

Investigating Properties of Pavement Materials Utilizing Loaded Wheel
Tester (LWT)

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ABSTRACT

Loaded wheel tester (LWT) is a common testing equipment usually used to test the permanent deformation and moisture susceptibility of asphalt mixtures by applying moving wheel loads on asphalt mixture specimens. It has been widely used in the United States since 1980s and practically each Department of Transportation or highway agency owns one or more LWT(s). Compared to other testing methods for pavement materials, LWT features movable wheel loads that allow more realistic situations existing on the actual pavement to be simulated in the laboratory. Due to its potential of creating a condition of repetitive loading, the concept of using LWT for characterizing the properties of pavement materials were promoted through four innovative or modified tests in this study.

- (1) The first test focuses on evaluating the effect of geogrids in reinforcing pavement base courses. In this test, a base course specimen compacted in a testing box with or without geogrids reinforced was tested under cyclic loading provided by LWT. The results showed that LWT test was able to characterize the improvement of the pavement base courses with geogrids reinforcement. In addition, the results from this study were repeatable and generally in agreement with the results from another independent study conducted by the University of Kansas with similar testing method and base materials.
- (2) A simple and efficient abrasion test was developed for characterizing the abrasion resistance of pervious concrete utilizing LWT. According to the abrading mechanisms for pervious concrete, some modifications were made to

the loading system of LWT to achieve better simulations of the spalling/raveling actions on pervious concrete pavements. By comparing the results from LWT abrasion tests to Cantabro abrasion tests, LWT abrasion test was proved effective to differentiate the abrasion resistances for various pervious concretes.

- (3) Two innovative LWT tests were developed for characterizing the viscoelastic and fatigue properties of asphalt mixtures in this study. In the test, asphalt beam specimens are subjected to the cyclic loads supplied by the moving wheels of LWT, and the tensile deformations of the beam specimens are measured by the LVDTs mounted on the bottom. According to the stress and strain, the parameters associated to the viscoelastic and fatigue properties of the asphalt mixture can be obtained through theoretical analyses.

In order to validate the concepts associated with the above mentioned tests, corresponding conventional tests have also been conducted to the same materials in the study. According to the comparisons between the conventional and the LWT tests, the LWT tests proposed in this study provided satisfactory repeatability and efficiency.

TABLE OF CONTENTS

CHAPTER 1 RESEARCH BACKGROUND	1
1.1 Introduction.....	1
1.2 Geogrids Reinforced Base Course	1
1.3 Durability of Pervious Concrete	3
1.4 Stiffness and Fatigue Strength of Asphalt Mixtures	5
1.5 Testing Methodology	6
1.6 Objectives and Scope.....	9
1.7 Arrangement of the Dissertation.....	11
CHAPTER 2 EVALUATING REINFORCEMENT EFFECT OF GEOGRIDS IN PAVEMENT BASE COURSES.....	13
2.1 Introduction.....	13
2.2 Objectives and Scope.....	15
2.3 Laboratory Experiment	15
2.4 Specimen Preparation	16
2.5 Testing Protocol.....	18
2.6 Evaluation Approach	20
2.6.1 <i>Traffic Benefit Ratio (TBR)</i>	21
2.6.2 <i>Rut Reduction Ratio (RRR)</i>	21
2.6.3 <i>Rate of Deflection (ROD)</i>	22
2.7 Results and Discussions.....	22
2.8 Summary and Conclusions	29

CHAPTER 3 INNOVATIVE TEST FOR EVALUATING ABRASION RESISTANCE OF PERVIOUS CONCRETE	30
3.1 Introduction.....	30
3.2 Objectives and Scope.....	33
3.3 Materials and Specimen Preparation	33
3.4 Laboratory Experiments	36
3.4.1 <i>Physical and Mechanical Performance Tests</i>	37
3.4.2 <i>Cantabro Abrasion Test</i>	38
3.4.3 <i>LWT Abrasion test (Loaded Wheel Abrasion Test)</i>	40
3.4.4 <i>Loaded Sweep Abrasion Test</i>	43
3.5 Abrasion Evaluation Parameters.....	45
3.6 Results and Discussions.....	46
3.6.1 <i>Physical and Mechanical Performance Tests</i>	46
3.6.2 <i>Cantabro Abrasion Test</i>	49
3.6.3 <i>LWT Abrasion Test (Loaded Wheel Abrasion Test)</i>	52
3.6.4 <i>Loaded Sweep Abrasion Test</i>	55
3.7 Sensitivity and Repeatability	55
3.8 Summary and Conclusions	57
3.9 Acknowledgement	58
CHAPTER 4 CHARACTERIZING VISCOELASTIC PROPERTIES OF ASPHALT MIXTURES UTILIZING LWT	59
4.1 Introduction.....	59
4.1.1 <i>Creep Test</i>	59

4.1.2 <i>Complex Modulus Test</i>	60
4.2 Viscoelastic Experimental methods for Asphalt mixtures.....	62
4.2.1 <i>Direct Tension Test</i>	64
4.2.2 <i>Uniaxial Compressive Test</i>	65
4.2.3 <i>Indirect Tension Test</i>	66
4.2.4 <i>LWT Viscoelastic test (Loaded Wheel Viscoelastic Test)</i>	66
4.3 Objectives and Scope.....	67
4.4 Loaded Wheel Testing System of APA.....	68
4.4.1 <i>Mechanical Analysis</i>	68
4.4.2 <i>Stress Analysis</i>	71
4.4.3 <i>Modeling Viscoelastic Properties</i>	75
4.5 Laboratory Experiments	80
4.5.1 <i>Uniaxial Viscoelastic Test</i>	80
4.5.2 <i>LWT Viscoelastic Test</i>	84
4.5.3 <i>Indirect Tension Creep Test</i>	86
4.6 Data Processing.....	87
4.7 Materials and Specimen Preparation	89
4.8 Results and Discussions.....	90
4.8.1 <i>Creep Compliance</i>	90
4.8.2 <i>Dynamic Modulus</i>	93
4.8.3 <i>Phase Angle</i>	97
4.9 Summary and Conclusions	100

CHAPTER 5 CHARACTERIZING FATIGUE BEHAVIORS OF ASPHALT MIXTURES UTILIZING LWT	102
5.1 Introduction.....	102
5.2 Fatigue Tests for Asphalt Mixtures	102
5.2.1 <i>Flexural Beam Fatigue Test (Four-Point Beam Fatigue Test)</i>	103
5.2.2 <i>Direct Tension Fatigue Test</i>	105
5.2.3 <i>LWT Fatigue Test (Loaded Wheel Fatigue Test)</i>	106
5.3 Interpreting Fatigue Behavior of Asphalt Mixtures.....	109
5.3.1 <i>Stiffness Approach</i>	110
5.3.2 <i>Dissipated Energy Approach</i>	111
5.3.3 <i>Pseudostiffness and Pseudo-Strain Energy Approach</i>	114
5.4 Materials and Specimen Preparation	117
5.5 Results and Discussions.....	120
5.5.1 <i>Reduction of Stiffness</i>	120
5.5.2 <i>Ratio of Dissipated Energy Change (RDEC)</i>	123
5.6 Discussions on LWT Fatigue Test.....	126
5.7 Summary and Conclusions	127
CHAPTER 6 CONCLUSIONS AND RECOMMENDATIONS	129
6.1 Conclusions.....	129
6.2 Recommendations.....	131
REFERENCES	134

LIST OF FIGURES

Figure 1.1 Interlocking actions between geogrids and aggregates (Wrigley, 1989)	2
Figure 1.2 Pervious concrete pavement.....	5
Figure 1.3 APA	7
Figure 1.4 Current APA tests.....	8
Figure 1.5 Data acquisition system and programs	9
Figure 2.1 Grain-size distribution of base course materials	16
Figure 2.2 Geogrids used for testing.....	17
Figure 2.3 Specimen preparation	18
Figure 2.4 Test setup.....	18
Figure 2.5 Schematic diagram of testing	19
Figure 2.6 Base course specimen before and after test.....	20
Figure 2.7 Rut depth vs. load cycles for river sand base	23
Figure 2.8 Rut depth vs. load cycles for original AB-3 gravel base.....	24
Figure 2.9 Rut depth vs. load cycles for adjusted AB-3 gravel base.....	25
Figure 2.10 Rate of deflection results for gravel base course.....	28
Figure 3.1 Gradations for coarse aggregates.....	33
Figure 3.2 Effective air voids test for PCPC.....	37
Figure 3.3 Permeability test for PCPC.....	38
Figure 3.4 Cantabro abrasion test	39
Figure 3.5 LWT abrasion test (loaded wheel abrasion test)	41
Figure 3.6 Studded steel wheels for LWT abrasion test	42

Figure 3.7 Raveling action and LWT simulation (not to scale).....	43
Figure 3.8 Loaded sweep abrasion test	44
Figure 3.9 Specimen compaction for loaded sweep abrasion test	45
Figure 3.10 Strength results of PCPC mixtures	49
Figure 3.11 Test Specimen at different cycles in Cantabro abrasion test	50
Figure 3.12 Typical growth of weight loss in Cantabro abrasion test (#89 mixtures).....	51
Figure 3.13 Weight loss of Cantabro test after 300 revolutions	52
Figure 3.14 Specimens after LWT abrasion test.....	53
Figure 3.15 Results of LWT abrasion test	55
Figure 4.1 Creep test at different temperatures.....	60
Figure 4.2 Typical master curve for dynamic modulus.....	62
Figure 4.3 Loading distribution of the surface layer in pavement system.....	62
Figure 4.4 Viscoelastic tests for asphalt mixtures in this study.....	64
Figure 4.5 Simple performance test (SPT)	65
Figure 4.6 Schematic diagram of APA loading system.....	69
Figure 4.7 Establishment of movement equation	69
Figure 4.8 Movement function of APA Loading System ($T_r = 1$)	70
Figure 4.9 Simplified calculation model for stress analysis	70
Figure 4.10 Finite element model and stress distribution	72
Figure 4.11 ISO-stress lines on cross sections of the specimen (unit: psi).....	73
Figure 4.12 Normal stresses on the bottom and midspan of the specimen (3 cycles)	74
Figure 4.13 Typical sinusoidal stress induced by loading wheel in APA ($T=0.5s$)	79
Figure 4.14 Uniaxial viscoelastic test setup.....	81

Figure 4.15 MTS loading system with environmental chamber	82
Figure 4.16 Typical recording in uniaxial viscoelastic test	83
Figure 4.17 LWT viscoelastic test	84
Figure 4.18 Typical stress and strain in LWT dynamic modulus test	85
Figure 4.19 Typical hysteresis loop in LWT dynamic modulus test	86
Figure 4.20 Indirect tension test (IDT)	87
Figure 4.21 Stress-strain curves before and after FFT smoothing process	88
Figure 4.22 Creep test results from LWT test (10°C)	91
Figure 4.23 Creep compliance results	92
Figure 4.24 Dynamic modulus master curves for different tests	94
Figure 4.25 Dynamic modulus results from different tests (LS-3)	95
Figure 4.26 Dynamic modulus results at 10°C, 2Hz and 40°C, 0.1Hz	97
Figure 4.27 Phase angle results for different tests	99
Figure 4.28 Phase angle results from different tests (LS-2 & LS-3)	100
Figure 5.1 Test setups for the fatigue tests	103
Figure 5.2 Flexural beam fatigue test	104
Figure 5.3 Direct tension fatigue test	106
Figure 5.4 LWT fatigue test	107
Figure 5.5 Typical hysteresis loops with the change of load cycles (LWT fatigue test)	108
Figure 5.6 Typical developing phase of stiffness in fatigue test	110
Figure 5.7 Typical stress-strain hysteresis loop	111
Figure 5.8 Typical RDEC vs. load cycle curve (Carpenter et al. 2003)	114
Figure 5.9 Stress-pseudosstrain hysteresis loops in a fatigue test (Kim 2009)	116

