The wealth of knowledge accumulated during the 17th IRF World Meeting & Exhibition in Riyadh was the driving force behind our decision to launch the IRF Examiner as a freely available resource for the industry. With this fourth issue, the International Road Federation confirms its role as a leading provider of applied knowledge in areas of vital importance for the global community of road professionals.

Pavements form the backbone of our transportation infrastructure and are essential in fully unleashing these benefits. By highlighting promising research in the areas of pavement engineering, stress, roughness and their collective impacts for vehicular travel, this issue of the IRF Examiner embodies our goal to help policy-makers, planners and infrastructure operators translate challenges into concrete policy and planning decisions.

H.E. Eng. Abdullah A. Al-Mogbel
IRF Chairman
Minister of Transport, Kingdom of Saudi Arabia

Roads are the world’s first “social network”. They are fundamental building blocks for human and economic development whose impacts transcend national borders. The benefits of investments in roads have shown how transformative an infrastructure they can be for a wide range of beneficiary communities.

At the International Road Federation, we have tried to capture these connections with a simple slogan “Better Roads. Better World”. Since we were established 1948, our primary purpose has been to transfer the latest technologies and knowledge from those who have it to those who need it, and in doing so, promote an agenda of shared prosperity that flows from accessible, affordable and sustainable road networks. The IRF Examiner is an essential vehicle to this ambitious agenda.

C. Patrick Sankey
IRF President & CEO

In the pavement community, we have come a long way in improving and advancing our current designs and performance prediction of pavements. We continue to make these advances mindful of important environmental, social and economic issues that we face today.

Better ride quality, safer roads, reduced fuel consumption, lower emissions and longer service life are among the tangible benefits that derive from research into pavement properties, lower energy in mixtures production and recyclability of existing pavements. Other pavement design considerations, such as the selection of surface or mixture type, typically result in lower impacts on tire wear emissions, reduced tire/pavement noise and mitigating impacts of the urban heat island effect.

To achieve these efforts, we need to continue our research efforts and ensure existing research is put into practice. This issue of the IRF Examiner highlight many promising approaches of interest to road agencies and practitioners worldwide.

Dr. Kamil Kaloush
Associate Professor, Arizona State University
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Wyndham Grand Hotel

“Corridors for Shared Prosperity”
Marginal Pavement Damage Cost Estimation Using Field Data: Issues and Opportunities

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ABSTRACT
For equitable highway user charging, highway agencies utilize different approaches for estimation of pavement damage cost. This paper addresses the issues involved with the use of field data for estimation of marginal pavement damage cost. The unavailability of appropriate data, temporal inconsistencies between the cost and traffic data, and the exclusion of reconstruction and routine maintenance cost are the major limitations of field data approach. Study results revealed that it is not possible to estimate marginal pavement damage cost with inadequate field data because marginal pavement damage cost requires a complete picture of the maintenance and rehabilitation profile between consecutive pavement reconstructions. However, it was also revealed that this issue can be partially resolved, and maintenance and rehabilitation marginal cost can be estimated either by using the estimated or observed treatment service lives of pavement preservation treatments.

STUDY METHODOLOGY
The true MPDC can only be estimated when the actual MR&R expenditures are equated with the actual traffic and climatic loading sustained by the pavement. To reinforce the expectation further, consider the case where a treatment costing $H is applied at a road section in year X (Figure 1). It is desired to estimate how much traffic loading can be sustained by this treatment and how much of a fee vehicles should be charged for using the highway segment. For doing this, the time taken for the pavement condition to reach a specified terminal state or the time of the application of the subsequent pavement maintenance treatment (treatment B in year Y), should be known. Knowing the service life of individual treatments, the total traffic and climatic loading over
the treatment service life can be estimated. Using this information for a number of segments within a given jurisdiction, a cost model can be estimated to relate the total cost to traffic and other factors.

**M&R Cost Estimation using Estimated Service Life of M&R Treatments**

Studies using field data suffer from the disadvantage of the lack of quality data or its availability. Data for individual M&R treatments are typically available through highway agency databases as it is relatively easy to obtain the information on the treatment application years and the agency cost of the treatments. However, information on the time of application of the next similar or higher treatment typically is unavailable, often because many of such treatments have not yet reached their service lives. In such a scenario, an appropriate approach is to extrapolate the available performance data to a point where the pavement condition reaches a threshold value.

To illustrate this point, consider a pavement segment that received rehabilitation intervention at a certain year (Figure 2). Information on the cost of the rehabilitation treatment applied is available; however, the pavement service life can only be obtained if its performance is extrapolated until the application of the next treatment or it reaches threshold condition. In order to derive a fair estimate of service life, the use and extrapolation of performance models is a viable option. Using a performance model, the treatment service life can be estimated, which can then be used as a basis to estimate the total traffic and climatic loading sustained by the pavement over its service life. Figure 4 presents a scenario where complete pavement condition data are not available.

**DATA COLLECTION AND COLLATION**

For the present study, maintenance and rehabilitation data for state of Indiana from 1994 to 2006 were used for model estimation. Data on the road inventory, pavement condition, contracts, and traffic and climate were collected from contract database of Indiana Department of Transportation (INDOT), INDOT’s pavement management system, INDOT’s traffic monitoring section and INDIPAVE-2000 and the National Oceanic and Atmospheric Administration (NOAA) database (NOAA, 1995; Labi, 2001; INDOT, 2011). The data on the treatment service life estimates of some commonly-used M&R treatments were obtained from past Indiana studies (Ahmed A., 2012).

For flexible pavements, the rehabilitation and periodic maintenance treatment include microsurfacing, thin Hot Mix Asphalt (HMA) overlay, functional HMA overlay, structural HMA overlay, resurfacing partial 3R standards, and mill asphalt concrete and bituminous overlay. The rigid pavement maintenance treatment includes rubblize Portland cement concrete (PCC) pavement, HMA overlay, crack-and-seat PCC, and HMA overlay. The cost data were converted into cost per lane-
mile for all of the contracts in 2010 constant $ using highway consumer price indices (CPI) (Sinha and Labi, 2007). The total traffic and climatic loading sustained by the pavement during one complete life cycle of M&R treatment were estimated on the basis of the service lives of the individual treatments.

**MODEL ESTIMATION AND RESULTS**

**OLS Models for M&R Marginal Cost Estimation using Estimated Service Life**

It may be recalled that the main objective of this paper is to highlight the issues associated with pavement M&R marginal cost estimation using field data. With that objective in mind, an OLS model was estimated that used the total M&R expenditure per-lane mile as its dependent variable. The model was estimated using LIMDEP (statistical software package) (Greene, 2007). A number of functional forms were investigated and the best estimated model is discussed.

The estimated model (Table 1) has a reasonable fit for the rather highly varied data that were collected over several years (1994-2006). To check the possible violations of the OLS regression assumptions, the plot of the residual vs. fitted values of the OLS model were examined (Washington et al., 2010). From this plot, there was no evidence of any distinct trend between the observed and fitted values, and thus it was concluded that it is appropriate to specify a linear relationship between the response and explanatory variables. However, the model result did reveal that the disturbances were correlated, which indicates a possible violation of OLS assumptions. As per Washington et al. (2010), a Durbin-Watson statistic close to 2.0 should be used to establish that the disturbances are uncorrelated. However, the developed model has a Durbin-Watson statistic of 1.74, suggesting that the error terms may be correlated; thus, the estimated model violates the OLS assumptions.

**TABLE 1: OLS Model Results Based on Estimated Treatment Service Life**

<table>
<thead>
<tr>
<th>Variable</th>
<th>Coef.</th>
<th>t-stat</th>
</tr>
</thead>
<tbody>
<tr>
<td>Constant</td>
<td>269,240</td>
<td>8.793</td>
</tr>
<tr>
<td>Total Traffic (Total ESALs over treatment service life)</td>
<td>0.0030</td>
<td>2.914</td>
</tr>
<tr>
<td>Microsurfacing indicator variable (1 if treatment is microsurfacing, 0 otherwise)</td>
<td>-33,419</td>
<td>-2.61</td>
</tr>
<tr>
<td>Pavement surface indicator variable (1 if pavement surface is flexible, 0 otherwise)</td>
<td>-218,659</td>
<td>-18.17</td>
</tr>
<tr>
<td>Total precipitation over treatment service life (inches)</td>
<td>154.34</td>
<td>4.31</td>
</tr>
<tr>
<td>Annual average freeze index (degree-days)</td>
<td>47.54</td>
<td>4.15</td>
</tr>
<tr>
<td>District indicator variable (1 if district is Vincennes, 0 otherwise)</td>
<td>13,896</td>
<td>1.94</td>
</tr>
<tr>
<td>District-US route indicator variable (1 if US route is from Fort Wayne district, 0 otherwise)</td>
<td>27,397</td>
<td>5.177</td>
</tr>
<tr>
<td>Functional HMA overlay indicator variable (1 if treatment is Functional HMA overlay, 0 otherwise)</td>
<td>22,309</td>
<td>4.14</td>
</tr>
<tr>
<td>R2</td>
<td></td>
<td>0.55</td>
</tr>
<tr>
<td>Adjusted R2</td>
<td>0.54</td>
<td></td>
</tr>
<tr>
<td>Durbin-Watson Stat</td>
<td>1.735</td>
<td></td>
</tr>
<tr>
<td>Number of observations</td>
<td>508</td>
<td></td>
</tr>
</tbody>
</table>

The presence of correlation in the error terms suggests that there is some additional information in the data, which have not been completely explained in the model estimation. Correlation of the error terms across adjacent time periods is referred to as serial correlation or auto correlation (Washington et al., 2010). Serial correlation may occur due to omitted variables or correlation over time. Serial correlation leads to OLS estimates that are inefficient, but which are unbiased and consistent (Washington et al., 2010). In the presence of serial correlation, the OLS estimates are no longer efficient; however, they still remain linear and unbiased. Since the model observations are from different time periods (1994-2006), there might be some spending patterns that are causing correlation over time. For the estimated model in this study, auto-correlation correction was applied and a new model was estimated (Table 2). The model results show that by applying auto-correlation correction, the Durbin-Watson statistic improved to 2.028.

The model results (Table 2) showed that the significant variables are traffic loading (total ESALs over the treatment service life), total precipitation, annual average freeze index, and indicator variables for microsurfacing, pavement surface type, Vincennes district, US route in Fort Wayne district, and functional HMA overlay.
TABLE 2: OLS Model Results with Auto-correlation Correction

<table>
<thead>
<tr>
<th>Variable</th>
<th>Coeff.</th>
<th>t-stat</th>
</tr>
</thead>
<tbody>
<tr>
<td>Constant</td>
<td>261,832</td>
<td>8.72</td>
</tr>
<tr>
<td>Total Traffic (Total ESALs over treatment service life)</td>
<td>0.0029</td>
<td>2.86</td>
</tr>
<tr>
<td>Microsurfacing indicator variable (1 if treatment is microsurfacing, 0 otherwise)</td>
<td>-36,307</td>
<td>-2.84</td>
</tr>
<tr>
<td>Pavement surface indicator variable (1 if pavement surface is flexible, 0 otherwise)</td>
<td>-270,280</td>
<td>-17.45</td>
</tr>
<tr>
<td>Total precipitation over treatment service life (inches)</td>
<td>147.05</td>
<td>4.16</td>
</tr>
<tr>
<td>Annual average freeze index (degree-days)</td>
<td>50.11</td>
<td>3.19</td>
</tr>
<tr>
<td>District indicator variable (1 if district is Vincennes, 0 otherwise)</td>
<td>15,636</td>
<td>2.20</td>
</tr>
<tr>
<td>District-US route indicator variable (1 if US route is from Fort Wayne, 0 otherwise)</td>
<td>24,520</td>
<td>4.68</td>
</tr>
<tr>
<td>Functional HMA overlay indicator variable (1 if treatment is Functional HMA overlay, 0 otherwise)</td>
<td>22,083</td>
<td>4.14</td>
</tr>
<tr>
<td>R²</td>
<td>0.550</td>
<td></td>
</tr>
<tr>
<td>Adjusted R²</td>
<td>0.543</td>
<td></td>
</tr>
<tr>
<td>Durbin-Watson Stat</td>
<td>2.028</td>
<td></td>
</tr>
<tr>
<td>Number of observations</td>
<td>508</td>
<td></td>
</tr>
</tbody>
</table>

Model results revealed that total ESALs over the treatment service life, total precipitation and annual average freeze index have a direct relationship with the total M&R expenditures during the treatment service life, that are all intuitive. The positive sign of traffic loading indicates that the pavement segments with higher traffic loadings are associated with increased pavement M&R expenditure. Similarly, pavements that are subjected to higher precipitation or higher annual average freeze index are likely to deteriorate faster and thus incur M&R expenditures. These findings are also consistent with past studies (Khurshid, 2010). The model results indicate that those pavement segments that received microsurfacing were associated with a lower M&R cost per lane-mile, which is intuitive. It was revealed that flexible pavements are associated with lower M&R expenditure, which is not consistent with past studies. The counter-intuitive results might be due to conservative service lives of rigid pavement rehabilitation treatment used for model estimation. The indicator variable for Vincennes district and US routes in Fort Wayne indicates district specific characteristics which might be the result of specific policies or the local maintenance practices at such locations.

**OLS Models for M&R Marginal Cost Estimation using Observed Service Life**

In this approach, the models were estimated for flexible pavements only using the actual service lives of individual treatments. The observed actual service life was obtained by tracking the pavement condition (IRI) between consecutive rehabilitations, or between a rehabilitation and a period maintenance, or between a rehabilitation/periodic maintenance and reconstruction. Similar to the case of estimated service life, an OLS model was estimated using the total M&R expenditure per lane-mile as a dependent variable. A number of functional forms were investigated and the best estimated (Table 3) model is presented and discussed in the ensuing paragraphs.

The positive sign of traffic loading (total ESALs over the treatment service life) indicates that pavement segments with higher traffic loadings are associated with increased pavement M&R expenditure. The model results also indicate that a higher annual average freeze index results in higher total M&R expenditures during treatment service life, which is consistent with previous studies (Khurshid, 2010), indicating that pavement segments located in high freeze-index zones are likely to deteriorate faster, thus resulting in higher expenditure. The indicator variable for state roads in Greenfield district indicates district specific characteristics which might be the result of specific policies or local maintenance practices at such locations. The positive sign of the indicator variables for the southern districts of Indiana (Seymour and Vincennes) revealed that these two districts have higher M&R expenditures. These two districts have the highest precipitation rate in Indiana (CIWRP, 2003; Volovski, 2011). Higher precipitation can result in faster pavement deterioration and higher maintenance and rehabilitation expenditure, which is quite logical. The model results indicate that those pavement segments that received microsurfacing are associated with a lower cost per lane-mile, which is intuitive. This indicates a treatment-specific characteristic indicating the higher suitability/effectiveness of microsurfacing. Lastly, the model revealed that those pavement segments which received maintenance or rehabilitation treatment before 2002 had less effective results.
TABLE 3: OLS Model Results Based on Observed Treatment Service Life

<table>
<thead>
<tr>
<th>Variable</th>
<th>Coeff.</th>
<th>t-stat</th>
</tr>
</thead>
<tbody>
<tr>
<td>Total Traffic (Total ESALs over treatment service life)</td>
<td>0.001</td>
<td>2.554</td>
</tr>
<tr>
<td>Annual average freeze index (degree-days)</td>
<td>283.99</td>
<td>6.939</td>
</tr>
<tr>
<td>District-State Route (SR) route indicator (1 if maintenance treatment is from state route of Greenfield district, 0 otherwise)</td>
<td>-154,255</td>
<td>-6.967</td>
</tr>
<tr>
<td>Southern districts indicator variable (1 if maintenance treatment is from Seymour or Vincennes districts, 0 otherwise)</td>
<td>70,963</td>
<td>2.919</td>
</tr>
<tr>
<td>Microsurfacing indicator variable (1 if treatment is microsurfacing, 0 otherwise)</td>
<td>-179,935</td>
<td>-1.961</td>
</tr>
<tr>
<td>Treatment application year indicator variable (1 if treatment was applied before 2002, 0 otherwise)</td>
<td>62,399</td>
<td>2.017</td>
</tr>
<tr>
<td>R2</td>
<td>0.41</td>
<td>1.94</td>
</tr>
<tr>
<td>Adjusted R2</td>
<td>0.40</td>
<td>5.177</td>
</tr>
<tr>
<td>Durbin-Watson Stat</td>
<td>0.634</td>
<td>4.14</td>
</tr>
<tr>
<td>Number of observations</td>
<td>194</td>
<td></td>
</tr>
</tbody>
</table>

Model findings are all intuitive and results can be reasonably used to estimate the M&R marginal cost. However, the model results revealed that the disturbances are correlated, which is a possible violation of the OLS assumptions. The developed model has a Durbin-Watson statistic of 0.634, indicating that it is highly likely that the error terms are correlated, thus the estimated models violated the OLS assumptions. For the estimated model (Table 3), auto-correlation correction was applied but Durban-Watson statistic could not be improved to desired level. This indicates that data used for model estimation (M&R data for 194 pavement segments) is inadequate or there is some additional information in the data, which have not been completely explained in the model estimation. This finding leads to the conclusion that other model specifications should be investigated, or other methodologies for M&R marginal cost estimation should be used that do not encounter such issues.

