Thermal Properties of Some Asphaltic Concrete Mixes

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ABSTRACT

To obtain an increased understanding of the energy transfer in asphaltic concrete recycling processes that use surface heat, laboratory tests were carried out on four asphaltic concrete mixes (three having limestone aggregate and one with expanded shale lightweight aggregate) to determine thermal properties. It was found that the thermal conductivity of the three limestone mixes depended on asphalt content and aggregate gradation, but that the conductivity of the lightweight aggregate mixes varied little as the asphalt content was increased from 3.5 to 6.5 percent. The specific heat and diffusivity of the mixes varied with mineralogy and gradation but changed little with asphalt content.

In the 1930s construction equipment was developed that would plane off the high surfaces of uneven, deteriorating asphaltic concrete pavements. It was found that, by first applying heat to the pavement surface to reduce the shearing resistance of the asphaltic concrete, the planing operation proceeded faster and easier and could be accomplished at less expense. These heater planers are still in use today and provide a cost-effective option for some types of pavement maintenance.

Heater scarifiers are also used in maintenance operations of asphaltic concrete pavements. These machines apply heat to the pavement surface to reduce the shear strength and then scarify or remove the degraded uppermost part of the pavement. This process allows new materials (or recycled materials), including aggregates, modifiers, and asphalt cement, to be thoroughly mixed into the scarified asphaltic concrete in situ or enables the spent pavement to be easily removed and hauled to a central location where additional materials are added. Recycling existing pavement materials can be an attractive alternative in pavement maintenance strategies because of the increased cost of asphaltic concrete in the past 10 years, which has been commensurate with the skyrocketing costs of petroleum products.

Although pavement heating techniques and equipment development have evolved over several decades, little research has been undertaken to provide an understanding of the heat transfer process in asphaltic concrete with a view toward optimizing the field procedure with respect to equipment costs and fuel consumed. Much of the research done in this field has been proprietary in nature and carried out by contractors on a trial-and-error basis. This work has been results-oriented, with success being the development of a method of pavement maintenance that is cost effective in competition with alternative nonheater methods.

Civilian and military airfield and highway pavement budgets have been and will continue to be heavily oriented toward maintenance of existing facilities rather than dedicated to new construction. Pavement recycling can be expected to play an increasingly important role in maintenance in the future, and, as the cost of the fuel used to heat pavements increases, the capability of predicting the temporal spatial-temperature field setup by a known heat source in an in-service pavement will be imperative in order for recycling methods using field heaters to remain competitive with recycling methods not using heat.

Accordingly, the subject of this paper is concerned with the first step leading to this goal: the measurement of the thermal properties of asphaltic concrete mixes that have different asphalt contents, aggregate densities, and aggregate gradations.

BACKGROUND INFORMATION

That some properties of asphaltic concrete vary with temperature has for many years fostered interest in the distribution of temperature within an in situ pavement. Interest in predicting frost penetration beneath pavements, minimum and maximum temperatures caused by ambient conditions, and the cooling rates of hot-laid asphaltic concrete has motivated research in heat transport through pavements.

Carlson and Kersten (1) applied heat transfer theory to the pavement and soil subgrade to develop a model used to predict frost penetration below asphaltic pavement. Input parameters included the thermal conductivity of the pavement (1.44 w/m°C), the latent heat of fusion of water, and the surface freezing index of the pavement. Aldrich (2) improved on Carlson and Kersten's approach of calculating frost penetration by including the effect of volumetric heat. Aldrich used a thermal conductivity value of 1.45 w/m°C in his analysis.

Barber (3) predicted in situ pavement temperatures by using the diffusion equation. Because the strength and stability of an asphaltic pavement are related to its temperature, research has been conducted to determine the minimum and maximum temperatures of pavement in natural field conditions. Southgate and Deen (4) analyzed the pavement temperatures recorded at various depths in a 0.3-m-thick asphaltic pavement. They found that a fourth-order polynomial fit the temperature as a function of depth data. No attempts were made to analyze the data by using heat transfer theory.

Rumney and Jimenez (5) recorded pavement temperatures with depth for a year in Arizona. Based on the observed data, they presented empirically derived curves whereby pavement temperature at depth could be estimated by knowing the maximum air temperature and average daily radiation rates.

Straub et al. (6) recorded pavement temperatures with depth for a 1-year period in 0.15- and 0.3-m-thick asphaltic concrete pavements and also measured corresponding air temperature and solar radiation. They found that the pavements had a substantial temperature gradient and no one temperature was representative of that of the pavement. They noted that changes in solar radiation have a larger effect on pavement temperatures than that caused by changes in air temperature. Straub et al. used a forward difference, one-dimensional transient heat flow program to predict pavement temperature with depth.