Figure 5.10 Stiffness vs. loading cycle for different tests.....	121
Figure 5.11 N_f results from different tests for various mixtures.....	122
Figure 5.12 Curve fitting process for dissipated energy vs. loading cycle	124
Figure 5.13 RDEC vs. load cycle.....	124
Figure 5.14 PV results from different tests for various mixtures	125
Figure 5.15 Relationship between PV and N_f	126
Figure 5.16 Deformation vs. loading cycle in fatigue test.....	127

LIST OF TABLES

Table 2.1 Rut, TBR and RRR results for various geogrids	27
Table 3.1 Mix proportions for PCPC (unit: kg/m ³)	35
Table 3.2 Laboratory test summary	36
Table 3.3 Results of effective air voids and permeability	47
Table 3.4 Comparison of investigated abrasion tests.....	56
Table 4.1 Asphalt mixtures tested in this study	89
Table 4.2 Specimen designed for the tests.....	90
Table 5.1 Fatigue test summary	109
Table 5.2 Specimens for fatigue tests	117
Table 5.3 Summary of the properties of asphalt mixtures	119

LIST OF ABBREVIATIONS

2-D	2-Dimension
3-D	3-Dimension
AASHTO	American Association of State Highway and Transportation Officials
ACI	American Concrete Institute
AMPT	Asphalt Mixture Performance Test
ASTM	American Society for Testing and Materials
AVC	Asphalt Vibratory Compactor
BFA	Beam Fatigue Apparatus
COV	Coefficient of Variation
DAQ	Data Acquisition System
DCSE _f	Dissipated Creep Strain Energy
DE	Dissipated Energy
DTT	Direct Tension Test
FE	Fracture Energy
FEM	Finite Element Method
FFT	Fast Fourier Transform
FPRT	French Pavement Rut Tester
GD1	Geogrids #1
GD2	Geogrids #2
GD3	Geogrids #3
GD4	Geogrids #4
GDOT	Georgia Department of Transportation
GLWT	Georgia Loaded Wheel Tester
GN	Granite
GN-1	Granite, PG64-22
GPL	Generalized Power Law

GR	Granite
HMA	Hot-Mix Asphalt
HRWR	High range water reducer
HWTD	Hamburg Wheel Tracking Device
IDT	Indirect Tension Test
LS	Limestone
LS-1	Limestone, PG64-22
LS-2	Limestone, PG70-22
LS-3	Limestone, PG76-22
LVDT	Linear Variable Displacement Transducer
LVE	Linear Viscoelastic
MPL	Modified Power Law
Mr	Resilient Modulus
MTS	Material Testing System
NCHRP	National Cooperative Highway Research Program
NI	National Instrument
NRMCA	National Ready Mixed Concrete Association
OGFC	Open-Graded Friction Course
PCA	Portland Cement Association
PCC	Portland Cement Concrete
PCPC	Portland Cement Pervious Concrete
PUR Wheel	Purdue University Laboratory Wheel Tracking Device
PV	Plateau Value
RAP	Reclaimed Asphalt Pavement
RDEC	Ratio of Dissipated Energy Change
ROD	Rate of Deflection
RRR	Rutting Reduction Ratio
SBR	Styrene Butadiene Rubber
SGC	Superpave Gyratory Compactor
SHRP	Strategic Highway Research Program

SPT	Simple Performance Test
SX	Normal stress in X direction(longitudinal direction)
SY	Normal stress in Y direction (vertical direction)
SZ	Normal stress in Z direction (transverse direction)
TBR	Traffic Benefit Ratio
TDOT	Tennessee Department of Transportation

LIST OF SYMBOLS

$J(t)$	Creep compliance
$\varepsilon(t)$	Strain varies with time
σ_0	Constant stress
σ_i	Initial stress
ϕ	Phase angle
σ_{amp}	Amplitude of sinusoidal stress
ε_{amp}	Amplitude of sinusoidal strain
ω	Angular velocity
Δt	Time lag between stress and strain
E_∞	Long time equilibrium modulus
E_m	Regression coefficient
ρ_m	Prony series regression coefficients
$\Delta H(t)$	Horizontal deformation with time change
GL	Gage length of the extensometer
S^R	Pseudo stiffness
γ_m^R	Peak pseudo strain in each physical stress–pseudo strain cycle
τ_m	Physical stress corresponding to peak pseudo strain in each cycle
$N_{reinforced}$	Number of cycles for geogrids reinforced specimen
$N_{non-reinforced}$	Number of cycles for geogrids non-reinforced specimens
t_n	Time period of the nth cycle
t'	Integration variable related to time
$u_{reinforced}$	Rut depth (deflection) at specific cycle for reinforced specimen

$u_{non-reinforced}$	Rut depth (deflection) at specific cycle for non-reinforced specimen
u_n	Deflection of the nth cycle
$ E^* $	Complex modulus
a	1/3 of the beam span
b	Width of the specimen
$E(t)$	Relaxation modulus
$D(t)$	Creep compliance
D_0	Glassy compliance
D_e	Long-time equilibrium or rubbery compliance
D_g	Glassy compliance
D_m	Regression coefficient
$E(t)$	Relaxation modulus related to time
E^*	Dynamic modulus
E^R	Reference modulus
f	Loading frequency
h	Height of the specimen
i	Imaginary component
l	Length of the loading path
L	Beam span
N_f	Failure Life
N_{f50}	Number of cycle of 50% reduction of initial stiffness
N_t	Number of cycle of initial stiffness
P	Wheel load or applied load
S	Stiffness
t	Time of interest
T	Testing period

T_r	Rotation period
w	Beam width
δ	Beam deflection at neutral axis
ε	Strain
ε^R	Pseudo-strain
ε_t	Peak-to-peak tensile strain
σ	Stress
σ_t	Peak-to-peak stress
τ_m	Retardation time
W_1	Initial sample weight
W_2	Final sample weight

CHAPTER 1 RESEARCH BACKGROUND

1.1 Introduction

Pavement materials can be categorized into surface materials and base course materials in the pavement system. Rigid and flexible pavements are the two major types of pavements in the world, in which portland cement concrete and asphalt mixtures are mainly used as surface materials, respectively. Portland cement concrete is generally created by a mixture of portland cement, coarse aggregates (gravel, limestone or granite etc.), fine aggregates (natural sand, manufactured sand etc.), water and additives.

Base Course is the sub layer material placed directly on the top of undisturbed and stabilized subgrades to support the top layers in the pavement. Generally, it is constructed by a specific type of granular aggregate, such as gravel, crushed limestone, sandstone and/or granite, etc. It has the following abilities in a pavement system: (1) control vertical deformation and improve the lateral extrusion resistance of the pavement; (2) distribute loads on the pavement; (3) control of pumping; (4) control of frost action; (5) permeable for the drainage of the pavement; (6) resistance to shrink and swell of the subgrade. An unbound base course could contribute some structural capacity to the pavement system, but due to the low strength its contribution to the capacity of load-carrying is insignificant (Yoder and Witczak 1975).

1.2 Geogrids Reinforced Base Course

Geogrid is a commonly used material for reinforcing pavement base courses so that they may carry more strength in pavement structure. A Geogrid consists of parallel sets

of stiffened tensile ribs with connected apertures. It is usually fabricated by extruding material sheets made of polyethylene or polypropylene. Because of the potential benefits of geogrids on decreasing permanent vertical deformation, increasing tensile strength and durability, and reducing long-term maintenance cost of pavements, it has been widely used as reinforcement in earthwork constructions, such as pavement bases, bridge abutments, and geo-environmental engineering applications.

It is well known that cracking is one of the biggest challenges for pavement systems, which is usually caused by traffic loading, age hardening, and temperature cycling. When an overlay is applied to an existing cracked pavement, geogrids may be used to prevent the spreading of existing cracks to the new asphalt overlay. As the energy from existing cracks moves upward through the cracks, towards the new overlay, the geogrids placed between them have the ability to dissipate the energy by distributing it horizontally.

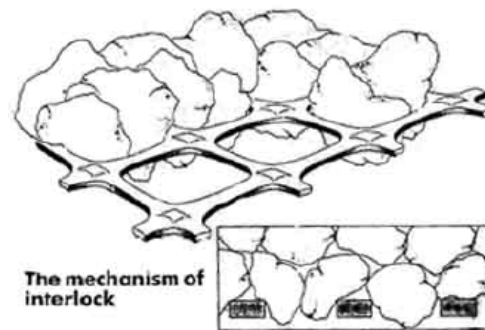


Figure 1.1 Interlocking actions between geogrids and aggregates (Wrigley, 1989)

Regarding to the reinforcement for pavement bases, the effect is mainly achieved through the interlocking actions between geogrids and aggregate particles (Wrigley, 1989). As shown in Figure 1.1, base aggregates are interlocked by geogrids, and the deflection of the pavement could be reduced and desired rut depth (permanent vertical deformation) could be maintained. Consequently, the service life of the whole pavement

system can be guaranteed or increased. Through years of applications and studies, geogrids have been proven to be able to extend the life of pavements by up to 80% (Haas et al. 1988; Chen et al. 2009).

1.3 Durability of Pervious Concrete

Since pervious concrete was first introduced into the United States in the mid of 1970s, pervious concrete has been used in many applications for over 30 years (Malhotra 1976). During the last few years, pervious concrete has attracted more and more attention in concrete industry due to the increased awareness of environmental protection.

Portland cement pervious concrete (PCPC) is an environmentally friendly concrete that features high interconnected air voids within the range from 15% to 25% (ACI 2006). Pervious concrete is a discontinuous mixture basically made of coarse aggregate, no or a small amount of fine aggregates, portland cement, and water. By reducing the fine aggregates and using single-sized coarse aggregates in the mixture, cementitious material in PCPC is reduced significantly, and substantial void content is created. Due to the pore structure with interconnected voids, high permeability can be achieved (commonly ranges from 0.2 to 0.5 cm/s). Thus PCPC has the capability to control storm water runoff, minimize wet weather spray, improve visibility, and avoid glare (Tennis et al. 2004; Collins et al. 2006; Wanielista et al. 2007). In addition, noise caused by traffics on pavements can also be absorbed partially by the porous surface of PCPC to improve the living environment (Boutin et al. 1998; Olek et al. 2003). As the emphasis on environmental protection and building green continues to increase, PCPC has been

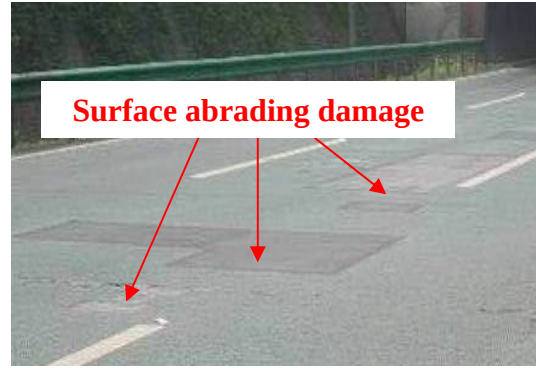
increasingly used in many kinds of constructions, such as low traffic pavements, parking lots, walkways, and recreation fields.

However, to obtain high porosity and water permeability, little or no fine aggregate is added into the mixture, which leads to aggregate particles in PCPC being bonded together through aggregate to aggregate contacts rather than being embedded in cement paste as in ordinary portland cement concrete (PCC). Therefore, PCPC has lower mechanical properties and durability than those of PCC, and it is more vulnerable to spalling and raveling under moving traffic loads. Currently, the application of PCPC is limited to parking lots, sidewalks, pavement subbases, and surface layers of low traffic pavements (Tennis et al. 2004; Montes 2006).

Figure 1.2 is a picture of a colorful pervious concrete highway pavement in China. It can be seen that the pavement had been smooth and in good shape after it was just built. However, after several years of service, the pavement surface was badly worn by moving vehicles and many potholes appeared. Therefore, abrasion and raveling resistance is a significant concern for pervious concrete to be used for high traffic pavements. How to make PCPC stronger and durable to support heavy traffic loading is a challenge for researchers and engineers in highway agencies.



