Laboratory Investigation of Hot Mix Asphalt Behaviour for Mechanistic-Empirical Pavement Design in Tropical Countries

T. Chompoorat[1] and S. Likitlersuang[2]
[1]*Department of Civil Engineering, School of Engineering, University of Phayao, Phayao, Thailand*
[2]*Department of Civil Engineering, Faculty of Engineering, Chulalongkorn University, Bangkok, Thailand*
[1, 2]*E-mail*: cthanakorn@gmail.com and fceslk@eng.chula.ac.th

ABSTRACT: To enhance the quality and efficiency of road design and construction it is necessary to elevate the comprehensive behaviour of the materials employed. Southeast Asian including Thailand, located in the tropical zone usually experiences an extreme weather condition of hot temperature and high humidity. This severe environmental condition is believed that it could reduce the performance and accelerate the aging process of the pavement. The material widely used in the construction of pavement in the Southeast Asian countries and perhaps all over the word is an asphaltic concrete mixed from the AC 60/70 asphalt binder and aggregates. Referring to the theory of mechanic it can be concluded that the asphaltic concrete is a thermo-visco-elastic material which its principle behaviours are caused by strain rate and temperature. To understand the mechanical behaviour of asphaltic concrete, this research aims to integrate the mechanical properties of asphaltic concrete, including stress-strain-strength characteristic, permanent deformation, and moisture damage. The experiments were run through an unconfined compression, an indirect tensile, a resilient modulus, and a dynamic creep tests under various temperatures and different strain rates. The results from this study lead to a high level of understanding in mechanical behaviours and failure mechanism influenced by strain rate and temperature as well as moisture. Moreover, the outcome of this research can be used to define the mechanistic-empirical equations for damage prediction. This research could hopefully enhance the development of pavement design in Thailand based on mechanistic design concept.

Keywords: Pavements and roads, Laboratory Tests, Hot Mix Asphalt, Strength, Creep, Moisture damage

1. INTRODUCTION

Good quality infrastructure is a key component of sustainable development and it reflects the quality of livelihood, growth and ability to trade in the global economy of its community, region, or country. One of significant infrastructures which is essential for the smooth running of many key economic sectors in the world development, including agriculture, industry and tourism is a road. A good quality road can improve the delivery of and access to vital social services, such as health and education. The crucial component of surface transportation infrastructure covering road, runway, parking lot and driveway are the pavements which are an engineered structure. By widely categorising pavements, there are two main types which are asphalt (flexible) pavement and concrete (rigid) pavement. Most of the roads around the world are asphalt pavements.

When disclosing the different layers inside asphalt pavements from the bottom up to top, it is made up of subgrade, subbase, base and asphalt surface. There are also certain pavements consisting of asphalt surface layers on top of concrete layers. The resistance to vehicle loading without excessive deformation is the most important performance sought out from the pavement. The layered structure of the pavement exists to ensure that the load is widespread below the tire such that the resultant stress at every layer does not get high enough to produce the damage. The flexible pavement is approached to scatter the load so that the stress happened in subgrade soil layer is low enough and does not lead to any major deformation. Other than that, the soil is necessary to enhance by using ground improvement when its stiff level is not enough to sustain the stress. The stress on subgrade soil can also be decreased when the asphalt surface is improved.

Refer to the World Bank (2008) data, 82% of the roads in Thailand are the asphalt pavement. Two major government organisations taking care of the pavement construction, maintenance and rehabilitation are the Department of Highways (DOH) and the Department of Rural Roads (DRR), in which the DOH is responsible for the 62,596 km road (DOH, 2013) and the DRR takes care another 41,276 km (DRR, 2014). Most of the pavement designs in Thailand are in accordance with the standard of Department of Highways, Thailand. A hot mix asphalt (HMA) is essentially paved to be the surface layer of pavement (or asphaltic concrete). The HMA is generally prepared from the combination of hot aggregates and asphalt binder and in Thailand limestone and AC 60/70 asphalt are typically utilised. The mixture needs to be designed properly to give it adequate stiffness, strength, and durability. A worldwide accepted method for the HMA mixture design is referred to the Marshall method. Due to the nature of the asphalt layers function that it is directly handled by the environment, the environmental factors must be concerned. To consider the environmental factors influencing the asphaltic concrete behaviour, three significant factors, which are strain rate, temperature, and moisture condition, are usually referred in the research. Especially in tropical area, the extreme weather condition of hot temperature and high humidity is considered that it could reduce the performance of asphalt pavement. The magnitude of strain rate affected from weight of vehicle and vehicle speed also lead to the deterioration of asphaltic concrete. However, when referring to the Marshall method, we learnt that it is partially accessed to these two factors. For the purpose of delineating the mechanical characteristics of the asphaltic concrete, the mechanistic-empirical design approach has been recently developed. The stress-strain and strength characteristics of the asphaltic concrete studied under this approach must use a technique of laboratory testing.