Dempsey and Thompson (7) used a model similar to that of Straub et al. to predict temperatures with depth in conjunction with highway frost studies. They reported that the accuracy of the temperatures predicted by the one-dimensional heat transfer model depended more on the quality of the input data than on the numerical method of calculation. They also pointed out the importance of accurately defining the boundary condition at the pavement surface because it is this input that is the major factor contributing to the heat transfer process.

Christison and Henderson (8) also used a finite difference approximation to predict temperatures in asphaltic concrete. They assumed a value of 1.45 w/m°C for the thermal conductivity of asphaltic concrete and stated that "the thermal conductivity (k) and heat capacity (c) of asphaltic concrete paving mixtures vary within narrow limits and for practical purposes can be considered independent." However, they did not indicate the basis for this statement or their source for the assumed thermal properties.

The references cited indicate that a numerical solution of the one-dimensional heat flow problem using a finite difference technique has excellent predictive capabilities for asphaltic concrete pavements under natural ambient conditions. However, the heat regime to which the pavement is subjected by a heater planer or heater scarifier is much different. The range of temperatures is greater, and the time scale is on the order of seconds, versus hours. Finally, the temperature level of the heat source and pavement surface is much different, which strongly affects the nature of the radiant heat exchange on the surface.

During paving operations a hot-laid asphaltic pavement must be sufficiently compacted before it cools below a specified temperature, which is about 90°C for most pavements. This temperature is required to achieve a specified density with a minimum amount of compactive effort. Compaction attempted when the pavement is too cool results in either a longer time period needed to reach a specified density or the inability to reach the specified density at all. Corlew and Dickson (9) used a one-dimensional transient flow of thermal energy equation to predict the temporal spatial-temperature field within a cooling layer of freshly placed hot-mix asphaltic concrete. By using a finite difference technique, they were able to predict temperatures in a 6-cm layer of asphaltic concrete within 7°C of measured temperatures in the range at which asphaltic concrete is compacted (greater than 90°C).

Frenzel et al. (10) developed a computer analysis to study the effect of preheating an existing asphaltic layer on the cooling of an overlay put down over the preheated base. They found, analytically, that preheating the base increased the cooling-down period in thin overlays sufficiently to allow a compaction window long enough to make paving in early spring and late fall feasible. Later, Corlew and Dickson (11) combined the base preheat model developed by Frenzel et al. and their previously described cooling model (9) to predict the temporal spatial-temperature field of an asphaltic concrete layer over a preheated base in a bench scale laboratory test. A sample with a 10-cm-diameter base was heated by a direct-fired propane heater. Agreement between predicted and measured temperatures was considered to be satisfactory. Assumed thermal properties of asphaltic concrete were used in the analysis. Another study of asphaltic pavement cooling rates was conducted by Wolfe and Colony (12). They developed a computer simulation method to predict cooling rates by using the same weather data and material property variables as Corlew and Dickson. The thermal properties of asphaltic pavement were taken from published values.

In-place surface recycling of asphaltic pavements involves reworking the surface to a depth of approximately 1 in. using a heater scarifier. This operation may involve the addition of new or recycled materials. The reworked material is then compacted, and sometimes a seal coat is applied. Typically, heater planers and heater scarifiers developed by contractors use propane as a fuel to fire a grid of torches. The grid is on the order of 3 m wide and 5 m or more long and is attached to a self-propelled machine. The torch grid is covered on the top and sides to diminish heat loss. Typically, the machine advances at the rate of about 4.5 m/min, giving a heat exposure time on the order of 1 min. To prevent combustion of the pavement, the temperature at the pavement surface should be limited to about 230°C. Contractors claim that, under these conditions, temperatures at a depth of about 2.5 cm in the asphaltic concrete are sufficiently high to allow easy scarification or removal of pavement to this depth. To increase the depth of influence of the heat source, a technique termed soaking is sometimes used. This method involves heating the surface as previously described and then insulating the pavement surface. This, in principle, allows the heat to soak in, thus producing greater heat penetration without exceeding the 230°C surface temperature requirement.

One of the few published studies undertaken on the subject of heat transfer in asphaltic concrete recycling appears to refute many of the contractor claims previously mentioned. Carmichael et al. (13) modeled an asphaltic concrete pavement as a semi-infinite solid and used a forward difference numerical method to solve the governing differential equation. They assumed that the thermal conductivity, density, and specific heat of asphaltic concrete were independent of temperature and did not vary from point to point. The authors, using realistic source and initial pavement temperatures along with realistic pavement parameters, found that even with an induced surface temperature of 540°C and a 30-sec exposure time, there was no increase in temperature of the pavement at a depth of 1.6 cm [see Figure 1 (13)].

