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How Good Are Linear Viscoelastic Properties of Asphalt Binder to Predict Rutting and Fatigue Cracking?

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This article evaluates the effects of linear viscoelastic properties of asphalt on pavement rutting and fatigue cracking. The parameters in the binder specification recently developed by the Strategic Highway Research Program (SHRP) were also compared for pavement performance. Two studies were conducted for asphalt-aggregate mixes. The first study was the wheel tracking test to evaluate the rutting of mixes containing three asphalts. The second study was a detailed field study of the effects of binder properties on the pavement performance of eight different sections. Results of both investigations indicated that SHRP parameters were not sufficient indicators for predicting the rutting and fatigue cracking of pavements. The discrepancies between performance data and existing parameters of the binder mainly resulted from the inherited assumptions made during the specification development, that is, stress- or strain-controlled mode and traffic loading frequency. In order to directly relate the linear viscoelastic properties of asphalt binders to pavement performance, calculating the dissipated energy per traffic cycle, W_d, became imperative. Fundamental derivation of W_d was developed in this study. Results indicated that W_d could predict the rutting and fatigue cracking of pavements reasonably well. This study, proposed the dissipated energy, W_d, as the single parameter for evaluating pavement rutting and fatigue cracking.

Keywords linear viscoelastic properties, pavement fatigue cracking, pavement rutting

1. Introduction

The American Association of State Highway and Transportation Officials (AASHTO) published its first asphalt binder specification based on penetration in 1931. Since then, empirical tests—that is, penetration and viscosity—have been used by highway agencies in Taiwan. The penetration test is commonly performed at 25 °C with a 100 g weight needle allowed to penetrate for 5 s and measured at 0.1 mm increments. Capillary viscosity is used to define viscosity of asphalt cements at 60 and 135 °C. These traditional binder properties are then used as a substitute indicator for predicting the pavement performance.

However, traffic flow has increased tremendously. There has been more than a 20 times increase in traffic volume in the past two decades in Taiwan. Overloading vehicles with 12,000 kg single axle loads are rampant, with 8,000 kg being the maximum tolerance. The high tire pressure pumped by truck drivers to carry more goods is close to 1.4 MPa (200 psi). Climatic effects such as temperatures and precipitation directly contribute to the deterioration of pavement performance. These demanding changes have troubled highway engineers all over the world. As a result, pavements are subjected to premature rutting and fatigue cracking. One third of all roadways, approximately one billion square meters, have to be rehabilitated annually in Taiwan (Ref 1).

Traditional methods used to test asphalt binder were developed in the early 1900s. These testing procedures include penetration, capillary viscosity, and softening point. Based upon these methods, three sets of asphalt grading systems, that is, penetration (PEN), viscosity (AC), and aged residue (AR), have been adopted. Conventional specifications are not, however, performance related. The measurements obtained from previous tests are (a) empirical in nature, (b) can be deceptive to pavement performance at higher or lower service temperatures, and (c) do not include fundamental engineering properties that can be related to pavement performance.

The established correlations between conventional binder properties and pavement performance fail to provide an appropriate indication for pavement distresses. Consequently, highway agencies realize that, to ensure the proper qualities for constructing endurable pavements, binder specifications should be related to pavement performance. Asphalt binders are viscoelastic materials that behave like elastic steel at low temperatures, show themselves as viscous honey at intermediate temperatures, and flow like water at high temperatures. Asphalt technology has to be changed to keep pace with a dynamic and ever growing transportation system that is placing severe demands upon the structure capacity of asphalt pavements. It is imperative to specify asphalts for construction on the basis of their potential performance.

From 1988 to 1992, the Strategic Highway Research Project (SHRP) was conducted to address these problems. The United States led SHRP program was a comprehensive study on highway facilities and cost $150 million. The major product of SHRP was a set of performance-related test methods and specifications for hot-mix asphalt concrete. These test methods and specifications are an integrated system for characterizing and specifying materials (binders and mixtures) for designing asphalt concrete mixtures, predicting maximum and minimum

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pavement temperatures, modeling pavement response to traffic load and the environment, and predicting pavement performance. The SHRP addressed a number of different pavement distress mechanisms: rutting, load-associated fatigue, and thermal cracking caused by low pavement temperatures.

Researchers in this $150 million SHRP project made tremendous efforts to determine viscoelastic behaviors of asphalt binders and recommended the performance-graded (PG) specification (Ref 2). Performance-graded binders are graded, such as PG 58-16. The first number, 58, is often called the “high temperature grade.” This means that the binder would possess adequate physical properties at least up to 58 °C. Likewise, the second number, 16, is often called the “low temperature grade” and means that the binder would possess adequate physical properties in pavements at least down to –16 °C. The new system for specifying asphalt binders is unique because it is a performance-based specification. It specifies binder on the basis of the climate and attendant pavement temperature in which the binder is expected to serve.

