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### **Author**

Pomerantz, M.

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## Durability and Visibility Benefits of Cooler Reflective Pavements

M. Pomerantz, H. Akbari, and J.T. Harvey

Environmental Energy  
Technologies Division

April 2000



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**Durability and Visibility Benefits of Cooler Reflective Pavements**

M. Pomerantz and H. Akbari

Heat Island Project  
Environmental Energy Technologies Division  
Ernest Orlando Lawrence Berkeley National Laboratory  
University of California  
Berkeley, CA 94720

And

J. T. Harvey  
Institute of Transportation Studies  
University of California, Berkeley,  
Richmond, CA 94804

April 2000

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## **Durability and Visibility Benefits of Cooler Reflective Pavements**

M. Pomerantz <sup>a</sup>, H. Akbari <sup>a</sup>, J. T. Harvey <sup>b</sup>

<sup>a</sup>Lawrence Berkeley National Laboratory, Berkeley, CA 94720

<sup>b</sup>Institute of Transportation Studies, UC Berkeley, Richmond, CA 94804-4603

### **Abstract**

City streets are usually paved with asphalt concrete because this material gives good service and is relatively inexpensive to construct and maintain. We show that making asphalt pavements cooler, by increasing their reflection of sunlight, may lead to longer lifetime of the pavement, lower initial costs of the asphalt binder, and savings on street lighting and signs. Excessive glare due to the whiter surface is not likely to be a problem.

*Key words:* pavements, asphalt, durability, illumination, visibility, surface treatment, albedo, reflectivity, cooler

# Durability and Visibility Benefits of Cooler Reflective Pavements

M. Pomerantz, H. Akbari, and J. T. Harvey

## 1. Introduction

Asphalt pavements are the most common form of street pavement in cities because they are relatively inexpensive to construct and maintain and the pavement gives good service. Asphalt concrete<sup>1</sup> (AC) is composed mostly of mineral aggregate, which gives it strength, and a binder of asphalt, to provide tensile strength, stiffness, and all-weather performance. The components are thoroughly mixed so that the aggregates will be coated and glued together by the asphalt. A newly constructed pavement thus has the color of the asphalt, which is quite black. As the surface asphalt is worn away by traffic and weather, the color of the aggregate is revealed. The color of asphalt itself lightens due to chemical reaction (oxidation), which increases with higher temperatures and sunlight.

The color of the pavement has several important environmental consequences. In hot cities, AC contributes to the heating of the air near the surface due to sunlight (Pomerantz *et al.* 1999). The dark color means that sunlight is not being reflected; the absorbed energy raises the temperature of the pavement and thus the temperature of the air that is in contact with it. This immediately contributes to the heating of the city. When the temperature gets high enough, the modern response is to turn on an air conditioner which further heats the outside air and requires energy. The atmosphere also reacts by using the thermal energy to drive the conversion of organic gases and nitrous oxides into smog. There is thus a cost in both energy consumed and degradation of the environment (Rosenfeld *et al.* 1998). These costs may be reduced by coating the pavement with a light-colored material (Pomerantz *et al.* 1997).

In addition to detrimental environmental effects, the heating of pavements may be harmful to the pavements themselves. The properties of asphalt binder are known to be temperature dependent. For example, the stiffness of asphalt decreases exponentially with temperature (Yang 1972). Likewise, the related property, viscosity, as measured by penetration of a sharp needle, decreases exponentially with temperature (Hunter 1994). The effects of pavement temperature on performance have been recognized by the recent Strategic Highway Research Program (SHRP)

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<sup>1</sup>In common usage, asphalt concrete is referred to as "asphalt" and Portland cement concrete is called "concrete." We use "concrete" in its technical sense of a composite of a binder (asphalt or Portland cement) with stony aggregate. Thus we refer to "asphalt concrete"; "asphalt" connotes only the binder.

which grades asphalt according to the temperature range *that the pavement itself* will endure (Cominsky *et al.* 1994). However, asphalt binders that function over wide temperature ranges are more costly. This opens possibilities for additional savings by constructing cooler pavements: by reducing the maximum pavement temperature, a lower grade of asphalt may be acceptable and/or some failure will be delayed. Ultimately, the lifecycle costs of maintenance and disposal of pavements will be reduced.

This paper will investigate these non-energy or non-environmentally related effects of cooler pavements, both the potential benefits and detriments. Such benefits, in addition to longer lifetime, may include better visibility. The danger of glare seems to be negligible for the suggested reflectivity. We also report some measurements on the relationship between the reflectivity of aggregates and chip seals made of them. The evidence suggests that cooler pavements may offer impressive benefits to society and thus warrant further study.

## 2. Effect of Pavement Temperature on Durability

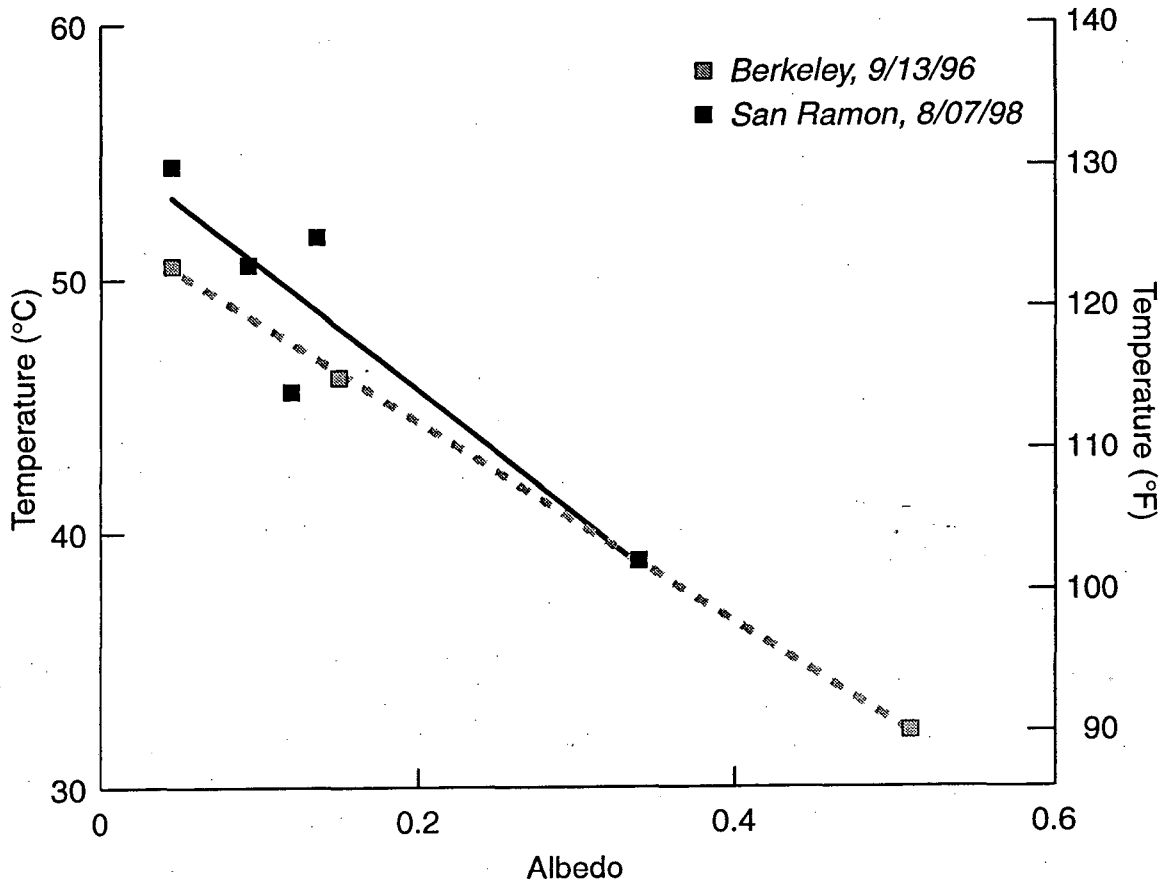
AC pavements fail by a variety of mechanisms, some of which are temperature dependent. Some failures might be delayed or eliminated if the pavement temperature were decreased by making it more reflective of sunlight. First, we establish the order of magnitude of the effect of the reflectivity of a pavement on its temperature. The reflectivity averaged over the solar spectrum is the albedo<sup>2</sup>,  $\hat{\alpha}$ . Measurements of albedo and temperature were made in Berkeley and San Ramon, CA (Pomerantz *et al.* 2000). In both places, in the afternoons when the solar energy fluxes were about 1000 W/m<sup>2</sup>, a decrease of about 4°C (7°F) was observed for an increase of albedo of 0.1. (Figure 1) (A change in albedo of 0.25 is the difference between fresh black AC, with  $\hat{\alpha} \approx 0.05$ , and Portland cement concrete, with  $\hat{\alpha} \approx 0.3$ .)