CONCLUSION

This paper highlighted the issues associated with MPDC estimation using field data. It was shown that it is not possible to estimate MPDC with inadequate field data because MPDC requires a complete picture of the M&R profile between consecutive reconstructions. However, it is duly recognized that the latter is a challenging task for most highway agencies due to the lack of past records. It was highlighted that the true MPDC can only be estimated when the actual expenditure (pavement maintenance, rehabilitation, and reconstruction) is equated/compared against the actual traffic and climatic loading sustained by the pavement. However, it was also demonstrated that this issue can be partially resolved, either by using the estimated or observed treatment service life, and M&R marginal cost (not the marginal cost of pavement damage) can be estimated. The M&R marginal cost was estimated by differentiating the estimated model (M&R cost function) with respect to the road-use variable. The M&R marginal cost was estimated as 0.003$ per ESAL-mile (0.0019$ per ESAL-Km).

Using data related to the estimated treatment service life for 508 flexible and rigid pavements, OLS regression models were developed. Similarly, OLS regression models were also developed by using data related to the observed treatment service life for 194 flexible pavements. The factors that were found to significantly influence the M&R marginal cost included traffic loading (total ESALs over the treatment service life), annual average freeze index, total precipitation, pavement surface type, geographical location (district), route type, and treatment type. It was observed that M&R marginal cost increases with increase in traffic loading, total precipitation and annual average freeze index over treatment service life. Thus, the pavement segments with higher traffic loadings, and/or located in higher precipitation and freeze index zones deteriorate faster and thus incurring higher M&R expenditures. Similarly, differences in effectiveness of individual treatments were also observed in the study indicating a treatment-specific characteristic.
REFERENCES


Hierarchical Markov Chain Monte Carlo for Pavement Roughness Estimation

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ABSTRACT
Traditionally, pavement roughness has been modeled to mimic changing characteristics across time periods. Due to uncertainty in roughness at different times, modeling these changes is challenging and can generate models that are unable to reflect true pavement conditions. This research seeks to simulate roughness of road pavements in Kansas using hierarchical Markov Chain Monte Carlo (MCMC) methods. The aim is to investigate how efficient this technique will be at estimating pavement roughness without neglecting inherent uncertainty. Using a nineteen year time span, a hierarchical MCMC model was used to simulate a history of pavement roughness. Estimated roughness values were comparable to original roughness values and generally followed similar deterioration trends. This simulation technique can be incorporated into pavement management systems and used as a check when making maintenance decisions about the level of roughness on a given road network.

INTRODUCTION
Pavement performance defines the adequacy of a pavement’s functional and structural service over a specified design period, and can be expressed in terms of distresses such as rutting, cracking and roughness. Of these, pavement roughness is one key indicator, which influences the acceptability of service provided by the roadway. The American Society of Testing and Materials (ASTM) defines pavement roughness as the deviation from a true planar surface with characteristic dimensions that affects vehicle dynamics, ride quality, dynamic loads, and drainage (1). Notably, roughness affects driving comfort, vehicle operating costs and safety. In most agencies, roughness is used as a benchmark in setting performance criteria in both design and management. Other agencies use roughness as a measure of serviceability or a primary indicator of serviceability. The International Roughness Index (IRI) is a standard measurement scale that is accepted widely in evaluating pavement roughness. The index unit for IRI is inch/mile.

Traditionally, roughness is modeled using either deterministic or probabilistic models. Deterministic models employ statistical methods such as regression analysis to relate roughness to other variables like pavement age, environment, pavement structural strength and traffic loading. Paterson (2) developed a deterministic model that combined structural effects, surface defects and environmental-age-condition influences to predict roughness progression. Concerns have however, been raised about the selection of explanatory variables for deterministic models and their inability at times to depict uncertainty (3).

Probabilistic models include Markov processes and survivor curves. Other probabilistic models use the Bayesian approach to model pavement performance. The Bayesian approach offers the flexibility to incorporate existing knowledge so that previous experience can be utilized rather than ignored (4). In addition, obtaining the probabilistic distribution of the parameters to reflect the performance heterogeneity is straightforward, and the output is a density function, which can provide comprehensive statistics of the individual parameters (5).

Other models such as back-propagation networks, functional networks and adaptive neural networks and back-propagation neural networks have also been developed to model pavement roughness (6; 7).

Modeling pavement roughness is characterized by uncertainty across different time periods. Incorporating uncertainty is thus fundamental to the modeling process as it affects how road conditions change continuously with corresponding time change. This is however challenging as models depicting uncertainty can sometimes produce unrealistic estimates due to the assumptions and initial boundary conditions used for the models. As such road administrators and other stakeholders are always on the lookout for models that produce genuine estimates without excluding uncertainty in their formulation so as to aid in decision-making.

This study hopes to add to the field of knowledge by examining the use of hierarchical Markov Chain Monte Carlo methods for simulating roughness.
Carlo (MCMC) simulation in estimating pavement roughness. Hierarchical MCMC models use Bayesian approach in their estimation process, which allows them to account for uncertainty in pavement roughness. They easily lend themselves to validation and can also be examined to see if they reflect roughness conditions on a specified length of roadway or a given network of roads. The research aims to simulate pavement history and compare simulated roughness values to values obtained from the field. If successful, this purely statistical method has the capacity to check for outliers in a dataset or provide estimates for missing data points for any pavement management system.

BAYESIAN INFERENCE

The Bayesian inference is based on Bayes’ Theorem and is described mathematically as follows: Assuming $y$ is a vector or matrix of data and $\theta$ is a vector or matrix that contains parameters that describe $y$, then

$$f(\theta|y) = \frac{f(y|\theta)f(\theta)}{f(y)} \propto f(y|\theta)f(\theta)$$

(1)

Where $f(\theta|y)$ is the posterior distribution, $f(y|\theta)$ is the likelihood and $f(\theta)$ is the prior distribution. In Bayesian inference, $\theta$ is considered to be a quantity whose variation can be described by its probability distribution $f(\theta)$. $f(\theta)$ is a subjective description based on the experimenter’s belief and is formulated before seeing the data. The likelihood is the distribution of the data conditional on the parameters. It is the data generating process. The posterior distribution is of fundamental interest. It summarizes all knowledge about $\theta$ after seeing the data.

MARKOV CHAIN MONTE CARLO (MCMC) SIMULATION

Markov Chain Monte Carlo (MCMC) methods are simulation techniques through which posterior distributions can be obtained accurately by specifying the prior and likelihood distributions. The MCMC is an iterative process that is based on the construction of a Markov chain, which eventually “converges” to a stationary, posterior distribution. Unlike direct simulation methods, the MCMC output is a dependent sample generated from a Markov chain (8).

A Markov chain is a stochastic process $\{\theta^{(1)}, \theta^{(2)}, ..., \theta^{(T)}\}$ such that

$$f(\theta^{(t+1)}|\theta^{(t)}, .., \theta^{(1)}) = f(\theta^{(t+1)}|\theta^{(t)})$$

(2)

that is, the distribution of parameter $\theta$ at sequence $t + 1$ given all the preceding $\theta$ values (for times $t, t - 1, ..., 1$) depends only on the value $\theta^{(t)}$ on the previous sequence $t$.

Gibbs sampling is an MCMC technique that is utilized in this paper. Gibbs sampling is a Markovian updating scheme (9). Sampling for each variable is done from a conditional distribution where all other variables are considered known and are given the values of the previous state of the chain.

The process of the algorithm used in the Gibbs sampling is described as follows (5): for a set of random variables $\theta_1, \theta_2, ..., \theta_m$, the joint distribution is denoted as $f(\theta_1, \theta_2, ..., \theta_m)$. With given arbitrary starting values of $\theta$’s, say $\theta_1^{(0)}, \theta_2^{(0)}, ..., \theta_m^{(0)}$ the first iteration of random draws of $\theta$’s is obtained as

$$\theta_1^{(1)} \text{ from } f(\theta_1| \theta_2^{(0)}, \theta_3^{(0)}, ..., \theta_m^{(0)})$$
$$\theta_2^{(1)} \text{ from } f(\theta_2| \theta_1^{(1)}, \theta_3^{(0)}, ..., \theta_m^{(0)})$$
$$\vdots$$
$$\theta_m^{(1)} \text{ from } f(\theta_m| \theta_1^{(1)}, \theta_2^{(1)}, ..., \theta_{m-1}^{(1)})$$

(3)

In a similar manner, the second set of random draws of $\theta$’s is obtained through the update process. After iterations, the series of $\theta$’s is obtained as $(\theta_1^{(r)}, \theta_2^{(r)}, ..., \theta_m^{(r)})$. It is shown that under mild conditions for each variable $\theta_i^{(r)} \rightarrow \theta_i \sim f(\theta_i)$ as $r \rightarrow \infty$ (10), which means that after enough iterations, $r$, $\theta_i^{(r)}$ can be regarded as a random draw from the distribution of $f(\theta_i)$.

HIERARCHICAL MODELS

Hierarchical models are an ordered set of sequential statements of conditional relationships whose estimation process is based on the Bayesian approach. Bayesian models portray an attribute of hierarchy because of the conditional structure of the posterior distribution, which can be decomposed to the data likelihood multiplied by the prior distribution. Given a prior distribution $f(\theta|a)$ of the model parameters $\theta$, prior parameters $a$ can be considered as one level of hierarchy and the likelihood as the final stage of a Bayesian model (8). The resulting posterior distribution is

$$f(\theta|y) \propto f(y|\theta)f(\theta|a)$$

(4)

In other complex cases, hierarchical stages of the prior distribution can be used to generate the posterior distribution. An example of this is a two level hierarchical model where for parameters $\theta$, $a$ and $b$ the first level is characterized by $f(\theta|a)$ and the second level is characterized by $f(a|b)$. The distribution $f(a|b)$ is identified as the hyperprior and $b$ are said to be the hyperparameters of the prior parameter $a$ (8). The posterior distribution $f(\theta|y)$ is represented as
\[ f(\theta| y) \propto f(y| \theta) \cdot f(\theta| a) \cdot f(a| b) \]

MODEL DEVELOPMENT

Data used for estimation was obtained from annual roughness of road pavements in Kansas. It spanned a period of 19 years, from 1989 to 2007. IRI values for 30 individual and independent pavement sections were provided. Two of these sections, Sections I and L were randomly selected for this paper and are shown in Table 1.

<table>
<thead>
<tr>
<th>Year</th>
<th>Section I (in/mile)</th>
<th>Section L</th>
</tr>
</thead>
<tbody>
<tr>
<td>1989</td>
<td>155</td>
<td>119</td>
</tr>
<tr>
<td>1990</td>
<td>112</td>
<td>84</td>
</tr>
<tr>
<td>1991</td>
<td>126</td>
<td>114</td>
</tr>
<tr>
<td>1992</td>
<td>115</td>
<td>96</td>
</tr>
<tr>
<td>1993</td>
<td>121</td>
<td>88</td>
</tr>
<tr>
<td>1994</td>
<td>120</td>
<td>83</td>
</tr>
<tr>
<td>1995</td>
<td>120</td>
<td>90</td>
</tr>
<tr>
<td>1996</td>
<td>116</td>
<td>85</td>
</tr>
<tr>
<td>1997</td>
<td>119</td>
<td>91</td>
</tr>
<tr>
<td>1998</td>
<td>120</td>
<td>94</td>
</tr>
<tr>
<td>1999</td>
<td>123</td>
<td>97</td>
</tr>
<tr>
<td>2000</td>
<td>127</td>
<td>98</td>
</tr>
<tr>
<td>2001</td>
<td>75</td>
<td>66</td>
</tr>
<tr>
<td>2002</td>
<td>79</td>
<td>67</td>
</tr>
<tr>
<td>2003</td>
<td>79</td>
<td>70</td>
</tr>
<tr>
<td>2004</td>
<td>78</td>
<td>73</td>
</tr>
<tr>
<td>2005</td>
<td>82</td>
<td>71</td>
</tr>
<tr>
<td>2006</td>
<td>88</td>
<td>76</td>
</tr>
<tr>
<td>2007</td>
<td>96</td>
<td>113</td>
</tr>
</tbody>
</table>

Time series plots were used to observe changes in pavement condition with time. Figure 1 shows how roughness progressed annually along Sections I and L. The plots depict changes in the observed IRI values for these two sections and display how uncertain these values are from one year to the next. For both pavement sections, drastic increases in roughness occur in 1990 and 2001. Section L also had a noticeable increase in 2007. These increases in pavement deterioration were investigated and found not to be errors and thus have to be adequately captured in any modeling process.

For Section I, two major rehabilitation efforts occurred in 1990 and 2001. Apart from these, the plot followed a general cycle of incremental increase and decrease in pavement roughness. A stable trend in IRI values is generally seen from year 1992 to 2000. The gentle slope from year 2001 to 2007 is indicative of a gradual increase in roughness for this time window. Section L underwent similar rehabilitation efforts in 1990 and 2001. Rehabilitation in 1990 did not last and significant deterioration in pavement roughness occurred in 1991. Rehabilitation on this same section was undertaken in 2001 after which roughness grew incrementally for 5 years. All other points depict a general cycle of deterioration and maintenance associated with any road pavement. Consequently, the roughness estimation model must attempt to model these changes without ignoring uncertainty exhibited at these points.

Figure 2 shows normal probability plots for Sections I and L respectively. The closer the points lie to the diagonal line, the better the normal distribution fits the observed data. The plots suggest IRI values for these 2 sections are normally distributed.
Normal hierarchical Markov Chain Monte Carlo (MCMC) model was used in estimating and predicting parameters for the sections. A diagrammatic representation of the normal hierarchical model is shown in Figure 3.

**FIGURE 3: Normal Hierarchical Model**

\[
p(y, \theta, \mu, \tau^2) = \prod_{j=1}^{n_j} N(y_j | \theta_j, \tau^2_j) \prod_{j=1}^{n_j} N(\theta_j | \mu, \tau^2)
\]

With \(\sigma^2 = \sigma^2 / n_j\) known, the model is

\[
 f(y | \theta) \sim N(\mu, \tau^2) \quad \text{and} \quad \tau = 1/\sigma^2
\]

where \(\mu\) is the mean, \(\tau\) is precision and \(\sigma^2\) is the variance.

WinBUGS 1.4.3 was the computational software used to develop normal hierarchical models for the data. WinBUGS is a Bayesian analysis software that uses Markov Chain Monte Carlo (MCMC) to fit statistical data.

**ESTIMATING IRI VALUES OF SECTIONS**

Estimation was done to determine the accuracy of the normal hierarchical model. Five arbitrary years for each pavement section were selected randomly and their IRI values were excluded from the model. The model was then run using the remaining fourteen IRI values and in the process new IRI values were replicated at various percentiles for all years including the absentee years. The results for the absentee years for Sections I and L are shown in Tables 2 and 3. Tables 2 and 3 show that generated IRI values are in close proximity to the observed IRI values from the field. Section I had 4 out of the 5 observed IRI field values within the 95% confidence band whilst Section L had 3 observed field values with its 95% confidence band. All other values for each of the sections just fell short of their respective confidence range.

**TABLE 2: Results for Section I**

<table>
<thead>
<tr>
<th>Year</th>
<th>Actual IRI (after run)</th>
<th>Estimated IRI 2.5 percentile</th>
<th>Estimated IRI 50 percentile</th>
<th>Estimated IRI 97.5 percentile</th>
</tr>
</thead>
<tbody>
<tr>
<td>1992</td>
<td>115</td>
<td>113.9</td>
<td>126.6</td>
<td>139.2</td>
</tr>
<tr>
<td>1996</td>
<td>116</td>
<td>104.7</td>
<td>114.2</td>
<td>123.5</td>
</tr>
<tr>
<td>1999</td>
<td>123</td>
<td>95.27</td>
<td>104.8</td>
<td>114.2</td>
</tr>
<tr>
<td>2003</td>
<td>79</td>
<td>79.54</td>
<td>92.3</td>
<td>104.9</td>
</tr>
<tr>
<td>2006</td>
<td>88</td>
<td>66.29</td>
<td>83</td>
<td>99.12</td>
</tr>
</tbody>
</table>

**TABLE 3: Results for Section L**

<table>
<thead>
<tr>
<th>Year</th>
<th>Actual IRI (after run)</th>
<th>Estimated IRI 2.5 percentile</th>
<th>Estimated IRI 50 percentile</th>
<th>Estimated IRI 97.5 percentile</th>
</tr>
</thead>
<tbody>
<tr>
<td>1991</td>
<td>114</td>
<td>80.73</td>
<td>94.92</td>
<td>108.9</td>
</tr>
<tr>
<td>1994</td>
<td>83</td>
<td>80.53</td>
<td>91.6</td>
<td>102.5</td>
</tr>
<tr>
<td>1997</td>
<td>91</td>
<td>79.09</td>
<td>88.32</td>
<td>97.31</td>
</tr>
<tr>
<td>2000</td>
<td>98</td>
<td>75.61</td>
<td>84.99</td>
<td>94.18</td>
</tr>
<tr>
<td>2004</td>
<td>73</td>
<td>67.96</td>
<td>80.52</td>
<td>93.17</td>
</tr>
</tbody>
</table>

Plots of all replicated IRI values for individual sections I and L are shown in Figure 4.

