Enhancing thermal properties of asphalt materials for heat storage and transfer applications

Andrew R. Dawson, Pejman Keikhaei Dehdezi, Matthew R. Hall, Junzhe Wang and Riccardo Isola

This paper considers extending the role of asphalt concrete pavements to become solar heat collectors and storage systems. The majority of the construction cost is already procured for such pavements and only marginal additional costs are likely to be incurred to add the necessary thermal features. Therefore, asphalt concrete pavements that incorporate aggregates and additives such as limestone, quartzite, lightweight aggregate, copper slag, and copper fibre are designed to make them more conductive, or more insulative, or to enable them to store more heat energy. The resulting materials are assessed for both mechanical and thermal properties by laboratory tests and numerical simulations and recommendations are made in regard to the optimum formulations for the purposes considered.

Keywords: thermo-physical properties; asphalt concrete; pavement energy system; heat storage and transfer applications

Background

Worldwide, asphalt pavement surfacings provide the vast majority of roads, parking lots, and airport runways. Given their dark colour, asphalt pavements can get heated up to 70°C due to solar irradiation in summertime because of their excellent heat-absorbing properties (Chen, Wei, & Wu, 2009).

Many modern industrial and commercial buildings have a high heating and/or cooling load. This load has a high, potential, environmental impact. Therefore, there is a strong pressure to obtain the necessary energy from a renewable source. Because such buildings frequently have large adjacent paved areas (roads and vehicle-parking lots), there is a great potential for collecting and/or storing solar energy using these adjacent surfaces which are already required and funded for operational purposes (e.g. from a transportation or parking budget). de Bondt (2003) reported a full-scale trial of such a ‘pavement energy system’ (PES), installing pipes close to a pavement surface, thereby optimising the pavement to collect solar energy in a pavement heat collection (PHC) configuration.

Pavements, by installing loops at greater depths, might also be used as a heat source during winter and as a heat sink during summer (an application similar to that of a ground-source
heat pump). Such pavement-source heat stores (PSHS) would exploit the fact that the seasonal temperature variation under the pavement is much less than the temperature fluctuation of ambient air because of the high thermal mass of the pavement.

Conceivably, the two arrangements could be combined to form a composite system in which solar heat, collected by the pavement surface in the summer, is transferred and stored at a shallow depth for subsequent re-use (Carder, Barker, Hewitt, Ritter, & Kiff, 2007). Figure 1 shows the different applications of thermally enhanced pavements. The heat passed to or from the pipes could also be used, directly or via a heat pump, for purposes such as

- the de-icing of roads in winter,
- the reduction of the urban heat island (UHI) effect,
- the reduction of rutting of asphalt,
- the supply of hot water, and
- the conversion of the energy to a transmittable form.

The efficiency of a PES in transporting large quantities of heat from the pavement surface to the embedded pipe network depends on several key factors:

1. the ability of the pavement to absorb heat at/near the surface–air interface;
2. the ability to conduct heat between the pavement surface and the pavement sub-surface;
3. the depth of the embedded pipe network;
4. the materials, geometry, spacing, and dimensions of the pipes;
5. the type of working fluid within the pipes;
6. the initial temperature and flow rate of the working fluid; and
7. the pavement material–pipe interface; that is, the ratio of specific surface area to area in contact with the embedded pipe.

**Objective and scope**

Thus, to realise the potential benefits of the concept just described, it would be necessary to have pavement materials that continue to provide pavement functionality, but that would also be optimised for their thermal properties. In a companion paper (Keikhaei et al., 2010), several of the current authors have described the assessment of concrete pavement materials in the application just described. Therefore, the objective of this paper is to investigate the thermal improvements of, and potential for using, asphaltic materials for heat storage, transmission, and/or insulation while still providing a useful paving function.
The thermo-physical properties of the pavement materials will, clearly, play a major role in Factor 2 in the list given above. For this reason, this paper studies the thermal properties of typical and modified asphalt materials. The three most important properties of a material in a thermal analysis are the thermal conductivity, specific heat capacity, and density. The rate of heat extraction and ultimately the size of the embedded pipe loop and cost of the system must, inevitably, be influenced by the thermo-physical properties of the pavement materials. However, it is the least understood aspect of a PES design because there are many factors affecting the thermal properties of a pavement and so little measured data available (ASHRAE, 1995; Rawlings & Sykulski, 1999).

The individual effects of the thermo-physical properties of a pavement on the variation of maximum and minimum pavement surface temperatures have been proven to be significant (Gui, Phelan, Kaloush, & Golden, 2007). In this study, the thermo-physical properties of asphalt pavement mixtures consisting of conventional and unconventional components, for the first time, are measured using accurate and reliable methods such as those using a heat flow meter (HFM) for measuring thermal conductivity and a differential scanning calorimeter (DSC) for measuring specific heat capacity. The thermo-physical properties, measured in this study, could also be used for the other applications where these parameters are critical, for example, for predicting freeze–thaw cycles in a pavement due to fluctuating environmental conditions (Dempsey & Thompson, 1970; Mrawira & Luca, 2002), predicting the UHI effect (Gui et al., 2007; Mallick, Chen, & Bhowmick, 2009), and road-condition forecasting for timely application of de-icing and anti-icing salt for winter road maintenance.

