	148½	
	1430	149½	
	1440	150	
	1450	150¼	
	1500	150½	Table top temperature, 117 deg. Room air temperature, 92 deg.
2	1530	130	Specimen pre-heated to 130 degrees and placed under sunlamp at 1525
	1600	139	
	1610	141	
	1620	143	
	1640	148	
	1650	149½	
	1710	150¼	
	1720	150¼	Table top temperature, 117 deg. Room air temperature, 92 deg.

TABLE VI

INTERNAL PAVEMENT TEMPERATURES

Type of Surface: Grey Limestone

Specimen No.	Time	Temperature (degrees F.)	Remarks
1	1230	141	Specimen pre-heated to 141 degrees and placed under sunlamp at 1226
	1250	144½	
	1310	146¼	
	1320	147½	
	1330	148½	
	1340	149½	
	1350	150	
	1400	150¼	
	1410	150¼	Table top temperature, 117 deg. Room air temperature, 89 deg.
2	1440	140	Specimen pre-heated to 140 degrees and placed under sunlamp at 1435
	1500	144	
	1520	147	
	1530	148	
	1540	149	
	1550	150	
	1600	150½	
	1610	150 3/4	
	1620	150 3/4	Table top temperature, 117 deg. Room air temperature, 88 deg.

TABLE VII

INTERNAL PAVEMENT TEMPERATURES

Type of Surface: White Limestone

Specimen No.	Time	Temperature (degrees F.)	Remarks
1	1140	126	Specimen pre-heated to 126 degrees and placed under sunlamp at 1135
	1210	134	
	1230	137	
	1240	138½	
	1300	140½	
	1310	141¼	
	1330	142¼	
	1340	142½	
	1350	142½	Table top temperature, 117 deg. Room air temperature, 86 deg.
2	1400	135	Specimen pre-heated to 135 degrees and placed under sunlamp at 1356
	1430	138 1/8	
	1500	151 5/8	
	1510	142½	
	1520	142 7/8	
	1530	143	
	1540	143	Table top temperature, 117 deg. Room air temperature, 90 deg.