Section I. For Section L, IRI for 2007 fell completely out of the 95% confidence interval.

**FIGURE 4: Plot of observed versus actual IRI values.**
Only 3 out of the 19 field values fell markedly out of the 95% confidence envelope for Section I. For Section L, IRI for 2007 fell completely out of the 95% confidence interval.

Sample nodes for each year were also monitored during the run. Figure 5 shows density plots of some monitored nodes for Section I that were used in the estimation process.

**DISCUSSION OF RESULTS**

The normal hierarchical MCMC model that was developed was validated based on its ability to model uncertainty without ignoring roughness conditions on the roadway. Using a confidence interval of 95%, estimated IRI values for Sections I and L from the fitted model corresponded strongly to observed IRI values from the dataset. Approximately 80% of observed IRI values fell within the confidence intervals for both sections. Points not captured were largely due to the immense magnitude of rehabilitation or deterioration that occurred within the corresponding time interval. This is seen from years 2000 to 2001 for both pavement sections and in year 2007 for Section L.

For the most part, the models captured a significant proportion of the distribution of roughness for the individual pavement sections. This statistical procedure could serve as a useful tool for Departments of Transportation that have huge datasets of IRI values to analyze as part of their pavement management systems. States collect data from their road networks annually and such data usually contain missing values or suspected outliers. To make sound planning and budgetary decisions these outliers or missing values must be investigated and the hierarchical MCMC models as used in this study represent a way by which this could be done. Once the model is run and sound estimates are produced, the model can then be used as a platform to predict roughness conditions for future years.

**CONCLUSION**

This paper used hierarchical Markov Chain Monte Carlo models in estimating IRI values for a given data set. The data used was annual roughness for Kansas. The model used the Gibbs sampler with MCMC simulation, and was able to reflect prevailing roughness conditions without neglecting uncertainty in road condition. This feature enabled the model to estimate and verify IRI values obtained from the field. This would ultimately help stakeholders during the use of data for maintenance scheduling and resource allocation for highway infrastructure. Furthermore, this estimation process can be extended to analyze larger datasets and can be applied to assessing the performance of other civil infrastructure.
REFERENCES


2. Paterson W. 1989. A transferable causal model for predicting roughness progression in flexible pavements. Transportation Research Record: Journal of the Transportation Research Board, No. 1215, 70-84


Application of Solar Heat Blocking Pavement for Improvement of Durability

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ABSTRACT
High surface temperatures adversely affect the longevity of pavement surfaces by accelerating the pace of rutting, aging and fatigue. Reducing the surface temperature is increasingly important in terms of durability and environmental concerns. In order to tackle the problem from a paving perspective, solar heat blocking pavement technology has been developed to achieve a reduction in surface temperature and increased pavement longevity. This paper describes the practical effects of solar heat blocking pavement technology through its development and case studies. With regard to the temperature, field results show that the reduction in surface temperature through use of the solar heat blocking pavement is approximately 16°C. In terms of durability, the laboratory test results show that the solar heat blocking pavement has an advantage in aggregate pop-out on the surface. Finally, its application to airport taxiways reveals that this technology effectively curtails rutting by half compared to dense graded asphalt surfaces.

INTRODUCTION
There is growing awareness in recent years that temperatures have been trending upward due to global warming and climate change. The problem has become more pronounced, especially in Japan, with the surface of asphalt pavements reaching 60°C or more during summer months setting new records and presenting serious challenges. Asphalt pavements cover approximately 20% of urban areas and are considered to be a prime factor in the urban heat island phenomenon, which significantly affects the thermal comfort of pedestrians. Past studies (1) suggest that asphalt pavement absorbs solar rays, including invisible solar radiation. According to Yoder and Witzak (2) rising temperatures from the paving surface affect the properties of the surface layer. Therefore a reduction in the surface temperature has become increasingly important in terms of sustainability as well as the environment.

Pomerantz et al. (3,4) indicated that making urban surfaces whiter to reflect both visible and infrared rays is the most practical way of mitigating surface heat. However, out of consideration for driver’s vision, darker surfaces would be preferred as they are more familiar. In addition, taking into account the workability on the existing pavement, surface treatment with a painted coating is one way to deal with the issue on the existing surface. For these reasons, a new surface treatment technology called ‘solar heat blocking pavement’, which aims to reduce surface temperatures, has been developed in Japan.

SOLAR HEAT BLOCKING PAVEMENT

Basic Concept
Solar reflective technology was originally developed to mitigate the rising temperature of building rooftops (5). Pomerantz et al. (3) suggested that sealing a reflective material onto the surface layer contributes to both a reduction in surface temperature and mitigation of surface damage due to ultraviolet rays. As a result, a reduction in pavement surface temperature during summer months was expected by applying this technology.
The function of this technology is based on the higher reflectivity of near infrared rays and lower reflectivity of visible rays. The reflectivity of solar rays and infrared rays are represented by albedo (6). Albedo is defined as the ratio between incoming and reflected solar rays. A higher albedo means that the surface layer has a higher reflectivity for infrared rays whereas a lower albedo indicates that infrared rays are absorbed into the surface layer increasing the surface temperature. Paint based materials with a higher albedo were coated onto the existing paving to prevent the surface layer from absorbing infrared rays as shown in Figure 1.

In order to examine the albedo characteristics a comparison was made between three surfaces: solar heat blocking pavement, conventional pavement (dense graded asphalt pavement) and normal paint material. The tests were conducted in accordance with the Japanese standard JIS A 5759 (7). In this case, the color for both the solar heat blocking pavement and the normal paint are grey, whilst that of the conventional pavement is black.

Figure 2 shows the comparison results. As shown in the figure, there are clear differences between the solar heat blocking pavement and conventional painting materials. Conventional painting material has almost the same or a less reflective ratio across the entire wavelength. However, in the case of the solar heat blocking pavement, the reflective rate for near infrared rays in the wavelength is much higher than others. This indicates that the solar heat blocking pavement has higher albedo, despite the fact that the normal paint material is the same color as the heat blocking pavement.

**Structure Of The Coating Layer And Construction Method**

The coating layer is about 1.0 mm in thickness. The coating layer consists of three components: primer layer, second layer and non-skid sand shown in Figure 3. First the prime layer is applied to cover the existing surface, then the non-skid sand is sprayed on immediately after the primary coating to ensure skid resistance, and lastly the second layer is applied as the grey surface which will sandwich the non-skid sand. After curing for only an hour the site can be reopened to traffic. The coating and spraying work is normally performed with a specialist spray gun. The material specification generally used for the pavement is as follows:

- Size of non-skid sand particles: 0.5 ~ 1.7 mm
- Density of non-skid sand: 0.5 kg/m²
- Density of coating: 0.4 ~ 0.6 kg/m² in each layer

**Skid resistance**

Skid resistance is a factor affecting the serviceability of paving and is also closely related to traffic accidents, especially on rainy days. Therefore, non-skid sand is sprayed on immediately after coating with the solar heat blocking material. For confirmation purposes the skid
resistance of both coated porous asphalt and uncoated porous asphalt surfaces were measured using a dynamic friction tester. The results are shown in Table 1.

<table>
<thead>
<tr>
<th>Dynamic Friction (μ)</th>
<th>Test Speed</th>
<th>Porous Asphalt Paving</th>
<th>Solar Heat-Blocking Pavement</th>
</tr>
</thead>
<tbody>
<tr>
<td>40 km/h</td>
<td>0.60</td>
<td>0.62</td>
<td></td>
</tr>
<tr>
<td>60 km/h</td>
<td>0.55</td>
<td>0.56</td>
<td></td>
</tr>
</tbody>
</table>

Tests were conducted as per ASTM E1911 (8). From this result, it was found that the skid resistance values of the two surfaces are almost identical. Therefore, it can be said that an appropriate amount of sand needs to be sprayed on to ensure adequate skid resistance.

**Stripping resistance**

Considering the service state, the coating layer of solar heat blocking pavement should be strong enough and have adequate bonding to the original asphalt surface. In order to investigate these factors, laboratory stripping resistance tests were conducted. A test load was conducted following the laboratory stripping resistance test method for solar heat blocking pavement material (9). A test load was applied to the specimen by turning the tire left and right. A summary of the test parameters is presented below:

- Loading condition: Turning the tire left and right
- Test temperature: 20°C
- Test load: 686 N
- Number of cycles: 650

A digital image of the specimen’s surface was taken with a digital camera after the test. Then the stripping resistance of the solar heat blocking layer is evaluated using computer image analysis, which can record the stripped area of the surface. The results are shown in Table 2 below. The coated layer demonstrates strong adhesion to the existing surface compared to the performance criteria set by one organization. As a result, a longer life cycle can be expected for solar heat blocking pavement.

<table>
<thead>
<tr>
<th>Stripping Area Rate (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Performance Requirement</td>
</tr>
<tr>
<td>Solar Heat Blocking Pavement</td>
</tr>
</tbody>
</table>

**Aggregate Pop-Out**

The bonding between each aggregate is important factor affecting the durability of pavement. Considering asphalt is thermo-plastic material it is vital that asphalt pavement is strong enough for torsional shear stress, especially at curved section on the road. Considering the strong coating layer, solar heat blocking pavement might be a countermeasure for this problem because it not only creates a reduction in surface temperature, but also provides reinforcement of surface layer. In order to confirm this effect laboratory torsional tests were conducted in accordance with Japanese Standard (10). A test load was conducted, following laboratory stripping resistance test method for solar heat blocking pavement material (13). A test load was applied to the specimen by rotating the tire and a summary of the test parameters are presented below:

- Loading duration: 120 minutes
- Test temperature: 50°C
- Test load: 686 N
- Rotation speed: 10.5 rotations per minute

Solar heat blocking pavement is strong enough to torsional shear stress, compared to conventional pavement. Particle loss rate for conventional pavement gradually increased during the test, whereas that of solar heat blocking pavement was almost 0% after the test. The results show that solar heat blocking pavement is highly effective for improvement of surface durability.

**EFFECT OF TEMPERATURE REDUCTION**

In order to examine the effect of temperature reduction, conventional pavement (i.e. dense graded asphalt pavement) and solar heat blocking pavement were compared in the field. The traffic classification of the field is more than 250 and less than 1000 vehicles/day for heavy traffic. Figure 4 below shows the temperature of the two surfaces. It can be seen that the maximum surface temperature of the solar heat blocking pavement was approximately 42°C, whilst that of conventional pavement was around 58°C; the difference between the two surfaces was about 16°C. The results clearly show the advantage of the solar heat blocking pavement in reducing the surface temperature in summer.
Figure 5 compares the surface temperatures of both solar heat blocking pavement and dense graded pavement through a thermographic image at a site. As can be seen from the thermographic image, the surface temperature of the solar heat blocking pavement was 35°C, whilst that of dense graded pavement was 48.3°C. Therefore, it is immediately evident that solar reflective coating can effectively reduce the surface temperature.

**APPLICATION OF AIRPORT TAXIWAY**

**Overview Of The Site**

In terms of reduction in surface temperature, the performance of solar heat blocking pavement was demonstrated in the field. However, bearing in mind its long term performance and further improvements in the technology, it is necessary to examine its durability as well as the surface temperature under severe traffic conditions for prolonged periods. In order to investigate these effects, solar heat blocking pavement was used for the taxiway of an international airport (11, 12). Both rut depth and temperature were examined for conventional dense graded asphalt and solar heat blocking pavements. In addition, thermocouples were set within the pavement (setting depths: 20 mm, 80 mm and 200 mm below the surface) and temperatures were measured and recorded for four years.

**Reduction In Surface Temperature And Rut Depth**

Figure 6 presents the temperature of the two pavements 20 mm below the surface. As can be seen in the figure, the maximum surface temperature of the solar heat blocking pavement was approximately 50°C, whereas that of conventional paving was around 62°C; the difference between the two 20 mm below the surface was about 12°C. The result also shows that this effect can be maintained for four years. The trend is consistent even at different depths. Figure 7 illustrates the reduction in temperature at the three depths. As shown in the figure, there is a tendency for the solar heat blocking pavement to reduce the temperature even at 80 mm and 200 mm below the surface, as compared to the conventional pavement section.

Figure 8 compares the maximum rut depth of the two pavements during the four year monitoring period. As can be seen from the figure, the solar heat blocking pavement can reduce the maximum rut depth by a half, as compared to that of dense graded pavement.

In addition, it should be noted that growth in the rut depth for the solar heat blocking pavement only gradually increases, whereas there is a steady increase.
in rut depth for the dense graded pavement. As a result, a clear difference is evident between solar heat blocking pavement and dense graded pavement, in terms of rut depth.

**FIGURE 8: Changes In Maximum Rut Depth Over Four Years**

CONSTRUCTION RESULTS AND THE FUTURE

In Japan, the total area of solar heat blocking pavement exceeds 500,000 m². Its application in urban areas has been increasing not only for roads, but also in parks, rest areas and parking lots.

Although the solar heat blocking pavement has been widely implemented, the drawback of this technology is cost. The construction cost of the surface is almost the same or slightly more than dense graded pavement. Some clients apply this technology expecting to mitigate urban heat whereas others are deferring its application due to the cost. Indeed, some cannot afford to treat existing surfaces at all because of the recent financial climate.

However, if we consider its advantages, the reduction in surface temperature afforded may prove to be a highly effective means of reducing the total life cycle costs of asphalt pavement. As described above solar heat blocking pavements can reduce rut depth, the coating layer can also serve as surface reinforcement and the cut in surface temperature may also contribute to delaying the aging of asphalt binder. If its effectiveness both environmentally and in terms of life cycle costs is proved, it may lead to increased applications of the technology, which may contribute to further cost reductions. Therefore, further investigations targeting both the environmental effect and life cycle costs need to be conducted for the future.

CONCLUSIONS

This paper presents the practical effects of solar reflective coating on pavement and addresses its potential for improvement of pavement durability. Through laboratory testing and field experiments the following conclusions are drawn:

- With regard to improvement of durability, the solar heat blocking surface contributes to improving the surface layer since the technology provides the surface a reduction in surface temperature and reinforcement of surface layer.
- With respect to the performance of solar heat blocking pavement, the surface temperature is reduced by approximately 16°C compared to conventional dense graded asphalt paving due to the curtailment of solar radiation.
- Solar heat blocking pavement can effectively reduce rutting, as the rate (based on rut depth) was approximately half compared to the dense graded asphalt surface used at an airport taxiway.
REFERENCES


Evaluation of the Internal Structure of Warm Mix Asphalt Using X-Ray Computed Tomography Images

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ABSTRACT
Recent innovations in warm mix asphalt (WMA) pavement engineering produced a reduction of asphalt production temperatures of 20-50 °C and are currently known for economic, technical, and environmental benefits. This paper aims to evaluate the internal structure of WMA mixtures performed by analyzing images obtained using X ray computed tomography. The internal structure analysis was performed in terms of the characteristics of the mixture’s air voids. Corresponding results suggest that WMA mixture specimens exhibited no significant variations in air voids characteristics as compared to conventional HMA mixture specimens. HMA mixture specimens had higher connectivity compared to a WMA mixture. WMA additives did not lead to significant differences in the internal structure of the mixtures as compared to HMA. It is recommended to cut 20 mm from the top and bottom sections of the WMA mixture specimens to obtain more homogeneous vertical distributions of air voids leading to more accurate results.

X-RAY COMPUTED TOMOGRAPHY (XR-CT) AND IMAGE ANALYSIS
The internal structure analysis of asphalt mixtures from XR-CT images can be developed by means of computational processes designed to evaluate the air voids characteristics (i.e., content, size, and connectivity) and aggregates structure characteristics (i.e., orientation, contact, and distribution).

The images used in this research were acquired from the XR-CT equipment at Texas A&M University (Figure 1), which essentially consists of an energy source, a mobile support for the specimens setting, a radiation detector, and a CPU that processes the information and subsequently produces grayscale images. Further information about technical specifications and features of the XR-CT equipment can be found at Shashidhar (1999) (17).