The Taiwan Ministry of Transportation and Communication (MOTC) began a three-year research program in 1997 to evaluate the new binder specification developed by the SHRP. If the concepts in the SHRP system would prove valid for Taiwan’s pavement conditions, the intention is to change the current binder specifications to the new system in the near future.

Products from SHRP nevertheless need to be further evaluated for their applicability to pavement performance. If the experiment data obtained from devices by SHRP can be successfully validated in the field, SHRP will become the future mixture design method.

The viscoelastic parameters used in the PG-based binders are, however, not intensively verified to the extent that they are suitable to all the environmental and loading conditions. It is a general concern that, without a thorough investigation, the direct application of SHRP parameters to select asphalt may cause adverse consequences that could deteriorate pavements early. Contradictory results on using these parameters to predict pavement performance have been reported (Ref 3-5). Furthermore, the theoretical development of the SHRP binder specification is not clearly presented anywhere in literature (Ref 3, 6-10). Asphalt technologists find it difficult to interpret the test results without understanding the theory. In addition, the effect of inherited assumptions on rutting and cracking has not been validated for in situ pavements.

In recognition of these shortcomings, the goal of this study was (a) to develop a binder rheological model that reflects pavement conditions, (b) to derive fundamental parameters that are related to asphalt pavement performance, and (c) to verify the derived parameters with rutting and cracking data. The transition from present grading systems to the new criteria would be possible if pavement performance and predicted values were in reasonable agreement. Based on pavement distresses in Taiwan, pavement performance across the temperature range was considered for permanent deformation at high service temperatures (40 to 60 °C) and fatigue cracking at intermediate service temperatures (10 to 30 °C). It should be noted that low-temperature cracking is not one of the distress modes discussed in this article because Taiwan is located in the subtropical zone.

2. Materials and Methods

2.1 Materials

Asphalt Binders. Three different types of asphalt binders manufactured by the Chinese Petroleum Corporation were selected for this study. These asphalts, shown in Table 1, represented a wide range of practical usage for pavement construction in Taiwan. The conventional tests, including penetration, soft point, and viscosity, were empirical in nature, and the performance models based on these tests are considered to be speculative (Ref 9-10).

Asphalt-Aggregate Mixes. Crushed limestone was selected for the research so that differences in rutting and fatigue cracking behavior could be attributed only to the asphalt binders used. This dense mix, as shown in Table 2, was typical of mixtures used on high-volume highways in Taiwan. Mix designs were developed following the 75 blow Marshall procedures currently employed by the highway authority. A grading of 19 mm maximum aggregates was mixed with a binder content of 5.4% and an air void of 4.7%.

2.2 Test Methods

Dynamic Shear Rheometer (DSR). The dynamic shear rheometer (DSR) was used to characterize the viscous and elastic behavior of asphalt binders at high and intermediate service temperatures. The DSR, a CSR-500 model (Carri-Med Ltd., Dorking, England), was used to characterize viscoelastic behavior of asphalt binders. It measured the complex shear modulus, G^*, and phase angle, δ, of asphalt binders by subjecting a small sample of binder to oscillatory shear stress. The G^* is a measure of the total resistance of a material to deformation when repeatedly sheared. The δ is an indicator of the relative amount of recoverable and nonrecoverable deformation.

Two measurements for each asphalt were obtained over a range of frequencies to determine the time dependency of the asphalt binder. Asphalt was sandwiched between the oscillating spindle and the fixed steel plate. The DSR measured G^* and δ by measuring the strain response of the specimen to a fixed torque. The normal procedure was to deposit a molten sample of a binder on the heated rotor and raise the plat so that a 2 mm

Table 1 Conventional properties of asphalt binders

Code	Grade	Viscosity at 60 °C, poise	Viscosity at 135 °C, cSt	Softening point, °C	Penetration at 25 °C, 0.1 mm
A	85/100 pen	1029	289	48	98
B	60/70 pen	1992	569	52	64
C	40/50 pen	3862	543	49	47

binder film was formed before testing. Asphalt was tested at 60 °C for rutting and 20 °C for fatigue cracking. A computer was used with the DSR to control test temperatures and record test results. By measuring G^* and δ, the DSR provided a complete picture of the behavior of asphalt at pavement service temperatures.