Literature review

The first application of pavement solar collectors in the literature seems to be that of Sedgwick and Patrick (1981). They experimentally studied swimming pool heating in summer by use of a grid of plastic pipes laid 20 mm under an asphalt surface in a tennis court in the UK. The air temperature and solar radiation for the period of the experiment reached 22°C and 610 W/m², respectively. They found that the system can provide heating to swimming pools which are usually operated at between 20°C and 27°C and, hence, concluded that the system was technically feasible, for UK conditions, and cost effective than a conventional swimming pool solar heater. Turner (1986, 1987) studied, theoretically, the performance of a pavement heat collector in winter (maximum pavement surface temperature of 15°C) and summer (maximum pavement surface temperature of 70°C) for applications varying from de-icing of roads and bridges to water heating for heat pumps and swimming pools. He conducted some analyses (using a simple 1D steady-state model) and found that pavement heat collector systems may be suitable for the application that he was considering. Nayak, Sukhatme, Limaye, and Bopshetty (1989) carried out experimental studies on a roof solar concrete collector, with glazing at the top, used for providing domestic hot water. They embedded PVC tubes 10 mm under the concrete which was painted black to increase its solar absorptivity. They concluded that solar concrete collectors can be used as cost-effective alternatives to conventional systems (e.g. flat-plate collectors) under the climate where the experiment was performed (maximum air temperature and solar radiation reached 35°C and 1000 W/m², respectively). Similarly, Chaurasia (2000) experimentally studied the performance of a concrete solar collector by laying down a network of aluminium pipes in the roof of a building (with no glazing at the top) in India (maximum air temperature and solar radiation reached 27°C and 650 W/m², respectively) to supply hot water for domestic use. They concluded that for an inlet water temperature of 15–28°C, hot water at a temperature of 36–58°C could be obtained during the daytime in winter. Hasebe, Kamikawa, and Meiarashi (2006) experimentally investigated an asphalt solar collector to produce electric power. The electric power was produced by temperature
differences between warm water (coming out of the embedded pipe in the pavement) and cool water (supplied from a river) as a thermoelectric generator. They investigated the effects of outlet temperature of the warm water on generated electricity and found that the output power significantly increases as the outlet water temperature increases.

Studies conducted in recent years have aimed to improve the efficiency of pavement heat collector systems. Mallick et al. (2009) experimentally and theoretically studied asphalt pavements for applications of harvesting energy and reducing the UHI effect. They showed, using a finite-element model, that near-surface air temperature could be reduced as much as 10°C by running water through the embedded pipes placed about 40 mm under the pavement. In addition, they performed small-scale laboratory testing on asphalt pavement samples. Their results showed that black acrylic paint on the asphalt surface and replacement of limestone aggregates with aggregates containing high percentages of quartz could increase the efficiency (rise in water temperature) of the system by 50% and 100%, respectively. Wu, Chen, Wang, and Zhang (2009) performed a similar study on small-scale asphalt pavement slabs and concluded that the pavement surface temperature could be significantly reduced as water flows in the embedded pipes. In another study, Wu et al. (2009) experimentally investigated the use of graphite powders in asphalt pavements to improve the thermal conductivity and energy exchange efficiency. They concluded that the addition of graphite could slightly increase the outlet water temperature, however, but realised that longer piping and a larger area of heat transfer are necessary to realise a meaningful temperature rise. In addition, the lubricant effect of graphite may have a negative effect on the mechanical performance of the asphalt pavements.

The Road Energy Systems® (RES) asphalt solar collector (see Figure 2) is a system for extracting energy from asphalt pavements. This system has been developed and partly commercialised in the Netherlands by Ooms Avenhorn Holding bv (de Bondt, 2003; Sullivan, de Bondt, Jansen, & Verweijmeren, 2007). The RES heats building and roads in winter and cools both of these in summer when it collects heat that is then stored at about 70 m below the ground. The water that circulates is the groundwater (de Bondt, 2003).

Figure 2. Asphalt solar collector operational modes (from the RES).
In order to commercialise the RES, Ooms has conducted different experiments to investigate possible detrimental effects of pipes on the lifetime of the pavement as well as the effect of the asphalt compaction process on the (plastic) pipes. van Bijsterveld and de Bondt (2002) experimentally showed that the presence of pipes in the asphalt has negative effects on the durability of the asphalt structure because of concentrated stresses near the pipes which can lead to crack initiation. To prevent this phenomenon, they developed a three-dimensional reinforcing grid to fix and protect the pipes during the laying and compaction of the asphalt mixture and to reduce the stresses around the pipes. In addition, a special polymer-modified bitumen was developed to achieve a high-quality asphalt mixture in between the pipes and the grid. The RES has been successfully installed in the Netherlands and the UK (Sullivan et al., 2007).

Another major instrumented trial of the solar energy collector from asphalt pavements was undertaken by Icax™ Limited (Carder et al., 2007) in the UK. The Icax system is very similar to the Ooms system since both of the systems can be considered as ‘composite systems’ (collect the solar energy and store it until required). In the Icax system, heat absorbed by the asphalt pavement will be collected through upper loops installed below the asphalt surface. The warm water in the upper loops will then be circulated and stored through lower loops installed below the asphalt surface. Carder et al. (2007) found that at the end of a full season of heat recovery (from May to September), ground temperature at the centre of the heat store remained about 9°C higher than that of the control area of the pavement (i.e. without any loops and insulation). Moreover, they carried out winter maintenance of the pavement from the heat recovered in the heat store. They found that the heated section of the road was maintained at a temperature about 3°C higher than the unheated area.