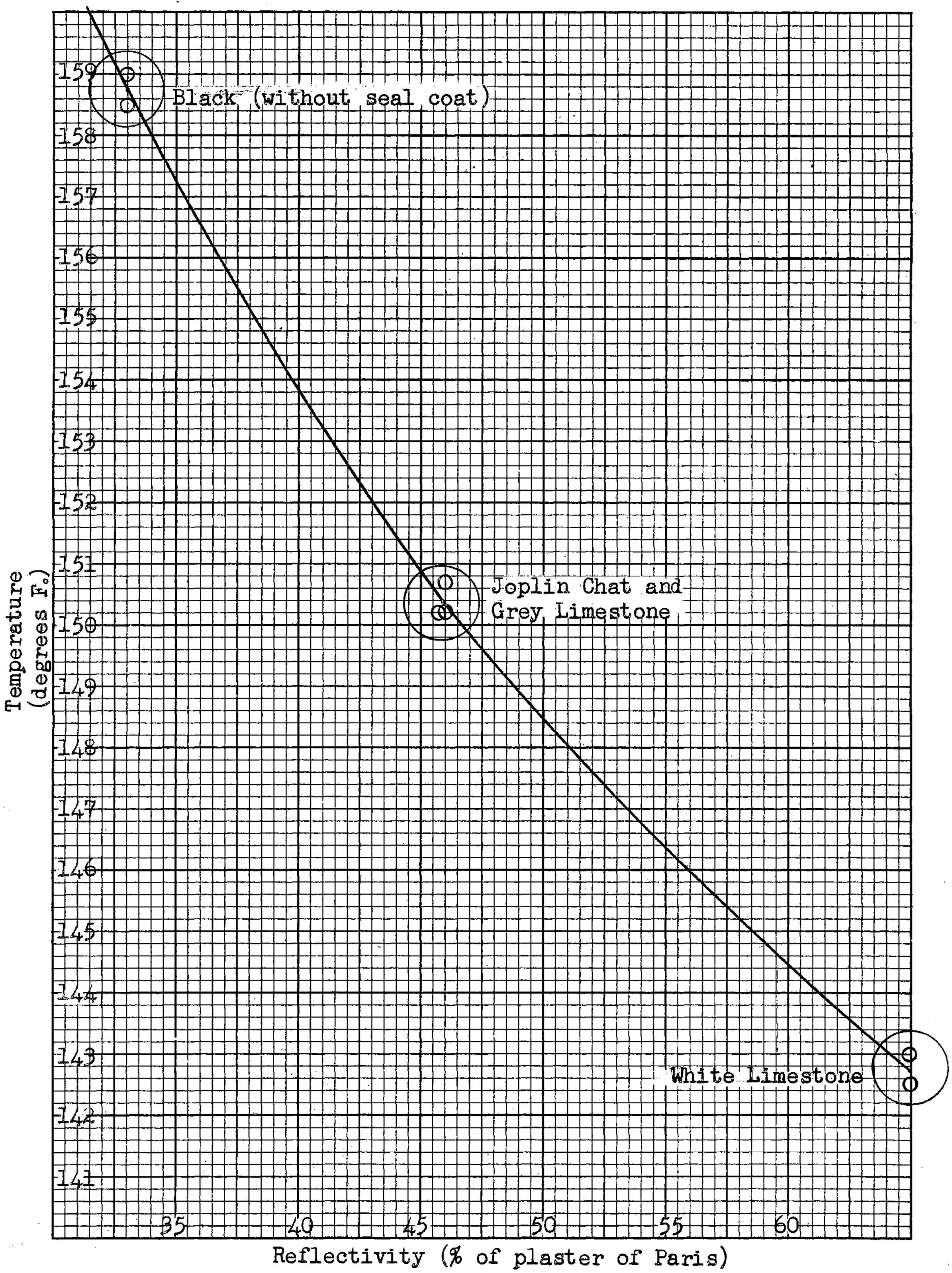


Fig. 7. Internal pavement temperature versus reflectivity of surface.

CHAPTER VI

MARSHALL STABILITY TEST

Discussion and Procedure

In actuality the title given to this phase of the research is a misnomer. In the Marshall test for stability of an asphaltic concrete specimen under load, the temperature is controlled in the specimen within one degree of 140 degrees F. The test was deliberately modified by the author in that the temperature was controlled at 6½ degrees below standard for half of the specimens tested, and 8 degrees above standard for the other half of the specimens. By so varying from the standard temperature, it was intended to demonstrate the significance of internal pavement temperature variations reported in the preceding chapter.

The Marshall method was chosen over other stability measurement methods for two reasons:

1. The specimen under test load is practically unconfined, and by virtue of this fact, temperature, which affects the viscous resistance and cohesion of the asphaltic cement, is a greater factor in the stability value determined than in other tests.

2. The Marshall method for the design and control of asphaltic concrete mix has been adopted by the U. S. Corps of Engineers and thus is used for a major portion of airfield paving design in the United States.

The Herrin-Martin publication previously referenced on page 18 was used for guidance in preparation of specimens and performance of tests. Only the main points in the procedure are described in this study.

Specimens for the test are 4 inches in diameter and 2½ inches (plus or minus $\frac{1}{4}$ inch) high. Based upon previous experience in molding specimens, the author decided upon a total weight of mix per specimen of 1235 grams. An asphalt content of six per cent by weight was used; therefore, the mix for each specimen consisted of 1161 grams of aggregate and 74 grams of asphalt cement (75-100 penetration grade). Grey limestone aggregate and fine sand were separated into the sizes shown below by use of a mechanical sieve shaker. The aggregates were then recombined in proportions specified by the Corps of Engineers for surface course asphaltic concrete mix:

TABLE VIII

SPECIFICATION OF AGGREGATES

Sieve Size	% Passing	% Retained	Weight in Grams	
3/4"	100	0	0	
1/2"	93	7	81	
#4	67.5	25.5	296	
#10	53	14.5	168	
#40	31	22	256	
#80	19	12	140	
#200	6.5	12.5	145	
Below #200	0	6.5	75	
		100.0	1161	Totals

The aggregate for each specimen was put into a small pan and into a 290 degree F. oven for a minimum of two hours. The asphalt cement was put into the oven and heated for one hour. At the end of the heating period, the aggregate was removed from the oven as needed; 74 grams of asphaltic cement was weighed and thoroughly mixed with the aggregate. Just prior to placement of the mix in the compaction mold, the temperature of the mix was checked to be certain that it was above 225 degrees F. Two mold sets were available. One set remained in the oven while the second set was used to mold a specimen, and the alternate set was used for the next specimen. The mix was compacted in the mold by 50 blows of a standard hammer, and then the collar was turned upside down and 50 blows were applied to the other side. The head of the hammer was kept hot over a Bunsen burner when not in use. After compaction the specimens were cooled in cold water for two or more minutes; then they were extruded from the collar of the mold by use of a hydraulic press.

As a check for uniformity of the eight specimens, the specific gravity of each specimen was found by the process of weighing a specimen in air, coating the specimen with paraffin and weighing in air again, and then weighing the specimen while it was suspended in water. Using this data, together with the known specific gravity of the paraffin and the asphaltic cement, the specific gravity and percentage of air voids for each specimen was calculated. The results of this check are contained in Table IX. A sample calculation for percentage of air voids is contained in Appendix A.

After the paraffin coating had been removed from the specimens and they had cured for one month, the stability and flow tests were accomplished. The apparatus for these tests is shown in Fig. 8. Prior to commencement of the tests, the rate of downward travel of the head of the hydraulic test machine was adjusted to two inches per minute. A thin film of oil was put onto the guide rods and all surfaces of the Marshall testing head which would be in contact with the specimen.

Four of the specimens were put into a 133.5 degree F. water bath for 20 minutes prior to start of the load tests. The specimens were taken from the bath one at a time and tested in rapid succession (reference Table X).

The temperature of the bath was then increased to 148 degrees F. and the process was repeated for the remaining four specimens. As a specimen was removed from the bath, it was shaken to remove excess water and was immediately placed in the testing head. A slight initial pressure was applied to the specimen, the flow measuring gauge was placed on top of the guide rod, zeroed, and held firmly against the Marshall testing head. Load was then applied until the specimen failed. At the time of failure, the maximum load applied and the flow distance were noted.

The results of the load tests are shown in Table X. Review of these results indicate a marked decrease of stability in the specimens which were tested at the higher temperature. The significance of these results is discussed in Chapter VIII.

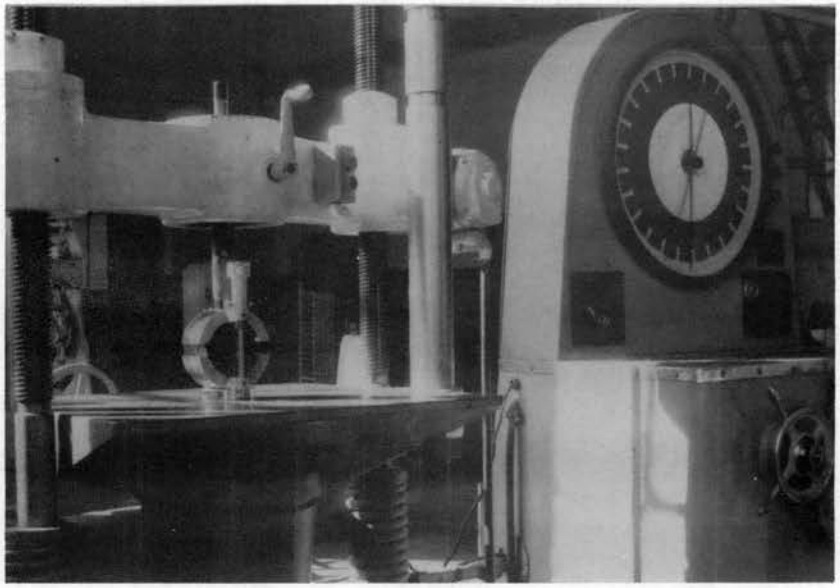


Fig. 8. Marshall Test Apparatus With a Specimen in Position for Testing.