Aging Tests. Two types of aging tests were performed in this study: thin-film oven test (RTFOT) and pressure aging vessel (PAV). The former test was used to simulate the early oxidation of asphalt in the pugmill according to ASTM D 2872. The RTFOT involved a moving film of asphalt material heated in an oven for 85 min at 163 °C. The PAV developed by SHRP aged asphalt in the laboratory to simulate the severe aging that occurs after the binder has served many years in a pavement. The PAV apparatus consisted of the pressure aging vessel and temperature chamber. A cylinder of dry, clean compressed air provided air pressure with a pressure regulator, release valve. In this study a 2.1 MPa pressure of air gas was applied to asphalts conditioned at 100 °C for 20 h.

Wheel Tracking Test. A wheel-tracking tester was performed to evaluate the susceptibility of a mixture to permanent deformation. This equipment is similar to the Hamburg wheel-tracking device, which was shown to be one of the adequate accelerated wheel test devices used to simulate the effect of traffic on pavements (Ref 11). Mixture samples of different binders were carefully controlled to have the same binder content, air void content, gradation, and aggregate type as used on the field. To predict rutting, the test was run at the mean highest weekly average temperature proposed by SHRP, which was set at 60 °C under dry conditions. A smooth solid steel track traveling at a speed of 1.44 km/h was used. Rut depths were measured every 200 wheel passes on 300 by 300 by 70 mm samples trafficking by a tire pressure of 540 kPa (80 psi).

Two rutting parameters were measured from the wheel tracking test: total rut depth and normalized rut rate. The total rut depth was the rut depth at the end of the test, that is, after 8000 passes. The normalized rut rate was the rate of increase in rut depth (0.01 mm/cycle) between 4000 and 8000 loading passes. The normalized rut rate was considered to be a more reliable indicator of permanent deformation because it is less likely to be affected by the initial compacting "errors."

3. Database and Field Study

A database administered by the Ministry of Transportation and Communication was employed in this study for the pavement performance. Data were collected from pavement survey reports and carefully documented by the engineer on the field, and later on, double checked for any ambiguity or error. Traffic volume and loading, environment conditions, and material properties were closely monitored. Under MOTC, a ten-year old database containing traditional properties of asphalt binders (penetration, viscosity, softening point, etc.), pavement distresses, and temperatures was maintained.

Table 2 Aggregate gradation

Sieve	Passing, %
1 in.	100
¾ in.	90
⅜ in.	70
No. 4	56
No. 8	42
No. 30	24
No. 50	18
No. 100	11
No. 200	4

Eight pavement sections constructed on a primary highway in Kaoshuing, a southern county in Taiwan, were selected for this study. The sections representing typical traffic flow and environment conditions were aged differently to assist in validating tests for rutting and fatigue cracking. Each pavement has a 500 m section designed according to Taiwan procedures. After traffic estimated to be approximately from 10^5 to 5×10^6 equivalent single axle load (ESAL), two cores were taken from each section of 10 cm thickness pavements to extract asphalts for further binder testing. Equivalent single axle load is the single traffic parameter for pavement performance purpose, that is, a summation of the equivalent effects of all axle loads resulting in pavement distresses. A single mixture on top of a crushed aggregate base was used for each section so that pavement performance is a function of asphalt binders. Distress surveys were conducted on each pavement during trafficking using the *Distress Identification Manual for the Long-Term Pavement Performance Project* (Ref 12). The surveys included transverse profile, longitudinal profiles, and the number and severity of cracks.

4. Theoretical Background

4.1 Development of Rheological Model

In order to understand how binders perform in pavements, it is important to characterize the linear viscoelastic response of asphalts under dynamic loading conditions. Asphalt is assumed to be subjected to a sinusoidal strain. When equilibrium is reached, both the stress and strain will vary sinusoidally, but the stress will lag behind the strain, that is:

$$\varepsilon = \varepsilon_0 \cdot \sin(\omega t) \qquad \text{(Eq 1)}$$

$$\sigma = \sigma_0 \cdot \sin(\omega t + \delta) \qquad \text{(Eq 2)}$$

where ω is the angular frequency and δ is the phase lag, that is, phase angle. Expanding the stress equation:

$$\sigma = \sigma_0 \cdot \cos(\delta) \cdot \sin(\omega t) + \sigma_0 \cdot \sin(\delta) \cdot \cos(\omega t) \qquad \text{(Eq 3)}$$

The storage modulus, $G' = (\sigma_0/\varepsilon_0) \cdot \cos(\delta)$, and the loss modulus $G'' = (\sigma_0/\varepsilon_0) \cdot \sin(\delta)$ were introduced, thus:

$$\sigma = \varepsilon_0 \cdot G' \cdot \sin(\omega t) + \varepsilon_0 \cdot G'' \cdot \cos(\omega t) \qquad \text{(Eq 4)}$$