A theory of maximum pavement temperature versus albedo predicts a decrease in temperature of 3.6°C (6.5°F) for a 0.1 increase in  $\hat{\alpha}$ , for conditions of insolation, time, and low wind-speed roughly similar to the measurements (Solaimanian *et al.* 1993). Their result is in reasonable agreement with the data in Figure 1. Other calculations more specific to the conditions of the experiments give similar results (Pomerantz *et al.* 1999).

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<sup>2</sup>Albedo here is the fraction of the incident solar energy reflected by a surface, averaged over the solar spectrum. Perfect reflectors have  $\hat{\alpha} = 1$  while perfect absorbers have  $\hat{\alpha} = 0$ . For opaque materials, absorptivity is  $(1 - \hat{\alpha})$ .



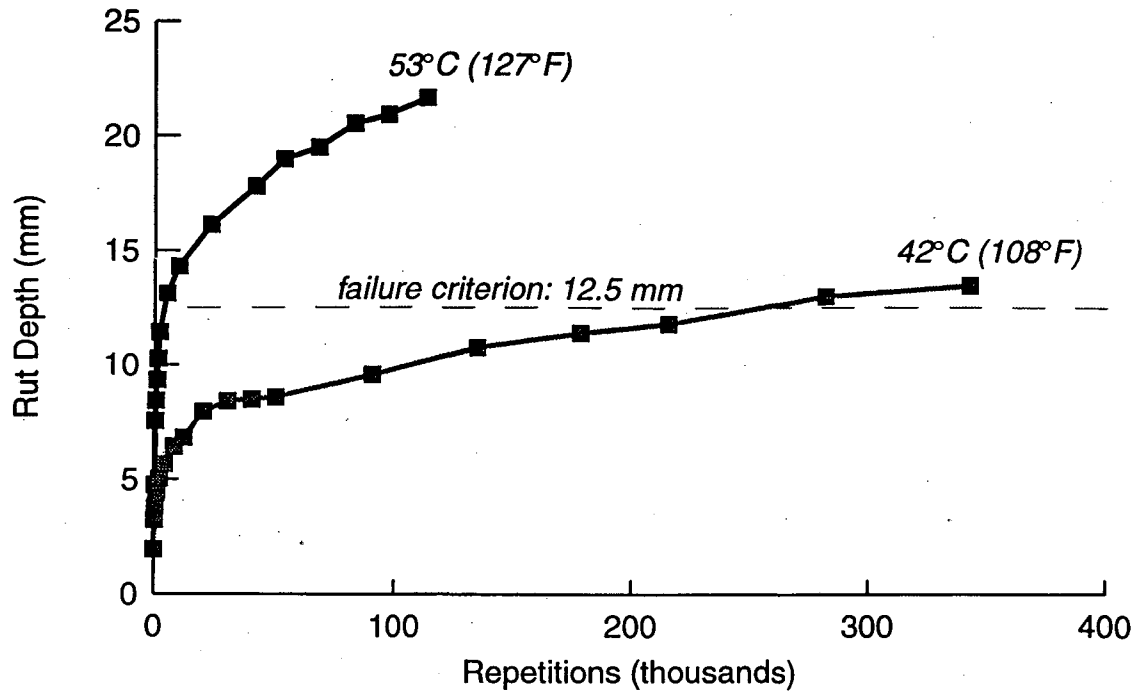


**Figure 1.** Example of the dependences of pavement surface temperatures on albedo. Data were taken at about 3 PM in Berkeley, CA, on new, old, and light-color coated asphalt pavements. The data from San Ramon, CA, were taken at about 3 PM on four asphalt concrete and one Portland cement concrete ( $\hat{\alpha} = 0.35$ ) pavements.

Thus it may be possible to reduce the peak pavement temperatures by about 8°C by increasing the albedo by a practical amount of about 0.2. The pavement temperature may affect the rate of pavement failures. There are several distress mechanisms of AC that are likely to be influenced by pavement temperature (Yoder *et al.* 1975) including:

- rutting: tires cause channel-like depressions in the pavement
- shoving: the AC is pushed in the direction of tire motion
- aging: asphalt becomes brittle and stiffer with age
- fatigue damage: gradual cracking of pavement
- bleeding: asphalt binder accumulates at the surface

It is well known (Croney *et al.* 1998) that the stiffness of asphalt depends strongly on temperature. The exponential dependence of viscosity on temperature results in about an order of



**Figure 2.** Depth of rutting vs. number of repetitions of a standard axle load, wide-base single tire, at pavement surface temperatures of 42°C and 53°C.

magnitude decrease in viscosity for a 10°C increase in temperature. The stiffness of AC also decreases exponentially as its temperature increases: a 10°C increase in temperature can halve the stiffness of AC. Stiffness is thought to be an indicator of pavement resistance to rutting and fatigue, which would suggest that the lifetime of the pavement might increase if the temperature of the pavement were lowered (Pomerantz *et al.* 1998).

Aging, the third on the list, is also believed to involve chemical and physical reactions that are speeded by higher temperatures. The following is some evidence on the effects of high pavement temperature on specific failure mechanisms.