(a) Immediately after construction



(b) After several years in service

Figure 1.2 Pervious concrete pavement

1.4 Stiffness and Fatigue Strength of Asphalt Mixtures

Two fundamental properties of asphalt mixtures, stiffness and fatigue strength (or fatigue resistance), are of paramount importance in the performance of an asphalt pavements.

Stiffness determines the strain levels in asphalt pavements induced by traffic loads. It can reflect the viscoelastic properties of asphalt mixtures, such as dynamic modulus and creep compliance. It is also a material input required in the AASHTO Mechanistic-Empirical Pavement Design Guide (MEPDG). Recent findings indicate that use of asphalt mixtures with better-performing dynamic modulus will improve the overall performance of pavements. However, too high stiffness may cause asphalt mixtures to become excessively stiff and brittle, resulting in susceptibility to disintegration and cracking failures.

Fatigue strength is the ability of asphalt mixtures to withstand repeated traffic loads within allowable tensile strain levels in pavement without cracking. When asphalt pavements reach their failure lives, some type of distress is expected to occur due to the

combined effects of environment and excessive repeated traffic loads (Monismith and Deacon 1969). Fatigue cracking is one of the major types of pavement distress (others include rutting, low-temperature cracking, and moisture damage) and is also one of the primary concerns when evaluating the service life of an asphalt pavement.

1.5 Testing Methodology

Loaded wheel tester (LWT) is commonly used to test rutting and fatigue potential of asphalt mixtures by applying moving wheel loads on asphalt mixture specimens. Several LWTs are currently used in the United States including the Georgia Loaded Wheel Tester (GLWT), the Asphalt Pavement Analyzer (APA), the Hamburg Wheel Tracking Device (HWT), the French Pavement Rut Tester (FPRT), and the Purdue University Laboratory Wheel Tracking Device (PUR Wheel) (Cooley et al. 2000).

APA, as shown in Figure 1.3, is an intelligent upgrade version of GLWT which was developed in 1996. It features three movable loaded wheels and contact pressure through three air pressurized rubber hoses to simulate vehicle actions on pavements (Collins 1995). Compared to the other testing machines for pavement materials, more realistic loading conditions can be simulated on the specimens by APA in the laboratory. A temperature environment ranging from 4 to 72°C (39 to 162°F) in the testing chamber of APA can be achieved under both dry and water submerged conditions. The loading frequency is able to be specified from 0.0167 to 1 Hz, and the magnitude of the wheel loads can reach up to 1112 N (250 lbs). Recently, a modified model of APA with a new control system was developed, allowing users to perform calibrations for temperature,

load, and air pressure, to operate the process of testing, and to analyze data more efficiently and accurately.



(a) Latest version of APA



(b) Movable wheels and load cell

Figure 1.3 APA

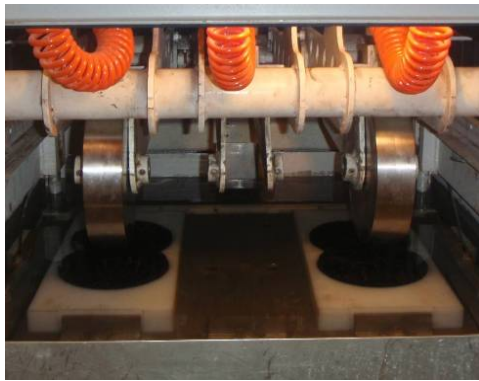
Two APA tests, rutting resistance and moisture susceptibility tests, are currently conducted by subjecting laboratory specimens to the repeated wheel loads under dry and water submerged conditions, as shown in Figure 1.4.



(a) Rutting test (permanent deformation test)



(b) Samples after rutting test



(c) Hamburg test (moisture susceptibility)



(d) Samples after test

Figure 1.4 Current APA tests

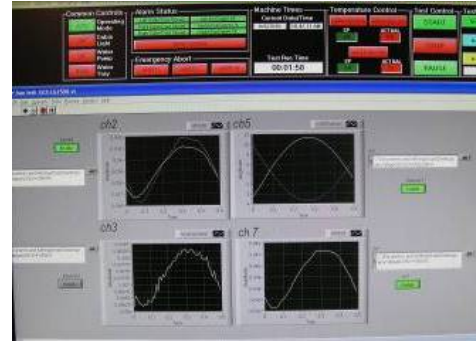
Because of the potential benefits of APA for simulating the traffic loading conditions as on real pavements, four innovative or modified tests are proposed in this study for testing pavement materials: (1) evaluating the improvement effect of geogrids reinforced pavement base courses; (2) abrasion resistance test for portland cement pervious concrete under cyclic loading; (3) characterizing viscoelastic properties of asphalt mixtures; (4) investigating fatigue properties of asphalt mixtures.

For the analysis in this study, some Linear Variable Differential Transducers (LVDTs) need to be installed on the specimens to obtain deformation data. The National Instrument (NI) data acquisition system (DAQ) and the corresponding LabView

programs were established and incorporated in the control system of APA for data collection and data processing (Figure 1.5).



(a) NI data acquisition system (DAQ)



(b) Data collection programs

Figure 1.5 Data acquisition system and programs

1.6 Objectives and Scope

As the development of science and technology, the testing method for pavement materials has been improved all the time. Many new or modified experimental approaches have been successfully applied for pavement systems. Throughout the development of all those tests, the philosophy for designing a testing method has always been focused on making the tests more efficient, accurate and in consistent with actual situations. Based on this principle, the concept of using LWT for characterizing the properties of pavement materials were promoted through four innovative or modified tests in this study:

1. A testing method was employed to evaluate the reinforcement effect of geogrids in pavement base courses. Cyclic loads were applied to specimens through the movable wheels of APA in the test. River sand and gravel were both tested as

base courses reinforced with four types of geogrids. Three different parameters were used to evaluate the reinforcement effect of geogrids.

2. Based on the mechanisms of abrading damage caused by traffic loads, a simple method for evaluating the abrasion resistance of pervious concrete under repeated loads was proposed. Some modifications were made to the steel wheels of APA to achieve a better simulation of the interaction of pavement surface and moving vehicles.
3. New methods to characterize the viscoelastic and fatigue properties of asphalt mixtures using LWT were developed. The study of these two sections can be summarized into the following three aspects:
 - a. The mechanical loading system of APA was analyzed and the stress solution of the specimen under cyclic loading was established.
 - b. Creep and complex modulus tests were conducted in tension conditions using APA. Creep compliance, dynamic modulus, and phase angles were calculated using the stress and the measured strain to evaluate the viscoelastic behaviors of asphalt mixtures.
 - c. A flexural tension LWT fatigue test (loaded wheel fatigue test) was proposed using the cyclic loading system provided by APA. Two theoretical analysis approaches in terms of change of flexural stiffness and dissipated energy were employed to evaluate fatigue failure of asphalt mixtures.

In addition to the tests proposed in the study, some other conventional or commonly used tests have been conducted to the same materials or mixtures for comparison purpose.

The feasibility of the proposed test methods with APA was validated through the comparison.

1.7 Arrangement of the Dissertation

This study is composed of six chapters. CHAPTER 1 is an introductory chapter outlining the problem statement and the objectives of the research work. The scope of the study is clearly stated in this chapter as well as the composition of the dissertation.

CHAPTER 2 introduces a modified testing method in evaluating the effect of geogrids in reinforcing pavement base courses. An overview is given in the introduction for using geogrids in pavement constructions as well as their reinforcement mechanisms. Evaluation of the reinforcement effect of various combinations of geogrids and pavement base courses are discussed in this chapter.

CHAPTER 3 describes a simple and efficient testing method in evaluating the abrasion resistance of portland cement pervious concrete (PCPC). A cyclic loading test is introduced in this chapter in accordance with the abrading and raveling actions for the pervious concrete pavements. The validity and feasibility of this test are discussed in this chapter by comparing to the conventional tests to the same pervious concrete mixtures.

CHAPTER 4 and CHAPTER 5 introduce two innovative experimental methods in investigating the viscoelastic and fatigue properties of asphalt mixtures utilizing LWT, respectively. The results from the proposed tests are compared to corresponding conventional tests for various asphalt mixtures with different aggregate types and asphalt binders.

CHAPTER 6 includes a list of conclusions drawn from the study as well as some recommendations for future research work.

CHAPTER 2 EVALUATING REINFORCEMENT EFFECT OF GEOGRIDS IN PAVEMENT BASE COURSES

2.1 Introduction

Due to the potential benefits of decreasing permanent vertical deformation, improving the ability of lateral restriction, and reducing long-term maintenance cost, geogrids have been widely used as an engineering material for reinforcement in many earthwork constructions, such as pavement bases, bridge abutments, and geo-environmental engineering applications. Three primary mechanisms have been proposed to attribute to geogrids' ability to stabilize and reinforce pavement system: aggregates separation, lateral confinement, and tensioned membrane effect (Berg et al. 2000). When pavements are subjected to traffic loads, base aggregates are interlocked by geogrids to confine the base and reinforce the subgrade. Therefore, pavement deflection can be reduced and desired base aggregate rut depth will be maintained (Haas et al. 1988, Chen et al. 2009). However, under repeated traffic loads, the interaction behavior in the base course and geogrids reinforcing system is very complicated. The overall behavior of geogrid-reinforced base depends on the properties of geogrids, characteristics of base aggregate, and the interface interaction between the two. In addition, the performance of geogrids are even more varied than their related applications, and an inappropriate geogrids for application could lead to insufficient reinforcement or money waste. Different geogrids and aggregates combinations could affect the interlocking effect significantly because of the diversification of aperture, shape and stiffness of geogrids.

Accordingly, to develop an effective method to investigate the interlocking actions between different geogrids and aggregates has great importance in evaluating the effects of geogrid reinforcement.

Currently, numerous experimental methods have been developed to appraise the effects of geogrids in pavement structures containing from small-scale laboratory tests to full-scale field tests. A field test on reinforced and unreinforced sections of unpaved road with a standard vehicle axle loading was carried out by Fannin and Sigurdsson (1996). The results showed that the geogrids reinforced sections exhibited significant improvement on the deformation resistance. Milligan and Love (1984) conducted a monotonic loading test for evaluating the performances of a geosynthetics reinforced soil-aggregate system through small-scale tests under plane strain conditions. The results indicated that geogrids-reinforced systems yielded a better performance than unreinforced systems, and the geogrids effectively resisted the tensile strains developed at the base of the aggregate layer. Perkins (1999) investigated the mechanistic response of geosynthetic-reinforced flexible pavement through laboratory cyclic loading plate tests. A significant improvement in reducing the permanent deformation was shown in the test results due to reinforcement of geosynthetic. Similar results were found in the laboratory test under cyclic loads conducted by Leng et al. (2002). A cyclic multi-plate loading test was proposed by Chen et al. (2009) on flexible pavement sections with and without geogrids reinforced, and the effects caused by the tensile modulus, aperture shape and location of geogrids were considered. The behavior of subgrade soils with a single layer of geogrids was tested under both static and cyclic loading system by Kamel et al. (2004). Most of the findings in previous studies indicate that the addition of geogrids in pavement

bases can potentially extend the pavement's service life by reducing the permanent deflection and restricting the propagation of reflection cracks. Recently, as a cyclic testing system, Asphalt Pavement Analyzer (APA) was introduced by Zhang (2007) and Han et al. (2008) in the University of Kansas to evaluate the effects of geosynthetics-soil confinement regarding different types of geosynthetics. With some modifications, the approach developed by the University of Kansas was employed in this study to evaluate the rutting resistance of pavement base courses with or without geogrid reinforced.