Focusing on this research, it aims to intensively enlighten on the results of laboratory testing and also the analyses of the asphaltic concrete. The HMA specimens of this study were composed of AC 60/70 asphalt binder and limestone aggregate and according to the Thailand's Department of Highways the mixtures were designed by employing Marshall method. In order to control the uniformity of the sample, a Superpave gyratory compactor (SGC) was however used for compacting the HMA specimens. A 14-kN dynamic universal testing machine (UTM) was also depended on for the unconfined compression (UC), the indirect tensile (IDT), the resilient modulus (Mr), and the dynamic creep tests. To simulate a severe environmental condition occurred in pavement surface a temperature of 55^{o}C and a wet condition was applied. By studying the experimental outcomes the mechanistic-empirical relationship can be explained and utilised to predict the stress-strain-strength behaviour of the HMA.

2. MATERIALS AND METHODS

The asphaltic concrete is formed of three various constituents which are of asphalt binder, aggregate, and air voids. In other word, it is a heterogeneous mixture and its properties can consequently be determined by the element properties and the component spatial arrangement. The mechanical properties of constituents as well as the method of mixing are exhibited in this part of the study.

2.1 Asphalt Binder and Aggregate

In Thailand a penetration-graded asphalt binder of AC 60/70 is utilised. The AASHTO and ASTM standards are applied to the test of penetration values, density, and viscosity of the AC 60/70 as summarised in Table 1. It is noted that the penetration value is measured at 25^{o}C according to AASHTO T49 and ASTM D5.

The temperature of measuring the asphalt stiffness in penetration testing is therefore an average temperature of the pavement. However, the fact is that the working temperature of asphalt binder such as 150^{o}C for mixing and 135^{o}C for paving, is obviously over than 100^{o}C. In order to determine the viscosity of asphalt at high temperature, the viscosity test from a rotational viscometer known as a Brookfield test was performed according to the standard of ASTM D4402. The penetration test appears to measure the stiffness of asphalt in solid state, but the viscosity test actually measures the shearing resistance of the asphalt in liquid state.

Table 1. Viscous behaviour of AC 60/70

Asphalt binder	Specific gravity at 25^{o}C	Penetration at 25^{o}C	Brookfield test (Pa.s)	
			135^{o}C	150^{o}C
AC 60/70	1.024	69	0.328	0.170

For aggregate, limestone was employed in this study as it is fundamentally used as a construction material in Thailand. A 12.5 mm nominal maximum aggregate size was selected in this study. A particle size distribution, density, and Los Angeles (LA) abrasion value of aggregate were tested and summarised as shown in Table 2. A particle size distribution curve can be also plotted as shown in Figure 1.

Table 2. Properties of aggregate

Aggregate type	Limestone
Specific gravity, G_s	2.62
Particle size	
d_{10} (mm)	0.18
d_{30} (mm)	0.90
d_{50} (mm)	3.00
d_{60} (mm)	4.75
LA abrasion value (%)	22.88

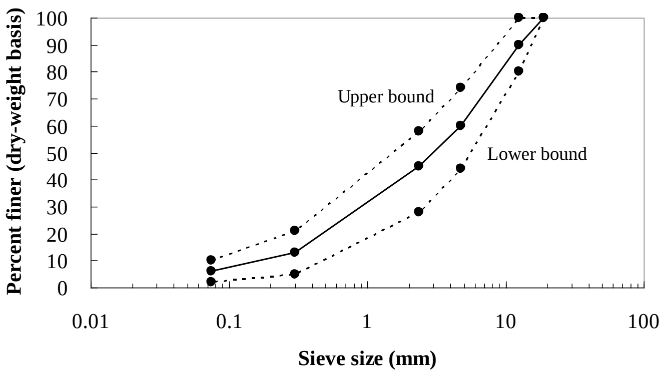


Figure 1. Gradation of aggregate

2.2 Marshall Mix Design

Since the 1940s the U.S. Army Corps of Engineers has improved the Marshall method and it has been extensively used for the pavement design. In Thailand, quality control test during the construction is also counted on the Marshall method and it is moreover indicated as the Thailand's standard (DH-T 604, 1974) by the Department of Highways. Following are the main steps of the Marshall method used in this study.

After the selection of aggregates and the asphalt binder, the hot aggregates shall be compounded with the asphalt binder at mixing temperature of 150^{o}C. A Marshall (handheld) compactor with an appropriate number of blows per side (considering the traffic level) is used at the compaction temperature of 135^{o}C for the asphaltic concrete specimen preparation. In each job mix the voids in the total mix (AV) and the bulk specific gravity of compacted asphalt mix (G_{MB}) were determined. The Marshall stability and flow test were also completed and the Marshall mix design method is plotted as displayed in Figure 2.

2.3 Sample Preparation

In accordance with Thailand's Department of Highways standards, the asphalt mixes were prepared from the AC 60/70 and a 12.5-mm nominal maximum aggregate size of limestone. All mixes were depended on the optimum asphalt content of 5%. For compaction of 150-mm diameter HMA specimens, a SGC was used in order that the uniformity of the samples can be controlled. A stress of 600 kPa is applied to the loading ram and a tilt of 1.25^{o} at 30 gyrations per minute is applied in the rotation of the base. Concerning the heavy traffic level, the compaction number was set to meet the final density of the specimens at the range of 2.40 – 2.45 g/cm^{3} and the air void content of approximately 4%.