FIGURE 1: X-Ray Computed Tomography Equipment (ACIM Lab, Texas A&M University).
The basic equations that allow the calculation of the air voids characteristics (TAV, CAV, AVR) using black and white images are:

Being $AV_i$ the air void content of an image $i$, $A_v$ the area of the air voids in the image $i$, $A_t$ the cross-sectional area of the image $i$, and $AV$ the content of air voids (either the TAV- or CAV-content) in a specimen represented by $n$ images. The parameter $AVR$ represents the average radius of the air voids and $M_i$ is the number of air voids in the image $i$. Therefore, equations 1 to 3 were used to calculate the characteristics of air voids in black and white images, categorized into total- and connected-air voids. A macro developed by Masad et al. (2007) (18) was used for the analyses of black and white images by means of the Software Image-Pro Plus® (19).

$$AV_i = \frac{A_v}{A_t}$$  \hspace{1cm} (1)

$$AV = \frac{\sum_{i=1}^{n} AV_i}{n}$$  \hspace{1cm} (2)

$$AVR = \sqrt{\frac{A_v}{\pi M_i}}$$  \hspace{1cm} (3)

### Table 1: Stages of Image Analysis for Air Voids Characterization

<table>
<thead>
<tr>
<th>Stage Description</th>
<th>Picture type</th>
</tr>
</thead>
<tbody>
<tr>
<td>Using XR-CT. Scanning of specimens (115+5 mm height, 152.4 mm in diameter) producing digital images at a 1 mm vertical gap.</td>
<td>Raw image in grayscale from the XR-CT equipment. Figure 2(a)</td>
</tr>
<tr>
<td>Preliminary image edition. Pre-processing of the images to remove and centralize edges (black zones in raw image—stage 1). For this process, the software ImageJ® 1.41o (20) was used. This software allowed for the elimination of the black color phase around the specimen image and the cut of unnecessary pixels around the borders of the circle defining the specimen border.</td>
<td>Grayscale image edited using ImageJ® 1.41o. Figure 2(b)</td>
</tr>
<tr>
<td>Evaluation of total air voids (TAV). Use of threshold values to produce black and white images of TAV. For this purpose, a macro developed by Masad et al. (2007) (18) was used and implemented into Image-Pro Plus® routines. In general, this software allowed for the automatic computation of the TAV by means of an iterative process developed by using a Visual Basic routine. For this, a threshold value has to be assigned until the black and white image visually matches the grayscale image obtained in the stage 2 (i.e., the contours of the solid phase as well as the black pixels—representing the air voids—become similar in both images). Based on the threshold value assigned to the gray scale images black and white images were produced, where the black areas represented air voids and the white areas represented aggregates bonded by asphalt cement. The threshold value seeks to equalize the TAV content calculated based on the image analysis and that obtained in the laboratory.</td>
<td>Black and white image that represents the TAV (black color phase). Figure 2(c)</td>
</tr>
<tr>
<td>Evaluation of connected air voids (CAV). This process involves the conversion of black and white images (stage 3) into .bit binary files by using a macro developed by Masad et al. (2007) (18) and implemented in Image-Pro Plus®. Subsequently, the .bit files are analyzed using an algorithm on Fortran language to determine those pixels that have connectivity along successive images from the surface to the bottom of the specimen. After running the analysis, an updated binary data file containing the connected pixels is obtained. As a final step, the binary files .bit are converted back into .tif pictures, where the black areas represent the CAV.</td>
<td>Black and white image depicting the CAV (black color phase). Figure 2(d)</td>
</tr>
</tbody>
</table>

Adapted from: Alvarez et al. (2011) (21).
METHODS AND MATERIALS

This section describes the materials, methods, and equipment used for analyzing the internal structure of WMA mixture and HMA mixture specimens in this study.

### TABLE 2: Characteristics of the Mixtures

<table>
<thead>
<tr>
<th>Type of mixture</th>
<th>SMA</th>
<th>Type D-F</th>
<th>Type D-L</th>
</tr>
</thead>
<tbody>
<tr>
<td>Type of asphalt</td>
<td>PG 70-22</td>
<td>PG 76-22</td>
<td>PG 64-22</td>
</tr>
<tr>
<td>Type of aggregate</td>
<td>91% limestone and 9% sand</td>
<td>91% limestone and 9% sand</td>
<td></td>
</tr>
<tr>
<td>OAC (%)*</td>
<td>6.4</td>
<td>5.0</td>
<td>4.6</td>
</tr>
<tr>
<td>WMA additive</td>
<td>Rediset®</td>
<td>Evotherm®</td>
<td>Aspha-min®</td>
</tr>
<tr>
<td>Laboratory compaction-temperature (°C)</td>
<td>HMA: NA</td>
<td>HMA: NA</td>
<td>HMA: 121</td>
</tr>
<tr>
<td></td>
<td>WMA: 104</td>
<td>WMA: 132</td>
<td>WMA: 104</td>
</tr>
</tbody>
</table>

The Stone Matrix Asphalt (SMA) mixtures are characterized by a high content of coarse aggregate (minimum 70%) that forms a structural skeleton. The average TAV content in these mixtures is 7% and the percentage of asphalt binder ranges between 4-6% (1; 22; 23; 24). The type-D dense mixtures are typically used as surface mixtures or overlays for high frequency and heavy traffic routes (25). These mixtures have TAV contents ranging between 3-7% and its asphalt content is in the range of 4-6%.

Four WMA additives were used, namely Aspha-min®, Evotherm®, Rediset®, and Sasobit®. Aspha-min® is a fine powder of zeolite that has undergone a process of hydro-thermal crystallization (26; 27; 28; 29) that allows reduction of the asphalt viscosity and provides the conditions for an adequate coating of aggregates, thereby increasing the workability of the mixes at temperatures below those of HMA production (30). Evotherm® is a combination of chemical additives and cationic emulsifying agents that increase the aggregate-asphalt-aggregate adhesion, as well as improving the workability of the mixture (30; 26). AkzoNobelTM Company introduced in 2007 the Rediset® WMA production system, formulated to reduce moisture damage (i.e., incorrect or poor adhesion between asphalt and aggregates in the mixture) (31). Finally, Sasobit® is a wax or synthetic paraffin that becomes completely soluble in the asphalt at temperatures above 115 °C (32; 33).

### Experimental Procedure

The design of the HMA mixtures was performed in accordance to the design guidelines of the Texas Department of Transportation (25) and the corresponding material specifications (34). WMA specimens—152.4 mm in diameter and 115+5 mm in height—were produced in accordance to Tex-241-F method (35) using the Superpave Gyratory Compactor (SGC). The SGC specimens were assessed in the laboratory to determine the TAV content (based on the standard method AASHTO T 269-11(36), subsequently scanned with the XR-CT equipment (Figure 1) and the corresponding images were analyzed as described in Table 1.

### RESULTS AND DISCUSSION

The results include: (i) summary of TAV, (ii) vertical distribution of TAV content, (iii) vertical distribution of AVR, (iv) vertical distribution of CAV content, and (v) the ratio of CAV content and TAV content.

#### Summary of Air Voids Content Values

The results in Table 3 below correspond to the analysis of one (1) specimen of each mixture.

### Analysis of Vertical Distribution of TAV Content and AVR

Figure 3(a) shows the vertical distribution of TAV content in asphalt mixtures produced through the inclusion of the additives Rediset®, Aspha-min®, Sasobit®, and Evotherm®. These distributions preserved the trends (i.e., “C”-shape distribution) referenced in previous research on conventional HMA mixtures (21; 37; 38). Furthermore, at the bottom and top portions (i.e., 0-20 mm; 100-115 mm) greater variability in the TAV content...
is exhibited as compared with the central portion of the specimens. This degree of variation in the voids content is the result of the compaction process in the SGC that generates volumes of higher concentration of energy in the central portion of the compacted specimens (38). Consequently, higher TAV contents are obtained at the surface and base of the cylindrical specimen, as depicted in Figure 3(a), especially with the D-F, D-L dense-graded mixtures and control-HMA mixture. The SMA mixture has a dissimilar tendency as compared to that of the dense WMA mixtures and control-HMA mixture, although exhibiting more variability along the entire height of the specimen and flattening of the “C-shape”. These significant differences between the SMA mixture and control-HMA mixture can be the result, among other factors, of the differences in the: (i) gradation of the aggregates, (ii) mixture structural packing (formed by stone-on-stone contact in the SMA mixture) as a function of the aggregate gradation and the particular geometrical properties of the aggregates, and (iii) design asphalt content.

Accordingly, the high variability at the outermost portions of the WMA mixture specimens suggests the need for cutting 20 mm from the top and bottom of the 115 mm in height specimens, in order to obtain 75-mm specimens in height with more homogeneous vertical distributions of TAV. This recommendation is consistent with those offered in previous research on permeable friction course mixtures and dense-graded HMA mixtures (21; 37).

Figure 3(b) shows the vertical distribution of air voids size and supports the existence of similar trends in both the WMA mixtures and control-HMA mixture, except for the SMA mixture. The most homogeneous distribution of air voids size values was reported for the ‘Type D-F mixture fabricated using Evotherm’. In addition, the “C-shaped” distributions in the WMA mixtures are coincident with those reported in previous research on HMA mixtures (21; 1, 37). On the other hand, the SMA mixture exhibits higher air voids size values representing differences close to 120% as compared to the other mixtures. Also, as shown in Figure 3(b), the greater sizes’ heterogeneity occurs at the bottom of the specimen, which may be result of the energy distribution during compaction with the SGC.

Overall, the results previously discussed suggest that the use of the additives evaluated for the production of WMA specimens did not lead to significant changes in the internal structure of the laboratory compacted specimens—as compared to that of the control-HMA mixture—, expressed in terms of the vertical distribution of TAV content and AVR. Finally, the size reduction of the specimens from 115 mm to 75 mm in height may be considered as an approach to obtain more accurate results on the mechanical tests applied for the evaluation of laboratory performance.

Analysis of Vertical Distribution of CAV Content and CAV/TAV Ratio

Figure 4(a) presents the results of the air voids connectivity computed by application of the algorithms implemented on ImageJ *(20) and Image-Pro* Plus (19). The results indicate that the control-HMA mixture and the WMA mixture specimens exhibited connectivity along the entire height of the specimen (i.e., from the upper to the lower surface). Therefore, the paths created along the connected air voids in the specimens may enable internal flow of water or other elements capable of modifying the physico-chemical properties and mechanical response.

In particular, the highest value of CAV content corresponds to the SMA-Rediset specimen and the lowest to the Type D-L produced with inclusion of Asphamin* (Table 3; Figure 4(a)). However, the specimen produced with Sasobit* exhibited no connectivity along the entire height of the specimen and for that reason the corresponding data are not presented in the Figure 4(a). This may indicate that Sasobit* plays a role in the development of a denser structural skeleton.
FIGURES 4a & 4b: Vertical distribution of (a) CAV content and (b) ratio of CAV content to TAV content of WMA mixtures and control-HMA mixture.

In order to have an approximation on the mixtures’ exposure to the moisture damage phenomenon, the CAV/TAV ratio was computed. Corresponding values are shown in the Figure 4(b) and Table 2 presents information on the average percentage values obtained for each type of WMA mixture. In general, the higher mean of the ratio corresponds to the mixture manufactured with Rediset® (59.0%), followed by those fabricated with Evotherm® (46.9%) and control-HMA mixture (44.8%). The lowest CAV/TAV ratio was obtained for the mixture produced with Aspha-min® (34.6%). In contrast, for the Sasobit® mixture, no connectivity along the specimen was evidenced (see Table 2), thus, no ratio was computed. The above results indicate that, the ease of access of water, oxygen, and other chemical into the mixtures, and aging may be greater for the mixture manufactured with Rediset® and less for that produced with Aspha-min®.

CONCLUSIONS

An evaluation of the internal structure of WMA mixtures and a control-HMA mixture produced in the laboratory was performed. This assessment was developed in terms of: (i) vertical distribution of TAV content, (ii) vertical distribution of AVR, (iii) vertical distribution of CAV content, and (iv) ratio of CAV content to TAV content in the specimens. Based on the particular WMA additives analyzed and corresponding results and analyses, the following conclusions are stated:

- The type of WMA additive had no considerable influence on the WMA mixtures analyzed in terms of the content of air voids (i.e., TAV and CAV) and size of air voids (AVR), as compared to the control-HMA mixture.

- The connectivity of air voids was observed in the control-HMA mixture as well as the WMA mixtures produced with the additives Aspha-min®, Rediset®, and Evotherm®. The mixture manufactured with Sasobit® exhibited no voids connectivity along the entire height of the specimen, which may be an indicator of better compactability, workability, and less susceptibility to the moisture exposure.

- The vertical distribution of TAV content in the WMA mixtures exhibited a "C-shape" trend, similar to that shown in research developed with HMA mixtures. In addition, it was perceived a high heterogeneity in the surface portions of the specimens produced in the laboratory using the SGC. Conversely, at the central portion of the specimens, a more uniform distribution of air voids size values (AVR) and air voids content (TAV, CAV) was observed. The aforementioned conditions may be the result of the compaction process by using the SGC, leading to a higher energy concentration at the central portion, rather than at the upper and lower portions, of the specimen.

- The manufacturing protocol for laboratory specimens could be modified in order to reduce the variability of the vertical distribution of air voids content and size. It is recommended to cut 20 mm at top and bottom to obtain shorter specimens, with less dispersion in the air voids characteristics. However, taller or shorter portions may be cut depending on the height of the specimens. With this approach, it may be possible to achieve more representativeness of the performance tests used to determine the mixture response.
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Effect of Air Temperature and Vehicle Speed on Tire/Pavement Noise Measured with On-Board Sound Intensity Methodology

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ABSTRACT
The objective of this paper is to show the variability of tire/pavement noise measured with the On-Board Sound Intensity (OBSI) method under different environmental conditions and vehicle speeds. Temperatures ranging from 40 to 90°F and speeds in the range in which tire/pavement noise becomes predominant for the overall vehicle noise were tested. The selected speeds were: 35, 45, and 60 mph. The paper presents a summary of different technologies used to determine noise measurements and introduces the OBSI methodology, its standards, and provides an overview of the system and software used for data acquisition. Findings confirm the previously observed trend that tire/pavement noise slightly decreases as air temperature increases. The model shows a gradient of approximately -0.05 dBA/°F. Regarding the influence of vehicle speed, the results showed that for the surface studied tire/pavement noise increases an average of 2.5 dBA for every 10 mph of increased speed.

NOISE AND VEHICLE SPEED
The noise generated from tire/pavement interaction is one of many noise sources created when a vehicle drives over a pavement. For light vehicles, it becomes the primary source of traffic noise for speeds greater than roughly 20-30 mph depending on the type of surface (1). Propulsion noise will dominate the total noise at low speeds. As speed increases, a crossover speed (the practical threshold above which quieter pavements will be most helpful) is reached at that point when the tire/pavement noise becomes the dominant source. Only at high speeds will aerodynamic sources begin to dominate (1).

Another study states that the major facet of noise emitted on roads by vehicles traveling in the medium to high speed range (speed > 30 km/h) is tire/pavement noise (2). However, it is important to note that, due to advances made in the car industry (e.g., quieter engines), the so-
called crossover speed is decreasing. Therefore the tire/pavement noise study is becoming imperative. Research summarized in the NCHRP Report 630 shows that the sound level increases as the vehicle speed increases. This phenomenon is similar across different vehicle types and different reference tires (3).

NOISE MEASUREMENT TECHNOLOGIES

Wayside Noise Measurements

Wayside noise measurements are taken “at the side of the road,” either at a fixed distance or at the location of target receivers such as a residential neighborhood. There are three types of wayside testing:

- Statistical pass-by (SPB) in which a microphone at a fixed position measures the maximum sound level (Lmax) of approximately one hundred individual vehicles. Using Lmax the sound level is calculated for an average car, a medium truck, and a heavy truck.

- Controlled pass-by (CPB) is similar to SPB in that a microphone at a fixed position measures Lmax. In this case, however, a test is conducted for one or more known test vehicle/tire combinations.

- Continuous flow traffic time-integrated model (CTIM) in which a microphone is set to record all of the traffic noise at a fixed time interval from 5 to 30 minutes. Traffic levels and speeds are simultaneously recorded, and an average equivalent sound level (Leq) is calculated (1).

Source Noise Measurements

Source noise measurements are taken “near the tire,” a method that is more accurate than wayside noise measurements when the interest is to design and build quieter pavements. There are two typical techniques for measuring tire/pavement noise using source noise:

- CPX uses a single microphone that measures sound pressure, and testing is often conducted in an enclosed trailer to isolate the microphone from other sources of sound.

- OBSI uses dual microphone probes that measure sound intensity. Because of the ability to identify the direction of a sound source, the enclosed trailer is not required.

Noise and Environmental Effects

The National Cooperative Highway Research Program (NCHRP) Report 630, 1-44 Project, indicates that previous work on the evaluation of test variables for OBSI measurements shows that there is a slight downward trend in noise with an increase in air and pavement temperatures, with a low coefficient of determination (R2) from 0 to 0.4 (4). Tests conducted with this project show a decrease of approximately 1 dB for an air temperature increase of 10°C, or 18°F.

Another study conducted in Spain illustrates the same trend of the influence of temperature on noise measurements. In this case the testing was conducted with the close proximity (CPX) trailer Tiresonic Mk4-LA2IC rolling at a speed of 50 km/h (31 mph). The analysis of the results shows that the increase in pavement temperature leads to a reduction in the CPX sound levels assessed at a rate of 0.06 dBA/°C (7). Sandberg (8) also reports that work conducted by Liedever in 1999 and Landsberger in 2001 also showed a decrease in noise (sound pressure level) with the increase in road and air temperatures. Therefore, research with both OBSI and CPX measurements indicate the same inverse relationship between the temperature and the noise generated by tire/pavement interaction.

The literature also suggests that the effect of temperature is different when measured across different surfaces. The temperature effect is comparatively larger for a rough-textured surface than for a smooth-textured surface (8). When considering the relationship between air, road, and tire temperatures with noise, studies show that the association between noise and road temperature or noise and air temperature is stronger than that between noise and tire temperature (8). This was used as a starting point to establish air temperature as the focus of this study.