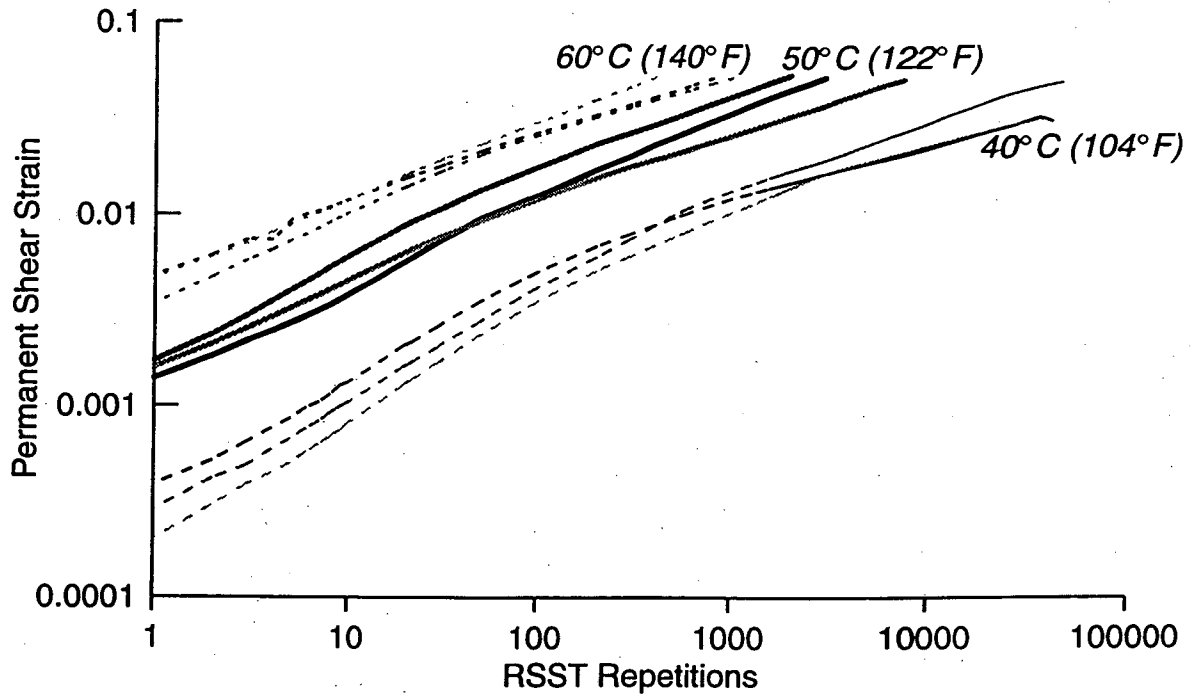
*a) Rutting as affected by pavement temperature.*

**Figure 2** shows the results of rutting experiments made with the Heavy Vehicle Simulator (HVS) at the Institute of Transportation Studies (ITS) of the University of California, Berkeley. A standard single-axle load with a wide-base single tire was repetitively driven at a speed of 7 km/hr without wander. The pavements, dense-graded AC, were maintained at various temperatures by heated ambient air and infrared lamps. The temperatures were measured by thermocouples embedded at various depths. The rut depth was measured from the top of the

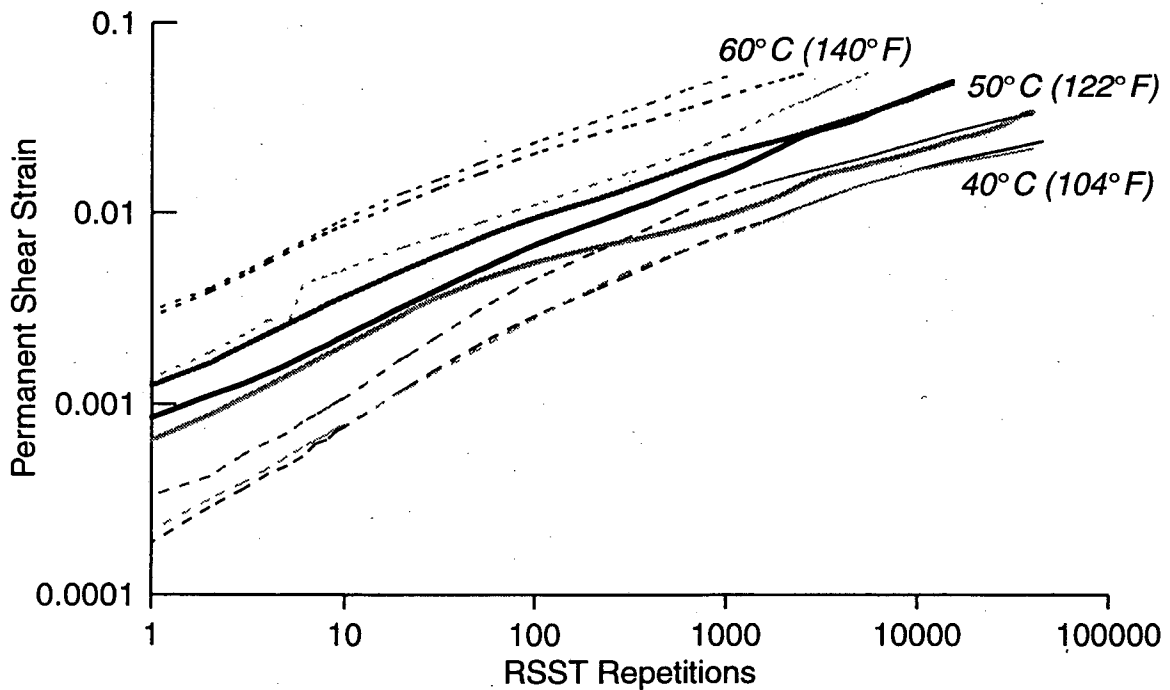
extruded material to the bottom of the rut. There is a striking increase in the ability of the road to resist rutting as the temperature was decreased. At a surface temperature of 53°C (127°F) the rut depth exceeded the failure criterion (12.5 mm ≈ 0.5 inch) in fewer than 20,000 repetitions. By lowering the temperature by about 10°C to 42°C (108°F) the road did not reach failure until about 270,000 cycles, a more than ten-fold increase in pavement life.

An experiment on the effect of pavement temperature on rutting was conducted on a busy highway in Paris, France (Loustalot *et al.* 1995). Three areas of pavement were resurfaced with different colored wearing surfaces: a normal black binder with gray aggregates (NN), a black binder with clear quartzite aggregates (NQ), and a special clear binder (with 1% white titanium oxide) with clear quartzite aggregates (CQ). Temperatures of similar pavements were measured for three summers representing normal, hot, and exceptionally hot weather. The maximum temperature of the NN pavement exceeded that of the CQ pavement (at 1 cm below the surface) by as much as 5°C for the normal summer. For the exceptionally hot summer conditions, they estimate the temperature difference might be 8°C. Their calculation of the expected rut depths of the three different pavements predicts that the hottest pavement will have about a 1-mm deeper rut than the cool pavement. (Unfortunately, the report does not make clear how much and kinds of traffic, the time of traffic, or how many years were assumed. Nor does the report specify the albedos of the pavements.) Side-by-side comparisons of this type, but with more details, are needed.

Another form of rutting is shoving, where the tires apply large forces in the direction of motion during braking. There are recent data from the ITS in Berkeley on the effect of temperature on the permanent distortion under simple shear stress as a function of temperature. In these repetitive simple shear tests (RSST), disks of AC were subjected to pulsed shear stresses,  $S$ , in the form of a haversine in time,  $S = S_0 \cdot (1 - \cos 2\pi t f)/2$ . The inverse of the time (0.1 sec) during which the stress is applied is denoted by  $f$ , i.e.  $f = 10/\text{sec}$ . Each pulse of stress is followed by 0.6 sec of recovery time. The repeated application of this uni-directional shear stress is similar to the pushing by tires that happens most strongly during stopping and starting. As shown in **Figure 3** for an amplitude,  $S_0 = 84 \text{ kPa}$  (12 psi), the number of repetitions required to produce a permanent strain of 0.01 was increased hundred-fold by reducing the temperature by 20°C, or an average of about an order of magnitude per 10°C reduction in pavement temperature. A similar effect is observed at lower stress  $S_0 = 56 \text{ kPa}$  (8 psi) (**Figure 4**). Reducing a pavement's temperature in this temperature range evidently enhances its resistance to failure by shoving.



**Figure 3.** The effect of pavement temperature on the permanent shear distortion caused by repeated simple shear test (RSST) with a peak shear stress of  $S_0 = 84$  kPa.