The specimens were then cored to arrange their diameters from 150 mm to 100 mm by employing a wet-type coring bit. The treatment of the model after shaping in 100 mm diameter was completed using a diamond saw blade to cut the top and bottom ends of the specimens. The preferred heights of the specimens from above cutting for the UC test and the dynamic creep test are approximately 150 mm and for the IDT test and the Mr test is approximately 65 mm. The specimens of 100 mm diameter and 150 mm length were applied for the simple performance test under NCHRP 465 (2002) while the 100 mm diameter and 65 mm thick samples were prepared according to the ASTM D6931 and ASTM D4867. The test system of the static and dynamic universal testing machine (UTM) was utilised for running all tests. Throughout aforementioned stages, all sample preparation and tests were completed at the Department of Civil Engineering's Geotechnical Laboratory of Chulalongkorn University.

2.4 Testing Programme

The UC test (or uniaxial compression test), the IDT test (or Brazilian test), the Mr test, and the dynamic creep test (or permanent deformation test) are covered in the testing programme of this research. A controlled temperature chamber is used to manage the temperatures desired for the testing samples which are respectively at 10^{o}C, 25^{o}C, 40^{o}C, and 55^{o}C as well as the hot temperature discovered on the pavement surface (Pearratda, 1989 and Jakpet *et al.*, 2011). For the magnitude of strain rate, not many literatures reported the nominal strain rates which measured or estimated from laboratory and numerical analysis results. Data from the Danish Road Testing Machine (Krarup, 1994) report a strain rate of 0.0125 s^{-1} measured at the bottom of the asphaltic concrete layer in the longitudinal direction while a wide base tire travelling about 20 km/hr. Strain rates on this order of magnitude were supported by results from simulated pavement strain response studies using numerical analysis results (Gillespie et al., 1993 and Zafir et al., 1994). Nominal strain rates from these studies were estimated around 0.0165 to 0.0066 s^{-1} at the bottom of the asphaltic concrete layer in the longitudinal direction of travel for pavements having thickness on the order of 150 mm to 200 mm. In this study, therefore the selected strain rates were assigned according to aforementioned literature and the recommended test rate from ASTM. Table 3 shows the strain rates targeted of the constant strain rate testing.

As for the study of moisture sensitivity, the wet specimens were prepared following ASTM D4867. The samples were handled under vacuum saturation as far as a saturation level was entered to 50% - 80%. The specimens were afterward soaked into the water at 55^{o} for 24 hours prior to the test started. The detail of testing condition can be concluded in Table 3.