It was also shown that for surface temperatures between 200° and 300°C, the temperature of the pavement at a depth of 1 cm hardly changed from its initial condition. This is illustrated in Figure 2 (13). Although it can be argued that the study by Carmichael et al. used restrictive assumptions, particularly with respect to the parameters used, and that the model had not been verified, it does indi-

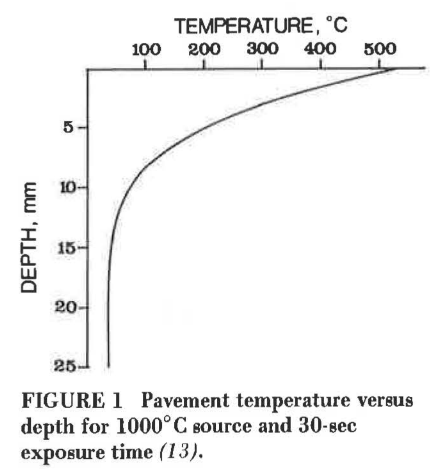

FIGURE 1 Pavement temperature versus depth for 1000°C source and 30-sec exposure time (13).

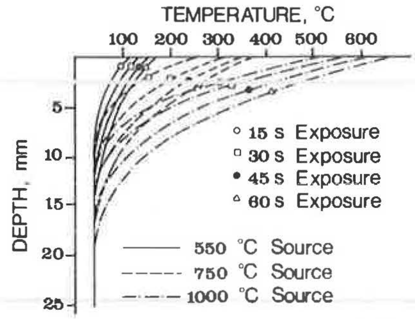

FIGURE 2 Pavement temperature versus depth as a function of source temperature and exposure time *(13)*.

cate that the current development of pavement heating techniques and the current use of heater planer and heater scarifier equipment ignores some of the basic constraints of asphaltic concrete recycling: namely, that to obtain sufficient temperature increases at depth in a reasonable period of time, it appears that the surface must be heated to a temperature greatly in excess of that at which damage to the pavement can occur. Carmichael et al. pointed out that it is important that the full depth of pavement be properly heated before scarifying because "cold asphalts do not bond well and any attempt to force cold, asphalt coated aggregates together by force after scarifying them by tensile failure of the asphalt binder can only lead to future problems" (13).

Much of the literature already cited used assumed values of the thermal properties of asphaltic concrete. However, as the data in Tables 1 and 2 indicate, measured values of conductivity and specific heat of asphalt and asphaltic concrete as well as minerals and rocks commonly used as asphaltic concrete aggregate are available. Reported values of thermal conductivity range from 0.74 to 2.88 w/m°C for asphaltic concrete and from 0.14 to 0.17 w/m°C for pure asphalt. Diffusivity values ranging from $5.8 * 10^{-7}$ to $14.4 * 10^{-7}$ m^2/sec have been reported. The range of reported values of specific heat (c) is 879 to 963 J/Kg°C for asphaltic concrete.

The predominant component of asphaltic concrete is aggregate. The thermal properties of the particular aggregate used in a given asphaltic concrete can be expected to have an important effect on the properties of the mix. The literature indicates that common minerals and rocks used as aggregate can have conductivities that range from about 1 to 8 w/m°C (Table 2); the range of c can be expected to be on the order of 650 to 1000 J/Kg°C. Wolfe et al. (14) suggest 920 J/Kg°C as an average value based on their tests. Limited data on the diffusivity of the fine aggregate component indicate α is on the order of 2 to $9 * 10^{-7}$ m^2/sec.

Thermal properties of asphaltic concrete reported in the literature do not indicate the mineralogy, grain-size distribution, or density of the aggregate used in the mix. Nor has the fraction of asphalt or density of the mix been reported. Certainly these factors can be expected to influence the resulting thermal properties of asphaltic concrete and thus the transfer of heat from a field heater through an asphaltic concrete pavement.

DESCRIPTION OF EXPERIMENTS

Thermal Properties Required

The heat transfer process in a solid is described by the transient energy equation:

$$(\partial/\partial x)[k_x(\partial T/\partial x)] + (\partial/\partial y)[k_y(\partial T/\partial y)] + (\partial/\partial z)[k_z(\partial T/\partial z)] = \rho c(\partial T/\partial t) \quad (1)$$

where

x,y,z = spatial coordinates (m),
T = temperature (°C),
k = thermal conductivity (w/m°C),
ρ = density (Kg/m^3),
c = specific heat (J/Kg°C), and
t = time (sec).