**Figure 4.** The effect of pavement temperature on the permanent shear distortion caused by repeated shear stress tests (RSST) with a peak shear stress of  $S_0 = 56$  kPa. The dotted curves are for 60°C, the solid curves for 50°C, and the dashed curves for 40°C.

The data indicate that pavements at temperatures higher than about 40°C tend to rut faster. Reduction of temperature gradients will result in less fatigue damage (McAuliffe *et al.* 1995). Relevant questions are how much of the time are roads at these temperatures and how much traffic is present at those times?

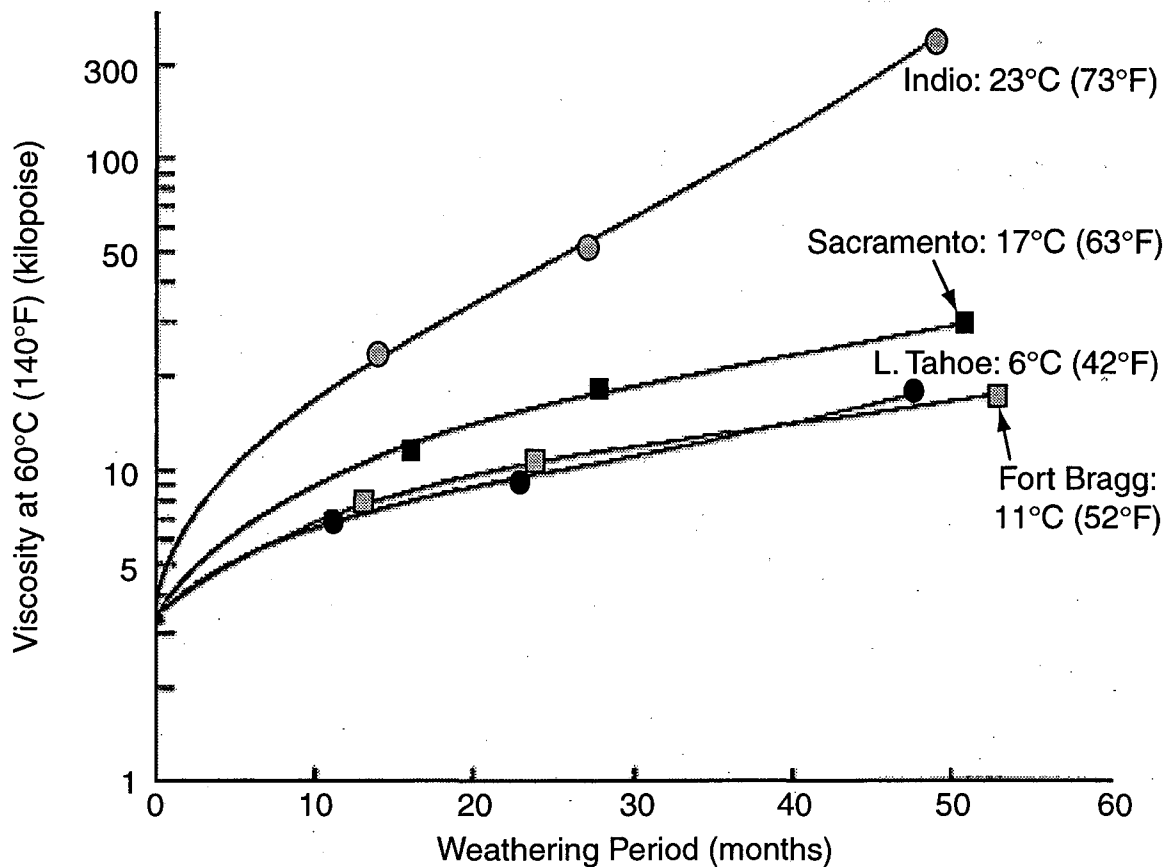
Pavement temperatures will depend on the climatic conditions.<sup>3</sup> A variety of pavements were measured in the eastern Bay Area of San Francisco (Pomerantz *et al.* 2000); afternoon temperatures in the summer ranged from 49°C (120°F) to 65°C (150°F); 54°C (130°F) was about the average peak. Roads in sunny, southerly latitudes are regularly hotter than 50°C. The theory of Solaimanian and Kennedy (Solaimanian *et al.* 1993), which agrees with our measurements, predicts that the maximum pavement temperature at lower latitudes will exceed the maximum air temperatures by about 25°C (40°F). This is confirmed by data (Dempsey *et al.* 1995) for Reno, NV, which shows a difference of 23°C (41°F) between maximum pavement surface temperatures and maximum air temperatures on sunny days during the summer of 1991. In southern regions, where air temperatures often reach 35°C (95°F), maximum surface temperatures of 60°C will be common. Measurements (Asaeda *et al.* 1995; Pomerantz *et al.* 2000) have shown that the pavement surface temperature peaks between one and two hours after the solar noon and then gradually falls<sup>4</sup>. Roads will be hotter than 50°C for a considerable part of the afternoons. At these times, when the roads may be susceptible to damage, there tends to be heavy commuter traffic.

It is desirable to have additional direct measurements of test sections of actual roads with different albedos because theories tend to neglect the effects of vehicles on the road temperatures. Vehicles will heat the roads with their tires as well as shading them and by stirring the air. Also, in real traffic the tires wander on the road. In the rutting and RSST experiments described above, the strains stayed in a single track.

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<sup>3</sup>Some data (Yang 1972) report that at Newark (NJ) Airport the highest pavement temperature reached in August was about 38°C (100°F). These temperatures seem too low, since we have measured pavement temperatures of about 49°C (120°F) in Berkeley in September (see Figure 1).

<sup>4</sup>In related experiments, temperatures of roofs were monitored continuously in sunny parts of the San Francisco Bay Area in Northern California. From June to September, dark roofs were hotter than 49°C (120°F) more than 20% of the time (Konopacki *et al.* 1997, Figures 3.1, 3.2, and 3.33), and reached at least 49°C on about 250 days of the year. Factors that must be considered in comparing roofs with roads are that the thermal conduction from the hot surface to cooler regions below are different and that there are cars passing on roads which stir the air and increase convective cooling of roads (Berg *et al.* 1983).



**Figure 5.** Effect of air temperature on hardening of asphalt as a function of time (from Kemp and Predoehl, 1981)

b) *Aging as affected by pavement temperature.*

Another cause of pavement failure is that, as it ages, the asphalt becomes stiffer and more brittle. Measurements on asphalt extracted from test sections of pavements (Page *et al.* 1985) showed that the viscosity increased with age. This “hardening” might be thought to enhance lifetime since “stiffness” is believed to be beneficial for thick pavements. Stiffening, however, is undesirable for thinner pavements where flexibility prevents cracking. Embrittlement leads to cracking in sudden, single events. The cause is some oxidation, loss of volatile hydrocarbons, and polymerization. It has been observed that the embrittlement increases with temperature and with the intensity of ultraviolet light (Kumar *et al.* 1977); the oxidation rate doubled for every increase of 10°C (Dickinson 1980).

Tests in various climates in California showed that desert conditions lead to relatively rapid decreases in ductility, as well as increased viscosity (described as “hardening”) (Kemp *et al.*

1981). **Figure 5** shows the dramatic effect of weathering in a hot and sunny desert climate. The average viscosity of several asphalts exposed to a desert climate with an annual average air temperature of 23°C (73°F) for about four years is ten times higher than when the average temperature was 17°C (63°F). The dependence on temperature seems to be non-linear; the hardening rate accelerates when the average air temperature exceeds about 13°C (55°F). In these studies, the embrittlement that contributes to road failure is assumed to be due to the same mechanism that increases the viscosity. Thus the process of embrittlement is likely to slow if the temperature of the pavement could be decreased. Kemp *et al.* describe their results in terms of air temperatures but they recognize that it is the asphalt temperature that is crucial and controllable. They conclude that the durability of asphalt can be improved by “the insulating of the asphalt concrete mat with a cover such as a reflective chip seal in hot areas.” A reflective seal has the benefits of both lowering the asphalt temperature and reducing the ultraviolet light damage.