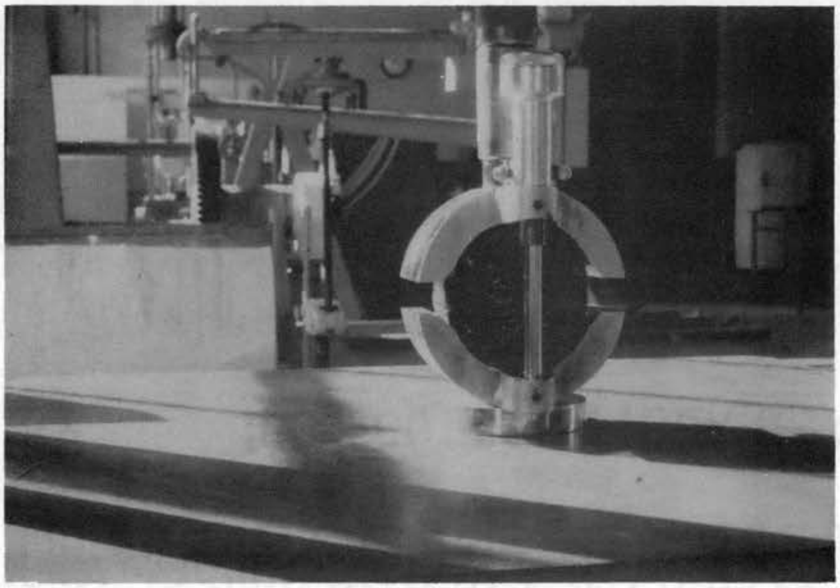


Fig. 9. Close-up of the Marshall Testing Head and the Flow Meter.

TABLE IX

DATA FOR MOLDED SPECIMENS FOR MARSHALL STABILITY TEST

Sample Number	Height (inches)	Weight (gm) (A)	Weight With Paraffin (B)	Weight in Water With Paraffin (C)	Specific Gravity (G)	Theoretical Maximum Specific Gravity (G_m)	% Voids (V_t)
1	2 9/16	1227.0	1255.0	702.0	2.35	2.41	2.5
2	2 5/8	1230.0	1260.0	698.0	2.33	2.41	3.3
3	2 5/8	1231.5	1287.8	695.0	2.33	2.41	3.5
4	2 9/16	1210.8	1245.2	685.0	2.32	2.41	4.0
5	2 9/16	1206.8	1236.0	685.0	2.33	2.41	3.5
6	2 1/2	1176.5	1207.8	667.0	2.32	2.41	4.0
7	2 9/16	1233.6	1273.5	703.5	2.35	2.41	2.5
8	2 9/16	1209.8	1254.8	684.0	2.32	2.41	4.0

TABLE X

RESULTS OF MARSHALL STABILITY TEST FOR SIX PER CENT ASPHALT SPECIMENS AT VARIOUS TEMPERATURES

Sample Number	Water Bath			Flow Value (1/100")	Maximum Load	Height Correction Factor	Marshall Stability
	Temperature (deg. F.)	Time In	Time Out				
1	133.5	1543	1607	13	1730	.96	1662
2	133.5	1543	1608	15	1635	.93	1520
3	133.5	1543	1609	18	1715	.93	1594
4	133.5	1543	1610	18	1680	.96	1612
5	148.0	1621		17	1410	.96	1352
6	148.0	1621		17	1215	1.0	1215
7	148.0	1621		16	1510	.96	1450
8	148.0	1621		21	1300	.96	1250