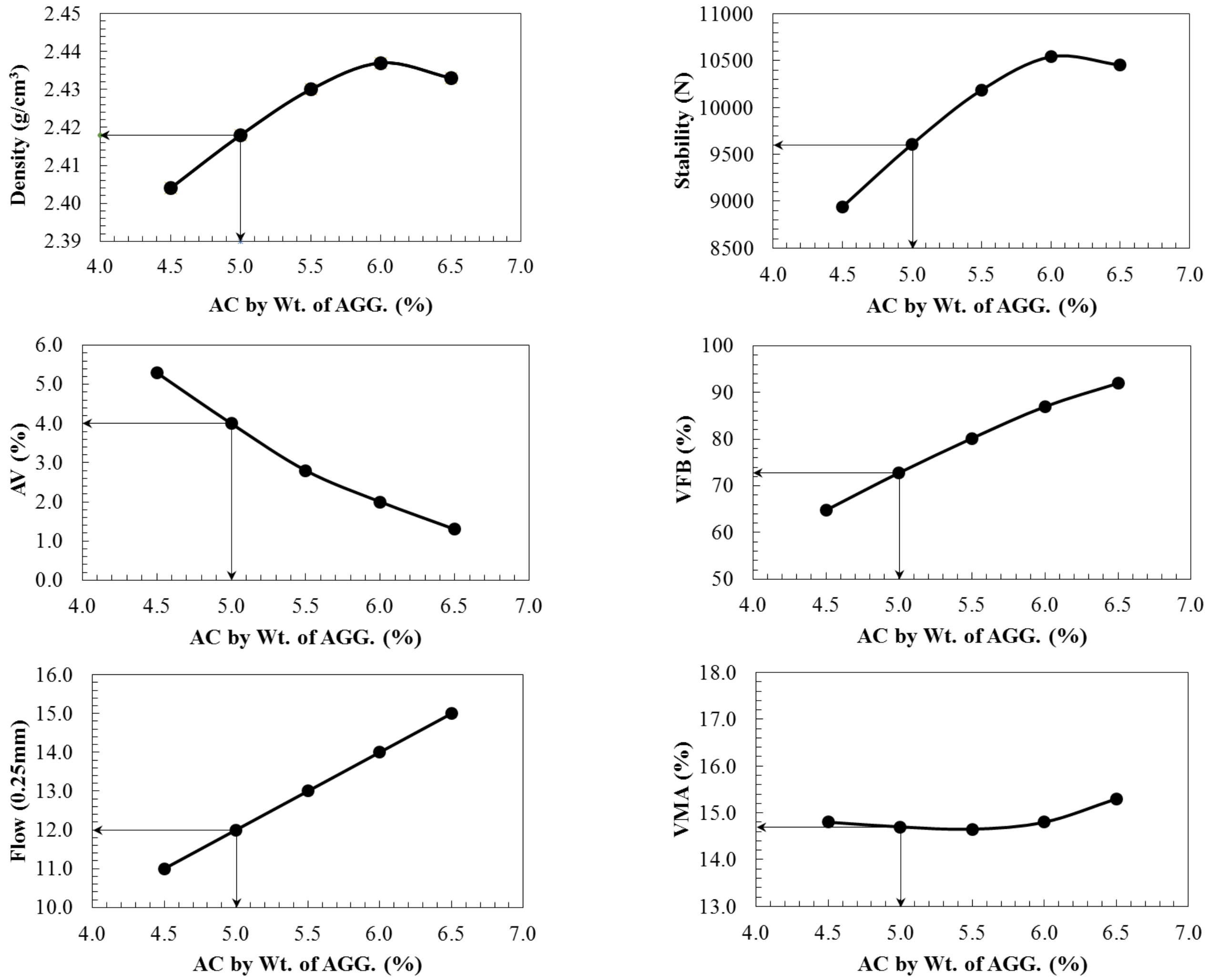


Figure 2. Required plots in the Marshall Mix Design Method

Table 3. Summary of experimental programme

Test	Temperature (^{o}C)	Loading	Condition
Unconfined compression test	10, 25, 40, 55	Monotonic load: rate of loading = 0.0006 s^{-1}, 0.0017 s^{-1}, 0.0056 s^{-1}, 0.0167 s^{-1}	Dry
Indirect tensile test (ASTM D6931, ASTM D4867)	10, 25, 40, 55	Monotonic load: rate of loading = 0.0008 s^{-1}, 0.0025 s^{-1}, 0.0083 s^{-1}, 0.0250 s^{-1}	Dry
	55	Monotonic load: rate of loading = 0.0083 s^{-1}	Dry/Wet
Resilient modulus test (ASTM D4123)	10, 25, 40, 55	Repeated load: Havesine load, frequency = 1 Hz, amplitude = 10% of ITS	Dry
Dynamic creep test (NCHRP 465)	10, 25, 40, 55	Repeated load: Havesine load, frequency = 1 Hz, amplitude = 207 kPa	Dry
	55	Repeated load: Havesine load, frequency = 1 Hz, amplitude = 207 kPa	Dry/Wet

3. RESULTS AND DISCUSSION

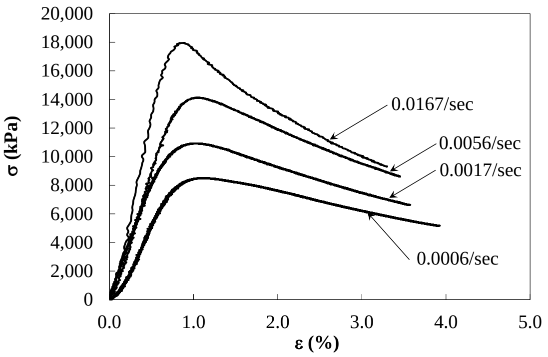
3.1 Unconfined Compression Test

The UTM was applied under monotonic manner to carry out the UC test. Prior to carrying on this test the specimens were held in the controlled chamber for 24 hours. The temperatures of 10^{o}C, 25^{o}C, 40^{o}C, and 55^{o}C were subsequently set in the functioning of the controlled chamber. The vertical displacement of the samples was measured by counting on two linear variable differential transformers (LVDTs). To investigate the effect of the loading rate, the selected strain rates of 0.0006, 0.0017, 0.0056, and 0.0167s^{-1} were used. All tests were performed well to strain levels beyond the peak of the stress-strain, typically to final strain magnitudes which approached 5% as much as it is possible. The summary of the UC test outcomes is shown in Table 4.

Figure 3 displays all relations between stress and strain obtaining from the UC tests. The obvious relation shows that the more strain rate increases, the higher peak strength will be. Conversely, the more temperature raises, the lower asphaltic concrete strength will be.