Experimental programme

In the present study, a wide range of heavyweight, lightweight, and normal aggregates, as well as other additives, were considered as potential inclusions within asphalt, being those that might deliver beneficial thermo-physical properties. Those considered in detail were limestone (as reference aggregate), quartzite, sintered pulverised fuel ash lightweight aggregate (known as Lytag®), air-cooled copper slag, and copper fibre. All the asphalt specimens were subjected to thermal conductivity test, specific heat capacity test, indirect tensile stiffness modulus (ITSM) test, indirect tensile fatigue test (ITFT) at different stress levels, and repeated load axial test (RLAT) (see British Standards Institution, 1993, 1996, 2003).

Mix design

The aggregate gradation and mix design was selected in accordance with the Construction Support Team (Defence Estates, 2008), for a wearing course using macadam concrete, as shown in Table 1.

The specifications listed in Table 1 are particularly for limestone, with there being no specification for the other aggregates used in the project. To generate alternative mix designs, the asphalt mixtures for the other aggregates were generated by replacing limestone with the same volume of the replacement aggregates while keeping the remaining parameters (bitumen type and content and aggregate grading) constant. Due to the non-availability of the larger sizes of Lytag and copper slag replacement aggregates, the asphalt mixes were produced according to Table 2. Although the bitumen content and aggregate grading need to be modified for each mix according to the aggregate type used in the mix, since the main focus of this paper was to study the effect of aggregate alteration on the thermal performance of the asphalt mixtures, it was decided to keep the other parameters (e.g. grading, bitumen content, and type) constant in order to eliminate
Table 1. Specification for wearing course AC14 (Defence Estates, 2008).

| Aggregate grading | Mass passing given sieve size (%) |
|-------------------|-----------------------------------|
| Sieve size (mm)   | 100                               |
| 14                |                                   |
| 10                | 77–83                             |
| 6.3               | 52–58                             |
| 2                 | 25–31                             |
| 1                 | 14–26                             |
| 0.063             | 4.5–6.5                           |
| Target binder content | Air voids in total mix   |
| 4.9 ± 0.4%        | 4 ± 1.5%                          |

Paving grade of bitumen

100/150

Table 2. Asphalt mixes used in the study.

| Mixes                        | Descriptions                                                                 |
|------------------------------|-------------------------------------------------------------------------------|
| Limestone (control)          | All the aggregates in the mix were limestone                                  |
| Quartzite (partially replaced) | Limestone smaller than 10 mm was replaced with quartzite (80% vol quartzite, 20% vol limestone) |
| Copper slag (partially replaced) | Limestone smaller than 10 mm was replaced with copper slag (80% vol copper slag, 20% vol limestone) |
| Quartzite (fully replaced)   | All the aggregates in the mix were quartzite                                  |
| Quartzite +2% Cu fibre (fully replaced) | All the aggregates in the mix were quartzite. In addition, 2% copper fibres (1 mm in diameter and 50-mm long) were added to the mix |
| Lytag (partially replaced)/LWA | Limestone smaller than 10 mm was replaced with Lytag (80% vol Lytag, 20% vol limestone) |

the effect of such parameters on the thermal performance of asphalt mixes. The study does not attempt to define material co-optimised for thermal and mechanical behaviour as it would be difficult to define successful co-optimisation, let alone achieve it. Instead, it aims to demonstrate that significantly improved thermal properties can be obtained with mixes that are still mechanically satisfactory for pavement application.

Two slabs each, with dimensions of 300 mm × 300 mm in area and around 60-mm thick, were manufactured for all mixes. The slabs were compacted until the target density was reached (as determined by slab volume) using a laboratory roller compactor. All the slabs were first subjected to thermal testing and then were cored across the plane of compaction and trimmed to produce specimens (100-mm diameter with a mean thickness of 40 mm) for the mechanical test evaluations. The core specimens were, first, subjected to the ITSM test, which is a non-destructive test; next, the same specimens were subjected to the ITFT test at different stress levels. In addition, three specimens (100-mm diameter with a mean thickness of 40 mm) for each mix were also produced in order to perform the RLAT.
Mechanical assessment

The Nottingham Asphalt Tester is a well-known test equipment used to carry out various performance tests on bituminous materials. It consists of a temperature-controlled cabinet containing a load frame, a sample support, and instrumentation cradle and a loading system comprising a pneumatic load actuator with load cell.

ITSM test

Stiffness in a pavement material is the principal measurement used to indicate the ability of a material to spread the traffic loading over an area. For comparative purposes, a fixed temperature of 20°C was used and the test was performed according to the 1993 British Standard (British Standards Institution, 1993). The stiffness modulus, $S_m$ (in MPa), can be calculated using Equation (1) (British Standards Institution, 1993):

$$S_m = \frac{F}{(D \times t)} \times (v + 0.27),$$  \hfill (1)

where $F$ is the peak value of the applied vertical load (N) with rise time of 124 ms, $D$ the peak horizontal diametral deformation resulting from the applied load (mm), $t$ the mean thickness of the test specimen (mm), and $v$ the value of Poisson’s ratio (0.35 for bituminous mixtures).