CHAPTER VII

PLATE BEARING TEST

Discussion and Procedure

The author felt that the results of the Marshall test were representative of action under load in the top few inches of the surface course of an asphaltic concrete pavement where the aggregate in the paving is relatively unconfined. However, it was believed that stability at lower levels in the pavement would be far less affected by viscosity, and hence, by temperature of the asphalt cement. It was decided that load tests should be made on partially confined specimens, a state somewhat similar to the condition at lower levels in a paving slab. Although the nature of the procedures used in this test is such that the results have no quantitative application, it is believed that they are of value in demonstrating, to some extent, the validity of the Marshall test in determining stability of asphaltic concrete pavements.

The main points in the procedure of specimen preparation are given below. A more detailed description of the procedure may be found in Chapter VII of the Herrin-Martin publication referenced on page 18 of this study.

Eight specimens were prepared, four of 6 per cent asphalt content, and four of 8 per cent asphalt content. The combination of limestone and fine sand aggregates was in the proportions of Table VIII, page 29. Based upon previous laboratory experience, it was decided to use 930 grams of mix per specimen to produce a size comparable to that normally used in the

Hveem stability test. The finished specimens were four inches in diameter and approximately two inches high. A Hobart C-100 mixer was used to mix together the aggregates and the asphalt cement prior to placement in the mold for compaction. The following proportions of aggregate to asphalt cement were used:

6 per cent asphalt specimens:

Total weight = 930 grams
Aggregate weight = .94 x 930 = 874 grams
Asphalt cement weight = .06 x 930 = 56 grams

8 per cent asphalt specimens

Total weight = 930 grams
Aggregate weight = .92 x 930 = 856 grams
Asphalt cement weight = .08 x 930 = 74 grams

The aggregate and the asphalt cement were heated to 255 degrees F. before they were mixed. The mix was placed in the Hveem mold and compacted to a final pressure of 1600 p.s.i. with a hydraulic press. The molded specimens were extruded from the mold collar and allowed to cure for four weeks.

A check on the uniformity of the specimens was made by determining the specific gravity and percentage of air voids for each specimen. A sample calculation is contained in Appendix B. The results of this check are contained in Table XI.

Two of the 6 per cent and two of the 8 per cent specimens were heated in a 100 degree F. oven. They were removed from the oven one at a time, as needed for testing, and were immediately placed in the Hveem mold collar. The head speed of the 60,000 pound hydraulic test machine was adjusted to one-fourth inch per minute. Each specimen was tested to failure using a round bearing plate of 2.67 square inches in area. The results of these tests are contained in Table XII. The procedure was repeated for specimens which were heated in a 119 degree F. oven.

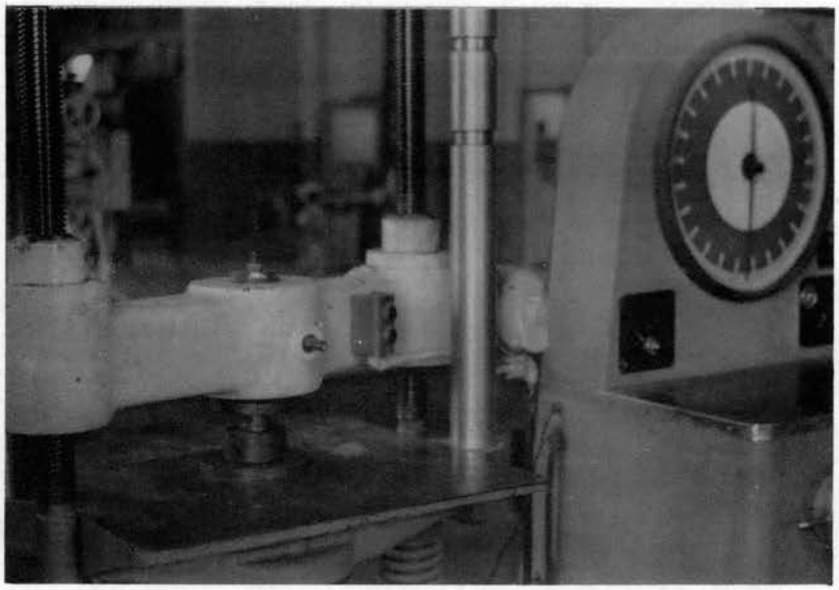


Fig. 10. Plate Bearing Test Apparatus With a Specimen in Position for Testing.