Because the dimensions of field heaters are much greater than the thickness of a typical asphaltic concrete pavement, the heat transfer can be assumed to be one dimensional. Therefore, with the z dimension being vertical, $\partial T/\partial x \cong \partial T/\partial y \cong 0$, and Equation 1 becomes

$$k_z(\partial^2 T/\partial z^2) = \rho c(\partial T/\partial t) \quad (2)$$

Letting $\alpha = k/\rho c$ [diffusivity (m^2/sec)] and $k_z = k$, Equation 2 reduces to the familiar equation:

$$\alpha(\partial^2 T/\partial z^2) = \partial T/\partial t \quad (3)$$

Equation 3 can be written in finite difference form so that the temporal spatial-temperature field can be calculated for a pavement system that has

TABLE 1 Some Published Thermal Properties of Asphaltic Concrete

k(w/m°C)	α(m^2/sec)	ρc(J/m^3°C)[a]	Remarks	Reference
1.454		$1.41 * 10^6$		*16*
2.88	$14.4 * 10^{-7}$	$2.00 * 10^6$	18°C, dry	*17*
2.28	$11.5 * 10^{-7}$	$1.97 * 10^6$	38°C, dry	*17*
1.21	$5.75 * 10^{-7}$		Obtained from the Asphalt Institute	*13*
0.74-0.76			20°-56°C	*18*
0.167-0.172			Pure asphalt 20°-80°C	*19*
0.65-0.75			Asphalt used in street paving	*19*
1.37-1.75	$7\text{-}8 * 10^{-7}$	c = 879-963		*14*
0.80	$10 * 10^{-7}$			*20*
		c = 879	100°C	*19*
1.21		c = 920		*3*
1.2	$5.9 * 10^{-7}$	$2.07 * 10^6$ c = 921	80°-149°C	*9,11*
1.5				*2*
0.85-2.32				*21*
0.14-0.17		c = 1582-2561	Asphaltic bitumen free of paraffin wax 0°-300°C	*22*

[a] c in J/Kg°C

TABLE 2 Some Published Thermal Properties of Asphaltic Concrete Aggregate

Material	$k(w/m^\circ C)$	$\alpha(m^2/sec)$	$c(J/Kg^\circ C)$	Reference
Calcite				*23*
0°C			790	
200°C			1000	
Dolomite 60°C			930	*23*
Quartz				*23*
0°C			698	
200°C			969	
Limestone 58°C			1000	*23*
Limestone, mean of 3 at 50°C			680	*23*
Limestone, mean of 10 at 65°C			830	*23*
Quartzite				*23*
0°C			700	
200°C			970	
Granite				*23*
0°C			650	
200°C			950	
Basalt				*23*
0°C			850	
200°C			1040	
Sandstone 59°C			930	*23*
Diabase				*23*
0°C			700	
200°C			870	
Slate				*23*
0°C			710	
200°C			1000	
Avg value for sand and gravel	1.82			*24*
Calcite 100°C	2.86			*25*
Quartz 100°C	6.45			*25*
Granite 100°C	2.37			*25*
Basalt	1.8-2.2			*25*
Compact limestone	2.0-3.4			*25*
Porous limestone	1.1-2.2			*25*
Slate 100°C	1.8			*25*
Dolomite 100°C	3.99			*25*
Quartzite 100°C	5.2			*25*
Granite gneiss	1.8-2.8			*25*
Granite schist	2.7			*25*
Hard sandstone	2.6-4.5			*25*
Diabase 100°C	2.1			*25*
Quartz sand	1.1	$2 * 10^{-7}$		*25*
Sandy soil		$9 * 10^{-7}$		*25*

several layers, provided that the thermal properties ρ, c, and k (or α) are known for each layer. The mathematics and computer coding for predicting the temperature profile with time for a layered asphaltic concrete pavement system for a known heat input is routine, provided that the asphaltic concrete thermal properties are known.

Measurement of Properties

Determining the bulk density (ρ) of asphaltic concrete is routine. Typically, the as-placed density of asphaltic concrete is on the order of 2250 Kg/m³. Aggregate mineralogy and gradation, compaction effort, compaction temperature, and asphalt content will affect the density.

Numerous methods for measuring thermal properties have been proposed over the years. Most are for measuring a single thermal property at a time and usually use closed-form solutions for steady-state and transient cases. The method described here has some special advantages.