2.2 Objectives and Scope

This study primarily focuses on assessing the feasibility of using loaded wheel tester (LWT) to investigate the effects of geogrids in reinforcing pavement base courses. The rut depth of the base courses was measured as the basic evaluation parameter of the improvement. Four types of geogrids with different apertures and stiffness were tested in river sand and gravel base courses.

2.3 Laboratory Experiment

In the test, cyclic loads were supplied by Asphalt Pavement Analyzer (APA) loading system which features three controllable loaded steel wheels and contact pressures through three air pressurized rubber hoses to simulate vehicle loads on actual pavements. According to different types of base materials, different load magnitudes were used. In terms of the rut depth of base courses, three evaluation parameters were adopted to evaluate the reinforcement effect of geogrids.

2.4 Specimen Preparation

River sand and gravel, similar to the materials tested in previous study of the University of Kansas were used as pavement base materials in this experiment. Besides, in order to reflect the effect of the interaction between the grain-size of aggregate and the aperture of geogrids, two different gradations of gravel aggregates were considered. One is the original AB-3 gravel, while the other one is the adjusted AB-3 gravel which satisfies the standard of the Tennessee Department of Transportation (TDOT) Gradation D. The grain-size distributions of the river sand and gravels used in this study are shown in Figure 2.1. It is evident that adjusted AB-3 gravel has bigger aggregate size than that of original AB-3 gravel.

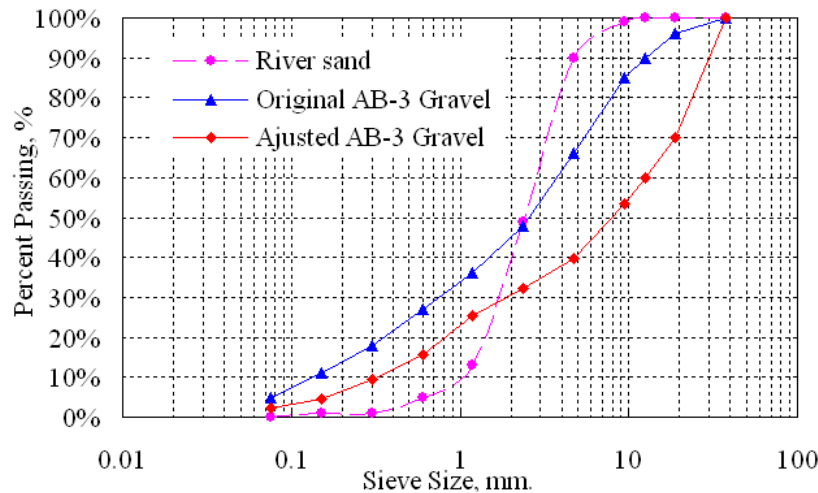


Figure 2.1 Grain-size distribution of base course materials

Four types of geogrids, geogrids#1 (GD1), geogrids#2 (GD2), geogrids#3 (GD3) and geogrids#4 (GD4), as shown in Figure 2.2 were evaluated in the study. All of these four geogrids were made of polypropylene material. In terms of fabrication technology, both GD1 and GD2 are multilayer geogrids, with GD1 being comprised of two layers and

GD2 of three layers, while GD3 and GD4 are punched-drawn biaxial geogrids with a single layer. The apertures of GD1 and GD2 are relatively smaller than those of GD3 and GD4 due to their multiplayer structures, but the stiffness and tensile strength of GD3 and GD4 are much higher than those of GD1 and GD2.

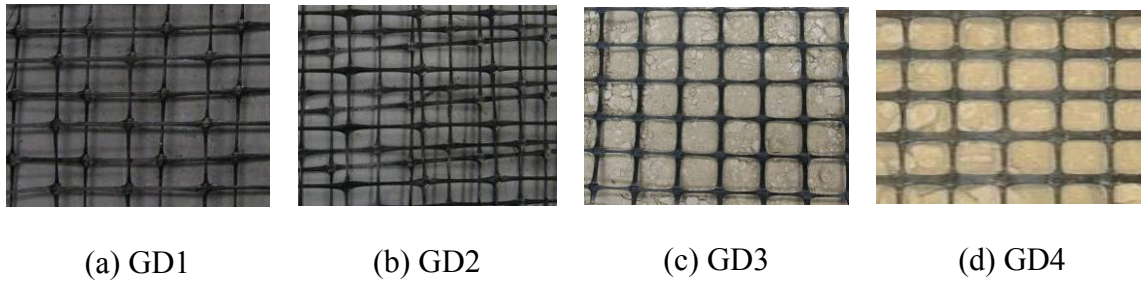


Figure 2.2 Geogrids used for testing

According to their respective physical properties and applications, GD1 and GD2 were tested only in river sand base material, while GD2, GD3 and GD4 were tested in gravel base materials. In the test, geogrids were placed 25 mm (1 in.) below the surface of base specimens. Each test was designed for running 8000 cycles, and the rut depth (vertical deformation) was measured at every cycle automatically. Prior to testing, the base material was compacted to 70% of the maximum dry density through tamping efforts. In order to achieve satisfied compaction effect for the specimen, the base material was placed and compacted in three layers. To control the density of the specimen, the aggregate mass for each layer was calculated and measured. After each layer of the base material was filled into the aluminum testing box, it was compacted by tamping efforts till the specimen's thickness reached the specified depth (Figure 2.3). In addition, it should be noted that geogrids should be buried even and level in the base course. Figure 2.4a shows the specimen with geogrids partially buried in the aggregates. After all the material layers were filled into the testing box and compacted, the specimen was ready to

be mounted into the APA testing chamber, as shown in Figure 2.4b. In order to compare the results from this study to the previous studies in Zhang (2007) and Han et al. (2008), the similar testing conditions were followed.



(a) Specimen with sand base course



(b) Specimen with gravel base course

Figure 2.3 Specimen preparation



(a) Base course with geogrids reinforced



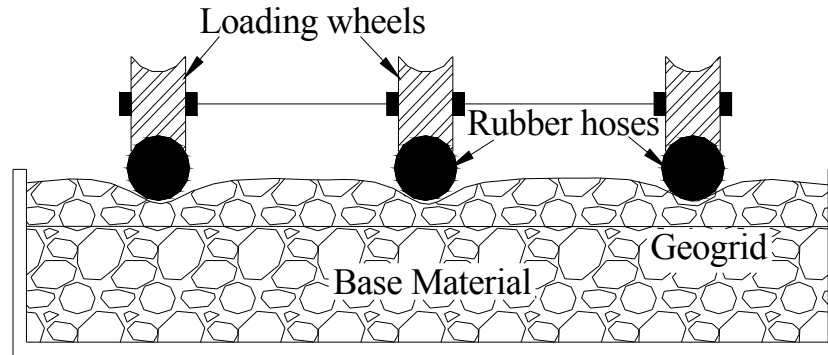
(b) Specimen for test

Figure 2.4 Test setup

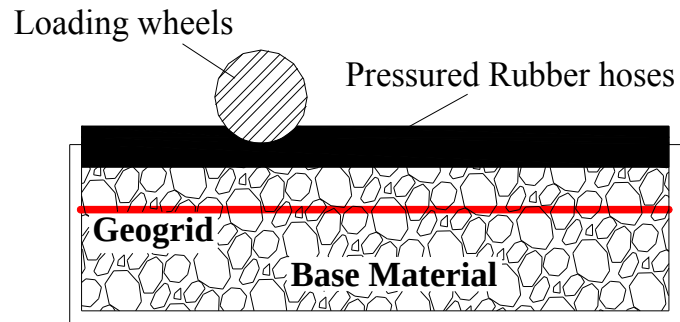
2.5 Testing Protocol

In the test, the base courses in the aluminum box were placed underneath the three rubber hoses with repeated wheel loads applied on them, as shown in the schematic

diagram in Figure 2.5. During the test, the rut depths were measured at five different locations along the pressurized hose. The average deflections were selected as the rut depths for analyzing and evaluating the rutting resistance of the specimen.



(a) Front view



(b) Side view

Figure 2.5 Schematic diagram of testing

88N (20lb) wheel loads with 138kPa (20psi) corresponding rubber hoses' pressures were designed for river sand base courses. While, 353N (80lb) and 552kPa (80psi) were used for gravel base courses. The specimen before and after test were presented in Figure 2.6, in which three rutting grooves can be seen clearly on the specimen after testing.



(a) Specimen before test



(b) Specimen after test

Figure 2.6 Base course specimen before and after test

2.6 Evaluation Approach

In order to quantify the benefits of the reinforcement effect of geogrids, three technical indices, the Traffic Benefit Ratio (TBR), the Rutting Reduction Ratio (RRR), and the Rate of Deflection (ROD) were adopted to appraise the testing results.

2.6.1 Traffic Benefit Ratio (TBR)

Traffic Benefit Ratio (TBR) is defined as the ratio of the number of cycles required to reach a certain rut depth in the reinforced specimen to the number of cycles required in the non-reinforced specimen. It can be expressed as the following formula:

$$TBR = \frac{N_{reinforced}}{N_{non-reinforced}} \quad (2.1)$$

where, $N_{reinforced}$ and $N_{non-reinforced}$ are the number of cycles for geogrids reinforced and non-reinforced specimens, respectively. The threshold rut depth selected for calculating the TBR is defined by the deflection point from the reinforced testing curve when the rut depth becomes stabilized. In this study, a rut depth of 7 mm for river sand base course and 3 mm for gravel base course were chosen for the calculation of TBR. Figure 2.7b, Figure 2.8b and Figure 2.9b illustrate the rationales for using the threshold rut depths to determine TBR of gravel base course, and the corresponding numbers of cycles were also marked out in the figures. Higher TBR usually shows more benefits of the geogrids in reinforcing base courses.

2.6.2 Rut Reduction Ratio (RRR)

The Rut Reduction Ratio (RRR) is defined as the ratio of the rut depth of the reinforced base course at a certain cycle to the rut depth of the non-reinforced base course at the same cycle. From the expression of RRR in the following formula, it is obvious that the specimen with lower RRR value has better performance in resisting rutting.

$$RRR = \frac{u_{reinforced}}{u_{non-reinforced}} \quad (2.2)$$

where, $u_{reinforced}$ and $u_{non-reinforced}$ are the rut depth (deflection) at specific cycle for reinforced and non-reinforced specimens, respectively. The rut depth at the terminal cycle (the 8000th cycle) is used for calculating RRR in this study.