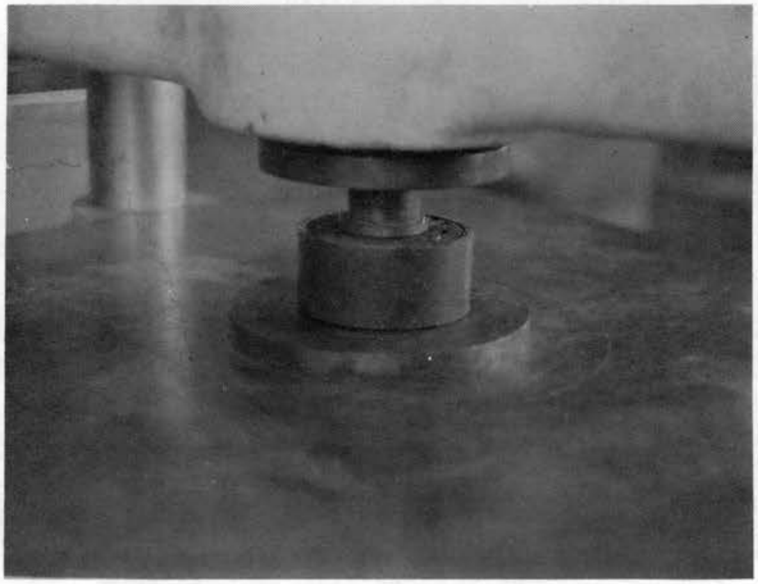


Fig. 11. Close-up of the Plate Bearing Test in Confining Collar.

TABLE XI

DATA FOR MOLDED SPECIMENS FOR PLATE BEARING TEST

Sample Number	% Asphalt (W_b)	Weight (gms.) (A)	Weight With Paraffin (B)	Weight in Water With Paraffin (C)	Specific Gravity (G)	Theoretical Maximum Specific Gravity (G_o)	% Voids (V_t)
1	6	926	989.0	509.0	2.27	2.42	6.3
2	6	931	1005.0	514.0	2.28	2.42	6.0
3	6	928	992.0	515.0	2.29	2.42	5.4
4	6	925	980.0	511.0	2.27	2.42	6.2
5	8	930	972.0	528.0	2.34	2.34	0
6	8	928	969.0	529.0	2.36	2.36	0
7	8	924	1023.0	519.0	2.35	2.35	0
8	8	932	983.0	530.0	2.35	2.35	0

TABLE XII

RESULTS OF PLATE BEARING LOAD TEST

Specimen Number	Temperature of Specimen (degrees F)	Asphalt Content (%)	Load at Failure (lbs.)	* Unit Load at Failure (p.s.i.)	Average Unit Load at Failure (p.s.i.)
1	100	6	3300	1240	1210
2	100	6	3150	1180	
3	100	8	1200	450	456
4	100	8	1250	468	
5	119	6	3000	1120	1105
6	119	6	2900	1090	
7	119	8	1500	560	570
8	119	8	1600	580	

* Using a round plate of 2.67 square inches in area.

CHAPTER VII

SUMMARY, CONCLUSIONS, AND RECOMMENDATIONS

The purpose of this chapter is to summarize and discuss the findings, to summarize conclusions, and to state recommendations.

Summary

The Airfield Temperature Test resulted in the finding that a variation of least six degrees F. in the temperature of the air above a runway surface may be effected by merely varying the color of the aggregate in the seal coat.