<p>| TABLE 1: Noise Measurement Technologies |</p>
<table>
<thead>
<tr>
<th>Technology</th>
<th>Noise Measurement</th>
<th>Type/Technique</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wayside noise measurement</td>
<td>At the side of the road</td>
<td>Statistical pass-by (SPB)</td>
</tr>
<tr>
<td>Controlled pass-by (CPB)</td>
<td>Continuous flow traffic time-integrated model (CTIM)</td>
<td></td>
</tr>
<tr>
<td>Source noise measurement</td>
<td>Near the tire</td>
<td>Close proximity (CPX)</td>
</tr>
<tr>
<td>On-Board Sound Intensity (OBSI)</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
METHODOLOGY

OBSI

Tire/pavement noise was measured following the American Association of State Highway and Transportation Officials (AASHTO) standard TP 76-12, “Measurement of Tire/Pavement Noise Using the On-Board Sound Intensity (OBSI) Method” (9). Results are reported as overall A-weighted sound ILS. Results were calculated using the A-weighted, one-third octave band levels.

The Virginia Tech Transportation Institute (VTTI) OBSI equipment was used during the test conducted on US-460 and was developed by Acoustical and Vibrations Engineering Consultants (AVEC), Inc. The aerodynamic configuration of the parts was conceived to reduce external noise picked up by the microphones in both probes. The main objective was the reduction of noise generated by wind hitting the parts while the vehicle was in movement during testing.

DESCRIPTION OF TEST PARAMETERS

Test Equipment

The VTTI OBSI system used to attain actual measurements includes:

- Hardware: Acoustic measurement instrumentations (one sound calibrator type 4231 Class 1 and LS, 94 and 114 dB, 1 kHz; four ½” microphone preamplifiers, two pairs of ½” condenser microphones for sound intensity, four 10 m extension cables, two spherical windscreens for ½” microphones, one microphone power module with 4 channels; and the physical parts for assembly [central plate sub-assy, intensity probe mounting assembly fixture, rim-mount sub-assy, shaft slide sub-assy, stabilization sub-assy, central bearing mounting plate_v2, coupling nut, microphone holding block_v4, rim-mounting plate, stabilization rod, tightening plate, flange-mounted steel bearing, pillow block housing]).
- Software: AVEC, Inc. OBSI software versions 1.00 and 1.43 licensed to VTTI, Copyright © 2007-2011 AVEC, Inc. (10).
- A 2011 sedan with a gross vehicle weight rating (GVWR) of 2,064 kg (4,549 lb) was used for all OBSI measurements.
- A P225/60R16 97S Radial Standard Reference Test Tire (SRTT) was used for all OBSI measurements. The tire was selected according to ASTM F 2493 (11).
- A type “A” Durometer was used for the hardness measurements performed on the SRTT before every set of OBSI testing. The Durometer was chosen according to Section X1, “Durometer Selection Guide,” from ASTM D2240 – 05 2010 (12).
- A Pocket Weather Tracker was used to calculate air temperature, wind speed, barometric pressure, air density, and relative humidity for all OBSI measurements.
- A Class II laser thermometer was used for pavement temperature measurements.

Test Procedure

Speeds were selected based on to test the range in which tire/pavement noise interaction was the most dominant noise generator. Three speeds were used: 35, 45, and 60 mph. The timeframes for each speed were chosen to test 440 ft of the sections according to AASHTO TP 76-12 (9). The tests were conducted as follows:

- Each set of runs was conducted in a timeframe during which environmental conditions were considered the same or within an acceptable 5°F range of variability.
- Temperatures in the 40s, 50s, 60s, 80s, and 90s (°F) were chosen for the testing. Where possible the temperatures were exactly 40°F, 50°F, 60°F, 80°F, and 90°F for the beginning of each set of runs.
- The tire inflation pressure (set before testing, cold inflation) was 30 ± 2 psi.
- The wind speed measurements, the hardness for the SRTT rubber, pavement temperature, barometric pressure, and air density were also recorded for each set of runs.

Sites

For the OBSI measurements two sites were selected on US-460 between Prices Fork Road and Toms Creek Road. The specific locations were 37º14’36.16”N - 80º26’2.5”W for US-460 eastbound (EB) and 37º14’6.93”N - 37º14’6.93”W for US-460 westbound (WB). Both sites had a 9.5 mm surface mix (SM9.5D) overlay placed in 2005 and presented an overall Critical Condition Index (CCI) of 78 with date rated 2/15/2011 (13). Note that a CCI of 100 refers to a pavement in perfect condition. The two test sections are shown in Figure 3. Only three set of measurements were possible in one of the directions because the section was overlaid during the study.
period. The three-dimensional graphs depicting the noise as orange walls were generated using the web-based software KML generator (14).

RESULTS

For both EB and WB sections on US-460 a limited number of runs was performed for each range of temperatures and for each vehicle speed.

Only valid results (at least 3 valid runs were taken per section) were used during the analysis and in the averaging of runs according to AASHTO standard TP 76-12 in terms of the following (9):

- Coherence: “The coherence of sound pressure between the two microphones of the sound intensity probe shall be equal to or greater than 0.8 for each one-third octave band with a center frequency between 400 and 4000 Hz and equal to or greater than 0.5 for the one-third octave band with a center frequency of 5000 Hz.”

- Pressure Intensity (PI) Index: “The PI index shall be less than 5.0 dB in each one-third octave band with a center frequency between 400 and 5000 Hz.”

- The direction of the sound intensity vector “must be positive for each one-third octave band with a center frequency between 400 and 5000 Hz.”

For each set of valid runs the average overall sound IL is reported (dBA referenced to 1 pW/m2). The results of the average overall IL (dBA) for the three vehicle speeds and for each temperature range are presented in Table 2, it is important to mention that westbound measurements for 90°F and 80°F were not taken due to changes on the test section. Table 3 summarizes the different conditions for all tests.

Figure 1 shows the variation of the sound IL with respect to the speed traveled on both directions of US-460 and at all temperatures. The partial differences (DdBA) shown in Figure 1 and summarized in Table 4 (fifth column) corresponding to the ratio of the speeds V1/V2 (Table 4, fourth column) were used to find the exponent “x” that is the indicator of the degree of influence of speed over the sound ILs (i.e., noise) following Equation 1:

$$ IL \propto \left( \frac{V_1}{V_2} \right)^x \Rightarrow \Delta \text{dBA} = x \log_{10} \left( \frac{V_1}{V_2} \right) $$

Table 4 shows the calculus of the exponent “x” for each temperature and its average. This exponent is calculated to state the degree of influence of speed on the Sound IL. Figure 2 shows the decrease of the sound IL when the temperature increases in both directions of US-460 and for the three selected speeds.

<table>
<thead>
<tr>
<th>TABLE 2: Average Overall ILs for All Runs (in dBA)</th>
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</thead>
<tbody>
<tr>
<td><strong>Air temp @ beginning of test (°F/°C)</strong></td>
</tr>
<tr>
<td><strong>Direction</strong></td>
</tr>
<tr>
<td><strong>Speed mph (km/h)</strong></td>
</tr>
<tr>
<td>45 (72.5)</td>
</tr>
<tr>
<td>60 (96.6)</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>TABLE 3: Meteorological, Pavement, and Tire Conditions for All Test Air Temperatures</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Air temp @ beginning of test (°F)</strong></td>
</tr>
<tr>
<td><strong>Barometric Pressure (inHg)</strong></td>
</tr>
<tr>
<td><strong>Air Density (lb/ft³)</strong></td>
</tr>
<tr>
<td><strong>Wind Speed (mph)</strong></td>
</tr>
<tr>
<td><strong>Pavement Temperature (°F)</strong></td>
</tr>
<tr>
<td><strong>Hardness (Durometer)</strong></td>
</tr>
</tbody>
</table>
ANALYSIS

Test Speed Effects

Figure 2 shows that there is a linear trend for this specific range of speed. The average slope of all measurements reflects an increase in the overall IL noise of 2.5 dBA for every 10 mph increment. Although this equation is applied for this speed domain and for slope comparisons, extrapolation is not recommended since tire/pavement interaction is not the main source of noise at lower speeds.

Again referencing Table 4, Equation 1 can be expressed as: IL a (V1/V2)^x, which is valid for the analyzed pavement, range of temperatures and speeds. The exponent x = 2.7 in the equation suggests that speed has a high degree of influence on the sound IL. For that reason it is essential to consider the speed differentials in any tire/pavement noise measurements and/or analysis made using the OBSI methodology.

Temperature Effects

Although the effects of air temperature changes with OBSI measurements were studied only on one type of surface (a two inch dense graded asphalt overlay placed in 2005), the results are in line with previous research. The sound ILs in Table 2 and the slope values of the tendency equations in Figure 2 shows a decrease in noise when temperature increases. It is possible to ascertain an average for these slope values and to determine that the average decrease in noise for all measurements reflected a gradient of -0.05 dBA/oF.

CONCLUSIONS

There is significant variation on the noise generated by tire/pavement interaction measured at different speeds. The change in the ILs for every 10 mph variation is in the range that may be detected by human hearing, taking into account that a human ear can perceive a change in noise level as low as 3 dB. The overall IL increases by approximately 2.5 dBA for every 10 mph increment for the surface studied.

A noise-temperature gradient of -0.05 dBA/oF was observed for the surface studied, which indicates a small influence of air temperature on tire/pavement noise levels. For this reason corrections to measurements taken at different temperatures are recommended, especially if measurements are conducted over a relatively high range of temperatures, as is the case for measurements taken during different seasons. This gradient is similar as the ones stated in previous literature.

---

TABLE 4: Calculus of the Exponent “x”

<table>
<thead>
<tr>
<th>Temp. °F (°C)</th>
<th>V1 mph (km/h)</th>
<th>V2 mph (km/h)</th>
<th>V1/V2</th>
<th>D dBA</th>
<th>“x”</th>
<th>“x” for each temp.</th>
<th>Average “x”</th>
</tr>
</thead>
<tbody>
<tr>
<td>90 (32.2)</td>
<td>45 (72)</td>
<td>35 (56)</td>
<td>1.29</td>
<td>2.2</td>
<td>2.0</td>
<td>2.5</td>
<td></td>
</tr>
<tr>
<td>60 (26.7)</td>
<td>45 (72)</td>
<td>45 (72)</td>
<td>1.33</td>
<td>3.8</td>
<td>3.1</td>
<td></td>
<td></td>
</tr>
<tr>
<td>80 (15.6)</td>
<td>45 (72)</td>
<td>35 (56)</td>
<td>1.29</td>
<td>2.9</td>
<td>2.7</td>
<td>2.7</td>
<td></td>
</tr>
<tr>
<td>60 (10.0)</td>
<td>45 (72)</td>
<td>45 (72)</td>
<td>1.33</td>
<td>3.4</td>
<td>2.7</td>
<td></td>
<td></td>
</tr>
<tr>
<td>50 (4.4)</td>
<td>45 (72)</td>
<td>35 (56)</td>
<td>1.29</td>
<td>3.1</td>
<td>2.8</td>
<td>2.8</td>
<td></td>
</tr>
<tr>
<td>45 (72)</td>
<td>45 (72)</td>
<td>45 (72)</td>
<td>1.33</td>
<td>3.6</td>
<td>3.0</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

FIGURES 2a, 2b & 2c: Variation of intensity level (IL) in A-Weighted Decibels (dBA) with air temperatures for different vehicle speeds
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Fatigue and Rutting Properties of Asphalt Binders Modified with Polyethylene and Polyphosphoric Acid

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ABSTRACT
This paper aims at evaluating the fatigue and creep-recovery properties of three asphalt cements (AC) modified with polyphosphoric acid (AC+PPA), low-density polyethylene (AC+PE) and a combination of both modifiers (AC+PE+PPA). The recent linear amplitude sweep (LAS) test for fatigue and the multiple stress creep and recovery (MSCR) test for permanent deformation were carried out. The modifier contents were such that the high-temperature performance grade is the same for all formulations in the Superpave® binder specification, namely, PG 76-xx. All the modified asphalt binders have better fatigue tolerance and higher resistance to rutting than the base material. In addition, the laboratory results indicated that PPA is a great alternative to be used as an asphalt binder modifier and that modified asphalt binders can have different rutting and fatigue behaviors, even if their high PG grades are the same.

INTRODUCTION
Asphalt binder modification is an alternative to improve the characteristics of the material to the main distress mechanisms that affect asphalt pavements, namely, fatigue cracking and rutting. There is evidence that the incorporation of polyphosphoric acid (PPA) to the asphalt binder can improve the rheological behavior of the material at high temperatures (1). Other authors have noticed that, despite this beneficial effect of the addition of PPA, it may bring problems to the resistance to fatigue and low temperature cracking (2). The polyethylene (PE) is one of the most popular plastics around the world and is renowned for its excellent chemical and good fatigue resistances (3). This polymer may also reduce creep rate of asphalt mixtures at high temperatures (4). With respect to fatigue cracking, it is caused by the cyclic loading action at intermediate temperatures. To describe the fatigue behavior of the material, an accelerated test called Linear Amplitude Sweep (LAS) test was proposed in the literature (5, 6). This test is performed on the dynamic shear rheometer (DSR) by applying reversal cyclic loading.

Another distress commonly found in asphalt pavements is the accumulation of permanent deformation in the wheel paths, which is referred to as “rutting” in the North American convention. The relatively recent multiple stress creep and recovery (MSCR) test analyzes the rutting susceptibility of the asphalt binder by applying subsequent loading-unloading cycles on DSR. The percent recovery (R) obtained in the test can directly measure the elastic response of the material, and also indicates the presence of a polymer network in the formulation (7). The nonrecoverable compliance \(J_{nr}\) is associated to the susceptibility of the binder to the accumulation of permanent deformation, whereas the
percent difference in nonrecoverable compliances \( (J_{nr} \text{ diff}) \) is used in the Superpave® specification to disconsider binders that are overly stress sensitive and highly susceptible to rutting in the field. This may be observed for a particular binder even when the other performance grade criteria – e. g., mass loss, flash point and rotational viscosity at 135 °C – are met (8).

By considering these recent developments proposed in the area of laboratory tests for the characterization of asphalt binders, the fatigue and rutting behavior of modified asphalt binders were analyzed in the LAS and the MSCR tests. In order to evaluate the behavior of the material under several test conditions, various test temperatures were used in MSCR tests and two aging conditions were considered in the LAS tests.

**EXPERIMENTAL DESIGN**

To produce the modified asphalt binders, the following materials were used: (a) a 50/70-penetration grade base binder supplied by the Replan-Petrobras refinery (Paulinia, Sao Paulo, Brazil) and graded as PG 64-xx; (b) low-density polyethylene designated as UB-160C and supplied by Quattor-Braskem (Santo Andre, Sao Paulo, Brazil); and (c) polyphosphoric acid designated as Innovalt® E200 and supplied by Innophos Inc (US). The mixtures were prepared using a Fisatom 722D low-shear mixer. Table 1 shows the modifier contents used to prepare the formulations, as well as the processing variables and their high PG grades according to a revised version of the Superpave® specification (AASHTO M320 standard, Table 3).

**Linear amplitude sweep (LAS) and multiple stress creep and recovery (MSCR) tests**

All of the LAS tests were performed on an AR-2000ex dynamic shear rheometer supplied by TA Instruments using parallel plate geometry of 8 mm in diameter and 2 mm in gap. The procedure that was recently standardized by AASTHO TP 101-12 UL consists of applying a reverse cyclic loading in two stages: 1) a frequency sweep with application of 0.1% deformation and frequencies in the range from 0.2 to 30 Hz; and 2) a linear amplitude sweep with linear strain increments from 0 to 30% in a time interval of 300 s and frequency of 10 Hz. In this study, tests were conducted at 25 °C under two aging conditions: short-term aging (RTFOT, ASTM D2872) and long-term aging (PAV, ASTM D6521).

Two analyses can be made from the LAS test results: 1) based on the viscoelastic continuous damage (VECD) approach; and 2) based on fracture analysis and damage tolerance index. In the first analysis, power models are obtained such as \( N_f = A35*\gamma B \), where \( N_f \) is the number of cycles to failure, \( \gamma \) is the shear strain applied and \( A35 \) and \( B \) are experimentally defined parameters. The criterion to failure in the second analysis is \( af \), the value of the crack length \( a \) which presents minimum local of the curve \( da/dN \) versus \( a \), where \( da/dN \) is the variation rate of crack length \( a \) with the number of cycles \( N \). This \( af \) value corresponds to the point before the rapid increase in the rate of crack growth. Higher \( af \) values indicate higher damage tolerance, that is, higher values are an indicator that the material could have a longer crack before the rapid propagation of cracks (6).

The MSCR tests were conducted on the same DSR used on the fatigue tests. Short-term aged samples (ASTM D2872-04) with a diameter of 25 mm and a gap height of 1 mm were prepared on a silicone rubber mold and sandwiched between the two parallel plates of the device. Standardized loading-unloading conditions – 1-s creep time, 9-s recovery time, 10 creep-recovery cycles and stress levels of 0.1 and 3.2 kPa – were used in the tests, and the average of the results of two replicates (R and \( J_{nr} \)) was calculated for each formulation. The percent differences in nonrecoverable compliances were determined based on the final results of the \( J_{nr} \) values at 0.1 and 3.2 kPa for all asphalt binders.

**RESULTS AND DISCUSSION**

**Fatigue behavior on LAS test**

FIGURES 1(a) and 1(b) show the fatigue law obtained for the materials using the VECD approach. These figures present the number of cycles to fatigue failure (NFf), which is an indicator of the volume of traffic that would support the material as a function of the applied shear strain. This represents the conditions which the material could be subjected within a given pavement structure.