Table 4. Summarise of stress-strain-strength values from the UC tests

Temp (^{o}C)	$\dot{\varepsilon}$ (sec^{-1})	ε_{peak} (%)	σ_{max} (kPa)	E^{50} (MPa)
10	0.0006	1.23	8600	1040
	0.0017	0.98	11040	1690
	0.0056	1.00	14200	1800
	0.0167	0.88	18080	2460
25	0.0006	1.69	3020	350
	0.0017	1.72	4160	470
	0.0056	1.46	5760	950
	0.0167	1.25	7110	1170
40	0.0006	1.92	1370	180
	0.0017	2.45	1720	190
	0.0056	1.89	1990	210
	0.0167	1.64	2980	370
55	0.0006	2.40	880	74
	0.0017	2.10	980	80
	0.0056	1.54	1000	220
	0.0167	2.11	1360	180



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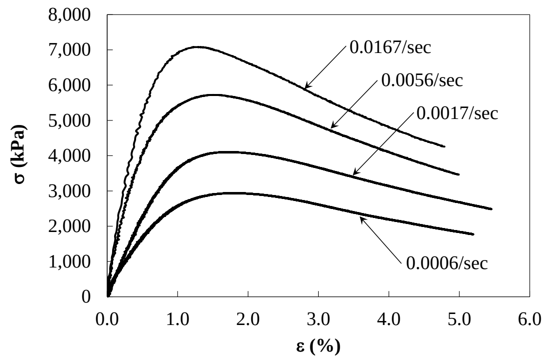

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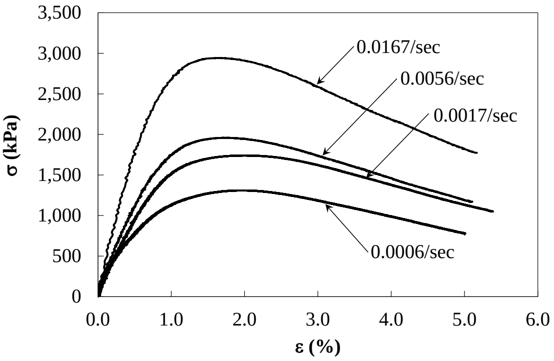

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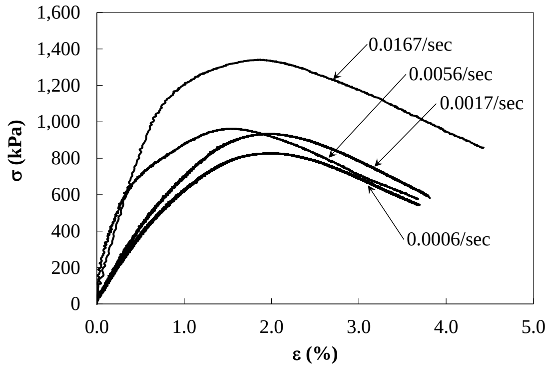

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Figure 3. Stress-strain relationships of HMA specimens with different strain rates under UC test (a) 10^{o}C (b) 25^{o}C (c) 40^{o}C (d) 55^{o}C

3.2 Indirect Tensile Test

The IDT tests were also performed using the UTM. It is the fact that the stress distribution in the IDT test is approximately uniform at the centre of the specimen, where the crack initiates. Two local strain gauges were attached at the centre on both sides of the specimen. A 50 mm/min loading rate which is comparable to 0.0083 s^{-1} strain rate is selected according to the recommendation from ASTM D6931. In this study the strain rates, which are 0.0008, 0.0025, 0.0083 and 0.0250s^{-1} were operated under control in order to observe the influence from strain rate. Prior to running the test, the specimens were managed for 24 hours in the controlled chamber which is performed under the temperature of 10^{o}C, 25^{o}C, 40^{o}C and 55^{o}C subsequently. All tests were executed well to strain levels, which were above the peak of stress-strain, as far as possible to final strain magnitude approaching 1.5%. In fact, only the quick test can be performed at 55^{o}C. It is because at high temperature the specimens behave as soft materials and they could not show the tensile failure mode as shown in Figure 4. Table 5 summarises the results acquiring from the IDT Test.

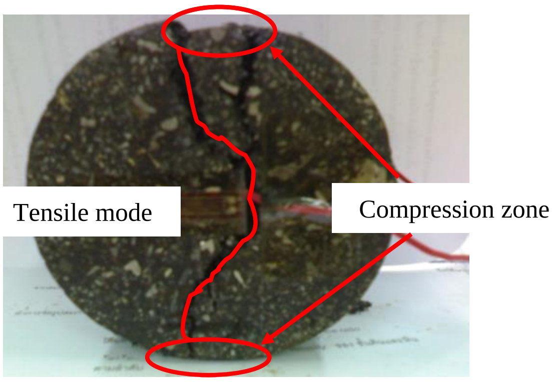


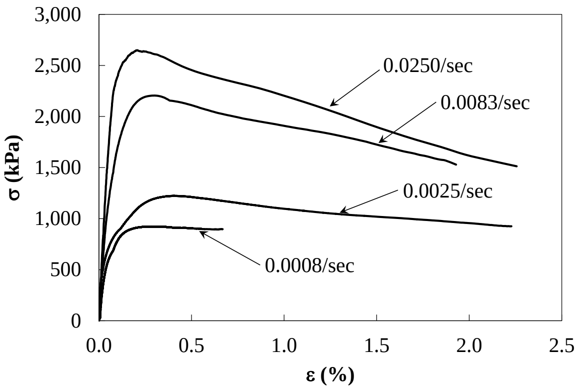
Figure 4. Failure mechanism of HMA specimen IDT test at 55°C (Chompoorat and Likitlersuang, 2009)

The stress-strain relations of the IDT tests are illustrated in Figure 5. The IDT test shows the similar outcome to the UC test that the low indirect tensile strength (ITS) values are impacted by the high strain rate and the low temperature (Chompoorat and Likitlersuang, 2009). The range of strain at peak is about 0.15 – 0.85% (approximately 0.5%) from all tests. It is noticed that the clear tensile failure only appeared on the samples of 10 °C and 25°C.