Indirect tensile fatigue test

Fatigue is the condition whereby a material cracks or fails as a result of repeated (cyclic) loading (stress or strain) applied below the ultimate strength of the material. For the purposes of evaluating the asphaltic specimens in this study, the test was performed according to DD ABF (British Standards Institution, 2003) using the stress mode. The maximum horizontal tensile stress and the maximum horizontal strain for each specimen can be calculated from Equations (2) and (3), respectively:

$$\sigma_{\text{max}} = \frac{2P}{\pi \cdot d \cdot t},$$ \hfill (2)

$$\varepsilon_{\text{max}} = \frac{\sigma_{\text{max}} \times (1 + 3v)}{S_m},$$ \hfill (3)

where $P$ is the applied compression load and $d$ the specimen diameter.

Repeated load axial test

Resistance to the development of permanent deformation is a property that is directly related to the stability of the aggregate skeleton. A highly deformation-resistant mixture requires a dense and well-interlocked aggregate skeleton. In this study, the RLAT was performed to assess this resistance according to the British Standard (British Standards Institution, 1996).

Thermal assessment

Thermal conductivity, $\lambda$, of the asphalt specimens was determined by the HFM technique using a computer-controlled P.A. Hilton B480 that complies with ISO 8301 (1996). The slab specimens were placed inside the apparatus between a temperature-controlled hot plate and a water-cooled cold plate. More details about the test can be found in previous publications (Keikhaei et al., 2010).
Two slabs were prepared for each mix design, and then the mean value of three independent readings was obtained for each slab specimen.

The specific heat capacity of each mix design, $c_p$, was calculated as the sum of the heat capacities of the constituent parts weighted by their relative proportions, with each being measured using a differential scanning calorimeter (DSC) (TA Instruments Model Q10 DSC). The overall specific heat capacity of asphalt can then be calculated from Equation (4):

$$c_p = \frac{1}{m_{\text{total}}} \left[ m_{\text{ Aggregate}} \times c_{\text{ Aggregate}} + m_{\text{ Bitumen}} \times c_{\text{ Bitumen}} + m_{\text{ Additive}} \times c_{\text{ Additive}} \right], \quad (4)$$

where $m$ is the mass of each constituent in kg and $c$ the specific heat capacity of each constituent in J/kg K.

Thermal diffusivity ($\alpha$) is the coefficient that expresses the rate of heat energy diffusion ($m^2/s$) throughout a material when it is exposed to a fluctuating thermal environment and is calculated as

$$\alpha = \frac{\lambda}{\rho \times c}, \quad (5)$$

where $\alpha$ is the thermal diffusivity (m/s), $\lambda$ the thermal conductivity (W/m K), $\rho$ the density (kg/m$^3$), and $c$ the specific heat capacity of each constituent in J/kg K.

**Results and discussions**

The test results of asphalt mixtures prepared with limestone, copper slag, and quartzite are presented and analysed as a group, while the results for Light Weight Asphalt (LWA) are discussed separately.

**ITSM test results**

Figure 3 shows the mean measured stiffness of all five mixes. Limestone has the highest stiffness modulus value of 1533 MPa, followed by copper slag and quartzite mixtures. The addition of metallic fibre seems to improve the stiffness by about 68% compared with the mix with no fibre (i.e. 100% quartzite mix). Criteria and limits for asphaltic concrete wearing course AC14 with conventional aggregates require a value for stiffness ranges 1500 to 2000 MPa at 20$^\circ$C (Kridan, Arshad, & Rahman, 2010; Thom, 2008). Thus, all alternative mixtures perform rather poorly compared with this value.

The low value for the stiffness of quartzite asphalt may be partly attributable to the relatively smooth faces of the quartzite aggregates as revealed by its low roughness value. For this reason, aggregate surface texture was measured using a surface profilometer (2D Mitutoyo Surftest SV 662 profiler). The roughness average ($R_a$) is the most commonly used parameter for expressing measurements of surface contour. The value represents the arithmetic average of the height of the roughness irregularities above the mean line along the sampling length and is normally measured in microns. The average readings for five profiles of the roughness ($R_a$) values for limestone, copper slag, and quartzite were 10.87, 8.63, and 5.52 $\mu$m, respectively. The lower ability of quartzite aggregates to absorb bitumen may also cause the reduction in the stiffness. Since the binder content for all the mixes is the same, lower bitumen absorption could cause a softer mix due to the excess of bitumen. The effect of copper fibres on the stiffness improvement of the mix may be due to the increased interconnection between the fibres.
Fatigue life is commonly defined as the number of load cycles to fail the asphalt concrete specimen at a certain stress or strain level. The specimens were subjected to the ITFT at different stress levels, namely 100, 150, 175, 200, 250, 300, 350, 375, 425, and 500 MPa. A fatigue regression analysis was performed using the relationship

$$N_f = k_1 \left( \frac{1}{\varepsilon} \right)^{k_2},$$

where $N_f$ is the number of load application to failure, $k_1$ and $k_2$ are the constants depending on the mixture characteristics, $\varepsilon$ is the resultant strain due to applied stress.