Beck and Al-Araji (15) developed a method of thermal property measurement capable of measuring the thermal conductivity, thermal diffusivity, and specific heat in a single test. The method requires the integration of some thermocouple signals, which can be accomplished by using available integrated circuits. The test procedure is quick and simple and requires minimal equipment.

To begin the test, a calorimeter (Figure 3) and a disk of asphaltic concrete (Figure 4) are placed in separate holders and insulated all around except for one face. The copper disk of the calorimeter is then heated to a uniform, elevated temperature (Figure 5) and brought into intimate contact with the asphaltic concrete sample (Figure 4). The differences in readings from thermocouples placed on the top (T_t) and bottom (T_b) of the sample are recorded at each time increment dt.

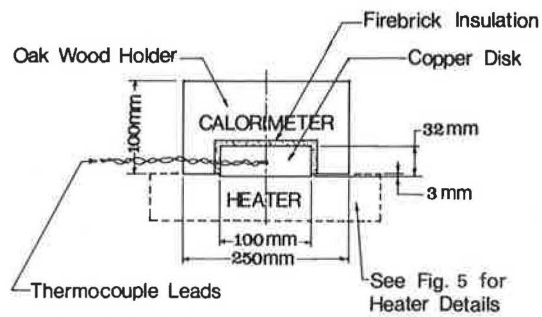

FIGURE 3 Calorimeter.

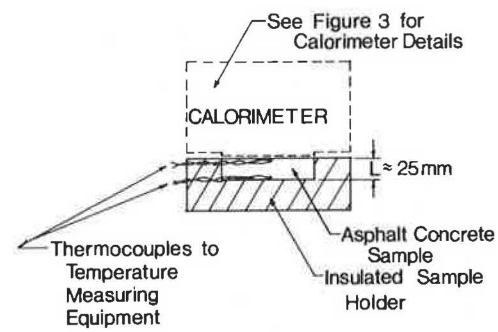

FIGURE 4 Calorimeter in contact with sample after being heated.

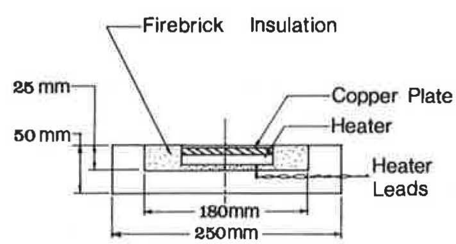

FIGURE 5 Calorimeter heater.

It has been found that, for a 30-mm-thick copper disk heated to a temperature of about 140°C and brought into contact with an asphaltic concrete disk about 25 mm thick at room temperature, the time required for T_b to equal T_t is about 30 min.

Beck and Al-Araji (15) showed that the thermal properties could be calculated from the following equations:

$$k = LQ/\left[2\int_0^\infty (T_t - T_b)dt\right] \tag{4}$$

$$\alpha = L^2(T_f - T_i)/\left[2\int_0^\infty (T_t - T_b)dt\right] \tag{5}$$

and

$$c = Q/[\rho L(T_f - T_i)] \quad (6)$$

where

Q = heat transferred from the copper disk to the asphaltic concrete sample (see Equation 7),
L = thickness of the asphaltic concrete disk,
T_f = final temperature of the asphaltic concrete sample (and the copper disk) when $T_t = T_b$,
T_i = initial temperature of the asphaltic concrete sample, and
ρ = previously measured density of the asphalt.

The integral in the denominators of Equations 4 and 5 are approximated by summing the temperature difference for each dt increment of time (dt = 15 sec was found to be appropriate) over the typical 30-min duration of the test and multiplying by dt. Because the surfaces are insulated, all the heat available in the copper disk is transferred to the asphaltic concrete sample, at which time both disks reach the same temperature. The heat-transferred Q can then be calculated from

$$Q = \rho_c L_c c_c (T_i - T_f)_c \quad (7)$$

where the subscript c refers to the copper disk.

Temperature Measuring System

The thermal property measuring device discussed in the previous section requires that temperatures be measured. Thermocouples were used to measure the temperatures. The TEMPSENSE Temperature Monitoring System (Interactive Microware, Inc.) in conjunction with an Apple II+ computer was used to calibrate the thermocouples and to select the frequency of sampling and the total duration of the test. The time of each measurement was displayed on the screen along with the temperature measured. The data were automatically stored on a disk for later analysis.