2.6.3 Rate of Deflection (ROD)

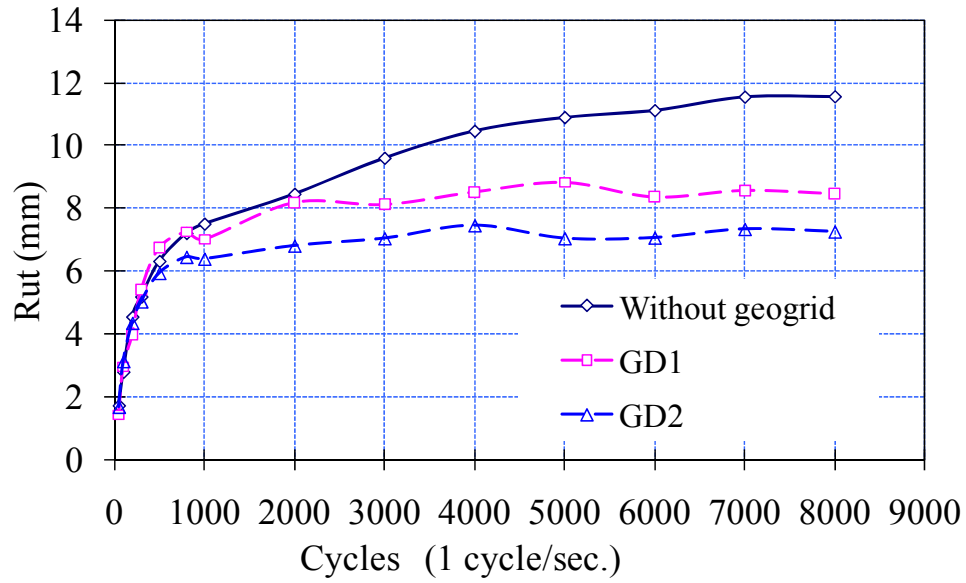
Rate of Deflection (ROD) has also been utilized to evaluate the rut resistance of various specimens, which is defined as the changing rate (velocity) of the vertical deformation. The lower the ROD value usually reflects better rutting resistance of the specimen. It can be expressed as the following formula:

$$DR = \frac{u_{n+1} - u_n}{t_{n+1} - t_n} \quad (2.3)$$

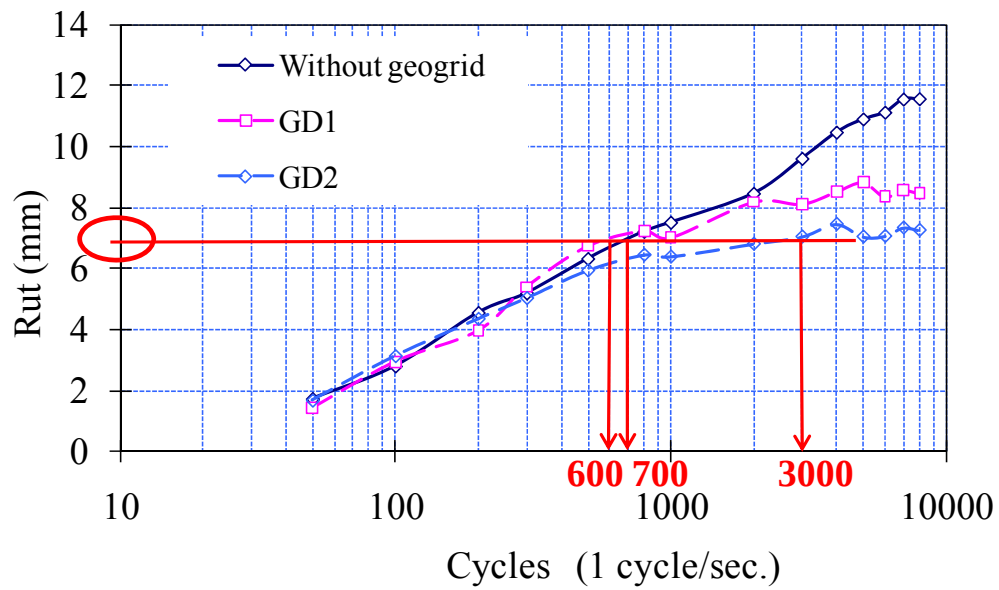
where, u_n is the deflection of the n th cycle; t_n is the time period of the n th cycle.

2.7 Results and Discussions

Due to the large amount of the collecting data, the results at specific cycles, as shown in Figure 2.7, Figure 2.8 and Figure 2.9, were selected for analysis. According to the results for both river sand and gravel bases, it is evident that all the bases reinforced by geogrids exhibited less rut depths than the control ones that without geogrids reinforced.

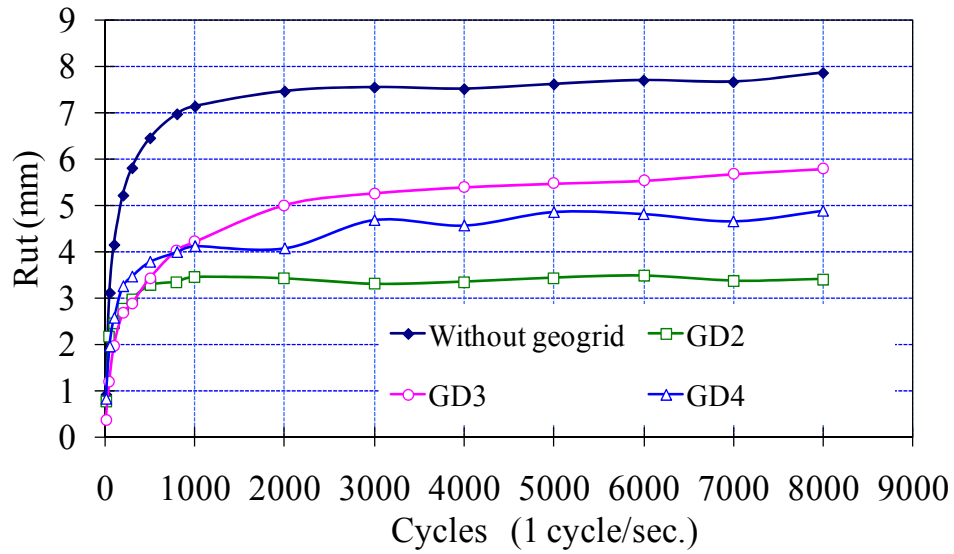


(a) Rut depth

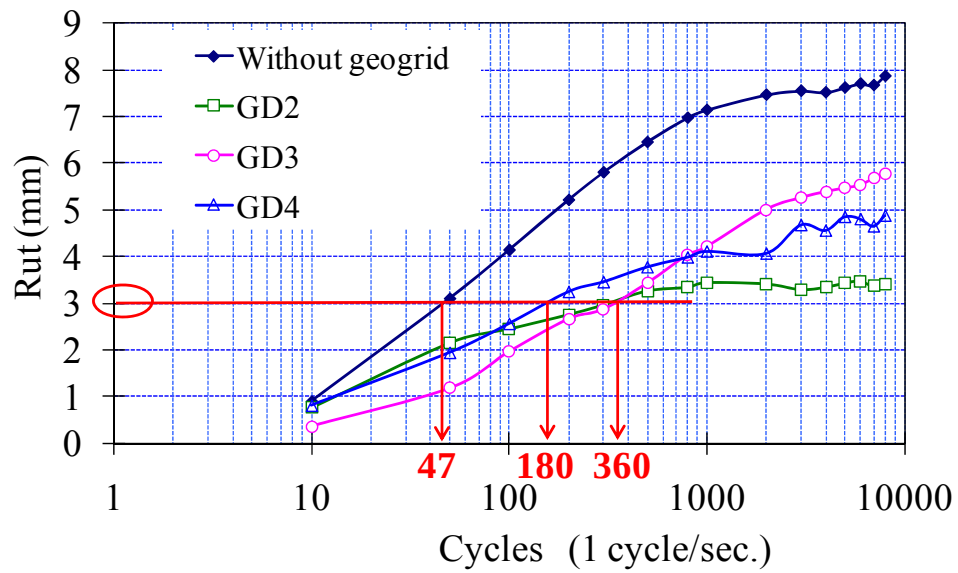


(b) TBR

Figure 2.7 Rut depth vs. load cycles for river sand base

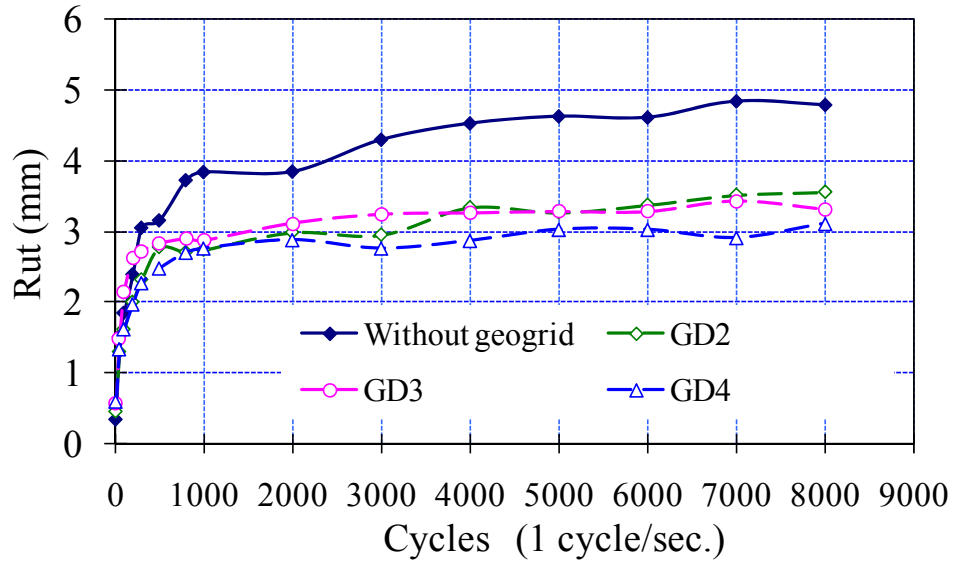


(a) Rut depth

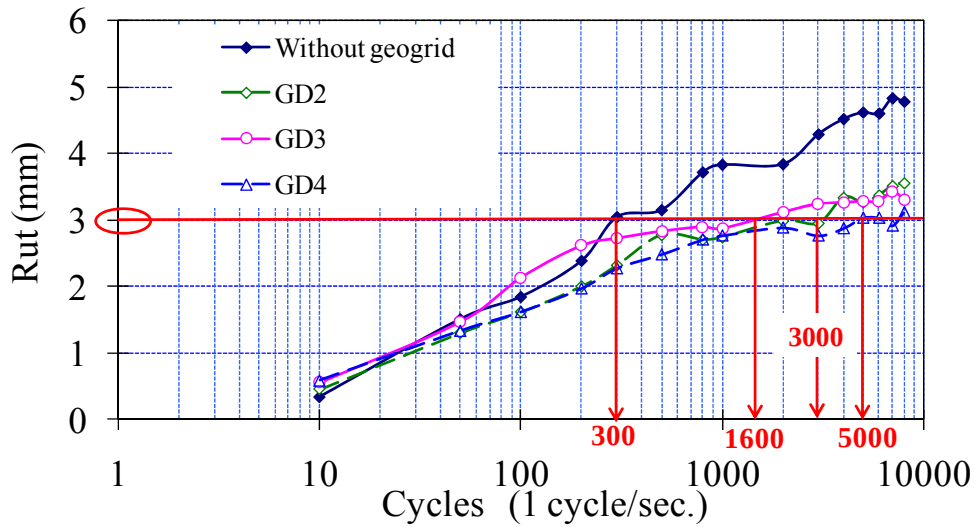


(b) TBR

Figure 2.8 Rut depth vs. load cycles for original AB-3 gravel base



(a) Rut depth



(b) TBR

Figure 2.9 Rut depth vs. load cycles for adjusted AB-3 gravel base

According to the results shown in the figures, it is clear that the gravel base, although subjected to higher wheel loading, exhibited lower rut depth than that of the river sand base. The rut depth at the 8000th cycle of river sand base without geogrids reinforced was 11.6 mm, while it was 7.9 mm for original AB-3 gravel base and 4.8 mm

for adjusted AB-3 gravel base. When applied in the adjusted AB-3 gravel base, all three geogrids showed similar reinforcement effects in terms of the final stabilized rut depths. It was observed that the rutting of gravel stabilized was much faster than that of the river sand. Basically, the gravel base obtained 80% of the total rut depth (within 8000 cycles) around 600 cycles; whereas the river sand base needed approximately 3000 cycles to reach this plateau. Based on the rut depth results, all of the base materials reinforced by geogrids had smaller threshold deflections than the control material without geogrids. The adjusted AB-3 gravel reinforced by geogrids had less rut depth than the original AB-3 gravel reinforced by the same geogrids, which indicates that the geogrids had better effects when using in adjusted AB-3 gravel base. This is due to the fact that the grain-size of adjusted AB-3 gravel is bigger than that of the original AB-3 gravel, so that the interlocking actions between these aggregates and geogrids played more active roles in restricting rotations and vertical movements of the aggregate particles.

With respect to river sand base course, both GD1 and GD2 resulted in significant improvement on reinforcement. But then, GD2 had a better effect than GD1 because of its relatively high stiffness and small apertures created by triple-layer structure. Regarding to all the geogrids tested for gravel base, GD2 had a better effect than the others in the original AB-3 gravel, while GD4 was the most efficient one for adjusted AB-3 gravel. The rut depths of the original and adjusted AB-3 gravel bases reinforced by GD4 were 4.89 mm and 3.12 mm respectively at the 8000th cycle, which were only 60% of the rut depth at the 8000th cycle of the controlled base.