The fatigue lines are plotted in Figure 4, with the enumerated values of Equation (6) for the five materials shown in the figure. The similarity of the values of $k_2$ reflect the parallel nature of the five lines, while the small differences in $k_1$ reflect the small spacing of the lines with respect to each other. From Figure 4, it can be seen that the asphalt mix containing copper fibre showed the best resistance against fatigue. Both quartzite mixes (i.e. partial and full aggregate replacement) achieved almost the same fatigue line, and the copper slag mix performed slightly better than the limestone mix. The higher fatigue life of the fibre mixture is most likely due to the high level of fibre interconnection. Thus, all of the unconventional mixtures show a fatigue relationship that is likely to represent satisfactory in situ performance. Given the lower stiffness modulus of the quartzite asphalt, the greater fatigue life is somewhat surprising. However, a lower stiffness layer would be expected to strain more in situ for the same level of system loading; hence, the actual fatigue life might be similar.

**RLAT test results**

Figure 5 presents the average axial permanent strain curves obtained from the results of the RLAT for the five mix materials. It can be seen that they exhibit a similar response during the loading.
Figure 4. Number of load cycles to failure versus strain.

The permanent strains all increase rapidly at the beginning, followed by a progressively reducing strain rate per cycle. The deformation occurring in the first 500 cycles is 69%, 69%, 85%, 82%, and 83%, respectively, for the limestone, copper slag, quartzite (partially replaced), quartzite + 2% fibre, and quartzite (fully replaced) asphalts. The quartzite asphalt experienced by far the largest permanent strain. Once again, this could be due to its smooth surface resulting in poor bond with bitumen. Although the addition of fibres to the asphalt mix slightly improved the permanent deformation, compared with the mix with no fibres, it did not result in a significant reduction of permanent deformation. This may be due to the horizontal fibre orientation (perpendicular to the direction of loading and, thus, probably largely ineffective as reinforcing elements) in the asphalt mix as shown in Figure 6.
**Test results of the thermal properties**

The thermo-physical properties of the asphalt mixtures are presented in Table 3. Table 3 shows that fully replacing limestone aggregates with quartzite can enhance thermal conductivity by about 135%. Surprisingly, the addition of copper fibres to asphalt mixtures did not increase thermal conductivity, and the thermal conductivity of fibre-modified asphalt was only enhanced by about 13%. This increase is unlikely to deliver a significant economic benefit given the typical cost associated with the purchase of copper fibre. The authors in another study (Keikhaei et al., 2010) along with other researchers (Cook & Uher, 1974) found that the addition of copper fibres to concrete mixes could significantly increase the thermal conductivity of the asphalt mixes. The ineffectiveness of the fibre addition on the thermal conductivity of the asphalt mix might be related to the fibre orientation in the asphalt mix. Figure 6 shows a 3D image reconstructed from the 2D slices taken across the height of the fibre-modified asphalt specimen at 1-mm slice spacing. Figure 6 shows that many fibres, possibly during the compaction process, are lying close to the

| Mix No. | Mix type                  | Thermal conductivity $\lambda$ (W/m K) | Specific heat capacity $c_p$ (J/kg K) | Density $\rho$ (kg/m$^3$) | Volumetric heat capacity $\alpha$ ($\times 10^{-7}$) (m$^2$/s) | Achieved air void (%) |
|---------|---------------------------|----------------------------------------|---------------------------------------|---------------------------|---------------------------------------------------------------|---------------------|
| 1       | Limestone (control)       | 1.21                                   | 919                                   | 2382                      | 5.53                                                          | 4.1                 |
| 2       | Quartzite (partially replaced) | 1.46                                   | 880                                   | 2351                      | 7.06                                                          | 4.1                 |
| 3       | Copper slag (partially replaced) | 1.05                                   | 814                                   | 3088                      | 4.15                                                          | 3.7                 |
| 4       | Quartzite (fully replaced) | 2.47                                   | 870                                   | 2314                      | 12.30                                                         | 4.9                 |
| 5       | Quartzite +2%Cu fibre (fully replaced) | 2.82                                   | 836                                   | 2477                      | 13.64                                                         | 3.7                 |
| 6       | Lytag (partially replaced) | 0.46                                   | 863                                   | 1504                      | 3.54                                                          | 4.9                 |
Table 4. Cycles to failure under 100 kPa and 200 kPa.

|          | Limestone | Copper slag | Quartzite | LWA |
|----------|-----------|-------------|-----------|-----|
| 200 kPa  | 915       | 1289        | 841       | 76  |
| 100 kPa  | 13781     | 7924        | 8728      | 738 |

horizontal direction (perpendicular to the direction of heat flow) and hence might not be able to convey heat as efficiently as possible in a fibre-modified asphalt mix.

Table 3 also shows that the thermal diffusivity of the asphalt mixes reduces due to the reduction of thermal conductivity as well as increase in volumetric heat capacity ($\rho \times c$). Values for the thermal properties (i.e. thermal conductivity and specific heat capacity) of unconventional aggregates (i.e. quartzite, Lytag, and copper slag) were not found in the literature. However, the thermal conductivity and specific heat capacity of the reference asphalt (i.e. limestone mix) are in a good agreement with the finding reported in the literature. Yavuzturk, Ksaibati, and Chiasson (2005) and Gui et al. (2007) reported values of 1.3 W/m K, 880 J/kg K and 1.21 W/m K, 921 J/kg K for the thermal conductivity and specific heat capacity of hot mix asphalts, respectively.