In the case of a linear viscoelastic response, the rheological model of asphalt can be expressed by energy dissipated per cycle, W_d. It was derived as follows:

$$W_d = \int_0^{2\pi/\omega} \sigma \cdot \mathrm{dt} = \int_0^{2\pi/\omega} \sigma \cdot \frac{d\varepsilon}{dt} \cdot dt = \int_0^{2\pi/\omega} \varepsilon_0 \cdot [G' \cdot \sin(\omega t)$$

$$+ G'' \cdot \cos(\omega t)] \cdot \frac{d\varepsilon}{dt} \cdot dt = \int_0^{2\pi/\omega} \varepsilon_0 \cdot [G' \cdot \sin(\omega t)$$

$$+ G'' \cdot \cos(\omega t)] \cdot [\omega \cdot \varepsilon_0 \cdot \cos(\omega t)] \cdot dt$$

$$= \omega \cdot \varepsilon_0^2 \int_0^{2\pi/\omega} [G' \cdot \sin(2\omega t)/2 + G'' \cdot \cos^2(\omega t)]dt = \pi G'' \cdot \varepsilon_0^2 \qquad \text{(Eq 5)}$$

where Eq 1 and 3 have been used. It can be seen that W_d is in proportion to the loss modulus of asphalts and the strain applied to materials. The larger the dissipated energy, the more susceptible the asphalt binder will be when subjected to pavement distress.

However, in the development of SHRP specification, no derivation can be found. The W_d was assumed to be (Ref 6-8):

$$W_d = \pi \cdot \sigma_0 \cdot \varepsilon \cdot \sin\delta \qquad \text{(Eq 6)}$$

Based on Eq 6, the parameters for rutting and cracking were then determined in the SHRP specification. They are discussed in the following sections.

4.2 Rutting Criteria of SHRP

At temperatures in the range of 40 to 60 °C, which are typical of the highest pavement in-service temperatures, the main distress mechanism is rutting. Pavement rutting is assumed to be stress-controlled, σ, cyclic loading phenomenon in the SHRP specification (Ref 5-7). It is considered that part of W_d is recovered in elastic rebound of the surface layer, while the remaining work is dissipated in the permanent deformation and heat. For a stress-controlled repetitive phenomenon, $\sigma = \sigma_0$ and $\varepsilon = \sigma_0/G^*$, W_d is shown as:

$$W_d = \pi \cdot \sigma_0 \cdot \varepsilon \cdot \sin\delta = \pi \cdot \sigma_0 \cdot \sigma_0/G^* \cdot \sin\delta$$

$$= \pi \cdot \sigma_0^2/(G^*/\sin\delta) \qquad \text{(Eq 7)}$$

where G^* is the complex modulus. During each traffic loading, a certain amount of work is being done in deforming the surface layer. To minimize rutting, W_d should be minimized. This means that $G^*/\sin\delta$ should be maximized to control permanent deformation. For rutting resistance a high G^* value is favorable because it represents a higher total resistance to deformation. A lower $\sin\delta$ is desirable because it reflects a more elastic component of the total deformation. The basic consideration behind the SHRP parameter $G^*/\sin\delta$ is that the contribution of the binder to rutting resistance can be increased by increasing its total resistance (G^*) and/or decreasing its nonelasticity ($\sin\delta$).

The SHRP, thus, used the parameter $G^*/\sin\delta$ to rank the rutting susceptibility of pavements. Asphalt needs to be measured on the original binder at average seven day maximum pavement temperature and at a frequency of 10 rad/s. A frequency of 10 rad/s was assumed to simulate the average frequency of a stress wave in the surface layer of a typical pavement as caused by a vehicle moving at 80 to 100 km/h.

4.3 Proposed Rutting Criteria

It is argued in this study that the assumption of the stress-controlled mode for representing permanent deformation in SHRP may not reflect the actual situation of pavement rutting. The frequency at 10 rad/s may not correspond to the traffic speed inducing deformation. Researchers were speculative about the adequacy of using the parameter $G^*/\sin\delta$ to predict pavement rutting (Ref 3, 9, 10). As derived in Eq 5, the mechanism of asphalts of contributing to rutting is directly related to the dissipated energy, W_d, regardless of the stress-controlled test. The more W_d, the more energy used for pavement deformation. The W_d should be limited to a certain value. In other words, under the same temperature and loading frequency, asphalts with lower W_d values are expected to better resist rutting.