It can be seen in FIGURE 1(a) that, in the short-term aging condition, the material that shows better fatigue behavior when subjected to low deformation levels is the AC+PPA. The AC+PE+PPA is the second best
material, which is also very close to the AC+PE. The worst performance to fatigue behavior was showed by the base binder. The aforementioned ranking is inverted for higher levels of shear strain, with the best behavior corresponding to the base binder and the worst behavior to the modified asphalt binders. In the short-term aging condition and low shear strains, the fatigue behavior of asphalt binders modified by PE and PE+PPA were very close, so there is not any clear different behavior. However, for high shear strains, the modified asphalt binders that do not include PPA in their formulation had a better behavior.

After the long-term aging of materials in the PAV (FIGURE 1(b)), the AC+PPA remained having better fatigue behavior when subjected to low levels of shear strains, followed by the AC+PE+PPA (very close to the CAP+PE). The unmodified asphalt binder had the worst fatigue behavior. However, for high shear strains, the three modified asphalt binders showed worse fatigue performance than the unmodified AC, and very similar results among them. Therefore, the best behavior in high strain levels was found in the base asphalt binder.

FIGURES 1a & 1b: Fatigue Models Obtained For the Materials Using the VECD Analysis

Rutting behavior in the MSCR test

FIGURE 3 depicts the percent recoveries of the unmodified and modified materials. The addition of one or two modifiers led to an increase in the elastic response of the asphalt binder, i.e., the presence of modifiers was responsible for the reduction in the amount of the unrecovered strain of the bituminous material at typical high pavement temperatures. The AC+PPA shows the highest R values, whereas the two
formulations with PE show the lowest ones. As expected, the unmodified material has very little or no recovery at these same temperatures and stress levels. One curious aspect of the results is the fact that the R values are null or close to zero (lower than 1%) under the most severe test conditions. One possible explanation for this is that the quantities of modifiers were not sufficient to greatly increase the elastic response of the unmodified asphalt binder, even though these quantities were able to boost the high PG grade of the material by 12 °C (from 64 to 76 °C). In this manner, higher R values may be obtained by selecting higher modifier contents or utilizing materials with different chemical and/or physical properties.

FIGURE 4 shows the nonrecoverable compliances of the binders. The presence of one or two modifiers decreased the J$_{nr}$ values of the base material, especially at higher temperatures and stress levels. These values are lower for the AC+PPA than for the AC+PE and the AC+PE+PPA, and this is more visible at temperatures higher than 58 °C. Although major differences among the J$_{nr}$ values of the formulations can be highlighted at temperatures of 64 °C or higher, the same cannot be said for the two lowest ones. The J$_{nr}$ values of the formulations are quite similar in such test conditions, and therefore it is difficult to examine the effects of the addition of modifiers in detail. This may be explained by the observation of very low strain levels at 52 and 58 °C, which makes it difficult to clearly distinguish between one formulation and the other. As the temperature and the strain level increase, the effects of each modification type become more noticeable.

TABLE 2 displays the J$_{nr, diff}$ values of the asphalt binders. None of the materials exceeded the maximum value of 75% at their high PG grades of 64 and 76 °C, and therefore they cannot be considered overly stress sensitive in the Superpave® specification. The AC+PPA was the only formulation that got closer to this upper limit, since its J$_{nr, diff}$ value is equal to 61.4% at 76 °C. These values are lower than or equal to 14% for the 50/70 base binder, and are no greater than 39% for the AC+PE and the AC+PE+PPA. In a general context, the addition of modifiers caused an increase in the stress sensitivity (higher J$_{nr, diff}$ values) of the binder, especially at 70 and 76 °C. These increases are more significant for the AC+PPA and the AC+PE+PPA at 70 and 76 °C, and they are greater for the AC+PE at the lowest ones (from 52 to 64 °C). In no case, however, was the presence of modifiers extremely detrimental to the susceptibility of the asphalt binder to rutting under unforeseen temperature and/or loading conditions.

**CONCLUSIONS**

The results of this paper presented rheological properties of asphalt binders at intermediate and high pavement temperatures. Considering a given pavement structure that admits small strains of the asphalt layer, the modification of asphalt binders can significantly improve the performance of the material against fatigue cracking. Taking the previous hypothesis into account, the AC+PPA would have the best fatigue behavior in both aging conditions evaluated (short- and long-term). The AC+PE+PPA would have the second best fatigue behavior, followed by the AC+PE and the base asphalt binder. Despite the beneficial effects that aging has in the fatigue life of modified asphalt binders at low shear strains, higher strains can lead to premature fatigue cracking of the material.

With respect to the MSCR test, the addition of one (PE or PPA) or two modifiers (PE+PPA) to the base asphalt binder increased the percent recoveries and decreased the nonrecoverable compliances of the material at high pavement temperatures from 52 to 76 °C. This can be translated into higher elastic responses of the
modified asphalt binders and lower susceptibility to the accumulation of permanent strain in the field, which is favorable to the resistance of the material to rutting. The best results can be found in the AC+PPA for the two rheological parameters, that is, asphalt mixtures with the AC+PPA on their composition would be less susceptible to rutting – lower $J_{nr}$ values – and would recover a higher portion of the total strain after each loading-unloading cycle – higher R values – than the ones with the AC+PE and the AC+PE+PPA. In addition, none of the formulations nor the unmodified asphalt binder can be considered overly stress sensitive because their $J_{nr}$, diff values do not overcome the upper limit of 75% set by the Superpave® specification.

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Follow-Up to a Five-Year Asphalt Pavement Trial: Better Performance with an Iron & Steel Slag Base Course

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ABSTRACT
It is empirically known in Japan that asphalt pavements having iron and steel slag base-courses provide longer service life than the ones with natural material. In order to understand a better performance of asphalt pavement with iron and steel slag base-course, an in-situ follow-up investigation was carried out on a trial pavement over five years from 2005 to 2009. Strain transducers, earth pressure gauges and thermocouples were installed in the trial section, and the development of horizontal strain and vertical earth pressure due to passage of a loaded vehicle was investigated at the critical locations. A conventional pavement survey accompanied with a falling weight deflectometer (FWD) was also carried out over three years. This paper first describes the in-situ investigation with a focus on resilient response of horizontal strain and vertical earth pressure, and the influence of hydraulic nature inherent in the slag material on the pavement response.

INTRODUCTION
Iron and steel slag is produced during the process of making iron and steel and is an industrial waste. In Japan, due to a social trend toward saving and effectively using natural resources and also due to persistent efforts of the iron and steel industry, iron and steel slag was added to the Japanese Industrial Standard in 1979 as “Iron and steel slag for road construction”. The standard has been revised several times and the latest edition has been just out in 2013 (1). The amount of iron slag (blast-furnace slag) and steel slag produced in Japan in 2011 is shown in Table 1 together with the their uses.
Table 2 summarizes the types of iron and steel slag used in the road construction together with some quality requirements designated in the latest Japanese Industrial Standard. The hydraulic graded iron and steel slag, graded iron and steel slag and crusher-run iron and steel slag are literally made by blending iron slag and steel slag. The blended iron and steel slag contains only slowly air-cooled blast-furnace slag in some cases, while in other cases it consists of slowly air-cooled blast-furnace slag, granulated blast-furnace slag and steel slag: the proportion depends on the manufacturers. In Table 2, the item “coloration” is to check whether or not sulfur in blast-furnace slag is sufficiently stabilized. The term “immersion expansion ratio” is to grasp the expansion of steel slag, which is attributed to the existence of free lime. The item “aging” is a treatment required for free lime in steel slag to be hydrated to suppress its expansion. The aging periods designated are applied to those, which piled in open yard but in the case that warm water or vapor is applied for accelerating the chemical reaction, the designated periods of time can be shortened (2).

FIGURE 1: Trial asphalt pavement site before and after completion of construction

(a) Before construction (b) After construction

In Japan, the structure of asphalt pavement is designed based mainly on an empirical method called the TA method which uses a subgrade CBR value and equivalence conversion coefficients of pavement materials since 1967. Regarding the iron and steel slag base-course material, the corresponding equivalence conversion coefficient, especially for the hydraulic graded iron and steel slag, appears underestimated. This is probably because detailed investigations have not been carried out so far on the mechanical properties and the response and performance of the asphalt pavement containing the iron and steel slag base-course. The situation is the same even in the Kansai region where the three major producers of iron and steel slag operate. The dynamic response of asphalt pavement with an iron and steel slag base-course has little been understood, although its favorable performance has been empirically grasped from occasional short-term on-site follow-up investigations with FWD and visual inspections on the road surface. For this reason, in order to use iron and steel slag base-course material more effectively, it was certainly needed to conduct at least a detailed investigation regarding the in-situ dynamic response of the asphalt pavement with an iron and steel slag, especially a hydraulic graded iron and steel slag.

In cooperation with Nippon Slag Association and Hyogo Prefecture, the authors had an opportunity of investigating the in-situ response of asphalt pavement with a hydraulic graded iron and steel slag base-course by constructing an instrumented trial asphalt pavement in a prefectural road and also the mechanical characteristics of the material used in the trial pavement. In the followings, the in-situ follow-up investigation over five years is described and the influence of the hydraulic nature of iron and steel slag upon the dynamic response of the asphalt pavement is illustrated.
TRIAL ASPHALT PAVEMENT AND INVESTIGATION

Construction Site, Structure and Instrumentation of Trial Pavement

A trial asphalt pavement was constructed at Nakashima-Anase line of a Hyogo prefecture road running through Himeji City, Hyogo Prefecture on occasion of an expansion work of the existing paved road in 2004. The Nakashima-Anase line is a four-lane road and designed for a one-way daily traffic volume of heavy vehicles ranging from 1001 to 3000 cars with a design vehicle-speed of 60 km/h. As shown in Figure 1, the trial asphalt pavement section is located at the near-centerline lane bound for Nakashima.

As shown in Figure 2 (a), the section of trial asphalt pavement consisted of a 50 mm thick surface-course of recycled dense-graded asphalt mixture (max. aggregate size of 20mm), a 50 mm thick mid-course of recycled coarse-graded asphalt mixture (max. aggregate size of 20mm), a 50 mm thick binder-course of recycled coarse-graded asphalt mixture (max. aggregate size of 20mm), a 100 mm thick base-course of hydraulic graded iron and steel slag (max. aggregate size of 25mm), a 100 mm thick subbase-course of recycled crusher-run (max. aggregate size of 40mm) and on-site subgrade with a blackish silty sand of 100 mm thick underlain by a yellow-whitish silty clay. The hydraulic graded iron and steel slag used in the base-course consisted of air-cooled blast-furnace slag alone. The trial asphalt pavement section was constructed in July, 2004, following an ordinal construction method.

During construction, embedded-type strain transducers, earth pressure gauges and thermocouples were carefully installed in order at pre-specified locations: the strain transducers were placed on three different depths, the earth pressure gauges at two different depths and the thermocouples were at five different depths in the location of Set 1, as shown in Figure 2 (b).

FIGURE 2 Trial asphalt pavement structure and instrumentation

(a) Cross-sectional view (b) Plan view

Follow-Up Investigation

On-site investigations with a loaded dump truck were carried out on August 2, 2005, September 5, 2006, September 10, 2007, February 27 and August 12, 2008 and February 24, 2009. The wheels of dump truck consist of a front single axle with single tire and tandem rear...
axles with dual tires. The total weight was 196 kN and was adjusted in such a way that the front axle weighed 96 kN and the tandem rear axles 100 kN. The dump truck was forced to run on the two courses: one was that the center of dual tires of tandem rear axles ran directly above the embedded instruments as shown in Figure 2 (a) (called “Center-passing”) and the other was that the outer tire of dual tires of tandem rear axles ran directly above the embedded instruments (called “Outer-passing”). Three targeted vehicle speeds were 5, 15 and 30 km/h but the actual speeds were computed from measured earth pressure pulses. A falling weight deflectometer (FWD) test was also conducted on August 2, 2005, September 5, 2006 and September 10, 2007. A road surface inspection was carried out on every occasion: the latest inspection was on May 2, 2013. In the following sections, only part of the investigation results is presented owing to the limited space without losing the essence.

RESULTS AND DISCUSSIONS

The typical variation of pavement temperature with depth during the on-site investigation is shown in Figure 3. It is seen that the pavement temperature measured in winter (February 2005 and 2008) was about 9°C and varied little with depth. In summer (August 2005 and 2008; September 2006 and 2007), on the other hand, a relatively large temperature gradient can be seen from the surface downward. It should be mentioned here that, in Japan, a relatively large gradient in pavement temperature in summer such as this is not unusual regardless of the type of base-course material used. In the figure, in August 2, 2005, temperature changes from 45.2°C at the pavement surface to 30.4°C at the top of the base-course and it appears not to vary below the layer. A quite similar variation in temperature can be observed in the summer of 2007 and 2008, though the values differ. Average temperatures in the asphalt mixture layers (surface-, mid- and binder-course) in the summer of 2005, 2007 and 2008 are 42.5°C, 42.6°C and 52.4°C, respectively. The temperature measured in September 5, 2006 is not as high as those in 2005, 2007 and 2008: an average temperature in the asphalt mixture layers is 35.2°C. This similarity and difference in temperature variation will be referred to later in analyzing the measurements of horizontal strain and vertical earth pressure.

**Horizontal Strain Response**

When the loaded dump truck passes directly over the embedded instruments, a response waveform with three peaks can be obtained: each peak corresponds to the single tire of the front axle, the dual tires of the front axle of tandem rear axles and the dual tires of the rear axle of tandem rear axles. The horizontal strain and vertical earth pressure presented hereafter are those corresponding to the passage of the dual tires of the rear axle of tandem rear axles.

![Figure 3: Temperature variation with depth](image_url)

Figure 4 summarizes the typical variations of peak horizontal strain with depth obtained in the winter and summer. It is seen that in winter, a small amount of tensile strain develops at the bottom of asphalt mixture layer (the binder-course), as speculated and no major difference between the investigation years. In summer, the situation quite differs. As seen in the figure, a large tensile strain is generated at the bottom of asphalt mixture layer. The magnitude of the tensile strain, however, decreases with the investigation years. Referring to the temperature variation shown in Figure 3 and also considering the likelihood of fatigue damage over four years of service, it can be said at least that the decrease of tensile strain attributes to the development with time of hydraulic nature inherent in the hydraulic graded iron and steel slag base-course. In fact, a laboratory study using a small-scale asphalt pavement with a hydraulic graded iron and steel slag base-course clearly indicated that the iron and steel slag base-course gradually hardened with time and that the behavior changed from a granular medium to a plate-like stiffened one (3) resulting in diminishing the tensile strain in the asphalt mixture layer above. This is certainly a favorable point as long as the fatigue failure of asphalt mixture layer is concerned.

It is also noticed in Figure 4 that a relatively large tensile strain is generated at the bottom of the hydraulic iron and steelslag base-course. It should be mentioned here that this relatively large tensile strain has also been observed in the location of Set 2 (Figure 2). This may suggest a possibility of fatigue failure of the layer just like
the asphalt mixture layer, which appears needed to be investigated in some detail (4).

**FIGURE 4: Variation of peak horizontal strain with depth**

(a) Winter  (b) Summer

**Vertical Earth Pressure Response**

The horizontal distributions of peak vertical earth pressure measured at the top of the subbase-course in the winter and summer are shown in Figure 5. In both the winter and summer measurements, the largest earth pressure develops directly beneath the center of dual tires and decreases with horizontal distance. In the winter, there is only a little difference in the earth pressure distribution between the measurement years.

**FIGURE 5: Horizontal distribution of peak vertical earth pressure**

(a) Winter  (b) Summer

In the summer, on the other hand, much larger earth pressure is generated as seen in Figure 5 (b). As time goes by, the earth pressure decreases drastically, this probably being attributed to the development of hydraulic nature of iron and steel slag, considering that the temperature distributions in the asphalt mixture layers are almost the same between the 2005 and 2007 (see Figure 3). It is noted that the earth pressure beneath the center of dual tires in 2008 is slightly higher than the 2006 and 2007. This appears to result from lowering of the load transfer function of the asphalt mixture layers due to its slightly higher temperature as shown in Figure 3.

**FIGURE 6: FWD deflection and back-calculated elastic modulus**

(a) FWD deflection at Location 2  (b) Elastic modulus

**FWD Deflections and Back-Calculated Elastic Modulus**

A FWD test was also conducted at the nine locations on the expanded road including the trial pavement section in the summer of 2005, 2006 and 2007 when the on-site investigation with a loaded dump truck was carried out. The peak load was adjusted as 49 kN. The locations 2, 3 and 5 correspond to the trial pavement section. Figure 6 shows the deflection curves obtained at Location 2 in each summer and the elastic moduli of base-course back-calculated from the deflections. Note that these deflections are the ones converted to the standard load and temperature conditions (49 kN and 20°C), following the procedure recommended by the Japan Road Association (5). It is seen that the deflection decreases year by year, and the deflections at the other two locations, though not shown here, decrease with time. This appears reflection of hardening of the hydraulic graded iron and steel slag base-course as seen from that the back-calculated elastic moduli increase year by year at all three locations.