Test results of LWA

The slabs generated with LWA were very weak. The aggregates did not bond well with the binder, especially at the corner of the slabs. Due to the poor quality, there were no meaningful results obtained for the ITSM test. Two samples were selected and tested under 200 kPa and 100 kPa for fatigue in order to give a general idea about the fatigue resistance of this mixture. As shown in Table 4, its performance is far behind the other materials. In terms of RLAT, the LWA experienced a deformation of 0.98% in strain, only a little higher than that of the quartzite asphalt. Though it is still the poorest performing, in terms of mechanical properties, it might have a useful role in forming a stabilised sub-base layer. One possibility to improve the performance of LWA could be to increase the binder content, since due to the higher porosity of LWA, a binder content of 4.9% (used in this study) seems to be not enough to provide an appropriate bond in the asphalt mixture. For the thermo-physical properties, one slab was tested, and the results are shown in Table 3. Thermal conductivity was approximately half the value obtained for the copper slag asphalt, reflecting the potential of LWA asphalt to act as an insulating layer. Given that one would not want to use an insulative layer at the surface of the pavement, but rather just above a lower pipe array (as in Figure 1, PSHS or composite), the stiffness, fatigue and deformation values would be unlikely to hinder its use at such a depth.

Numerical analysis of thermally enhanced pavements

Material design enhancement for PHC applications

In order to simulate the relative effects of pavement materials on surface temperature and temperature depth profile development in the pavements, a one-dimensional finite-difference (FD) transient heat transport model was used. The model is accurate to within 2°C variation, and it was found to give results at least as accurate as other similar models (Gui et al., 2007; Yavuzturk et al., 2005). The details of the model and its validation can be found in Hall, Keikhaei Dehdezi, Dawson, Grenfell, and Isola (2012) and Keikhaei et al. (2010), and a brief description is given here.

The factors influencing the pavement surface energy balance, as well as the heat transport processes that occur within a pavement, are short-wave and long-wave solar radiations, convection
heat due to wind, and conduction heat in the pavement. The absorbed short-wave radiation on the pavement surface is simply equal to $\alpha \cdot q_{\text{solar}}$, where $\alpha$ is an absorptivity coefficient. Thermal (long-wave) radiation heat flux between the pavement surface and the surrounding matter (i.e. the lower atmosphere and other buildings/objects) can be calculated as

$$q_{\text{thermal}} = \varepsilon \sigma (T_{\text{surr}}^4 - T_0^4),$$

(7)

where $\varepsilon$ is the emissivity (−), $\sigma$ the Stefan–Boltzmann constant = 5.67 × 10⁻⁸ W/m² K⁴, $T_0$ the surface temperature (K), and $T_{\text{surr}}$ the surrounding temperature (K).

The empirical Bliss equation, which estimates the surrounding conditions in the form of a hypothetical ‘sky temperature’ (an approximation of $T_{\text{surr}}$), is used (Gui et al., 2007; Yavuzturk et al., 2005):

$$T_{\text{sky}} = T_{\text{air}} \left(0.8 + \frac{T_{\text{dp}}}{250}\right)^{0.25},$$

(8)

where $T_{\text{dp}}$ is the dew point temperature (°C) and $T_{\text{air}}$ the air temperature (K).

One-dimensional vertical heat transport by transient conduction through the pavement can simply be modelled as a response to absorbed/desorbed energy at the pavement surface using an explicit form of the FD method. The cross-sectional pavement profile and the sub-soil beneath it can therefore be considered as a semi-infinite medium extending downwards from $d = 0$ (pavement surface) to $d = x$, at which point $\Delta T \to 0$. In reality, at a critical depth (usually several metres), the ground temperature is approximately constant as a result of thermal mass and so is largely unaffected by heating/cooling cycles at the pavement surface. In the FD method, the pavement comprises a column of nodes. The temperature at each boundary node is given by the energy balance equations as summarised in Table 5.

The boundary equations (summarised in Table 5) are solved by iteration in order to predicatively compute the temperature depth profile evolution at a given time interval. The environmental input parameters required for the model are hourly (or more frequently) solar irradiation, dry bulb air temperature, relative humidity (or dew point temperature), and mean wind velocity. The inputs were interpolated linearly across the hour period in order to achieve the interval required for the

| $T_{i+1}^{0}$ | $T_{m+1}^{i}$ | $T_{m}^{i+1}$ |
|---------------|---------------|---------------|
| $T_{0}^{i+1} = \frac{2\Delta t}{\rho_d c_p \Delta z} \left[ q_{\text{solar}} + h_c (T_{\text{air}} - T_0) + h_{\text{rad}} (T_{\text{sky}} - T_0) + \lambda \frac{T_i^{i} - T_0^{i}}{\Delta z} \right] + T_0^{i}$ | $T_{m}^{i+1} = \frac{\alpha \Delta t}{\Delta z^2} \left[ T_{m-1}^{i} - 2T_{m}^{i} + T_{m+1}^{i} \right] + T_{m}^{i}$ | $T_{m}^{i+1} = \frac{\lambda_1/\Delta Z}{\rho_d c_p 1 \Delta Z} T_{m-1}^{i} + \frac{\lambda_2/\Delta Z}{\rho_d c_p 2 \Delta Z} T_{m+1}^{i}$ |
| Surface node | Interior node | Interface node |