Table 2.1 summarizes the results of the rut depth, TBR and RRR. Compared to the controlled specimens without geogrids reinforced, all the base courses with geogrids

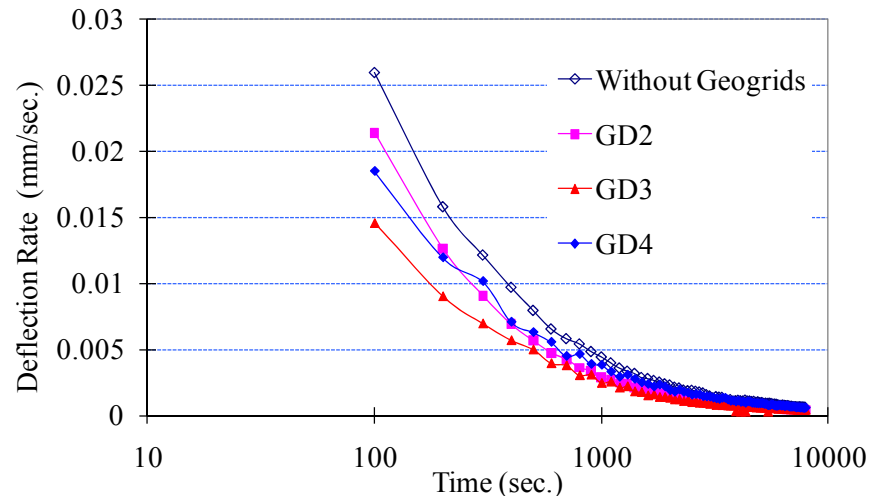
reinforced showed appreciable improvement in rutting resistance in terms of TBR and RRR. According to the TBR value for river sand base, GD2 also had a better effect than GD1, which is consistent with the results obtained through RRR. In sum, GD2 and GD4 had better reinforcement effects in reinforcing gravel base course according to TBR values. However, according to RRR values all the geogrids had similar improvement, which indicates that RRR is not sensitive for identifying the discrepancies of the effects between various geogrids.

Table 2.1 Rut, TBR and RRR results for various geogrids

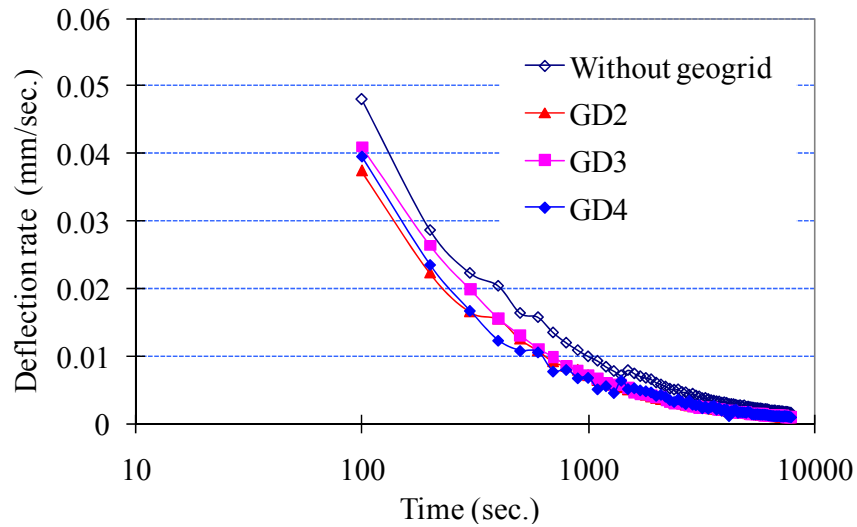
Base material	Item	Without Geogrids	GD1	GD2	GD3	GD4
River Sand	Rut (mm)	11.57	8.47	7.26	N/A	N/A
	RRR	1.00	0.73	0.63	N/A	N/A
	TBR	1.00	0.86	4.29	N/A	N/A
Original AB-3 Gravel	Rut(mm)	7.85	N/A	3.40	5.78	4.89
	RRR	1.00	N/A	0.43	0.74	0.62
	TBR	1.00	N/A	7.66	7.66	3.83
Adjusted AB-3 Gravel	Rut(mm)	4.78	N/A	3.56	3.31	3.12
	RRR	1.00	N/A	0.74	0.69	0.65
	TBR	1.00	N/A	10.00	5.33	16.67

As shown in Figure 2.10, the rate of deflection (ROD) results for the AB-3 gravel base, the deflection rate of the gravel base without geogrids reinforced were greater than

the gravel base with geogrids reinforced, especially in the initial stage of the test. The deflection rates became smaller with increased cycles, and basically no significant difference could be observed between the reinforced and non-reinforced base courses after 2000 cycles.



(a) Original gravel base



(b) Adjusted gravel base

Figure 2.10 Rate of deflection results for gravel base course

2.8 Summary and Conclusions

Laboratory experiments were carried out to evaluate the reinforcement effects of geogrids in river sand and gravel base courses utilizing LWT. Four types of geogrids were evaluated, two for the river sand base and three for the gravel bases. Based on results the following can be summarized:

1. The LWT system with a modified specimen box is appropriate to be used to evaluate the reinforcement effects of geogrids in pavement base courses. The results were repeatable and generally in agreement with the results from another independent study by the researchers at the University of Kansas for similar base materials.
2. The testing method proposed in this study is able to differentiate the effects existing among the aggregates and geogrids combinations. All the geogrids tested in this study exhibited significant improvement in the rutting resistance for both river sand and gravel base courses.

CHAPTER 3 INNOVATIVE TEST FOR EVALUATING ABRASION RESISTANCE OF PERVIOUS CONCRETE

3.1 Introduction

Portland cement pervious concrete (PCPC) is an open-graded friction course (OGFC) mixture with highly interconnected voids between aggregate particles. However, in order to obtain high porosity and water permeability, little or no fine aggregate is added into the mixture, which leads to aggregate particles in PCPC being bonded together through aggregate to aggregate contacts rather than being embedded in cement paste as in ordinary portland cement concrete. The general compressive and splitting tensile strength of PCPC is 25Mpa and 2.5Mpa, respectively, which are significantly less than those of ordinary portland cement concrete. Therefore, PCPC has relatively low mechanical properties and it is vulnerable to spalling and raveling under repeated traffic loads on the pavement. Although many studies have shown that the properties of PCPC can be improved by using small grain-size coarse aggregate, adding a small amount of fine aggregate, polymer modifiers, as well as chemical reinforcing agents (Ramakrishnan 1992; Yang and Jiang 2003; Kevern 2008), the major applications of PCPC are currently limited on parking lots, sidewalks, pavement subbases, and surface layers of low traffic pavements (Tennis et al. 2004; Montes 2006).

Numerous testing methods and devices have been developed to evaluate the abrasion resistance of ordinary portland cement concrete (Sadegzadeh and Kettle 1988; Dhir et al. 1991; Atis 2002). A test method for determining the abrasion resistance of

horizontal concrete surface by subjecting concrete specimens to the special abrasion machines has been introduced in ASTM C 779. Three abrasion test machines, disks abrasion test machine, dressing wheel abrasion test machine and ball bearing abrasion test machine are showed in this specification and 300 by 300-mm (12 by 12-in.) specimen is required for the test. The British Pendulum machine has been developed to measure the surface frictional properties of concrete through impacting actions in ASTM E 303. Moreover, ASTM C 418 describes a method for abrasion resistance of concrete using sandblasting device. This device simulates the actions of waterborne abrasives and abrasives caused by traffic loads on concrete surfaces.

Based on the facts that the abrasion resistance is mainly determined by the bonding strength between aggregates, freeze-thaw test was employed to examine the durability of pervious concrete (NRMCA 2004; Schaefer et al. 2006; Demille 2008). In freeze-thaw test, the resistance of concrete is characterized by subjecting the specimen in cyclic effects of freezing and thawing. The continuously expansive action caused by the volume change from water to ice can result in distresses in the concrete, and aggregate particles will continue to be lost off on the surface once the bonding strength is insufficient. Usually, freeze-thaw test uses the weight loss or the change of dynamic Young's modulus to quantify the durability of concrete. However, PCPC is high in air voids and permeability and, generally, it cannot be fully saturated. Generally, the water can easily drain away from PCPC, or PCPC has enough space within to accommodate the volume expansion due to freezing. Therefore, the freeze-thaw test may not be appropriate for characterizing the durability of pervious concrete (Schaefer et al. 2006).

For pervious concrete, the primary cause for the damage is that aggregate particles on the surface being worn off or crushed by the tires of moving vehicles. Due to the reduced strength and contact area between neighboring aggregate particles, PCPC is more vulnerable than ordinary portland cement concrete in this situation. Therefore, the major concern about the durability of PCPC is its ability to resist spalling and raveling under repeated traffic loads. Currently, none of the methods for testing ordinary concrete has been proven effective for pervious concrete due to their different mechanisms of abrasion failure. Most of the existing methods are surface wearing resistance tests rather than spalling and raveling resistance tests subjected to the long term effect by repeated impulsive traffic loads. How to address the abrasion resistance of PCPC is of great concern to researchers.

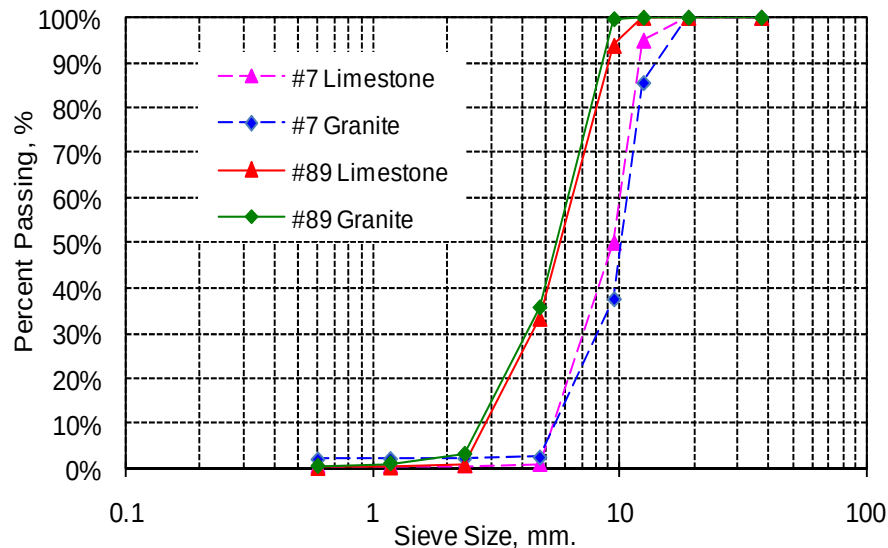
Recently, a standard test in ASTM C 944 for evaluating the abrasion resistance of concrete through a rotating-cutter machine was employed and modified by Kevern (2008) for pervious concrete. The Los Angeles abrasion machine, initially for testing the abrasion resistance of coarse aggregate, has been used for testing the abrasion resistance of asphalt open-graded friction course (OGFC), which is a porous asphalt mixture similar to PCPC in porosity (Watson et al. 2003; Alvarez et al. 2010). In this study, the Asphalt Pavement Analyzer (APA), which is one of the loaded wheel testers (LWTs) widely used in the United State for testing asphalt mixtures, has been used by the authors to evaluate the abrasion resistance of PCPC. During the loaded abrasion test by means of APA, beam specimens are subjected to repeated wheel loads with controllable magnitude and contact pressure, and both dry and water-submerged conditions can be considered for the specimens.