Table 5. Summarise of stress-strain-strength values from the IDT tests

Temp (°C)	$\dot{\varepsilon}$ **(sec^{-1})**	ε_{peak} **(%)**	σ_{max} **(kPa)**	E^{50} **(MPa)**
10	0.0008	0.26	1020	1460
	0.0025	0.31	1370	1800
	0.0083	0.32	2300	2470
	0.0250	0.18	2680	3500
25	0.0008	0.59	370	180
	0.0025	0.67	580	450
	0.0083	0.37	1010	1100
	0.0250	0.28	1080	1140
40	0.0008	N/A	N/A	N/A
	0.0025	N/A	N/A	N/A
	0.0083	0.11	340	290
	0.0250	0.39	400	310
55	0.0008	N/A	N/A	N/A
	0.0025	N/A	N/A	N/A
	0.0083	N/A	169	N/A
	0.0250	0.31	180	36

N/A is not available



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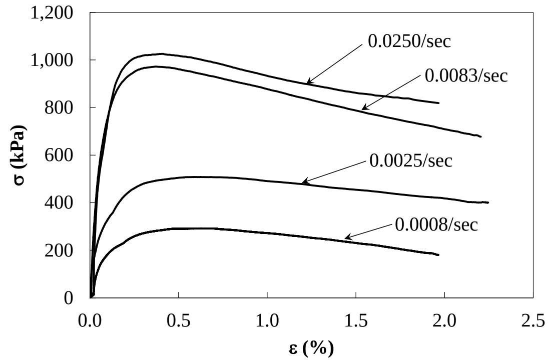

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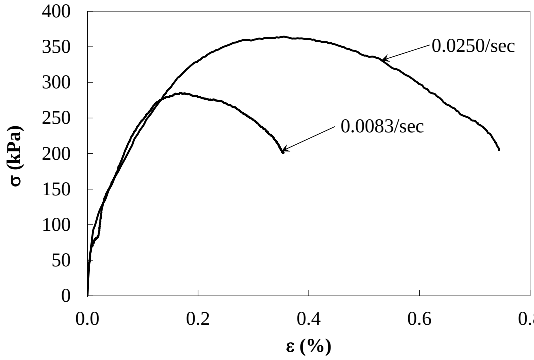

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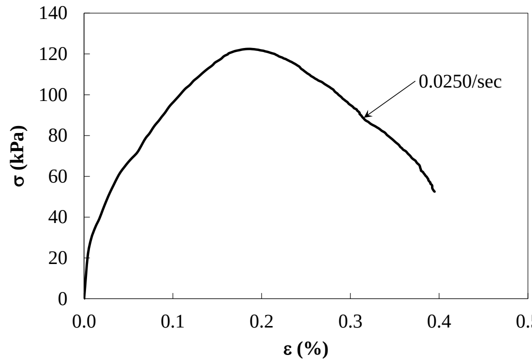

(d)

Figure 5. Stress-strain relationships of HMA specimens with different strain rates under IDT test (a) 10°C (b) 25°C (c) 40°C (d) 55°C

3.3 Resilient Modulus Test

To identify the occurring deformation and to also determine the resilient modulus defined as the ratio of stress and resilient strain ($M_r = \sigma/\varepsilon_r$), the specimens were loaded depending on a haversine loading (Seed *et al.*, 1955). The ASTM D4123 developed the simple equation which used to calculate the M_r as expressed below.

$$M_r = \frac{P(\upsilon + 0.2734)}{\delta t} \tag{1}$$

where P is dynamic load of IDT test, υ is Poisson's ratio which is set to 0.35 (ASTM D4123), δ is total recoverable deformation, and t is specimen thickness.

In general, the Mr test is operated with the indirect tensile mode and this study was also carried out the Mr test in this mode. A period of 0.1 s and a following of rest period 0.9 s were applied as the havesine load employed in the testing protocol. The magnitude of the repeated load is 10% of each ITS of each temperature. The samples were loaded uninterruptedly under the frequency of 1 Hz for 155 cycles. The first 150 cycles are preloading and final 5 cycles are for the Mr determination. The seating stress or approximately 10% of maximum stress is applied to ensure the reliable contacting between the actuator and the specimen for every loading cycle.