Table 3 SHRP classification of asphalts used in this study

Test	Property	Asphalt A	Asphalt B	Asphalt C	SHRP specification
Rotational viscosity, unaged	Viscosity at 135 °C	0.28 Pa · s	0.50 Pa · s	0.51 Pa · s	3 Pa · s max
Dynamic shear, unaged, 10 rad/s	$G^*/\sin\delta$ at 58 °C	1.29 kPa	2.53 kPa	3.29 kPa	1.00 kPa min
	$G^*/\sin\delta$ at 64 °C	0.56 kPa	1.07 kPa	1.47 kPa	1.00 kPa min
	$G^*/\sin\delta$ at 67 °C	…	0.73 kPa	1.02 kPa	1.00 kPa min
	$G^*/\sin\delta$ at 70 °C	…	0.50 kPa	0.71 kPa	1.00 kPa min
RTFOT, mass loss	…	0.09%	0.09%	0.07%	1.00% max
Dynamic shear, RTFOT, 10 rad/s	$G^*/\sin\delta$ at 58°C	2.36 kPa	4.83 kPa	…	2.20 kPa min
	$G^*/\sin\delta$ at 64 °C	1.00 kPa	2.14 kPa	3.29 kPa	2.20 kPa min
	$G^*/\sin\delta$ at 67 °C	…	…	2.29 kPa	2.20 kPa min
Dynamic shear, PAV, 10 rad/s	$G^* \cdot \sin\delta$ at 16 °C	6417 kPa	9848 kPa	…	5000 kPa max
	$G^* \cdot \sin\delta$ at 19 °C	3925 kPa	6807 kPa	6916 kPa	5000 kPa max
	$G^* \cdot \sin\delta$ at 22 °C	2500 kPa	5284 kPa	4691 kPa	5000 kPa max
	$G^* \cdot \sin\delta$ at 25 °C	…	3423 kPa	…	5000 kPa max
PG grade		PG 58-28	PG 58-22	PG 67-22	…

4.4 SHRP Fatigue Cracking Criteria

Within the intermediate temperature, 10 to 30 °C, asphalts are relatively harder and more elastic than at higher temperatures. The prevailing failure mode at these temperatures is fatigue cracking, which is caused by repeated cycles of loading at levels lower than the static strength of an asphalt. For viscoelastic materials like asphalts, both G^* and δ play a role in damage induced by fatigue. The SHRP specification stipulates that fatigue cracking is primarily due to repeated loading on the pavement surface, and a strain-controlled (ε) phenomenon is assumed (Ref 7-8). For a strain-controlled repetitive phenomenon, $\varepsilon = \varepsilon_0$ and $\sigma = \varepsilon_0 \cdot G^*$, W_d can be expressed as:

$$W_d = \pi \cdot \sigma \cdot \varepsilon_0 \cdot \sin\delta = \pi \cdot (\varepsilon_0 \cdot G^*) \cdot \varepsilon_0 \cdot \sin\delta$$
$$= \pi \cdot \varepsilon_0^2 \cdot (G^* \cdot \sin\delta) \qquad \text{(Eq 8)}$$

To prevent fatigue cracking, W_d should be minimized. This means that $G^* \cdot \sin\delta$ should be minimized to control fatigue cracking.

The thought associated with the parameter $G^* \cdot \sin\delta$ is that the amount of work dissipated is directly proportional to $G^* \cdot \sin\delta$. Asphalts with lower G^* value will be softer and, thus, can deform without developing large stresses. In addition, asphalts with lower δ values will be more elastic and, thus, will recover to their original condition without dissipating energy in any manner. The SHRP, thus, uses $G^* \cdot \sin\delta$ measured on the PAV aged residue at an intermediate pavement temperature and at a frequency of 10 rad/s to rank the fatigue cracking susceptibility of asphalts.

4.5 Proposed Fatigue Cracking Criteria

Because of its inherited assumptions, $G^* \cdot \sin\delta$ may not be a good indicator for evaluating fatigue cracking. When pavement is subjected to traffic loading, part of the work from the traffic loading is recovered in elastic rebound of pavement, and the remaining work is dissipated as a result of fatigue cracking. The more energy dissipates, the more likely pavements crack. The dissipated energy, W_d, represented by Eq 5 seems to be more in line with pavement cracking. It has nothing to do with whether the test is run under a strain-controlled mode or not. Unsatisfactory results on using $G^* \cdot \sin\delta$ to predict the pavement cracking have been reported (Ref 3, 4).