PRESENTATION AND DISCUSSION OF RESULTS

Asphaltic concrete samples were compacted as specified for the Marshall method of mix design (ASTM D 1559). Four different aggregate gradations were used: a base course, a dense-graded surface course, and an open-graded surface course, all using limestone aggregate; and a dense-graded surface course using lightweight aggregate (expanded shale). Samples of each mix were prepared at asphalt contents of 3.5, 5, and 6.5 percent. The 10.2-cm-diameter samples were cut into disks about 2.5 cm thick. Thermal properties of these disks were then obtained by the procedure described previously.

The particle-size distributions of the four asphaltic concrete mixes are shown in Figure 6. The base course and dense-graded surface course mixes were similar, except that the base course mix had particle sizes up to 38 mm, whereas the largest particles present in the dense-graded surface course were 12 mm. The open-graded surface course mix had the same range of particle sizes as the dense-graded surface course mix, but as shown in Figure 6 aggregate size is much more uniform.

The density-asphalt content relationship for each of the mixes is shown in Figure 7. Each point on the curves represents the average of the results of either four or five samples. Thermal properties of each mix were obtained from the disk samples, and

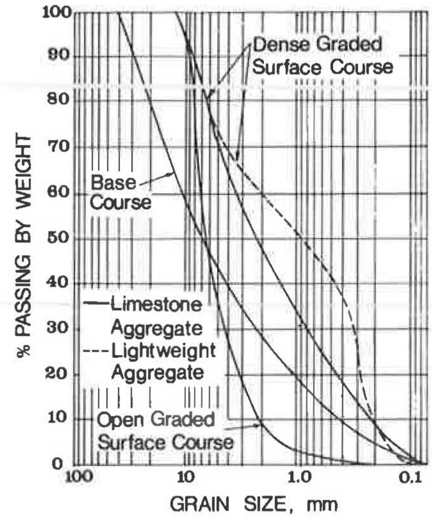

FIGURE 6 Grain-size distributions for the four aggregate mixes.

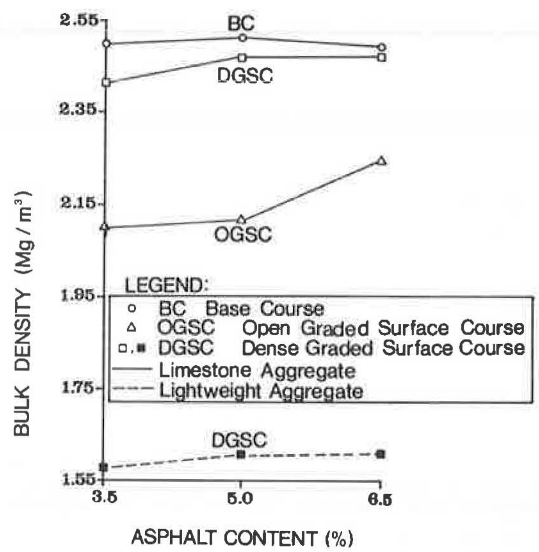

FIGURE 7 Bulk density as a function of asphalt content for the four aggregate mixes.

the results for thermal conductivity (k), specific heat (c), and diffusivity (α) are shown in Figures 8, 9, and 10, respectively. A comparison of Figures 7 and 8 indicates that thermal conductivity for the four mixes does not vary systematically with density or asphalt content.

For the dense-graded surface course mix with limestone aggregate, the thermal conductivity increased as the asphalt content increased from 3.5 to 5 percent. That the density of the mix increased as well (Figure 7) indicates that asphalt was replacing air in the mix voids. Because the thermal conductivity of asphalt is greater than air, the conductivity increased slightly (by about 4 percent). As the asphalt content of the mix was increased from 5 to 6.5 percent, the density remained essentially constant. This indicates that, in this range of asphalt content for this mix, asphalt had filled the voids and some of the mineral aggregate itself was being replaced by asphalt. Because the conductivity of asphalt is less than that of limestone (Tables 1 and 2), the thermal conductivity of the mix decreased, as shown in Figure 8.

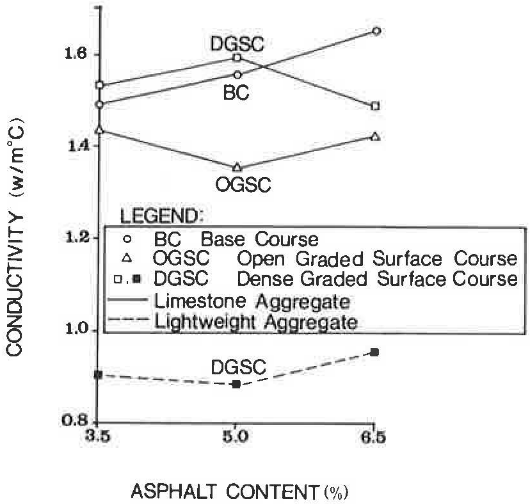

FIGURE 8 Thermal conductivity as a function of asphalt content for the four aggregate mixes.