The outcomes obtaining from the Mr test are subsequently 14.83, 6.91, 1.14, and 0.53 GPa at 10°C, 25°C, 40°C, and 55°C, respectively. It is discovered that the Mr exponentially decrease when the temperature increase as depicted by a semi-log plot in Figure 6. It can be moreover fit well with an exponential function as displayed in Figure 6.

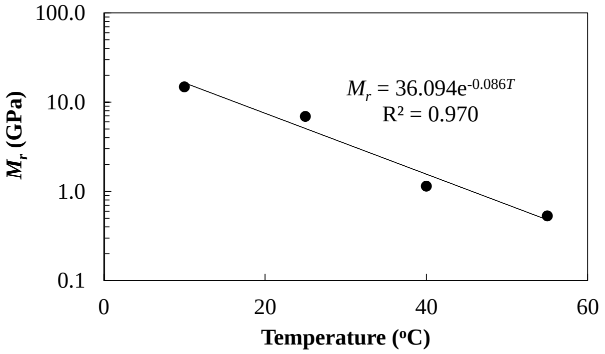

Figure 6. Variation of Mr with temperatures

3.4 Dynamic Creep Test

Permanent deformation (or rutting) results from the accumulation of small amounts of unrecoverable strain as a result of repeated loads applied to the pavement. Permanent deformation in HMA is caused by a deformation flow rather than a volume change (Eisenmann and Hilmer, 1987). The deformation caused by the influence of temperature and also the rate of loading-dependent visco-elastoplastic behaviour lead to rutting, and these two factors can be simulated through laboratory by utilising either static or repeated load creep test. A Simple Performance Test for Superpave Mix Design which is described in NCHRP 465 (2002) consisting of test methods for repeated load testing of asphalt concrete mixture in uniaxial compression is applied in this study for operating the dynamic creep test. This test is also performed by employing the dynamic UTM. Each specimen is loaded at 207 kPa in accordance with the stress level of passenger car. Figure 7 shows the dynamic creep test setup in dry condition.

Figure 7. Dynamic creep test setup in dry condition

During and after applying a load, the vertical strains which were undertaken repeated load at the temperature of 10°C, 25°C, 40°C, and 55°C were measured. The samples were continuously loaded for 12 hours under a frequency of 1 Hz and 40,000 cycles or continued it until they reached failure. This duration of load is formed in 1 by 9 and during the load the seating stress of 10% of maximum stress is applied. Figure 8 shows the plot between permanent strains and the number of cycles known as creep curves.

At 10°C it is manifest that the HMA specimen perfectly exhibits elastic behaviour and permanent strain could not be found along 40,000 loading cycles. At 25°C the sample shows the permanent deformation in the secondary range and do not display the failure. On the contrary, at 40°C and 55°C, when continued loading cycles of the specimens are approximately reached to 9,000 and 5,000 cycles respectively, the specimens display the failure. Under the flow number (FN) of 9,000 and 5,000, in other words, the failure occurred. The secondary creep is able to fit with an exponential equation known as a creep equation. All creep equations for 10°C, 25°C, 40°C, and 55°C are summarised in Table 6.

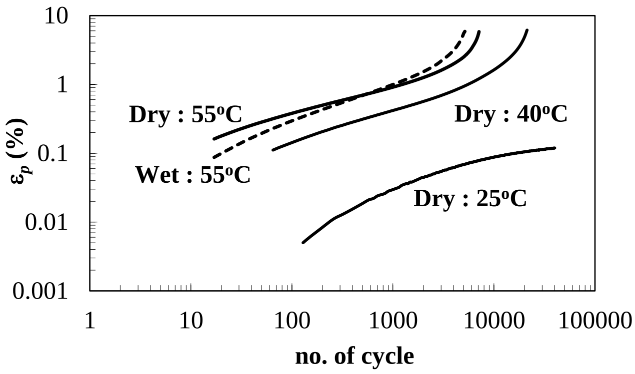

Figure 8. Relation of permanent strain and number of cycles

Table 6. Dynamic creep test results

Temp (°C)	Condition	FN	Creep equation
10	Dry	N/A	N/A
25	Dry	N/A	$\varepsilon_p = 0.0027N^{0.363}$
40	Dry	9,000	$\varepsilon_p = 0.0090N^{0.822}$
55	Dry	5,000	$\varepsilon_p = 0.0650N^{0.381}$
55	Wet	3,000	$\varepsilon_p = 0.0021N^{0.875}$

N/A is not available

3.5 Moisture sensitivity

Moisture damage is one of the important distress mechanisms leading to premature failure of asphalt pavements. Moisture could damage asphalt pavements due to stripping or loss of bond between asphalt and aggregate surface (Terrel and Al-Swailmi, 1994). The pore pressure effect is one of the stripping mechanisms where water may circulate freely through interconnected voids in the mixes (Majidzadeh and Brovold, 1968; Taylor and Khosla, 1983). The resulting loss of stiffness or strength is reversible when water is removed from the mix (Santucci 2002). Due to the complication of the moisture damage phenomenon, there are many researchers trying to study and develop the method of test such as Hicks (1991), Aschenbrener *et al.* (1995), Chen and Huang (2007), and Mehrara and Khodaii (2011).