3.2 Objectives and Scope

The objective of this study is to propose a LWT abrasion test to evaluate the abrasion resistance of PCPC using Asphalt Pavement Analyzer (APA). Various PCPC mixtures were considered in this study, and latex and fibers were added in some mixtures in order to improve the abrasion resistance of PCPC. Porosity, permeability, compressive, and splitting tensile strengths were conducted to investigate the fundamental properties of PCPC. In addition to LWT abrasion test, two potential abrasion tests, Cantabro abrasion and loaded sweep abrasion tests, were also performed to the same PCPC mixtures.

3.3 Materials and Specimen Preparation

Two types of coarse aggregates (limestone and granite) with two different grain-size distributions were considered in this study. The gradations for these coarse aggregates are shown in Figure 3.1.



Styrene Butadiene Rubber (SBR) and Ethoxylated Di-sec-butylphenol latex modifier (0.91 specific gravity and 53% water content) was added into the mixtures to improve the bonding strength. Polypropylene monofilament fibers with a length of 19 mm, a tensile strength of 680MPa, and a specific gravity of 0.9 were also added into the PCPC mixtures to improve their abrasion resistance.

According to the previous studies, all the mixtures contained a small amount of natural sand in order to guarantee the strength, and the sand was used to replace 7% coarse aggregate in the mixtures by weight. For the mixtures containing latex and/or fiber, the portion of latex was used to replace 10% cement by weight and the amount of fiber was 0.9 kg/m³ as recommended by the manufacturer. The mix proportions of the PCPC mixtures are presented in

Table 3.1 Mix proportions for PCPC (unit: kg/m³)

Aggregate	Mix Type	Cement	Latex	Coarse Aggregate	Natural Sand	Water	Fiber	HRWR (l)	
#7	LS	G1	360	--	1430	100	130	--	1.1
		G2	350	35	1410	100	110	--	1.1
		G3	360	--	1430	100	130	0.9	1.1
		G4	350	35	1410	100	110	0.9	1.1
	GR	G1	360	--	1420	100	130	--	1.1
		G2	350	35	1400	100	110	--	1.1
		G3	360	--	1420	100	130	0.9	1.1
		G4	350	35	1400	100	110	0.9	1.1
#89	LS	G1	370	--	1490	100	130	--	1.1
		G2	370	37	1460	100	120	--	1.1
		G3	370	--	1490	100	130	0.9	1.1
		G4	370	37	1460	100	120	0.9	1.1
	GR	G1	360	--	1460	100	130	--	1.1
		G2	360	36	1430	100	110	--	1.1
		G3	360	--	1460	100	130	0.9	1.1
		G4	360	36	1430	100	110	0.9	1.1

*LS-Limestone; GR-Granite; G1-Control; G2-Latex modified; G3-Fiber added; G4-Latex & Fiber modified; HRWR-High Range Water Reducer.

The samples were fabricated and cured in the standard curing room for 28 days and cut into specific size for testing. Commonly, a paving machine or weighed roller is used in actual field construction for pervious concrete pavements. Marshall Hammer compaction method has been developed for compacting pervious concrete samples in the laboratory by either high or low designed compaction by Crouch (2003). The results

indicated that relatively high strengths for PCPC were obtained by using this Marshall Hammer compaction method. In this study, all PCPC specimens were fabricated by applying rodding effort for compaction in the laboratory according to ASTM C 94. In order to avoid high variability of the specimens caused by the process of compaction, only one designated technician was allowed to do the compaction during the whole experiment.

3.4 Laboratory Experiments

The primary laboratory tests in this study include compressive and splitting tensile strength test, effective air voids test, permeability test, Cantabro abrasion test and LWT abrasion test. The fundamental information of these tests is summarized in Table 3.2.

Table 3.2 Laboratory test summary

Test	Specification	Testing machine	Specimen Dimensions
Compressive strength	ASTM C 39	INSTRON	10*20cm cylinder
Splitting tensile strength	ASTM C 496	MTS	15*7.5cm cylinder
Effective air voids	ASTM D 7063	Corelok	15*7.5cm cylinder
Permeability	Florida Method (2006)	Modified Karol-Warner permeameter	15*7.5cm cylinder
Cantabro abrasion	ASTM C 131	LA abrasion machine	15*10cm cylinder
LWT abrasion	N/A	APA	30*12.5*5cm beam
Loaded sweep abrasion	ASTM D 7000-08	Modified electrical mixer	30.5*5cm circular plate

3.4.1 Physical and Mechanical Performance Tests

Prior to all the abrasion tests, effective air voids, permeability, compressive strength, and splitting tensile strength tests were conducted to investigate the fundamental properties of those various mixtures. The compressive and splitting tensile strength tests were conducted in accordance with ASTM C 39 and ASTM C 496/C 496M, respectively. The effective air voids test was performed according to ASTM D 7063 by using a CoreLok device (Figure 3.2), which is usually used for testing the porosity of compacted asphalt concrete samples. Through this test, the degree of interconnectivity of the pores in the specimen can be determined.



(a) Corelok device



(b) Sealed specimen

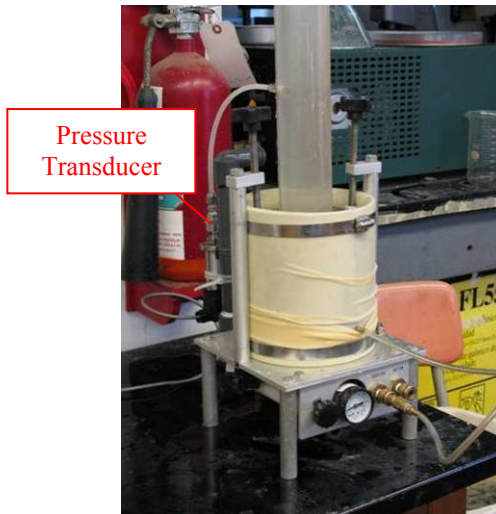
Figure 3.2 Effective air voids test for PCPC

Due to the high porosity and the interconnected air voids path, Darcy's law for laminar flow is no longer applicable for pervious concrete. According to the Florida Method (2006), a modified Karol-Warner flexible wall permeameter with a pressure sensor mounted on the water tube for measuring the water head pressure was used to test

the permeability of PCPC specimens (Figure 3.3), and a computer programmed with data acquisition system was used to collect the data automatically during the test. Permeability coefficients can be expressed in the function of hydraulic gradients and discharge velocity. And because the permeability coefficient varies with time, pseudo-permeability was propounded to represent the drainage ability for pervious concrete. Detailed information for the physical and mechanical performance testing can be obtained in Huang et al. (2010).



(a) Specimen and permeameter



(b) In the test

Figure 3.3 Permeability test for PCPC

3.4.2 Cantabro Abrasion Test

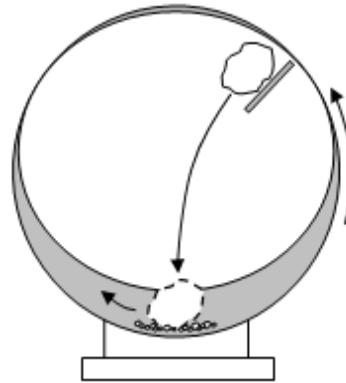
Cantabro test, initially used in Europe and South Africa to characterize the durability and resistance to stone loss for open-graded friction course (OGFC) mixtures of hot-mix asphalt (HMA), has been introduced into the United States to determine the resistance of HMA to abrasion (Ruiz et al. 1990; Watson et al. 2003; Alvarez et al. 2010). Since PCPC

has many similarities with HMA OGFC, the Cantabro test has the potential of testing the abrasion resistance of PCPC.

The Cantabro abrasion test in this study was carried out through a Los Angeles (LA) abrasion machine in accordance with ASTM C 131 but without steel ball charges, as shown in Figure 3.4. Prior to testing, the cylindrical specimen is weighed and placed in the steel drum to be tested at the rotating speed of 30 rpm. The weight loss of the specimen before and after the test is used to characterize the abrasion resistance of PCPC. Weight loss values were calculated every 50 revolutions. It should be mentioned that some trial tests were accomplished by using 100mm (4 in.) by 75 mm (3 in.) cylinders. However, those smaller specimens were crushed into pieces at very low cycles, and the discrepancies of the abrasion resistance from various mixtures were unable to be identified. Therefore, 150 (6 in.) by 100 mm (4 in.) cylinders were designed for Cantabro abrasion tests in this study, and all of them were cut from 150 by 300 mm (6 by 12-in.) cylinders through an electrical saw machine.



(a) L.A. abrasion machine



(b) Illustration of Cantabro test

Figure 3.4 Cantabro abrasion test

3.4.3 LWT Abrasion test (Loaded Wheel Abrasion Test)

In LWT abrasion test, repeated wheel loads are applied to the beam specimens by three movable loaded wheels to simulate actual loading situations on pavements (Figure 3.5). In the study, beam specimens were tested in water bath condition to consider the effect of hydraulic pressure caused by moving wheels on the abrasion performance of pavements. Prior to testing, the specimens were dried in ovens at low temperatures. After drying, the surfaces of the specimens were cleaned by steel brush to remove any loose aggregate particles. Subsequently, the specimens were subjected to 10000 cycles of repeated loads at the frequency of 2cycle/second. The width of the wear path is 35 mm, which is about 1/3 of the width of the specimen. In order to provide sufficient impacting and abrasive force to the specimen, the load for each wheel was determined as 890 N based on some trial tests with various loading levels.



(a) Dry condition



(b) Water submerged condition

Figure 3.5 LWT abrasion test (loaded wheel abrasion test)

Initially, the steel wheels with smooth surfaces were used in the study. However, only a small amount of abrasion was observed on the specimen's surface from the trial tests, and the weight loss was less than 0.5%. Besides, the smooth wheel can only provide an extrusion force on the specimen rather than a spalling/raveling force. Therefore, steel studs 7.5 mm in diameter and 1.5 mm high were soldered onto the initially smooth surface of the wheels, as shown in Figure 3.6. The addition of studs onto the steel wheels can better simulate the tires' abrasion action on pavement surface and thus better simulate the spalling/raveling failure on pervious concrete pavements.



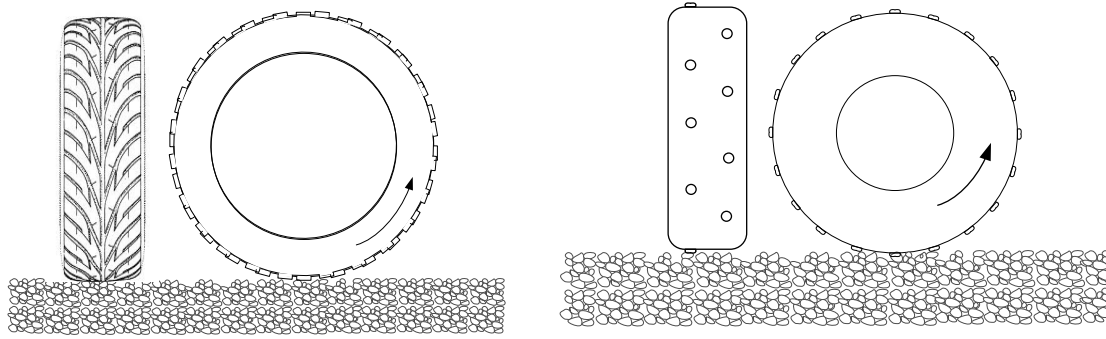
(a) Studded steel wheel



(b) Soldered studs

Figure 3.6 Studded steel wheels for LWT abrasion test

As can be seen from Figure 3.7, when a vehicle moves on the pervious concrete pavement, its tires will exert a horizontal force on aggregate particles. If these particles are not strongly bonded to the pervious concrete body, they will ultimately be worn off by the moving vehicle tires and spalling/raveling distress will occur. Once a small part has been created on the pavement surface, the damage will be continuously deteriorated due to the repeated impacting and raveling actions of vehicle loads. According to the abrasion mechanisms of the vehicle loads on pervious concrete pavements, the design of studded steel wheels are appropriate to consider the combined effects of abrasion in the test.