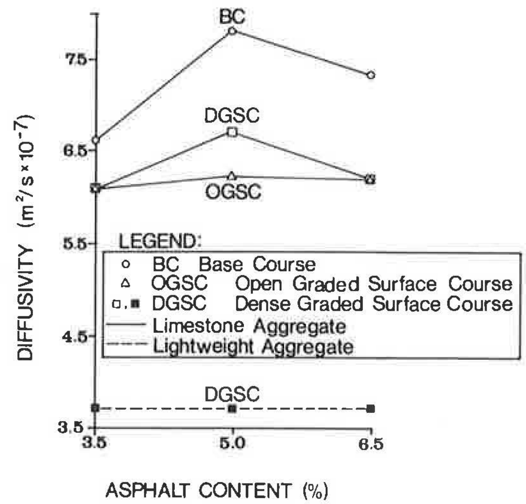

FIGURE 10 Thermal diffusivity as a function of asphalt content for the four aggregate mixes.

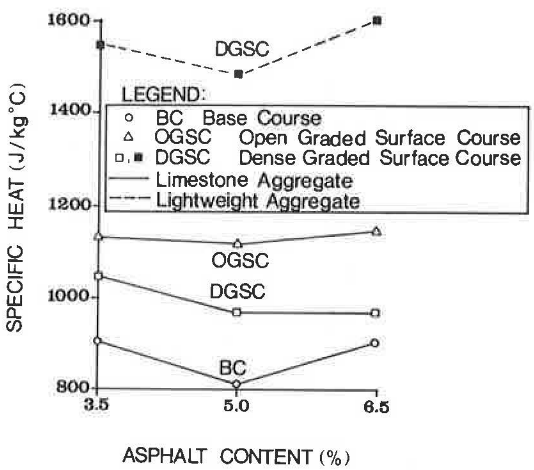

FIGURE 9 Specific heat as a function of asphalt content for the four aggregate mixes.

The thermal conductivity of the base course mix increased with asphalt content over the entire range of asphalt content (3.5 to 6.5 percent) at which samples were tested. This behavior was observed despite the fact that the density of the base course mix changed little with asphalt content. It is believed that this occurred because most of the heat transferred through this mix was through the larger aggregate (up to 38 mm), and that after the mineral aggregate voids were filled, the asphalt thickness around the larger particles was not increased substantially. The continuous increase in k with asphalt content may suggest a rearrangement of the mineral structure such that, although the mean distance between particles has increased (as suggested by a slight decrease in density), the nearest proximity points of individual aggregates may have decreased.

The open-graded surface course mix showed little change in conductivity with changing asphalt content. There are two opposing effects of adding asphalt to a uniform aggregate size. The first, as mentioned previously, is to replace air in the mineral aggregate voids with asphalt. This would tend to increase the conductivity of the mix. Working counter to this is the effect of increasing the asphalt coating around the particles as the asphalt content is increased. This would reduce the conductivity of the mix. The increase in asphalt coating thickness as asphalt content increases would be greater on the open-graded surface course mix than on the other two limestone aggregate mixes studied because of the smaller specific surface of the open-graded mix.

The thermal conductivity of the dense-graded surface course mix with the lightweight aggregate was much less than that of the mixes that have limestone aggregates. This mix consisted (by weight) of 25 percent expanded shade (from 5 to 13 mm in size), 30 percent limestone aggregate and filler, and 45 percent river sand (quartz). The expanded shale controlled the conductivity of the mix and the asphalt content had little effect on the conductivity (Figure 8). It is postulated that the air trapped in the expanded shale is an important factor controlling the conductivity of this mix.

Because the specific heat of asphalt is much higher than air or limestone and shale (Tables 1 and 2), it was expected that, as the asphalt content of the mixes was increased from 3.5 to 6.5 percent, there would be a corresponding increase in the specific heat of the mixes. However, Figure 9 shows that the specific heat of the mixtures changed little with asphalt content and that specific heat of these mixes is more dependent on the aggregate gradation and mineralogy than the asphalt content. Based on these results, it appears that an average value of c for each mix could be used; the actual number used would depend on the mix. The average specific heat of the lightweight aggregate mixes was much higher than the limestone aggregate mixes. This is primarily due to the decrease in density of the expanded shale. The products ρc of the lightweight mixes and the surface course limestone mixes are nearly the same ($2.5 * 10^{-6}$ J/m³°C). This product for the base course mix is about 15 percent less.