The resistance of HMA conditioned under moisture change was studied by the application of the moisture sensitivity test. According to the standard of AASHTO T283 and ASTM D4867, it was tested through the indirect tensile test and the dynamic creep test. On setting up the wet condition, the samples were placed in the triaxial cell which was put in the water below the top surface of specimens for 5 cm. In the operating unconfined test a deviator stress of 207 kPa was applied. Since 1 cycle/sec loading frequency is a typical frequency in the experiment for slow moving traffic, it was chosen for this test. Similar to dynamic creep test the maximum load was performed in 0.1 sec, and then removed. In order to ensure a continuous contact between the upper platen and the vertical load, the contact pressure of 10% of maximum load was run for the remaining 0.9 sec. The symmetrical haversine loading on the sample was carried on until reaching 40,000 cycles taking 12 hours or until the sample failed. In this case, the confining pressure did not utilise. Depending on this way, the water was able to freely get in and out the pores of the specimen in the application of every dynamic load.

From the setup of this test we can at the same time evaluate the effect of moisture damage and permanent deformation (Kanitpong *et al.*, 2012, Chompoorat *et al.*, 2011 and 2012). The test setting up of dynamic creep test assigned in water-exposed condition is illustrated in Figure 9.

The wet conditioned sample was tested at 55^{o}C with strain rate of 0.0083 s^{-1} to inspect the moisture sensitivity using the IDT test. A tensile strength ratio (TSR) is defined by the ratio of the ITS of the wet sample to the ITS of the dry sample. The TSR value of this study at 55^{o}C is 0.82 (140.0 kPa/169.00 kPa). The asphaltic concrete, which defined as good quality recommended by AASHTO T283, ASTM D4867, and Stuart (1986) that the TSR should be more than 0.7 – 0.8.

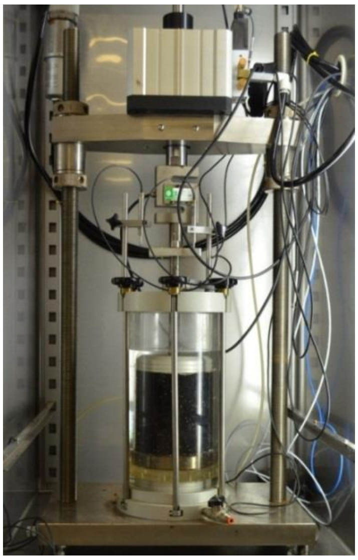

Figure 9. Dynamic creep test setup in wet condition

The HMA specimens operated in dynamic creep test was also tested in soak condition under 55^{o}C as displayed in Figure 9. Throughout this operation, the relation of permanent strain and the loading cycle number of wet conditioned HMA sample can be discovered. Table 6 shows the investigation of the FN and the creep equation. The ratio of flow number (FRN) acquired from the ratio of FN of the damp specimen to the dry specimen was determined to study the moisture susceptibility of the HMA sample. The FRN at the temperature of 55^{o}C in this research is 0.6 (3,000/5,000). It is discovered from the result that the HMA under hot and damped condition has approximately 60% less rutting resistance

4. CONCLUSIONS

The purpose of this research is to study the mechanical behaviours and damage mechanism of the HMA employed in Thailand. According to this study the concluding remarks can be summarised as follows.

1) The UC and IDT tests under the temperature change and rate of loading confirm the relation between strength and temperature that the strength will drop when the temperature is getting higher. On the other hand, the strength will increase when the rate of loading increases.

2) The results of the resilient modulus test provide the Mr values at four different temperatures. The relationship between the Mr values and temperatures can be used to build an empirical equation. This could support the development of pavement design based on mechanistic design approach.

3) Beside the Marshall test method, the dynamic creep test could provide the durability of HMA specimens under a variety of temperatures. This is because the dynamic creep test depends on repeated load which is simulated from the actual loading happened from the traffic load. By this test, it allows us to understand the permanent deformation at different temperatures in dry condition. Additionally, the creep equation obtained from this test can be employed for predicting permanent strain values of HMA specimens and can also be used to anticipate the permanent deformation in HMA under vehicle loading.

4) The moisture sensitivity test which is the imitation of road under a large amount of rainfall or even flooding, allows us to know when the permanent deformation would be happened beneath wet condition. The creep equation from moisture sensitivity condition can be utilised to forecast permanent deformation resulting from moisture condition. It is furthermore found from the comparison between this wet condition result and dry condition result above that resistance of permanent deformation of the HMA specimen at the same temperature will be reduced 40% when getting the moisture in.

5) Throughout a variety of factors causing the deformation of the pavement, by depending on the Marshall test method we able to conceive only the values of stability and flow under static condition while all above conclusions bring us closer to the real conditions affecting the pavement. From studying all previous tests we can better predict the mechanical behaviour as well as the durability of HMA material under various temperatures in different climates, diverse loading types of vehicles, and moisture condition.

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