The durability of roads against various modes of failure can thus be enhanced by preventing the pavement temperature from becoming too high.

### **3. Discussion of Cost and Benefits of Increased Durability of AC**

The evidence cited above indicates the importance of the pavement temperature in determining the lifetime for several failure mechanisms. There remains the question of whether the cost of cooling the road is less than the lifetime cost-savings of such a road. To analyze completely the lifecycle cost requires a comprehensive comparison of various construction, maintenance and disposal options. This is beyond the scope of the present paper. Such an analysis is currently underway (Ting, Koomey & Pomerantz 2000). As a simple example, we here compare higher-grade binders with a cooler surface constructed by a chip seal.

Durability is a prime concern for pavements. Rutting failure occurs within the first five years after construction, after which aging stiffens the asphalt, which slows rutting of thick pavements. Thereafter, fatigue becomes a more likely risk. It is important to prevent rutting, however, because its repair is a safety issue and requires the removal of the AC and repaving. This type of repair is very expensive. The traditional approach to achieving greater pavement durability is to use higher-grade binders. An example is the recommendation of the Strategic Highway Research Program (SHRP) (Cominsky *et al.* 1994). Beginning in 1987, this program brought pavement experts to the task of researching and then recommending the best methods of constructing AC pavements. A result of this study is specifications of the asphalt binder. The specification form, reproduced in **Fig. 6** from a SHRP report (Cominsky *et al.* 1994), shows that a primary

consideration is the temperature range to which the pavement will be exposed. The solution suggested is to employ binders rated by their “performance grade” or PG.

PERFORMANCE GRADE	PG 46-			PG 52-						PG 58-					PG 64-					
	34	40	46	10	16	22	28	34	40	46	16	22	28	34	40	10	16	22	28	34
Average 7-day Maximum Pavement Design Temperature, °C	<46			<52						<58					<64					
Minimum Pavement Design Temperature, °C	>-34	>-40	>-46	>-10	>-16	>-22	>-28	>-34	>-40	>-46	>-16	>-22	>-28	>-34	>-40	>-10	>-16	>-22	>-28	>-34
ORIGINAL BINDER																				
Flash Point Temp, T44: Minimum °C	230																			
Viscosity, ASTM D4402 <sup>a</sup> Maximum, J Pa·s, Test Temp, °C	135																			

Figure 6. Copy of part of SHRP binder specification form.

The PG is specified by two extremes of temperature the pavement may experience: the average 7-day *maximum* temperature and the mean 1-day *minimum* temperature. (Note, importantly and logically, that the *pavement* temperatures and not the *air* temperatures are considered.) There is a rule of thumb in the industry—the “Rule of 90”—that when the difference between these extreme temperatures is greater than 90°C some kind of modified asphalt will be needed; this adds to the cost. For example, if a binder is specified as PG 58-22, it is intended to function between 58°C and -22°C. The difference,  $58 - (-22) = 80$ . An ordinary grade of asphalt binder, at a cost of about \$125 per ton, will suffice. If, however, the pavement must function between 76°C and -16°C, or PG 76-16, the difference  $76 - (-16) = 92$ , and the price of such a binder is about \$165 per ton (Bally 1998), a 30% increase in price. The Rule of 90 arises because ordinary asphalt has difficulty in performing over wide temperature ranges. Additives, such as polymers, are needed to attain performance over a wide range. Unmodified asphalts that are suitable for high maximum temperatures regardless of the range (such as any PG 70) are scarce, less than 2% of available asphalts, and thus more expensive (Kennedy *et al.* 1994). The solution may be to lower and compress the temperature range that pavements will experience by decreasing the maximum temperature. This can be done directly by increasing the pavement reflectivity. As noted above, a change of 10°C in pavement temperature can be affected by an increase in  $\hat{\alpha}$  of about 0.25. This suggests that the pavement grade’s maximum temperature specification could be decreased by perhaps 10°C. For example, if a grade of PG 76-16 were previously recommended, now PG 70-16 would suffice. By the “Rule of 90,” the lower grade does not need polymer modification and consequently is cheaper.



Recommended AC mixes (Cominsky *et al.* 1994) are about 6% asphalt by weight, or about 16% by volume. The cost of ordinary asphalt (1998 prices) is about \$125 per ton, and the price of aggregate is about \$20 per ton, exclusive of transportation costs. Thus, in one ton of mixed AC the cost of materials only is about \$28/ton, of which about \$9 is in the binder and \$19 is in the aggregate. For a pavement about 10 cm thick (4"), with a density of 2.1, the cost of the binder alone is about \$2.00 per m<sup>2</sup>. If the higher-grade binder is used, at a surcharge of 30%, the extra cost is about \$0.60 per m<sup>2</sup>. Thus the cost of \$2.60 per m<sup>2</sup> for higher grade asphalt in a 10-cm pavement could be reduced to the usual \$2.00 per m<sup>2</sup>, for a savings of \$0.60 per m<sup>2</sup>. The economic question is: can one lower the surface temperature at a cost lower than this \$0.60/m<sup>2</sup>?

To decrease the surface temperature one need only cover the surface with a reflective layer. In the case of repairing a road, the well-known method of "chip seal" is a possibility. Onto the faulty pavement, a layer of asphalt emulsion is spread and, before it dries, a layer of aggregate of uniform size (typically about 3/8") is placed on top. The aggregate is pressed into the surface by rollers and later by the traffic over time. This technique is used for maintenance, since it increases the lifetime of the road, but it also gives a cooler road. Placing aggregate on the binder has heretofore been used as a repair technique, but we suggest that it could also be used as the final treatment of a new road. The additional cost of adding this final layer can be estimated from experience in applying chip seals. Typically, labor plus equipment cost about \$0.34/m<sup>2</sup> more for a chip seal than for a simple single spreading of emulsion (Means 1996), due to the extra costs of spreading, rolling, and sweeping the aggregate. Suppose, however, that a lower temperature were thereby achieved. A lower grade of asphalt might be used where the SHRP specifications call for a higher grade of asphalt. The savings of \$0.60/m<sup>2</sup> in binder cost could be applied against the additional costs of the equipment, labor, and materials, allowing \$0.26/m<sup>2</sup> to be spent toward the cost of the required 0.016 tons/m<sup>2</sup> of aggregate<sup>5</sup>. This allows for a cost of \$16/ton of extra aggregate. This is a typical cost of aggregate; white aggregate would likely cost more.

The question then arises, how white does the aggregate have to be? Test samples of chip seals were made by spreading aggregates onto an asphalt emulsion (Pomerantz *et al.* 2000). Using a "Solar Spectrum Reflectometer" purchased from Device and Services Corp., the albedos of the aggregates alone were measured. A chip seal was made using each of the aggregates, and its albedo was measured. The albedos of different aggregates ranged from 0.10 to 0.28; the observed chip seal albedos ranged from 0.08 to 0.20. On average the albedo of the chip seal was

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<sup>5</sup>The amount of aggregate needed can be estimated from the example of Chula Vista, CA, where chip seals on 10,500 m<sup>2</sup> of pavement required about 170 tons of aggregate or 0.016 ton/m<sup>2</sup> (Maruffo 1998).

about 70% of the albedo of the aggregate. Extrapolating this to higher albedos, a chip seal with the albedo of cement concrete (0.35) would require aggregate with albedo of 0.5. The cost of such a high- $\hat{a}$  aggregate depends on proximity to quarries from which such aggregate is available. The cost of transporting aggregate is about \$0.10 per ton-mi.