**FIGURE 7: Increase of uniaxial compressive strength with time of five different hydraulic graded iron and steel slag (modified after (6))**

Second, the hydraulic nature inherent to iron and steel slag certainly may attract a pavement designer but there remains unknown in its development. As stated in the Introduction, hydraulic iron and steel slag consists of slowly air-cooled
blast-furnace slag alone or mixtures of blast-furnace slag and steel slag with or without additives, which suggests that the development of hydraulic nature with time would vary. In fact, as seen in Figure 7, an investigation on the uniaxial compressive strength of hydraulic graded iron and steel slag base-course materials obtained from five different producers clearly illustrates that, there exists relatively large difference in the magnitude of strength and the strength increases with curing period among the producers (6). Thus, in adopting the material, this variety in strength development should be considered in some way.

CONCLUSIONS

An attempt was made to grasp why the pavement exhibits a better performance and the followings can be pointed out:

- The horizontal strain and vertical earth pressure generated in the pavement due to passage of a loaded dump truck decrease year by year and elastic moduli back-calculated from the FWD deflections increase with time.
- These favorable characteristics result from the development of hydraulic nature inherent in the iron and steel slag with time. That is to say, the base-course changes from a granular material-like behavior to a stiff plate-like one; thus, the load transfer function of the layer becomes strengthened. By this, the magnitude of tensile strain at the bottom of asphalt mixture layer also diminishes, reducing the risk of its fatigue failure.
- Two concerns are also addressed: one is a necessity of investigating a likelihood of fatigue failure of the iron and steel slag base-course and the other is to cope with a variety in the development of hydraulic nature in the base-course between different iron and steel slag producers.

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Effect of Aggregate Composition and Freeze-Thaw Cycles on the Stiffness Modulus of Asphalt Concrete Mix

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ABSTRACT
Aggregate samples from five different quarries in Sweden were collected to manufacture asphalt concrete specimens by gyratory compaction. The program involved soaking the samples in water and exposure for up to 35 freeze-thaw cycles. The resilient modulus tests were performed at 10 °C on the conditioned samples. The results show the importance of subjecting the asphalt concrete samples to a high number of freeze-thaw cycles to simulate the excessive decrease in the asphalt concrete performance with time. A regression analysis has been performed to model the effect of the mineralogical composition of the filler aggregate and the freeze-thaw cycles on the resilient moduli of the asphalt concrete samples and a regression equation has been detected with a high coefficient of correlation. The developed equation assists pavement engineers to predict the retained stiffness moduli of asphalt concrete samples subjected to high number of freeze-thaw cycles taking into account the mineralogical composition.

INTRODUCTION
Taylor and Airey (2008) defined filler as a fine powder with a particle size distribution in the range of 0-100 µm and they can be naturally occurring materials or driven from industrial wastes. Correspondingly, it is important to highlight the fact that the mineral filler can be a major component influencing the mechanical properties of the asphalt concrete materials used in the construction of asphalt pavement layers.

Together with the great influence of fillers on pavements performance, the relationship between filler properties and pavement performance has been reported by many researchers and to date several attempts were made to tackle this problem (Kim et. al., 1992; Kandhal and Parker, 1998; Kandhal, 1999; Dâmiean and Gugiuman 2005; and Said et. al., 2009). In principle, several studies showed that fillers significantly influence the asphalt mixture performance, depending mainly on their mineralogical composition as well as finer size particles (Shahrour and Saloukeh, 1992, and Grabowski and Wilanowicz, 2008).

Anderson and Goetz (1973) evaluated the stiffening effect of a series of one-sized fillers in one of the first studies that focused on determining the mechanical properties of asphalt mixtures. As a result, Anderson and Goetz (1973) concluded that the size of the filler particles in producing asphalt mastic (binder + filler) had an important influence on the stiffening of an asphalt mix.

Filler in asphalt mixtures may act as a bitumen extender and it may stiffen the asphalt mastic (bitumen + filler) depending on the type of filler used, correspondingly, the fillers which extend the asphalt-cement effectively can increase the binder volume in the mix (Shahrour and Saloukeh, 1992). Along with the fact that the quantity and quality of filler used in hot asphaltic mixtures greatly affect its performance, the function of mineral filler has been recognized to be more than filling voids (Shahrour and Saloukeh, 1992). On the other hand, Shahrour and Saloukeh (1992) added that stiffening of mixes caused by certain types of fillers is not easy to demonstrate.

As expected, in bituminous mixtures, the finer fraction of the filler has the highest surface area and the surface related physic-chemical properties are known to largely influence the performance of asphalt mixtures (Chapuis and Légaré, 1992). However, a certain amount of filler is
necessary in bituminous mixtures to obtain the required density and strength (Chapuis and Légaré, 1992). Basically, the filler particles fill a portion of the space between sand and gravel particles, and thus contribute to increase density (Chapuis and Légaré, 1992). Incorporation of fillers within the asphalt mixes also influence the optimum bitumen content in bituminous mixtures by increasing the surface area of mineral particles (Chapuis and Légaré, 1992). In fact, the presence of large amount of filler can affect negatively the performance of asphalt materials under certain environmental conditions. This finding is in good agreement with Miskovsky (2004) and Said et.al. (2009) who reported considerable deteriorations of the mechanical properties of the asphalt mixes with an increasing content of fillers in terms of free mica in the fine fraction depending on the interaction between different mineralogical and chemical influencing factors.

**EXPERIMENTAL PROCEDURES**

This paper presents laboratory tests carried out on samples selected from five quarries in Sweden. The micaceous fine aggregates from various quarries were collected and used in preparing the asphalt concrete mixture (AC) specimens in order to evaluate a satisfactory knowledge and prediction of the effect of filler composition and freeze-thaw cycles on the stiffness moduli of asphalt concrete samples.

Usually, stiffness of hot mix asphalt (HMA) has been used as an indicator of the pavement’s ability to bear traffic loads without undergoing excessive deformation (Singh, 2011). In addition, early deterioration of pavements due to rutting or fatigue cracking may be attributed to inadequate stiffness (Singh, 2011). Therefore, in this study, changes in stiffness moduli after conditioning have been considered as an indicator for water susceptibility of the asphalt mixtures.

**Properties of the Selected Materials**

Fine aggregates from five Swedish quarries with different percentages of mica have been involved in this study. Correspondingly, an extensive characterisation of the mechanical properties of the asphalt mixes with an increasing content of fillers in terms of free mica in the fine fraction depending on the interaction between different mineralogical and chemical influencing factors.

In this study, analysis of the mineralogical composition was carried out on representative samples of 24–42 µm grain fractions as recommended by Loorents and Said (2009). The aggregates fractions were examined under a polarizing microscope in order to estimate the proportion of free mica content in the fine aggregates adopted in this study according to Sims and Nixon (2003).

To further quantify the content of free mica grains, an additional analyzing step was performed by determining the SSA by the Brunauer–Emmett–Teller method (BET) of nitrogen adsorption (ISO method 9277, 1995) for grain size fractions 24–42 µm (Table 1). Since the roughness of the external surface, shape, porosity and mineralogy can affect the specific surface area (Brantly and Mellot 2000), it is important to mention that the shape of small particles of the filler aggregate studied tends towards platy and rod-like geometries (Santamarina et. al. 2002). Here, it is worth to notice that all of the minerals presented in Table 2 have some effect on each other and jointly with other factors; their influence on asphalt mixes performance can be increased.

<table>
<thead>
<tr>
<th>Quarry name</th>
<th>Mica content, % by weight</th>
<th>SSA, BET method (m²/g)</th>
<th>Rigden number volume (%) FAS method 252 (1998)</th>
<th>Maximum theoretical density (filler) in g/cm³, according to FAS method 228 (1998)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Karlstad</td>
<td>9.7</td>
<td>0.527</td>
<td>35.8</td>
<td>2.74</td>
</tr>
<tr>
<td>Hallsberg</td>
<td>15.4</td>
<td>0.4655</td>
<td>36.0</td>
<td>2.67</td>
</tr>
<tr>
<td>Hössjö</td>
<td>19.8</td>
<td>0.4016</td>
<td>35.1</td>
<td>2.65</td>
</tr>
<tr>
<td>Svarberget</td>
<td>31.8</td>
<td>0.3813</td>
<td>38.1</td>
<td>2.84</td>
</tr>
<tr>
<td>Ödsberget</td>
<td>22.3</td>
<td>0.5297</td>
<td>44.8</td>
<td>2.74</td>
</tr>
</tbody>
</table>
As shown in Table 2, greater amounts of black biotite mica and fewer amounts of white muscovite mica were observed for aggregate samples from different quarries. In addition, reasonable quantities of chlorite and quartz were also noticed in the collected materials.

**Preparation and Compaction of the Test Specimens**

The collected aggregate samples were used to manufacture five new asphalt mixture series according to the selected gradation shown in Table 3. As a result, these mixtures have approximately the same particle gradation as desired.

**Conditioning Procedure and Water Susceptibility Test**

The conditioning procedure started by determining the density and dimensions of each asphalt concrete sample by dimension measurements according to FAS Method 448-98. Then the specimens were soaked in distilled water and vacuum saturate for 10 min to a pressure of 6.7 kPa for 3 h and then 30 min at atmospheric pressure. After that the conditioned specimen's weight and volume were measured by dimension measurements and the resilient modulus tests were carried out using the Material Testing System (MTS). The tests were performed in accordance with FAS method 454-98 by determining the dynamic moduli of dry specimens as well as the dynamic moduli of soaked specimens as shown in Figure 1. It is important to note that all the stiffness modulus measurements were carried out at 10°C on AG16/160-220 asphalt mixes.

In order to have a better idea about the effect of environmental conditions on the stiffness of asphalt mixtures under study, the water susceptibility of the asphalt mixtures were tested by exposing the specimens to several freeze-thaw cycles followed by stiffness modulus measurements. Therefore, different asphalt specimens were stored in a plastic bag with distilled water and exposed to various freeze–thaw cycles at a temperature of -20 ± 1°C to +20 ± 1°C for 24 h and at each temperature, 12 h were supposed to be enough according to thermometer measurements on a dummy specimen.

Here, it is important to note that the specimens’ conditionings were checked visually for physical damage and if none was noticed the specimens were exposed to extra freeze-thaw cycles and then the stiffness modulus test were performed.

The purpose of cyclic freeze-thaw test consists of evaluating the effect of continuous freeze-thaw periods on the stiffness of asphalt mixtures by determining the percentage reduction of a dynamic modulus value of specimens after specified conditioning. The related results were calculated as the average of five test specimens for each quarry under specific soaking or freezing condition. The results from stiffness modules tests carried out on the adopted asphalt mixes are presented in Figure 2. The values shown in Figure 2 are the average values of five tested samples of each asphalt mix. It is almost certain from Figure 2 that the stiffness moduli decrease after soaking in water as well as after each conditioning process.

<table>
<thead>
<tr>
<th>Quarry name</th>
<th>Biotite (Particle%)</th>
<th>Muscovite</th>
<th>Chlorite</th>
<th>Quartz</th>
</tr>
</thead>
<tbody>
<tr>
<td>Karlstad</td>
<td>9.7</td>
<td>0</td>
<td>12.2</td>
<td>13.0</td>
</tr>
<tr>
<td>Hallsberg</td>
<td>14.8</td>
<td>0.6</td>
<td>3.0</td>
<td>20.8</td>
</tr>
<tr>
<td>Hössjö</td>
<td>17.8</td>
<td>2.0</td>
<td>3.1</td>
<td>17.3</td>
</tr>
<tr>
<td>Svarberget</td>
<td>31.8</td>
<td>0</td>
<td>2.7</td>
<td>9.5</td>
</tr>
<tr>
<td>Ödsberget</td>
<td>22.1</td>
<td>0.2</td>
<td>5.7</td>
<td>19.6</td>
</tr>
</tbody>
</table>

**TABLE 3: Aggregate Particle Gradations of the Asphalt Mixtures Adopted in this Study**

<table>
<thead>
<tr>
<th>Sieve size (mm)</th>
<th>.063</th>
<th>.125</th>
<th>.25</th>
<th>0.5</th>
<th>1</th>
<th>2</th>
<th>4</th>
<th>5.6</th>
<th>8</th>
<th>11.2</th>
<th>16</th>
</tr>
</thead>
<tbody>
<tr>
<td>Passing weight (%)</td>
<td>4.4</td>
<td>7.1</td>
<td>12.4</td>
<td>19.0</td>
<td>27.3</td>
<td>38.2</td>
<td>44.6</td>
<td>57.5</td>
<td>70.1</td>
<td>82.3</td>
<td>99.5</td>
</tr>
</tbody>
</table>

Regarding this study, the asphalt concrete samples series were manufactured in the VTI’s laboratory and each prepared series consisted of five specimens compacted using the gyratory compactor according to EN 12697-31 (2007).

In this investigation, the mix proportioning was carried out according to the Swedish Road Norms (2005), which illustrates mix design by weight percent. As a result, the bitumen content was 4.5 % and air void content was 7 % according to the recipe. More details about the average void contents with the corresponding number of gyrations for the studied asphalt mixtures are illustrated in Table 4 below.

**TABLE 4: The Average Void Contents with the Corresponding Number of Gyrations for the Studied Asphalt Mixtures**

<table>
<thead>
<tr>
<th>Asphalt Mixture Type</th>
<th>Air Void Content (%)</th>
<th>Number Of Gyrations</th>
</tr>
</thead>
<tbody>
<tr>
<td>Svarberget</td>
<td>7.2</td>
<td>37</td>
</tr>
<tr>
<td>Hössjö</td>
<td>7.3</td>
<td>35</td>
</tr>
<tr>
<td>Hallsberg</td>
<td>7.1</td>
<td>35</td>
</tr>
<tr>
<td>Karlstad</td>
<td>7.0</td>
<td>27</td>
</tr>
<tr>
<td>Ödsberget</td>
<td>7.0</td>
<td>26</td>
</tr>
</tbody>
</table>
FIGURE 2: The average stiffness moduli of dried and conditioned asphalt specimens

Based on this study, it has been found that the asphalt concrete specimens have relatively high resistance to relatively few freeze-thaw cycles.

Together with the current study, Höbeda (2000) reported that usually seven freeze–thaw cycles weakens most mixes but he manufactured the specimen with 3% higher air void content than recipe. It is observed that increasing the number of freeze and thaw cycles above that recommended by Höbeda (2000) causes a significant drop in the stiffness modulus values. To find out significant differences between tested mixes, the specimens were subjected to 35 freeze-thaw cycles. Concerning Figure 2, it can be noticed that after 35 freeze-thaw cycles, the stiffness moduli of asphalt samples decreased by 75.2, 74.5, 67.2, 74.9 and 62.31% for asphalt mixtures containing fillers from Svartberget, Ödsberget, Karlstad, Hallsberg and Hössjö respectively. As a result, asphalt mixtures containing fillers from Svartberget suffered from an optimal weakening in its performance regarding the average resilient modulus values after exposing to 35 freeze-thaw cycles on the other hand the average resilient modulus of asphalt mix containing filler from Hössjö was less affected by the similar number of freeze-thaw cycles. However, the interaction of influencing factors is behind such an observation.

MODELING OF THE RETAINED STIFFNESS MODULI

In this study, the role of SSA, mineralogical composition as well as the effect of freeze-thaw cycles on the retained stiffness moduli of asphalt samples have been predicted, and the validity of the predicted equations has been verified for data available from this study as well as previously published data by Said et. al. (2009).

In order to get a better idea about the factors influencing the dynamic characteristics of the tested samples, multi regression analysis has been performed. The results of the current study together with the previously published results by Said et. al. (2009) suggests the following multiple regression equation:

\[
\text{Retained MR} = 82.69 - 1.8 N + 0.375 M - 6.7363 \text{SSA} + 0.255 Q + 0.664 C \quad (1)
\]

where,

- Retained MR = The resilient modulus after conditioning divided by the resilient modulus of the corresponding dried asphalt samples in (\%),
- N = Number of freeze-thaw cycles,
- M = Percentage of mica particles in grain fraction 24–42 μm,
- SSA = Specific surface area of fine grain fractions, 24–42 μm in (m²/g),
- Q = Percentage quarts in mineral aggregate;
- C = Percentage Chlorite in mineral aggregate.

Moreover, the scattering points of measured and predicted retained stiffness moduli of the tested asphalt samples were plotted and the graphical comparison of the results is shown in Figure 3.

Figure 3 demonstrates that the calculated retained stiffness moduli are quite consistent with the measured retained stiffness moduli of tested asphalt mixtures. Nevertheless, the high coefficient of determination of Equation 1 (i.e. about 0.94) confirms the validity of the strong relationship between the durability of asphalt samples, conditioning and the mineralogical composition of the filler content. Consequently, Equation 1 takes into account the influence of Quartz and Chlorite in addition to the above-mentioned factors on the retained stiffness moduli of the tested samples.

FIGURE 3: Validity of Equation 1 in Modeling the Retained Stiffness Modulus of Asphalt Concrete Samples
mixes since the retained stiffness modulus of an asphalt mixture has an important role in indicating the rutting and fatigue characteristics.

**CONCLUSIONS AND RECOMMENDATIONS**

An overall conclusion drawn from this study is the significant influence and interaction between the fillers mineralogical composition with the number of freeze-thaw cycles and their influence on the retained stiffness moduli of asphalt samples.