5. Results and Discussions

5.1 Binder Properties

Because the low-temperature cracking is not a concern in Taiwan, the SHRP PG grading is only based on unaged RTFOT and PAV asphalts tested by the dynamic shear rheometer (DSR) in this study. Asphalt B (Pen 60/70) was expected to be classified as higher grade (i.e., PG 64-xx) than the asphalt A (Pen 85/100). Examination of Table 3 showed that asphalt B is stiffer at all temperatures than asphalt A; however, asphalt B barely failed the RTFOT residue stiffness requirement ($G^*/\sin\delta > 2.2$ kPa) at 64 °C with a value of 2.14 kPa. The physical properties of asphalts A and B belonged to the upper and lower limits of a PG 58 grade respectively. Although both asphalts A and B were graded the same (PG 58) at high temperatures, they may perform differently for rutting.

Asphalt C passed both rutting requirements at 67 °C. By comparing the southern region of Taiwan where high pavement temperature (>60 °C) and heavy traffic ($>3 \times 10^7$ ESAL) occur, it is reasonable to assume that a PG 64 or higher grade will be needed. Current SHRP grades show a big gap between PG 64-xx and PG 70-xx. To be on the safe side of pavement distresses, a PG 67 grade was recommended in the study to meet the specific requirements of Taiwan climate and traffic. Asphalt C, graded as PG 67-22, appeared to be the better selection of binders that can be used in the southern Taiwan. Asphalts A and B were recommended to apply to the northern and middle regions of Taiwan respectively, where temperature in summer is relatively mild.

5.2 Wheel Tracking Test Results

The SHRP binder specification was based on traffic speeds of 80 to 100 km/h while the wheel tracking test traveled at 1.44 km/h (Ref 8). It was reasoned that the slow speed of the wheel tracking would make the subjected pavements too severe. Figure 1 shows that the relationship between rutting rate and $G^*/\sin\delta$ at 10 rad/s is not good, with a coefficient of determination, R^2, value of 0.46. Rutting and fatigue cracking are a function of frequency of loading. The rate of loading of pavement under traffic, thus, needs to be simulated in the measurement to obtain a reliable estimate of the binder's contribution to pavement performance.

In this study the DSR frequency corresponding to the pavement and mixture tests was chosen with 10 rad/s being equivalent to 80 km/h. In order to make the comparison at the same loading rate, asphalts should be tested at a frequency of 0.18 rad/s (corresponding to the track speed 1.44 km/h) under a temperature of 60 °C. However, the closest frequency of loading that could be obtained experimentally was 0.6 rad/s. The improvement of using parameter $G^*/\sin\delta$ to predict rutting rate is indicated in Fig. 2 in which the R^2 value is 0.57. The correlation was improved by the change in loading frequency, but the parameter $G^*/\sin\delta$ was found not to be very reliable to predict pavement rutting. This finding is in good agreement with other researchers (Ref 9, 10).

Figure 3 shows a good relationship between rutting and the dissipated energy, W_d, with an R^2 equal to 0.72. The proposed parameter, that is, W_d, appeared to be a better indicator for pre-

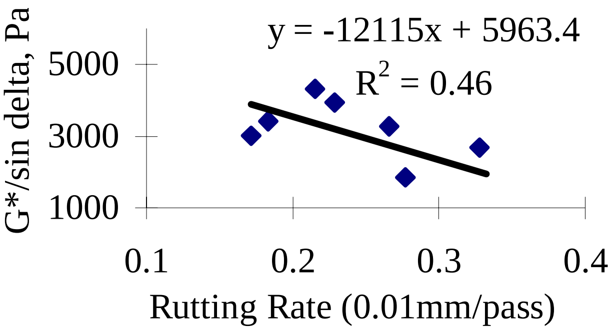

Fig. 1 Rutting rate and $G^*/\sin\delta$ at 10 rad/s from wheel tracking test

dicting the rutting rate than $G^*/\sin\delta$ at the same loading rate. Decreasing W_d confirms the direction of decreased permanent deformation; thus, a minimum W_d value needs to be set to prevent rutting. The implication is that, if the W_d value exceeds this limit, the binder would provide acceptable resistance to permanent deformation in asphalt mixes, whereas binders with lower W_d value may contribute less to rutting.