Savings might also accrue for resurfacing of pavements. Some engineers find that chip seals last longer than simple black slurry seals by about 25% (Donnelly 1998). However, the extra labor and equipment costs of a chip seal makes it about 20% more expensive than a slurry seal (Means 1996). Longer pavement life may compensate for the additional cost.

It should be mentioned that chip seals can have undesirable properties. In roads where tires are repeatedly turned, such as in cul de sacs, the aggregate tends to be dislodged. It can then be thrown by tires or tracked into homes. The practice is, thus, not to use chip seals in such locations (Donnelly 1998; Maruffo 1998). Construction of a chip seal requires the coordination of a precise three-step process and is thus is more demanding and a little more expensive than simple slurry seals. Part of the resistance to light-colored pavement is that it is often associated with being old or worn (Donnelly 1998). Sometimes carbon black is added to asphalt to darken it in order to make the surface appear newer. This attitude does not exist where chip seals are used extensively (Maruffo 1998). In some up-scale communities, black roads are disfavored because they look too much like urban surfaces. Such matters of taste may be modified by education in the practical and aesthetic advantages of lighter-colored pavements. Development of low-cost, highly-reflective surface treatments without the negative aspects of chip seals should be investigated.

#### **4. Effects of Reflective Pavements on Illumination**

If pavements are more reflective, illumination at night is enhanced by the light reflected off the pavement. Thus both traffic signs and pedestrians may be more visible. According to the International Commission on Illumination (CIE 1984), "In order to make asphalt pavements lighter, some countries (e.g. Denmark) stipulate the inclusion of a proportion of white stones in the bituminous concrete. In Belgium, the use of light-colored stones for chip sprinkling is obligatory on the major roads of the State network." The need for better lighting will become greater because of the aging of the population. Demographers estimate that, by the year 2020, in the United States over 50 million drivers will be at least 65 years old. As people age their night vision worsens and the risk of accidents increases (FHWA 1994). Statistically this older population has slower reaction times and has reduced visual acuity and depth perception.

Enhanced illumination due to reflective pavements may help to avoid accidents and reduce the costs of automobile insurance. Another consequence of the aging of drivers is that it becomes more important that traffic signs and their supports be larger and clearer, increasing their costs. In addition, better illumination probably reduces auto theft and other street crimes.

A quantitative estimate of the contribution of pavement reflectivity to the illumination of a subject is calculated for a particular geometry in the Appendix. It is shown there that for a subject placed on an illuminated area of a pavement, the ratio of the light reflected off the pavement,  $q_r$ , to the light arriving directly from the street light,  $q_d$ , is

$$q_r/q_d \approx \mathcal{R} \quad (1)$$

where  $\mathcal{R}$  is the reflectivity for the spectrum of visible light emitted by the lamp<sup>6</sup>. For a reflectivity of 0.1, the contribution of reflected light is about 10%; a reflectivity of 0.3 increases the reflected light to about 30% of the direct light. This offers the possibility of using fewer or less powerful street lamps, or leaving these unchanged and receiving greater illumination. Our estimate of about a 20% reduction in the strength of the light sources by changing reflectivity from 10% to 30% is similar to the results of a rather different calculation of the number of light fixtures required to achieve a desired level of illumination with different pavement reflectances (Stark 1986).

The actual visibility depends not only on illumination but also on contrast, which is not an issue here because it depends on the background which is uncontrolled.

Data taken inside a building with cement concrete floors show that an increase of reflectivity of 61% (from gray at 27% to white at 44%) increased in the illumination on a vertical plane of about 20% (Atlas 1942). The data are not complete enough for us to compare with Eq. 1.

Higher reflectivity does not imply unacceptable glare. The maximum albedos contemplated here are about 0.35, similar to cement concrete. Cement concrete roads are in widespread use around the world; the reader of this report has likely ridden on some. One does not hear that the users of such roads suffer from glare. It seems likely that AC pavements with such reflectivities will not cause problems from glare.

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<sup>6</sup>For illumination we are concerned with the reflection of only the visible light emitted by the luminaire. Thus, we use the letter  $\mathcal{R}$  to distinguish it from the albedo, which is the reflectivity over the solar spectrum.

## 5. Discussion of Future Research

Experience in the construction of AC roads, as embodied in the SHRP specifications (Figure 6), points to pavement temperature as a major controlling factor in durability of roads. Our own more recent data—Figures 2, 3, and 4—are indicative of a rapid decrease in the durability when the pavement temperature is higher than 40°C (104°F). Our data (Figure 1) and other literature show that the temperature of roads regularly exceed this temperature even in temperate climates. Theory indicates that the maximum pavement temperature will exceed the maximum air temperature in sunny southern latitudes by 25°C (45°F). We suggest that more durable roads can be built if the heating of the roads is reduced by making them more reflective of sunlight. The peak temperature of the pavement surface can be reduced by about 4°C (7°F) by increase in albedo of 0.1 (Figure 1). The cost/benefit of constructing cooler roads will depend on local conditions, such as proximity to light-colored aggregate and climate. Research is needed to quantify these conclusions in various localities.

A focus of future research should be on the effects of pavement temperature, in both laboratory and field studies. We hypothesize that the parameter that governs the damage to the roads is the product of the pavement temperature,  $T(t)$ , above some critical temperature  $T_c$ , the length of time above that temperature,  $t$ , and the amount of traffic at that time  $N(t)$ , integrated over time. In general, we expect that the dependences may not be linear, so we write a more general parameter with power law dependences:

$$\text{Damage} = K \int (T(t) - T_c)^a \cdot t^b \cdot N(t)^c dt$$

The constants  $K$ ,  $a$ ,  $b$ , and  $c$  must be determined from fitting to data. Such data would ideally be obtained from controlled experiments in which successive sections of limited access highways (such that traffic would be identical on all sections) would be constructed identically, except for surface albedos. Albedos could be varied by the use of suitable aggregates in chip seals. The traffic and the time of exposure to sunlight would be the same. Then the difference in distress could be attributed to the differences in pavement temperatures that result from differences in albedos. Such experiments could look for the effect of temperature on, for example, bleeding of asphalt. Bleeding, the movement of asphalt to the surface, makes a pavement more slippery and is hazardous. It is plausible that cooling the pavement, and thus increasing the viscosity of the asphalt, will reduce bleeding. Such experiments would require the cooperation of public works departments and the results would not be apparent for years. We have an alternative suggestion that may approach to the truth in a shorter time.

Evidence for the effects of pavement temperature on durability might be obtained by examining the maintenance records of heavily traveled commuter roads. We would examine roads in which the afternoon side of the road, when it is hotter, is used much more heavily than the morning side, when it is cooler. We predict that the afternoon rush-hour side would develop a greater need for maintenance than the morning rush-hour side. Such an “experiment” incorporates the real-life effects that moving cars produce: shade, stirred air, and heating by tires. It requires that the failure mechanisms of the morning and afternoon sides of the road be identified by a uniform method. Such record keeping, if it does not already exist, should be a high priority for those concerned with economizing on the huge budget for road construction and repair. With the help of computers and the Internet, such a data base could help discern what is successful and what is not and could save a great deal of money. A source of this information may be the Long Term Pavement Performance Program. Funded by the Intermodal Surface Transportation Efficiency Act, it is a program of the Federal Highway Administration. Begun in 1987, the project monitors the behavior of 2400 test sections of asphalt and cement concrete pavements in the US and Canada (Churilla 1998). Their data may contain the information we require.