The diffusivity (α) of the limestone aggregate mixes is shown as it varies with asphalt content in Figure 10. For the open-graded surface course mix, α is essentially independent of asphalt content; for the base course and dense-graded surface course mixes, the diffusivity varied about 10 percent over the range of asphalt contents used in the mixes. The values shown in Figure 10 for the limestone aggregate mixes are well within the range reported by other researchers (Table 1). The diffusivity of the dense-graded surface course that has lightweight aggregate was much less than that of the limestone aggregate mixes and was found to be independent of asphalt content. Because the product ρc for the lightweight mixes is similar to that of the limestone mixes, the reduction in α is due to the smaller thermal conductivity of the lightweight mixes (Figure 8).

SUMMARY AND CONCLUSIONS

The conductivity (k) of the three limestone mixes of asphaltic concrete was found to vary as much as 20 percent over the range of asphalt contents (3.5 to 6.5 percent) used in this study. Two opposing mechanisms come into play that influence the thermal conductivity of an asphaltic concrete mix as the asphalt content is increased. On the one hand, the replacement of air in the voids in the mix by asphalt tends to increase the conductivity of asphaltic concrete because the conductivity of asphalt is much higher than that of air; on the other hand, the additional asphalt increases the thickness of the coating around aggregates, which tends to decrease the conductivity of the mix because the conductivity of asphalt is much less than that of the aggregate. The dominant mechanism depends on the asphalt content and the mix properties, such as the largest particle sizes present, the gradation, and the specific surface.

The conductivity of the lightweight aggregate mix was about 60 percent of that of similarly graded limestone aggregate mixes and varied little with asphalt content. It is believed that the conductivity of the lightweight mixes is controlled by air trapped in the expanded shale.

The specific heat (c) of the four mixes did not increase systematically with asphalt content, as was expected. The specific heat of the lightweight aggregate mixes was about 60 percent higher than that of the limestone aggregate mixes primarily because of the decrease in bulk density. The products ρc for similarly graded lightweight and limestone aggregate mixes were found to be similar.

The diffusivity (α) of each of the three limestone mixes varied less than 10 percent with asphalt content. The open- and dense-graded surface course mixes had similar diffusivities--the average was about $6.2 * 10^{-7}$ m^2/sec. The average α of the base course mix was about $7.3 * 10^{-7}$ m^2/sec. The diffusivity of the asphaltic concrete mixes that had lightweight aggregate was found to be essentially independent of asphalt content. The reduction in α from limestone to lightweight aggregate dense-graded surface course mixes was nearly the same in magnitude as the corresponding reduction in thermal conductivity.

Based on the results of this study, it appears that in analyzing heat transfer in asphaltic concrete having limestone aggregate, an average value of specific heat and an average value of diffusivity can be used that are independent of asphalt content but depend on the aggregate gradation. The conductivity used in analysis must reflect the asphalt content as well as the gradation of the aggregate. It appears that average values of diffusivity and specific heat can be used for lightweight aggregate mixes similar to that investigated, and it appears that little accuracy would be lost if an average value of conductivity, independent of asphalt content, were used.

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Evaluating Moisture Susceptibility of Asphalt Mixtures Using the Texas Boiling Test

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ABSTRACT

A description of the development and use of the Texas boiling test to evaluate stripping of materials susceptible to moisture damage is presented. Based on a review and comparison of boiling tests currently in use by several agencies and a limited test evaluation program, a tentative test procedure was prepared and used for all subsequent testing. Tests were performed on eight mixtures, of which five had stripped in the field and three had not. Each mixture and its individual aggregate components were tested to determine if the results could be used to differentiate between stripping and nonstripping mixtures. Because antistripping additives are commonly used in stripping-prone mixtures, a few additives and aggregate combinations were tested to determine if test results were affected by the presence of these additives. Test results indicate that valuable information is provided by the Texas boiling test. The test is simple and easy to perform; it can be performed either in the laboratory during mixture design or on field-mixed mixtures. Evaluation of known aggregates and various antistripping additives indicates that the Texas boiling test generally can be used to detect moisture-susceptible mixtures.

Water-induced damage of asphalt mixtures has produced serious distress, reduced performance, and increased maintenance for pavements in Texas as well as in other regions of the United States. Moisture-induced damage produces several forms of distress, including localized bleeding, rutting, shoving, and ultimately complete failure because of permanent deformations and cracking. This damage occurs because of stripping of asphalt from aggregate and in some cases possibly because of softening of the asphalt matrix.