5.3 *In Situ Rutting Evaluation*

The plot of $G^*/\sin\delta$ versus rutting depth from SHRP, shown in Fig. 4, has a poor R^2 value of 0.44. The SHRP researchers based their specification on the presumption that the inverse shear loss compliance of the binder plays an eminent role in preventing rutting (Ref 6, 7). The SHRP assumptions of stress-

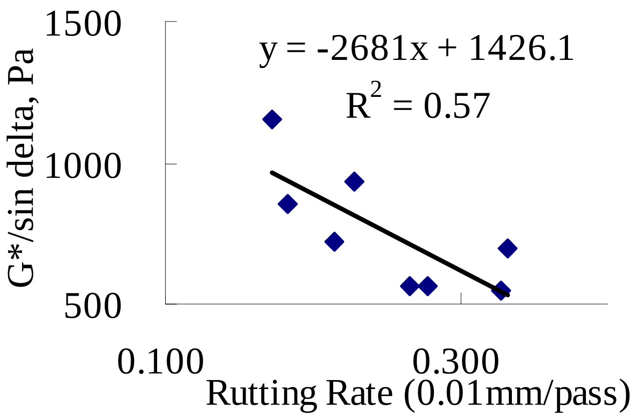

Fig. 2 Rutting rate and $G^*/\sin\delta$ at 0.6 rad/s from wheel tracking test

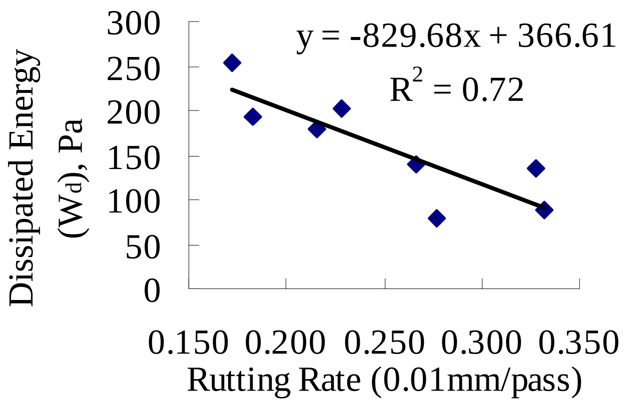

Fig. 3 Rutting rate and dissipated energy at 0.6 rad/s from wheel tracking test

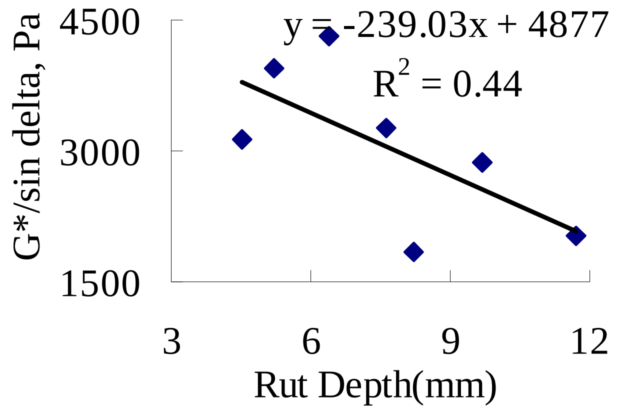

Fig. 4 In situ rutting depth and $G^*/\sin\delta$ at 10 rad/s

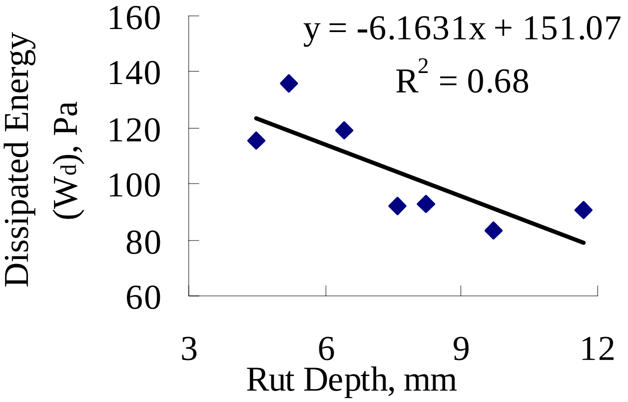

Fig. 5 In situ rutting depth and dissipated energy at 5 rad/s

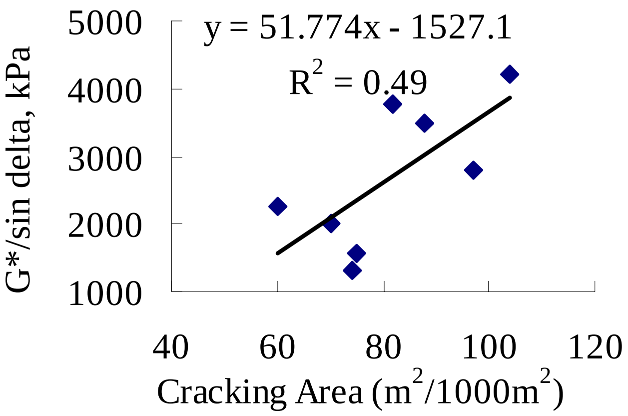

Fig. 6 In situ rutting fatigue cracking and $G^*/\sin\delta$ at 10 rad/s

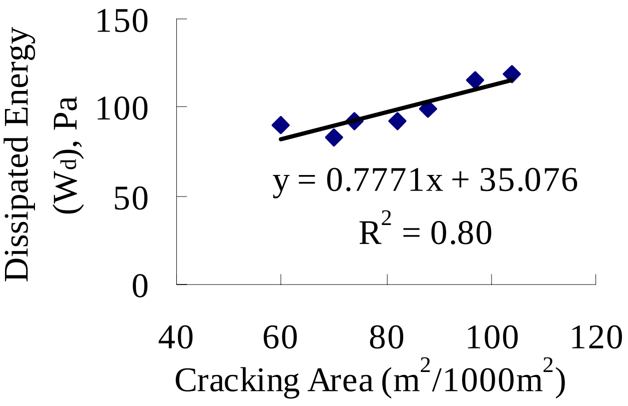

Fig. 7 In situ fatigue cracking and dissipated energy at 5 rad/s

controlled mode and 10 rad/s frequency may not be representative for in situ rutting. It is important to determine the viscous behavior of asphalt binders under the same traffic speed to predict permanent deformation in pavements. Average speed of traffic was conservatively set at 40 km/h in the primary highway because of traffic congestion. The corresponding frequency for traffic speed 40 km/h was set at 5 rad/s.

This study proposed using the dissipated energy, W_d, as a measure for the ability of the binder to resist permanent deformation. The dissipated energy with a corresponding frequency 5 rad/s tested for asphalt was then plotted against the rut depth, as shown in Fig. 5. The R^2 value of 0.68 implied that W_d can reasonably well predict in situ rutting. Significant improvements in the rut resistance were observed when W_d decreased. Therefore, a minimum value of W_d needs to be set for evaluating pavement rutting. This approach combined the linear viscoelastic properties of the binder in one parameter via the dissipated energy. It also accounts for the rate dependence of the material functions because W_d is time dependent, as shown in Eq 5.

5.4 In Situ Fatigue Cracking Evaluation

Use of the parameter $G^* \cdot \sin\delta$ to control fatigue cracking was based in part on the controlled-strain fatigue tests as results of SHRP research efforts (Ref 7, 8). This study indicated in Fig. 6 that fatigue cracking observed from in situ pavement is a poorly correlated function of the asphalt parameter $G^* \cdot \sin\delta$ with an R^2 value equal to 0.49. Other researchers also showed that the parameter $G^*/\sin\delta$ cannot predict pavement cracking accurately (Ref 4, 9).