In addition, in an attempt to verify the correlation between the content of mica, SSA and durability, a regression analysis was performed and verified using the data obtained from this study together with previously published data. As a result, strong correlation has been observed between the measured and predicted retained stiffness moduli of asphalt mixtures.

It will be interesting to validate the relationship between the mineralogical composition, freeze and thaw cycles, and retained stiffness moduli by new measurements. Surely, there are other factors influencing the performance and the durability of asphalt mixtures but they need also to be carefully inspected in future studies.

Furthermore, it would be valuable to correlate the retained stiffness of various asphalt mixes, determined in the laboratory, to the climate related deterioration of bituminous pavement layers under various climatic conditions encountered in the field.

The results from these investigations have given useful experience regarding the effect of material properties of asphalt mixes on durability of asphalt mixes.

Beside the fact that this study was focused on the Swedish aggregates and quarries, the developed regression equation may be applied to aggregates of similar compositions from other countries. However, if the aggregate composition is considerably differs from that used in this study, the influencing factors may be different and therefore require more relevant or custom study.

**REFERENCES**


Evaluation Of Airside Pavement Elements At Domestic Airports In Saudi Arabia

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ABSTRACT
The pavement structure of airside elements represented by runways, taxiways, and aprons at domestic airports within the Kingdom of Saudi Arabia have undergone deterioration due to increased traffic, environmental effects, construction related factors and age effects. It became imperative to develop a systematic approach to ensure that these airside elements operate at the highest level of safety and comfort. This paper describes an integrated functional, structural and safety program carried out at 13 domestic airports using advanced non-destructive techniques supplemented with field investigations and laboratory testing. The non-destructive evaluation included distress survey, deflection testing, friction testing, and rut and roughness determination including the development of Boeing Bump Indices (BBI) for heavy aircraft loading and the use of Ground Penetrating Radar (GPR) to determine pavement layer thicknesses to supplement and verify core/bore investigations. The paper presents the results and its use in developing a customized Airside Pavement Management System (APMS).

PROJECT METHODOLOGY
The project methodology as shown pictorially in Figure 1 consisted of the following:

- Information Collection through review of historic reports, construction records and air traffic data.
- Conducting topographic surveys of the paved and unpaved strip areas.
- Development of an inventory in the form of pavement network definition.
- Comprehensive pavement condition evaluation through non-destructive survey, field destructive testing and laboratory testing.
- Engineering Analysis and calculation of pavement performance indices and Pavement Classification Number (PCN) using industry-standard software programs and engineering judgment.
- Database development; preparation of short/long term maintenance/rehabilitation strategies.
- Implementation, Installation and Set-up of a customized APMS.
INFORMATION COLLECTION

Prior to conducting pavement evaluation surveys and field and laboratory investigations, an exercise was carried to acquire all relevant information related to the thirteen airports. These included the following:

- Construction and maintenance history obtained through as-built drawings, upgrades to the existing airside elements if any, previous design and evaluation reports, and maintenance records.
- Airfield physical data, which included location, airside layout, geometry and other features of the airside elements.
- Air Traffic data consisting of total air traffic volume in terms of arrivals and departures for the last ten years, aircraft types and weights, landing gear configurations and forecasted air traffic were obtained.
- Climatic data in terms of temperature and precipitation were also obtained and are the main factors considered with respect to pavement performance and pavement design.

PAVEMENT EVALUATION SURVEYS

A comprehensive non-destructive and destructive pavement evaluation survey program was carried out at each of the thirteen domestic airports to determine the following conditions:

- Functional Condition through visual distress survey.
- Structural Condition through Heavy Weight Deflectometer (HWD) survey, Ground Penetrating Radar (GPR) survey and coring/boring/Dynamic Cone Penetrometer (DCP) testing.
- Safety Condition through Roughness, Rut Depth and Surface Friction measurements.

Visual Distress Survey

Surface distresses in pavements are a manifestation of the inadequacies of the system and pavement distress surveys are often used to catalogue existing pavement deficiencies. A pavement condition survey is a visual inspection of airfield pavements to determine their present condition based on the types, severities and quantities of pavement distresses which is then converted into a numerical index termed as Pavement Condition Index (PCI) with a scale from 0 to 100 (1). The PCI forms the basis for determining engineering needs through functional evaluation for correcting these deficiencies through M&R strategies.

Based on the data collected from the pre-survey information and the topographic survey, the airside elements were divided into branches, sections and sample units and as per the procedure detailed in (1) to obtain statistically representative number of samples. Additional sample units were identified at each airport airside during actual field surveys based on ground conditions and engineering judgment.

Structural Condition Surveys

The structural evaluation of airfield pavements was carried out for determination of intrinsic material properties of various pavement layers which have undergone deterioration and aging over an extended service life and to determine existing layer thicknesses and the conditions of cracking. Each of the survey/testing is briefly described below.

HWD Testing

Non-destructive deflection measurements at all airports were taken using a HWD equipment capable of applying a dynamic force depending on the stiffness of the pavement structure and equivalent to a load up to approximately 260 kN (26 tons) that simulates the loading of a wheel of a fully loaded wide-body aircraft. The purpose of the HWD measurements was to measure the actual structural condition of the pavements and generate the requisite load-deflection data.
**GPR Survey**

GPR survey was conducted at all airports for the identification & measurement of pavement layer thicknesses as well as to detect voids and potential moisture related problems. Although cores are needed to calibrate the system, it reduces the number of cores, which is especially advantage at busy airports where air traffic may not allow long closures of the airside elements. The GPR testing was conducted in all tracks of HWD testing and along core locations at scan density of one scan every 33.3 cm with operating speed of 30 kph.

**Coring/DCP Testing/Laboratory Testing**

In order to obtain reliable information on as-built thicknesses of pavement layers and their in-situ strengths, coring and DCP testing was carried out at each airport. The cores also provided ground truths for calibrating the GPR results. The cores were extracted from different sections of the airside elements from asphalt and PCC pavements and were tested in the laboratory to determine their residual strengths.

**Safety Condition Surveys**

A detailed safety condition assessment was carried out at all airports included in the project and the methodology of each survey is described in the subsequent sub-sections.

**Friction Testing**

The condition of runway pavement surface should be such that an aircraft can land safely without skidding. The skid-resistance of runway pavements deteriorate due to a number of factors, the primary ones being mechanical wear and polishing action from aircraft tires rolling or braking on the pavement and the accumulation of contaminants, chiefly rubber, on the pavement surface. The effect of these two factors is directly dependent upon the volume and type of aircraft traffic. Other influences on the rate of deterioration are local weather conditions, the type of pavement; the materials used in original construction, any subsequent surface treatment, and airport maintenance practices.

**Roughness Survey**

Roughness is a safety issue and primarily causes discomfort. However, it can also cause fatigue damage to aircraft undercarriage, blow tires and influence the braking distance during a landing or an aborted take-off. For the aircraft manufacturers, the roughness of runways has always been a cause for concern regarding the fatigue life of the aircraft frame and landing gear. A criterion was developed for measuring roughness of civil runways, which were based on a single bump event and used for newly constructed pavements (4). Based on the findings, the maximum allowable vertical acceleration at the center of gravity of an aircraft is close to 0.35 g which agrees with the tolerance limits of g-forces for human beings at which nuisance and discomfort is experienced.

**DATA ANALYSIS AND DISCUSSION**

**Development of PCI**

Upon completion of the field distress survey, data was loaded and analyzed in terms of percentage breakdown distribution for distresses caused due to load, climate/durability/age, and others and to provide PCI values for airside elements at each airport indicating the effect of these distresses on the functional condition. Load related distresses for AC pavements include, but are not limited to, rutting, alligator cracking and corrugation and for PCC pavements, the predominant distresses are corner breaks, shattered slabs and faulting. Non-load related distresses include, but are not limited to, longitudinal and transverse cracking for both asphalt and PCC pavements, block cracking and reflection cracking for asphalt pavements, joint seal damage and spalling for PCC pavements.

A comparison was made of the PCI ratings based on the recent and previous surveys (1984-88, 1998-2000) carried out over a span of thirty years at some of the airports. The results revealed that the average pavement rating has dropped down to one or two levels and not drastically to a serious or failed condition. Keeping in view the average pavement age of 20 years and routine maintenance carried out at these airports, the PCI ratings are within the expected ranges.

**Structural Analysis**

Deflection data collected from HWD test points of the surveyed pavements were included in a detailed analysis procedure using a software program to determine the effective stiffness of various pavement layers. Figure 2 shows the deflection profiles and stiffness values along a runway measured at 3m left of centerline. A general observation is that the deflection levels range from medium to high for an airport pavement, although this depends on the type of aircraft using the pavements. There are some variations in longitudinal direction when looking at the deflection and back-calculated stiffness that is reflected from the different thickness layers along the runway and taxiway sections at some airport airside elements. This is either due to braking and turning effects at exits or due to sections constructed and receiving maintenance on different dates as well as having different pavement cross-sections.
In order to model the pavement structures into a linear multi-layer model, layer thicknesses and material types have to be known. General information was supplied based on historic as-built and coring information from previous studies as well as the coring and GPR survey as part of the present study.

The HWD deflection data was analyzed to determine the residual life of the pavement structures to withstand the aircraft traffic loadings based on loading of future aircraft. It is to be noted that these residual life estimates are essentially governed by the choice of fatigue equations and the predicted aircraft traffic loading.

In addition to residual life, PCN for each branch of each airside element of the thirteen airports was calculated using the following three methods:

- International Civil Aviation Organization (ICAO) Method (6) based on equivalent total pavement thickness and subgrade CBR.
- Federal Aviation Administration (FAA) Method (7).
- Based on in-situ measurements and back-calculated layer moduli values.

Table 1 shows typical PCN results for airside elements of one of the thirteen airports. The numbers highlighted in bold indicate that the particular airside element have PCN values less than the Aircraft Classification Number (ACN) for the particular aircraft. This indicates the need for either M&R treatment for increasing the PCN value in order for the aircraft to use the facility or apply weight limitations for the particular aircraft to use the particular airside facility such that the PCN is either equal to or greater than the ACN of the aircraft.
TABLE 1: PCN Results for Sharurah Airport

<table>
<thead>
<tr>
<th>Airside Element/ Sub-sections</th>
<th>PCN 85th Percentile Value</th>
<th>ACN B747-400ER</th>
</tr>
</thead>
<tbody>
<tr>
<td>RWY 08-26</td>
<td>57/F/A/X/T</td>
<td>57</td>
</tr>
<tr>
<td>PTWY A</td>
<td>57/F/A/X/T</td>
<td>57</td>
</tr>
<tr>
<td>CTWYs A to J</td>
<td>57/F/A/X/T</td>
<td>57</td>
</tr>
<tr>
<td>CTWY K</td>
<td>78/F/C/X/T</td>
<td>78</td>
</tr>
<tr>
<td>CTWY L, M, N</td>
<td>57/F/A/X/T</td>
<td>57</td>
</tr>
<tr>
<td>APRON A (Branch AA1 - AC)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>All Sections</td>
<td>57/F/A/X/T</td>
<td>57</td>
</tr>
<tr>
<td>APRON A (Branch AA2 - PCC)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>G-1</td>
<td></td>
<td></td>
</tr>
<tr>
<td>G-2 and G-3</td>
<td>69/R/B/X/T</td>
<td></td>
</tr>
<tr>
<td>81/R/C/X/T</td>
<td>70</td>
<td></td>
</tr>
<tr>
<td>82</td>
<td></td>
<td></td>
</tr>
<tr>
<td>APRON B (Branch B1 – AC Layer Over PCC)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>All Sections</td>
<td>7/R/A/X/T</td>
<td>59</td>
</tr>
<tr>
<td>APRON B (Branch B2 – PCC)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Centerline</td>
<td>48/R/D/X/T</td>
<td>92</td>
</tr>
<tr>
<td>APRON</td>
<td></td>
<td></td>
</tr>
<tr>
<td>All Sections</td>
<td>78/F/C/X/T</td>
<td>78</td>
</tr>
<tr>
<td>DISPERSAL APRONS</td>
<td></td>
<td></td>
</tr>
<tr>
<td>F-1</td>
<td>78/R/C/X/T</td>
<td>82</td>
</tr>
<tr>
<td>F-2</td>
<td>58/R/C/X/T</td>
<td>82</td>
</tr>
<tr>
<td>F-3</td>
<td>68/R/D/X/T</td>
<td>92</td>
</tr>
<tr>
<td>F-4</td>
<td>41/R/D/X/T</td>
<td>92</td>
</tr>
<tr>
<td>F-5</td>
<td>41/R/D/X/T</td>
<td>92</td>
</tr>
<tr>
<td>F-6</td>
<td>20/R/C/X/T</td>
<td>82</td>
</tr>
<tr>
<td>F-7</td>
<td>48/R/D/X/T</td>
<td>92</td>
</tr>
<tr>
<td>F-8</td>
<td>31/R/C/X/T</td>
<td>82</td>
</tr>
</tbody>
</table>

TABLE 2: Friction Level Classification for Runway Pavement

<table>
<thead>
<tr>
<th>Testing Speed</th>
<th>65 kph</th>
<th>95 kph</th>
</tr>
</thead>
<tbody>
<tr>
<td>Friction Value (Mu Value)</td>
<td>Minimum</td>
<td>Maintenance Planning</td>
</tr>
<tr>
<td>0.42</td>
<td>0.52</td>
<td>0.72</td>
</tr>
</tbody>
</table>

Skid Resistance

Friction values measured by CFME can be used as guidelines for evaluating the surface friction deterioration of runway pavements and for identifying appropriate corrective actions required for safe aircraft operations. Table 2 provides the friction values for three classification levels for CFMEs operated at 65kph and 95kph test speeds. These guidelines take into account that poor friction conditions for short distances on the runway do not pose a safety problem to aircraft, but long stretches of slippery pavement are of serious concern and require prompt remedial action.

Surface

Figure 3 shows a typical skid profile along a runway track at testing speed of 95kph. It can be observed that friction values are approaching the minimum friction level (0.26) at touchdown areas and at 3m from runway centerline location where the landing gears of most of the aircrafts using the airport are located. Similar analysis was carried out for all airside elements at each airport, which helped in determining, and applying corrective maintenance treatments related to skid resistance.
Roughness and Roughness Indices

The International Roughness Index (IRI) is a worldwide accepted standard for road pavements. Only recently more attention has been paid to the roughness of airfield pavement, especially directed to the dynamic behaviour of aircraft. The Boeing Bump Index (BBI) based on the guidelines and procedures set forth by the FAA (8) is the more widely used method for reporting roughness on airfield pavements as compared to the IRI. To be able to judge roughness, the longitudinal profile has to be measured based on a reference height, an offset of this reference height and horizontal distance.

DEVELOPMENT OF CUSTOMIZED APMS

An APMS customized to KSA airport requirements was developed as an end-product of this project and is an effective tool to quantify current pavement condition, anticipate future problems, as well as select and schedule the best M & R strategy at the appropriate time (9). It is a decision support tool, which provides a systematic, objective and consistent procedure to evaluate existing and future pavement condition and identify sections requiring preventive and corrective treatments. The APMS with the combination of technologically advanced pavement evaluation procedures and engineering analysis software programs can provide GACA with the necessary tools to maximize pavement condition, manage pavement maintenance expenditure more economically and efficiently and allow cost-effective prioritization and allocation of resources. The key elements of the GACA-APMS are as follows:

- Create and build Airports airside element database
- Aircraft traffic data inventory
- Pavement evaluation data analysis and reporting
- Budget estimating
- APMS-GIS Integration

CONCLUSIONS

General observations related to all the airports based on analysis of results are summarized as follows:
• The functional and structural analysis indicates that non-keel sections of runway and taxiway pavements are in comparative better condition than the keel sections. This is expected as landing gears and wheelbases of aircrafts are concentrated in the keel section with maximum landing gear spacing of 11m for a wide-body aircraft.

• Most of the distresses noted on the pavement surfaces are related to the effects of climate, durability and aging.

• A comparison of the deflection profiles of the loaded tracks at 3m on either side of the runway centerline indicate similar variation in deflection levels or structural condition except at exit taxiways where the deflection is slightly more on the side from where the aircraft is exiting.

• Friction Testing results at both testing speeds of 65kph and 95kph indicate that at all airports, the values are within the Maintenance Planning Level for all track lanes. Exceptions were noted on the touchdown areas of runways at few airports where the friction values were approaching the Minimum Friction Levels.

• The results of the roughness measurement show that the pavement in the keel section tracks have a higher IRI in comparison to the tracks of the un-loaded pavement.

• Results based on the BBI are within the range of acceptable smoothness over the total length of the runway longitudinal profiles at all airports. The relative roughness at the beginning and end of each track is most probably caused by the difference between the concrete runway ends and the asphalt mid section and at the end is most probably because of braking forces.

• The layer thicknesses measured from GPR Testing, coring/boring and information extracted from previous reports for different tracks of runway, taxiways and aprons showed a good correlation.

• The overall conclusion is that at majority of the asphalt pavements require either localized corrective maintenance or rehabilitation in terms of milling and paving of the asphalt layers to correct surface distresses and improve skid resistance. The PCC pavements at aprons are in good condition with corrective maintenance treatment such as joint and crack re-sealing, spall repair and shallow patches to be carried out at specific locations.

REFERENCES
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