$T_{m}^{0}$ is the temperature at the ground surface and $T_{m}^{n}$ is the temperature at the bottom boundary node, $n$ being the number of nodal points. The terms in the above equations are defined as follows:

- $\lambda_1$, thermal conductivity (W/m K); $\alpha$, thermal diffusivity (m²/s); $m$, number of nodal points (1, 2, ..., $n$); $i$, counter for time step ($i = 0$ corresponding to specific initial condition); $\Delta t$, time step (s); $\rho_d$, dry density (kg/m³); $c_p$, specific heat capacity (J/kg K); $\Delta z$, node distance; $h_{\text{rad}}$, radiation heat transfer coefficient (W/m² K) = $\varepsilon \sigma (T_0 + T_{\text{sky}})/(T_0^2 + T_{\text{sky}}^2)$; $T_1$, temperature at first node beneath the surface node (K); $\alpha$, solar absorptivity; $h_c$, convective heat transfer coefficient (W/m² K); $h_c = 5.6 + 4.0 \times v_w$ for $v_w \leq 5$ m/s and $h_c = 7.2 \times v_w^{0.78}$ for $v_w > 5$ m/s.
model. In addition to surface absorptivity and surface emissivity, the thermo-physical properties of the pavement material required are experimentally determined.

The climatic data and pavement section for simulations were extracted from the seasonal monitoring performance conducted under the Long-Term Pavement Performance (LTPP) programme (US Department of Transportation–Federal Highways Administration, 2009) for the state of Arizona, USA. This was chosen as it is a prime location for a PHC installation where solar radiation exceeds 1000 W/m² in summer and approaches a ‘best-case’ performance scenario. The Arizona LTPP pavement climatic data were collected at weather station number 0100, between 1 June 1996 and 31 August 1996. The pavement section consisted of a 100-mm wearing course on top of a granular base. Figure 7 shows the predicted surface temperature variations for two cases:

1. where the pavement wearing course was constructed using limestone aggregates (Mix No. 1 (see Table 3) used in the model),
2. where the pavement wearing course was constructed using quartzite aggregates (Mix No. 4 (see Table 3) used in the model).

As can be seen from Figure 7, the maximum surface temperature can be reduced when quartzite aggregates are used in the wearing course of the pavement. The maximum surface reduction is approximately 4°C with an average reduction of more than 2°C of the peak temperature. The reduction in surface temperature is because of the higher thermal conductivity of quartzite mix that could increase the rate of heat transfer towards the bottom of the pavement. Reductions of pavement surface temperature could, potentially, minimise the rutting in asphalt pavements and extend their life. The UHI effect could also be minimised as a result of pavement surface temperature reduction and subsequently air temperature reduction in the adjacent urban area.
Figure 8. Predicted temperatures at a 50-mm depth for the limestone and quartzite asphalt mixes.

Figure 8 shows the temperature variations at a 50-mm depth for the two pavements mentioned above from 1 June 1996 to 1 July 1996 (top) and 1 October 1996 to 1 November 1996 (bottom). Figure 8 shows that using the quartzite mix could increase the average temperature by more than 2°C at a 50-mm depth in the pavement. This is because, when the pavement is a net heat recipient from the environment, a highly conductive surface material facilitates heat movement within the pavement away from the surface to the interior. This could, potentially, increase the performance of an installed PHC system.
Material design enhancement for PSHS applications

In order to determine the effects of pavement materials on heat storage in the pavement (i.e. PSHS), two pavement cross sections were considered as follows:

1. The conventional pavement consisted of 100-mm limestone as a surface on top of a 200-mm compacted aggregate as a base.
2. The modified pavement consisted of 100-mm copper slag mix (Mix No. 3) on top of a 200-mm Lytag mix (Mix No. 6) as a base.

Figure 9 shows the temperature distribution on typical summer day (top) and winter day (bottom) within conventional and modified pavements. Figure 9 shows that under the modified pavement, the temperature remains lower in summer and higher in winter by about 1.6°C. The stable temperature at a shallower depth is due to the use of a low thermal diffusivity pavement, which can be achieved by using high volumetric heat capacity aggregates and/or low conductivity aggregates. A more stable temperature at a shallower depth enables easier heat storage in the pavement as well as minimises the risk of damage due to freeze–thaw cycling in cold climates.

Mechanical analysis of thermally enhanced pavements

In order to investigate the structural performance of a quartzite asphalt layer (Mix No. 4) in comparison to that of a limestone asphalt layer (Mix No. 1), Shell Pavement Design Method (SPDM) software Version 3.0 was used. The analyses performed with the SPDM were an estimation of the asphalt thickness required to withstand fatigue and rutting under certain traffic and climate. Although the SPDM was never intended to work with the ITFT data, here it was employed to permit a comparison of pavements with different materials. Thus, the relative performance estimate should be valid even if the absolute values must be read with caution. To take into account the effect of climate, the software requires the monthly mean air temperature for 1 year and applies correction factors to estimate the resultant temperatures within the asphalt layer. Therefore, mean air temperature (in °C) multiplies by the correction factor of 1.47 for the rutting calculation and by 1.92 for the fatigue calculation (Brown, Brunton, & Stock, 1985). In order to recognise that the two pavements have different thermal properties, the air temperatures for the SPDM input were calculated by dividing the previously calculated average asphalt temperatures (see Figure 7) by 1.47 for the rutting calculation and by 1.92 for fatigue.