Coolness and reflectivity of AC pavements may have other benefits, which should be investigated. These include:

- 1) Tires may run cooler and thus last longer if the pavements were cooler. It is plausible that rubber is removed from tires more rapidly when the tire is hotter. Tires are heated by friction with and conduction from hot pavement. Also heat is generated from the internal friction of the flexing of the tire. This heat is removed partly by contact with the pavement. The effect of pavement temperature on tire lifetime should be studied. The cost and disposal of tires might also be improved by cooler roads.

- 2) Improved skid resistance. Some measurements (Yang 1972, p. 161), indicate that the coefficient of friction of both rock asphalt and AC decrease as the pavement temperature increases. Thus skid resistance seems to improve if the pavement is cooler. This effect is worthy of study as a safety improvement.

## **6. Conclusions**

That high pavement temperatures lead to more rapid deterioration of roads is anticipated by civil engineers. The concept is embodied in the Superpave specifications for the choice of the grade of asphalt binder: one of the criteria for the grade of asphalt is the highest temperatures the

pavement is expected to endure. The experiments with the Heavy Vehicle Simulator reported here show quantitatively that at pavement temperatures exceeding 40°C the amount of rutting increases dramatically. Similarly, under simple shear stress, samples suffer larger permanent shear distortion when their temperatures are elevated. Temperatures greater than 50°C, at which the pavements degrade more rapidly, are known to occur in actual roads even in temperate climates. The traditional means to strengthen pavements is to use a modified or high grade asphalt binder. An alternative is to make the pavement cooler by reflecting sunlight before it is absorbed. If the surface of the pavement is kept cooler, the gradient of the temperature inside the pavement will obviously be smaller. Reduction of temperature gradients will result in less fatigue damage (McAuliffe *et al.* 1995). The peak pavement temperature can be reduced by about 4°C for each increase of 0.1 of albedo.

One suggested method of increasing the albedo is to cover the pavement with a single layer of aggregate. When used as a repair technique this procedure is known as a chip seal. Experiments show that the albedo of chip seals are about 70% of the albedo of the aggregate. A similar technique may also be applicable to new AC construction—by spreading white aggregate as the final layer and rolling it into the pavement. In cases where a high grade or modified asphalt is called for, it may be less expensive to place an additional layer of aggregate. The possibility of covering a road with a thin layer of cement concrete—thin white topping—is being researched in the industry.

However, to check the effect of cooler pavements on durability in realistic conditions we believe that tests must now be extended to actual functioning roads, with time dependent flows of traffic that are representative of actual patterns.

More-reflective pavements have the benefits of increasing the effectiveness of street lighting and automobile headlights. Our result for a representative case is that the ratio of reflected light to direct light is approximately equal to the visible reflectivity. Changing from surfaces that are 10% reflecting to 30% would result in 20% more light from luminaires reaching a subject in the middle of a street.

There are likely to be benefits of longer tire life and better road traction if pavements are cooler. These potential benefits of cooler roads, achieved by making them more reflective, need to be quantified.

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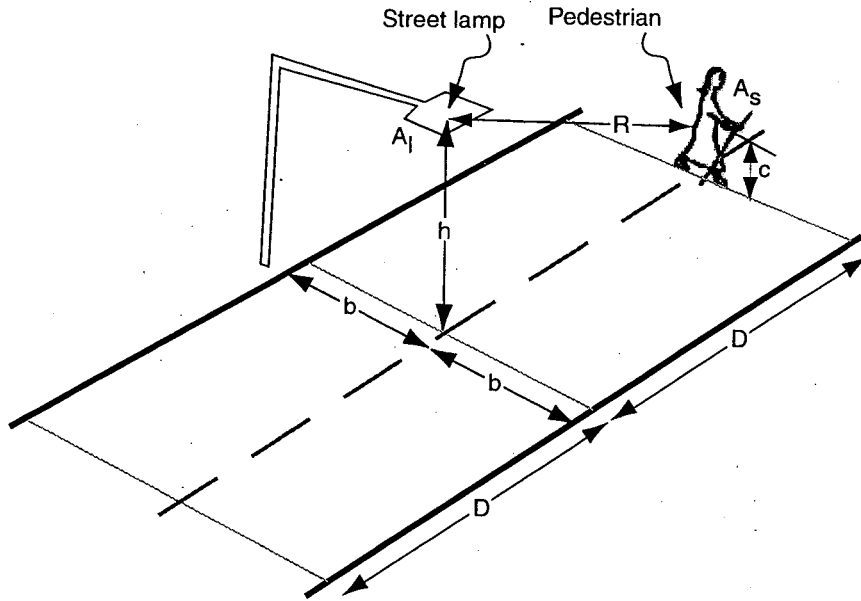
## Appendix—The Contribution of Pavement Reflectivity to the Illumination of Objects on a Road.

In this appendix we estimate the importance of the pavement reflectivity on the illumination of a subject standing on a road. (See Figure A.1) The subject is illuminated by a street lamp—partly by light that comes directly from the lamp and partly by light reflected off the pavement. We estimate the relative amounts of light that comes directly from the street lamp (luminaire) compared to the amount from the luminaire that is first reflected from the pavement. The light that comes directly from the luminaire takes the shortest path and is scattered least, but only a small fraction of the emitted light is intercepted by the subject (S). The light that is reflected from the pavement is intercepted by the entire pavement and thus is abundant, but the pavement reflects only a portion of the light and sends it in all directions. The question is: what is the ratio of the amount of light that arrives at S that has been reflected by the pavement to the amount that arrives directly. Our main interest is to estimate the effect of the pavement reflectivity,  $\mathcal{R}$ , on this ratio, which will indicate the importance of reflectivity on the visibility of the subject.

The quantity we seek is the ratio of energy flow on the subject from the reflected light,  $q_r$ , to the energy flow by the direct light,  $q_d$ , i.e.,  $q_r/q_d$ . Key to the solution is the use of the fraction of light emitted by one surface that is intercepted by another surface. This amounts to the solid angle subtended at one surface by the other. It has been variously called the “view factor” (Incropera *et al.* 1985), “configuration factor,” and other names (Siegel *et al.* 1981).

We formulate the method in general terms and then apply it to a specific the case of interest. We employ the notation of Siegel and Howell’s book (Siegel *et al.* 1981). The luminaire is assumed to have a luminous intensity of  $i$ , have an area of  $A_L$ , have a height of  $h$ , and be over the center of a road of width  $2 \times b$  (Figure A.1). The subject is assumed to have a height of  $c$ , have an area of  $A_s$ , stand in the center of the road, and be at a distance  $D$  along the road from the location of the

luminaire. For simplicity, we assume the luminaire to be a diffuse, or Lambertian, source (Siegel *et al.* 1981).



**Figure A.1.** The geometry of a subject illuminated by a street lamp. The subject is partly illuminated by light originating directly from the luminaire and partly illuminated by light reflected off the pavement.