The Internal Pavement Temperature Test resulted in the finding that the temperature of asphaltic concrete paving may be decreased as much as 16 degrees F. by using a seal coat with high reflectivity instead of an unsealed surface or a seal coat of dark colored aggregate. It was found that the relationship of surface reflectivity to pavement temperature is close to a straight line. Commonly used grey limestone and Joplin chat aggregate are both about midway between an unsealed surface and a white limestone chip surface in reflective quality.

It was determined in the Marshall Stability Test that an increase of 14.5 degrees F. in the temperature of surface course asphaltic concrete may result in an 18 per cent decrease in stability. The stability loss for the normal summer temperature range amounts to an average of 1.24 per cent per degree F.

The Plate Bearing Test results for specimens of 6 per cent asphalt content indicated a decrease in bearing capacity of 8.7 per cent for a temperature increase of 19 degrees F., or 0.46 per cent per degree F. In the 8 per cent asphalt specimens, a 53 per cent decrease in bearing capacity was found for a temperature increase of 19 degrees F., or an average of 2.8 per cent per degree F.

Conclusions

As a result of the author's experiences during the Airfield Temperature Test, it is apparent that whatever the findings of the test might be, that the seriousness of the problem considered would vary widely depending on climatology of the airfield site. However, for the conditions reported upon at Tinker Air Force Base, the increased temperature found above the pavement of low surface reflectivity is of important significance in airfield design. The 400 foot increase in average runway length requirement caused by the higher temperature indicates that lack of consideration of surface reflectivity could have serious consequences.

The findings in the Internal Pavement Temperature Test, the Marshall Stability Test, and the Plate Bearing Test indicate that surface reflectivity is a factor of major importance to stability in the surface course of asphaltic concrete pavement. The decrease in the effect of temperature in six per cent asphalt specimens in the Plate Bearing Test, as compared with the Marshall Test, indicates that the Marshall Test is not representative of the action in the lower levels of pavement slabs. The marked effect of temperature increase on the bearing capacity of the eight per cent asphalt specimens, even when confined as they were in the Plate Bearing Test, is an indication of the importance of controlling the asphalt cement content of pavement mix.

Application of the results of the Marshall Test to the temperature variations in the Internal Pavement Temperature Test indicates that as much as a 20 per cent increase in stability of asphaltic concrete surface courses may be attained by use of light colored seal coat aggregate.

A load imposed upon a properly designed asphaltic concrete pavement is transferred to the sub-grade through the pressures transmitted from aggregate to aggregate. The asphalt serves only as a cementing agent to bind the aggregate in proper position to transmit the load.[12] If an excess of asphalt is present, the load on the pavement will be transferred through pressure in the asphalt; and, if the asphalt is in a soft plastic or a liquid state due to excessive heat, the result will be slippage between the surfaces of the aggregate. The outward evidence of this state is "bleeding" of asphalt on the pavement surface, and shoving or rutting of the surface.

An excess of air voids in asphaltic concrete results in accelerated hardening and deterioration of the asphalt.[13] Keeping the temperature of the pavement as low as possible will permit use of a higher percentage of asphalt cement without loss of stability.

Recommendations

From the findings in this study the following recommendations are offered:

1. That aggregate of the lightest color economically available be specified for use in seal coats for asphaltic concrete pavements in all but the most temperate areas in the United States.
2. That existing pavement which shows evidence of "bleeding" be treated by brooming fine white limestone chips into the surface

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[12] J. Rogers Martin and Hugh A. Wallace, Design and Construction of Asphalt Pavements, (New York, 1958), p. 1.

[13] Ibid., p. 57.

and rolling lightly prior to brooming off excessive chips. Any suitable material, available locally, might be used; in Gulf Coast areas, white coral sand might be used.

3. That a "Reflectometer" method be used for checking for compliance with specifications as to surface reflectivity. This would be required in most areas due to greater abundance, and thus lower cost, of darker colored aggregates.

4. That the frequency of occurrence of conditions of zero wind and simultaneous high temperature be determined as basic data for airfield sites; and, if economically feasible, runway length for the critical aircraft to be accommodated should be provided considering the maximum temperature which will be experienced above the runway surface.