Fatigue simulation

The ITSM tests performed on the limestone and the 100% quartzite mixes have shown that both are not particularly resistant to fatigue (Thom, 2008); hence, the pavements simulated were subjected to a relatively low design traffic of one million equivalent standard axles over a design period of 5 years.

Four different structures were simulated, namely limestone over a 200-MPa and 800-MPa sub-base (L200, L800) and quartzite over a 200-MPa and 800-MPa sub-base (Q200, Q800). The stiffness value and fatigue line for limestone and quartzite asphalt layers (i.e. Mix No. 1 and Mix No. 4) were imported to the model shown in Figures 3 and 4, respectively, while other inputs were kept constant (see Table 6). The results from simulations L800 and Q800 show that the limestone mix would need a thickness of 220 mm, while the quartzite mix would only need 207 mm. This is probably due to the fact that, for these first examples, a stiff sub-base, in the case of quartzite, which might be bearing most of the stresses is employed. If the stiffness of the sub-base is lowered...
from 800 MPa to 200 MPa (simulations L200 and Q200), the limestone would require a thickness of 400 mm against the 520 mm of quartzite, which reflects the difference in stiffness.

*Rutting simulation*

The structures that were obtained from the fatigue simulations were also investigated from the rutting point of view, with the addition of two new structures. As can be seen from Table 7, two
Table 6. General inputs for fatigue and rutting simulations.

|                       | Fatigue simulations | Rutting simulations |
|-----------------------|---------------------|---------------------|
| Daily equivalent standard axles | 400                 | Daily equivalent standard axles | 400 |
| Design period         | 5 years             | Axle load            | 80 KN |
| Lateral distribution factor | 2                   | Design period        | 5 years |
| Healing factor\(^a\)  | 5                   | Wheels per axle      | 4 |
| Sub-base thickness    | 600 mm              | Contact stress       | 570 kPa |
| Sub-base Poisson’s ratio | 0.35               | Sub-base thickness   | 600 m |
| Subgrade stiffness    | 150 MPa             | Sub-base Poisson’s ratio | 0.35 |
| Subgrade Poisson’s ratio | 0.35               | Subgrade stiffness   | 150 MPa |
| Mass % of binder      | 5                   | Subgrade Poisson’s ratio | 0.35 |
| Volume % of voids     | 4                   | Mass % of binder     | 5 |
| Asphalt Poisson’s ratio | 0.48               | Asphalt Poisson’s ratio | 0.48 |
| –                     | –                   | Bitumen creep characteristic \(Q\) | 0.42 |
| –                     | –                   | Bitumen creep characteristic \(B\) | 0.60 MPa |

\(^a\)Allows for recovery on site between loads and for beneficial effects of wheel wander.

Table 7. Rutting simulation labels and asphalt layer thicknesses.

| Structure                        | Thickness (mm) | Rutting depths (mm) |
|----------------------------------|----------------|---------------------|
| L800                             | 220            | 21.7                |
| L200                             | 400            | 25.4                |
| Q800                             | 207            | 18.9                |
| Q200                             | 520            | 22.3                |
| New_ Quartzite on 200 MPa        | 220            | 18.3                |
| New_ Quartzite on 800 MPa        | 400            | 23.1                |

quartzite structures have been given the same asphalt thickness as the limestone structures in order to isolate the effect of materials on rutting (i.e. since the thicknesses are the same, the only differences observed will be due to the thermal properties of the two materials). These six structures were simulated by keeping a large number of parameters constant. These general settings for the rutting simulations are summarised in Table 7.

The results obtained from these simulations are given in Table 5. As can be seen, quartzite is constantly performing better than limestone, thanks to its superior thermal properties. Although a thicker asphalt layer would always be a disadvantage in terms of total rutting compared with a thinner one, it can be seen that even in the case of structure Q200, the total rutting still remains lower. As can be expected, this is even more evident for L800 and Q800 where the quartzite layer does not need to be as thick as the limestone one.

Conclusions

This study has investigated the desirable mechanical and thermo-physical properties of asphalt concrete pavement materials, their effects on the evolution of temperature depth profile, and the implications for mechanical pavement design and performance. The following conclusions can be drawn on the basis of the results and analysis presented in this paper:

1. Fully replacing limestone aggregates with quartzite can enhance the thermal conductivity by about 135%. In addition, the quartzite mixture improved the fatigue performance while showing a negative effect on the stiffness.
The addition of copper fibres improved the thermal conductivity slightly, while it offered a significant improvement in the stiffness and fatigue performance.

The use of LWA and copper slag decreased the thermal diffusivity of asphalt pavements, inducing a more stable temperature at a shallower depth, which would enable easier heat storage in the pavement as well as lower the risk of damage due to freeze–thaw cycling.

Quartzite asphalt mixes showed the potential to reduce the maximum surface temperature, by up to 4°C. This could, potentially, lessen rutting and the UHI effect while increasing the performance of the installed PHC.

Comparison of the quartzite and limestone wearing courses for their structural performance revealed that the quartzite mix would experience less rutting; however, it would need to be placed thicker in order to compensate for its lower stiffness.

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