This means that it emits with an emissive power  $= e = iA_L \cos\theta$  in the direction  $\theta$ .<sup>7</sup> The power it emits into an entire hemisphere is (Siegel *et al.* 1981, Eq. 2.8a)

$$q_L = \pi i A_L \quad (\text{A.1})$$

The configuration factor,  $F_{L \rightarrow 2}$ , is defined as the fraction of  $q_L$  that is intercepted by surface #2. Thus

$$F_{L \rightarrow 2} \equiv q_{L \rightarrow 2} / q_L \quad (\text{A.2})$$

where  $q_{L \rightarrow 2}$  is the total power flow from the luminaire, L, to a surface #2 (e.g., the pavement, P). Of the total power from the luminaire to the pavement

$$q_{L \rightarrow P} = q_L F_{L \rightarrow P} \quad (\text{A.3})$$

<sup>7</sup>Actual luminaires can have a variety of highly anisotropic radiation patterns (IES 1983), each of which must be considered individually.



a fraction,  $\mathcal{R}$ , of the visible light is reflected diffusely. Of the amount leaving the pavement, a fraction  $F_{P \rightarrow S}$  is intercepted by the subject, S. Thus the visible light power from the luminaire that arrives at the subject after being reflected from the pavement is (multiplying Eqs. A.1 by A.2 and A.3)

$$q_{L \rightarrow S, r} = \pi i A_L F_{L \rightarrow P} \mathcal{R} F_{P \rightarrow S} \quad (\text{A.4})$$

For our geometry, we find the fraction of light intercepted by the road,  $F_{L \rightarrow P}$ , by applying the analytic solution for the configuration factor of a small planar diffuse source of area  $A_L$  located at a height  $h$ , parallel to and over the corner of a rectangle (Siegel *et al.* 1981, configuration C-4). In our case, we want the luminaire to be over the *center* of a rectangular pavement. This is obtained by placing four rectangles of size  $b \times D$  as indicated in Figure A.1. Four times the light intercepted by one of them is the light intercepted by all, which is

$$F_{L \rightarrow P} = \frac{2}{\pi} \left( \frac{X}{\sqrt{1 + X^2}} \tan^{-1} \frac{Y}{\sqrt{1 + X^2}} + \frac{Y}{\sqrt{1 + Y^2}} \tan^{-1} \frac{X}{\sqrt{1 + Y^2}} \right) \quad (\text{A.5})$$

where  $X = b/h$  and  $Y = D/h$ .

To calculate  $F_{P \rightarrow S}$ , the fraction of light emitted (i.e., reflected) from the pavement that is intercepted by the subject, we start with the configuration factor from Siegel and Howell's Appendix C, configuration C-6. This is the fraction of light *from* a small planar area ( $A_S$ , aligned perpendicular) *to* a rectangular area (of length  $2 \times D$  in the direction of the normal to  $A_S$  and side  $b$  parallel to the plane of  $A_S$ ), which is denoted by  $F_{S \rightarrow P}$ . The fraction we need is not  $F_{S \rightarrow P}$ , but rather  $F_{P \rightarrow S}$ , the fraction of light reflected from the pavement that is intercepted by the subject. To obtain this, we use the reciprocity relationship (Siegel and Howell, Eq. 7-19):  $A_P F_{P \rightarrow S} = A_S F_{S \rightarrow P}$ . From which follows

$$F_{P \rightarrow S} = A_S F_{S \rightarrow P} / A_P \quad (\text{A.6})$$

We substitute  $F_{P \rightarrow S}$  of Eq. A.6 into Eq. A.4 to give

$$q_r = \pi i A_L F_{L \rightarrow P} \cdot \mathcal{R} \cdot A_S F_{S \rightarrow P} / A_P \quad (\text{A.7})$$

The configuration factor (C-6) applies to a subject at a corner of the rectangle. It needs to be doubled for our case in which the subject is taken to be at the midline of the pavement. Thus, the fraction of light that leaves S and arrives at rectangle ( $2b \times 2D$ ) is

$$F_{S \rightarrow P} = \frac{1}{\pi} \left( \tan^{-1} \frac{1}{y} - \frac{y}{\sqrt{x^2 + y^2}} \tan^{-1} \frac{1}{\sqrt{x^2 + y^2}} \right) \quad (\text{A.8})$$

where  $x = 2D/b$  and  $y = c/b$ . Similarly, the power that flows *directly* from luminaire to the subject is

$$q_{L \rightarrow S, d} = \pi i A_L F_{L \rightarrow S} \quad (\text{A.9})$$

As given by Siegel and Howell (their Eq.7-11)

$$F_{L \rightarrow S} = \cos\theta_1 \cos\theta_2 A_S / \pi R^2. \quad (\text{A.10})$$

Substituting Eq. A.6 in Eq. A.5 gives

$$q_{L \rightarrow S, d} = i A_L \cos\theta_1 \cos\theta_2 A_S / R^2. \quad (\text{A.11})$$

$\theta_1$  and  $\theta_2$  are the angles between the normals to areas  $A_L$  and  $A_S$  and the line  $R$  between them, respectively. This is the same answer derived directly from the inverse square law and the cosine dependences of subtended areas (Siegel *et al.* 1981, Eq. 7-3).

The ratio of the reflected light to the light that arrives directly, i.e., Eq. 11 divided by Eq. A.7, is

$$q_r/q_d = \frac{\pi i A_L F_{L \rightarrow P} \cdot \mathcal{R} \cdot A_S F_{S \rightarrow P} R^2}{i A_L A_S A_p \cos(90 - \theta) \cos\theta} = \frac{\pi \mathcal{R} \cdot F_{L \rightarrow P} F_{S \rightarrow P} R^2}{A_p \cos\theta \cos(90 - \theta)} \quad (\text{A.12})$$

Appropriately for this ratio, the luminaire intensity and the areas of the luminaire and the subject cancel out.

To obtain a numerical value, we make the observation that a diffuse luminaire casts light in equal intensity circles. The intensity is 0.5 of the maximum at a radius that is 1.7 of the height of the light post ( $q_d/q_{d, \max} = \cos\theta = 0.5$  when  $R = 1.7 h$ ). The configuration factor of Eq. A.8 applies only to *uniformly* emitting surfaces (Siegel & Howell 1981, p. 183), which the pavement is not; it is neither uniformly illuminated nor is it circular. To estimate, we define the distance  $D$  as the distance over which the illumination and thus the luminance of the pavement are within  $0.75 \pm$

0.25 of the maximum. Thus we take  $D = 1.7 h$ . We make the reasonable approximations that  $b = h$  and  $h \gg c$ , so that  $R^2 \approx D^2 + h^2 \approx 4h^2$ . Thus we take  $X = b/h = 1$  and  $Y = D/h = 1.7$ . This allows us to evaluate Eq. A.12 for the special case of the subject being at the edge of the illuminated part of the pavement, as indicated in Figure A.1. With these values

$$q_r/q_d \approx 0.36\pi\mathcal{R} \approx 1.1\mathcal{R} \quad (\text{A.13})$$

If one makes the criterion for “uniform illumination” to be that the intensity of light on the *horizontal* pavement surface be one-half maximum,  $q_{d, \text{horiz}}/q_{d, \text{max}} = \cos^2\theta = 0.5$ , then  $D = 0.7 h$ , and the ratio  $q_r/q_d \approx 0.25\pi\mathcal{R} \approx 0.8\mathcal{R}$ . Thus, the ratio of reflected to direct light is not strongly sensitive to the assumption of how large is the area of “uniform” illumination for this geometry. We conclude that the reflected contribution to the illumination is a noticeable fraction of the direct illumination and is directly proportional to the visible reflectivity of the pavement for this example.

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**ERNEST ORLANDO LAWRENCE BERKELEY NATIONAL LABORATORY  
ONE CYCLOTRON ROAD | BERKELEY, CALIFORNIA 94720**