As shown in Fig. 7, the dissipated energy, W_d, seemed to be a good indicator for predicting fatigue cracking with a R^2 of 0.80. Research reports by other investigators showed that dissipated energy is related to the fatigue response of asphalt-aggregate mixes (Ref 5, 9). It appeared logical to use the W_d of the binder as the parameter to control pavement cracking because fatigue life is attributable to cracking in the binder phase of the mix. Assuming a strain-controlled mode as suggested in the SHRP cannot control fatigue response in a pavement structure. Dissipated energy is independent of the mode of testing, stress-controlled versus strain-controlled, thus, W_d is more representative. This study thus proposed that a maximum value of W_d should be set for evaluating pavement cracking.

6. Conclusions and Recommendations

Examination of asphalt data from the dynamic shear rheometer suggested that another PG 67-xx grade be introduced in SHRP binder specification for the different climatic conditions. The wide temperature gap existing in the SHRP specification can be improved by inserting middle grades in between. The parameters defined in the SHRP binder specification were shown to be inadequate to predict pavement rutting and fatigue cracking. To investigate the effects of linear viscoelastic properties of asphalt binders on pavement performance, samples needed to be tested at the same temperature and frequency experienced on pavements. Based on the theoretical derivation, the dissipated energy, W_d, was proposed as a fundamental indicator for pavement distresses in Taiwan. Both the wheel tracking test and the in situ pavement study clearly demonstrated that the W_d of the binder had a pronounced influence on the ability of the mixture to resist permanent deformation and cracking. This study has shown that there existed a reasonably good relationship between W_d and pavement rutting and cracking. Other contributing factors, including mix characteristic and pavement structure, need to be further investigated to determine the effects of asphalt mixtures on pavement performance. Attention should also be given to applying the results to other mixtures that are different from this study.

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