5. That temperatures, supplied to flight crews for planning permissible take-off weights for aircraft, be obtained by equipment which records in a pavement area which simulates the temperature conditions of the runway. Correction of free air temperatures by what might be called a "runway temperature calibration factor" might be used, in lieu of actual runway temperature recordings, during periods of zero wind.

Recommendations for Further Study

The following subjects are recommended for further study:

1. Additional airfield temperature tests of the type described in this study, to determine if the maximum temperature differential above pavements of varying surface reflectivity has, in fact, been determined. Such testing could best be accomplished by a person or agency permanently located on or near a large airfield.

2. Determination of the effect of aggregate size upon reflectivity of seal coat surfaces.

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APPENDIX A

Sample Calculations for the Air Voids Ratio for Marshall Test Specimens

G = Specific gravity of the specimen
A = Weight of the specimen
B = Weight of the specimen when coated with paraffin
C = Weight of the specimen when suspended in water
G_p = Specific gravity of paraffin 0.89

G_m = Theoretical maximum specific gravity of the specimen
W_b = Percentage of asphalt by weight
W_a = Percentage of aggregate by weight
G_b = Specific gravity of asphalt 1.0
G_a = Specific gravity of aggregate 2.65

V_t = Percentage of air voids in specimen

Step 1. $$G = \frac{A}{(B - C) - \frac{B - A}{G_p}} = \frac{1230}{(1260 - 698) - \frac{1260 - 1230}{.89}}$$

$$G = 2.33$$

Step 2. $$G_m = \frac{100}{\frac{W_a}{G_a} + \frac{W_b}{G_b}} = \frac{100}{\frac{94}{2.65} + \frac{6}{1.0}}$$

$$G_m = 2.41$$

Step 3. $$V_t = 100 - 100\frac{G}{G_m} = 100 - 100\,\frac{2.33}{2.41}$$

$$V_t = 3.3\%$$

APPENDIX B

Sample Calculations for the Air Voids Ratio for Plate Bearing Test Specimens

G = Specific gravity of the specimen
G_o = Maximum theoretical specific gravity of the specimen
A = Weight of the specimen
B = Weight of the specimen when coated with paraffin
C = Weight of the specimen when suspended in water
G_p = Specific gravity of paraffin 0.89
G_a = Specific gravity of the aggregates
G_b = Specific gravity of the asphalt 1.0
W_b = Percentage of asphalt in specimen
V_t = Percentage of air voids in the specimen

Step 1. Find the specific gravity of the specimen (specimen No. 2)

$$G = \frac{A}{(B - C) - \frac{(B - A)}{G_p}} = \frac{931}{(1005 - 514) - \frac{(1005 - 931)}{.89}}$$

$$G = 2.28$$

Step 2. Find the specific gravity of the aggregate. Based on the assumption that an 8 per cent asphalt specimen has no air voids, the specific gravity of an 8 per cent specimen would be the same as the maximum theoretical specific gravity. Using the data for specimen No. 7 (an 8 per cent specimen):

$$G_o = \frac{100}{\frac{100 - W_b}{G_a} + \frac{W_b}{G_b}}$$

$$2.35 = \frac{100}{\frac{100 - 8}{G_a} + \frac{8}{1}}$$

$$G_a = 2.68$$

Step 3. Find the theoretical maximum specific gravity (specimen No. 2)

$$G_o = \frac{100}{\frac{100 - W_b}{G_a} + \frac{W_b}{G_b}} = \frac{100}{\frac{100 - 6}{2.68} + \frac{6}{1}} = 2.42$$

Step 4. Find percentage of air voids in specimen No. 2.

$$V_t = 100 - 100\frac{G}{G_o} = 100 - 100\,\frac{2.28}{2.42} = 6\%$$

VITA

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Master of Science

Thesis: SURFACE COLOR AS A FACTOR IN ASPHALTIC CONCRETE AIRFIELD PAVEMENT DESIGN

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