(a) Vehicle tire on pavement

(b) Studded wheel on specimen

Figure 3.7 Raveling action and LWT simulation (not to scale)

3.4.4 Loaded Sweep Abrasion Test

A loaded sweep abrasion test based on ASTM D 7000-08 is proposed for appraising the surface abrasion resistance for PCPC. This test is based on the sweep test originally used to measure the curing performance characteristics of bituminous emulsion and aggregates for asphalt pavement surface treatment. Some modifications have been made to make the abrading effect more severe. As shown in Figure 3.8, an additional load could be installed on the rotating shaft, and a wearable rubber hose is attached on the loading head.



(a) Sweep test device



(b) Loaded rubber head

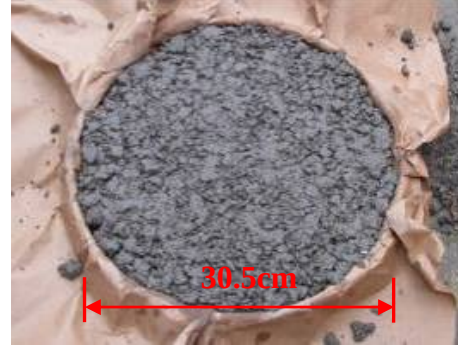
Figure 3.8 Loaded sweep abrasion test

With this design, the vertical load could be increased by adding extra weight to make the test more severe for abrading action. Meanwhile, a solid rubber hose with 16.5cm in length is attached to the removable brush holder to simulate the effect of car tires. Before testing, the loaded head and the rubber hose can be forced to contact with the specimen tightly by pushing the rotating shaft down with some preload given by the machine. During the test, the shaft will rotate randomly on the surface of the specimen as a specified speed.

According to the compaction method used for pervious concrete pavement in the field, rolling compaction method was used for the specimen fabrication in this test (Figure 3.9).



(a) Rolling compaction



(b) Specimen

Figure 3.9 Specimen compaction for loaded sweep abrasion test

3.5 Abrasion Evaluation Parameters

Two parameters, weight loss and wear depth, were used to characterize the abrasion resistance of PCPC in this study. Weight loss refers to the loss of specimen weight occurring during the test, and it can be calculated by the following formula:

$$\text{Weight Loss} = \frac{W_1 - W_2}{W_1} \times 100 \quad (3.1)$$

where, W_1 = Initial sample weight;

W_2 = Final sample weight.

While, wear depth is related to the vertical abrading depth on the surface of the specimen caused by abrasion, which can be measured automatically and continuously by computer through displacement sensors integrated in the machine.

3.6 Results and Discussions

3.6.1 Physical and Mechanical Performance Tests

Above all, it should be mentioned that the average effective air voids and permeability of the specimens cut from the top part of the original PCPC cylinders were 80% and 50% of those cut from the middle part. These differences were caused by the effect of surface finishing method used for specimen fabrication in the laboratory.

It can be observed from Table 3.3 that most of the PCPC mixtures made with #7 aggregates had effective air voids within the range from 20% to 30% while those mixtures made with #89 aggregates had effective air voids of 15% to 20%. Commonly, PCPC mixtures with an interconnected void content of 15 to 25% will exhibit acceptable drainable ability (ACI 2008). Furthermore, the mixtures of #7 aggregates had higher effective air voids and permeability than those of #89 aggregates, which was consistent with the findings from a previous study by the authors that the larger the grain-size of coarse aggregate, the higher the porosity and permeability of pervious concrete mixtures (Huang et al. 2010). In addition, the addition of latex polymer resulted in a slightly reduction in porosity and permeability of PCPC mixtures, on the account that latex incorporated into cementitious matrix increased the coating thickness of particles. Yet the mixtures containing fibers did not present any remarkable difference on the property of porosity.

Table 3.3 Results of effective air voids and permeability

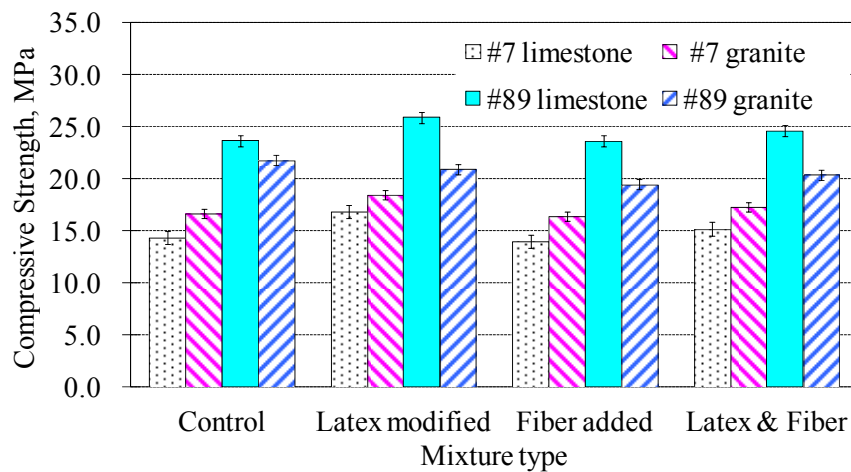
Aggregate	Mix type	Average test results	
		Effective air voids (%)	Permeability (cm/s)
#7	LS	G1	27.4
		G2	21.6
		G3	28.2
		G4	23.0
	GR	G1	25.3
		G2	22.9
		G3	24.2
		G4	23.5
#89	LS	G1	16.4
		G2	15.0
		G3	16.5
		G4	10.0
	GR	G1	21.9
		G2	18.2
		G3	21.3
		G4	18.0

* LS-Limestone; GR-Granite; G1-Control; G2-Latex modified; G3-Fiber added; G4-Latex & Fiber modified.

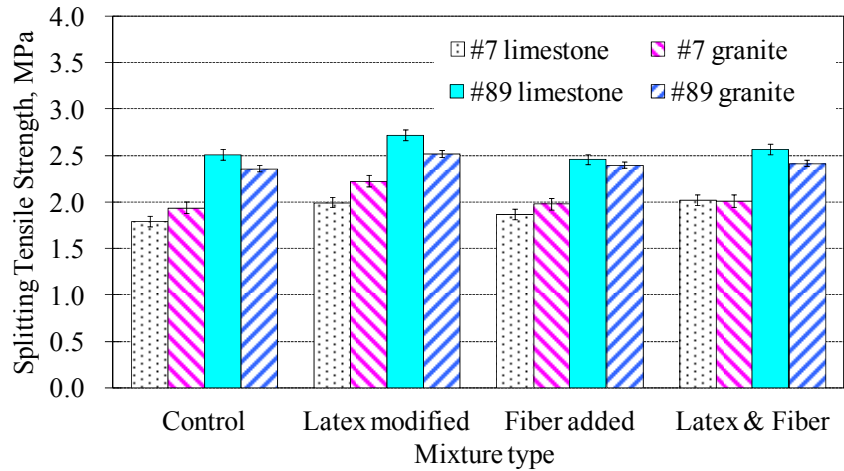
Figure 3.10 shows the compressive and splitting tensile strength results of PCPC mixtures. It is obvious that the smaller the grain-size of coarse aggregate the higher the compressive and splitting tensile strength of the mixtures. The inclusion of latex increased the strength of pervious concrete mixtures to a certain extent, which can be attributed to the improved bonding strength and contact areas between neighboring

aggregate particles. Latex and cement hydration products commingle and create two interpenetrating matrices which work together, resulting in the improvement of strength.

The mixtures containing fibers had no contribution towards compressive strength, and they just showed slight improvement in the splitting tensile strength as well, which is because that the fibers were not well distributed during mixing. Furthermore, for pervious concrete, fibers could not be fully incorporated or wrapped into the cementitious because of the high porosity and thinner coating material, which could diminish the function of fibers and even has negative influence on the bonding strength. Hence, the mixtures with latex and fiber modified did not show any better improvement than the mixtures only with latex modified.



(a) Compressive strength



(b) Splitting tensile strength

Figure 3.10 Strength results of PCPC mixtures

3.6.2 Cantabro Abrasion Test

In the Cantabro abrasion test, in order to feature the process of impacting loss, the specimens were tested at every 50 revolutions until 300 revolutions were reached. From Figure 3.11, it can be seen that the edges of the cylindrical specimen were smashed off during the first 100 revolutions. As the revolution number increasing, the shape of the specimen became more and more spherical until some damage happened in the major part of the specimen.



(a) Before test



(b) After 50 cycles



(c) After 100 cycles



(d) After 150 cycles



(e) After 200cycles



(f) After 250 cycles

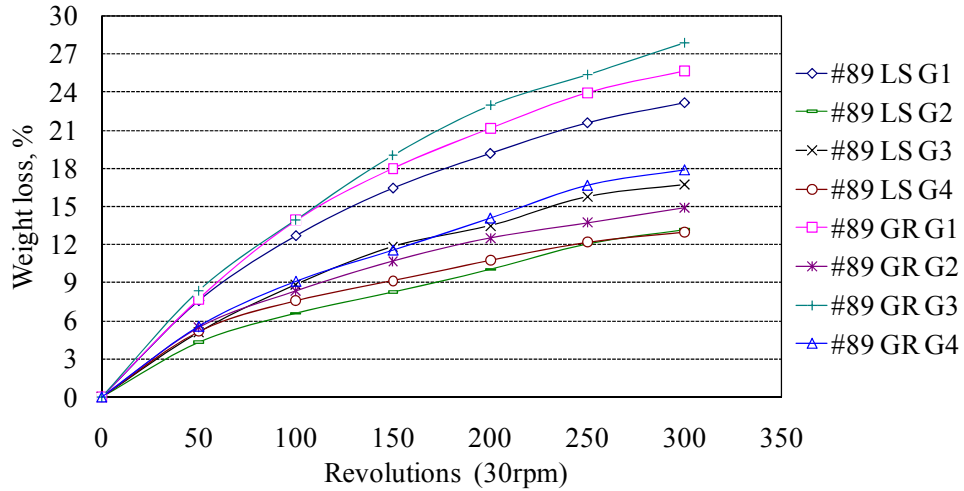


(g) After 300cycles

Figure 3.11 Test Specimen at different cycles in Cantabro abrasion test

Figure 3.12 shows the typical change rate in weight loss of PCPC mixtures with various revolutions. It can be observed that the weight loss of the PCPC mixtures kept increasing with the increase in the revolution number. However, the loss rate slightly

decreased with the increasing revolutions, because the shape of the specimens became more and more spherical and the edge parts have already been broken away from the major part.



* LS-Limestone; GR-Granite; G1-Control; G2-Latex modified; G3-Fiber added; G4-Latex & Fiber modified;

Figure 3.12 Typical growth of weight loss in Cantabro abrasion test (#89 mixtures)

Figure 3.13 presents the final weight loss values after 300 revolutions. It shows that most of the mixtures suffered weight loss from 10% to 35%. The unexpected high weight loss for the control mixture made with #7 limestone aggregate was due to the fact that the specimens were smashed into several pieces at around 200 revolutions during testing and only the big pieces were taken out and weighed. All the remaining small pieces were counted into the weight loss. Therefore, the final weight loss value was much higher than expected. Compared to the control mixtures, the addition of latex reduced the weight loss values dramatically, which demonstrates that latex could significantly improve the abrasion resistance of PCPC mixtures. The reason for the improvement in abrasion resistance could be attributed to the intermingled and interpenetrated matrix structure formed by latex and cement hydration products. The matrix structure of latex and cement

hydration products was much stronger than that of just cement paste and thus made the latex-modified pervious concrete more resistant to abrasion. However, mixtures containing fiber did not show any effect in improving abrasion resistance for the mixtures made with #89 aggregates, while a slight improvement in abrasion resistance was observed for mixtures made with #7 aggregates. Therefore, fiber seemed to have better effect on abrasion resistance of PCPC mixtures with larger grain-size aggregate.

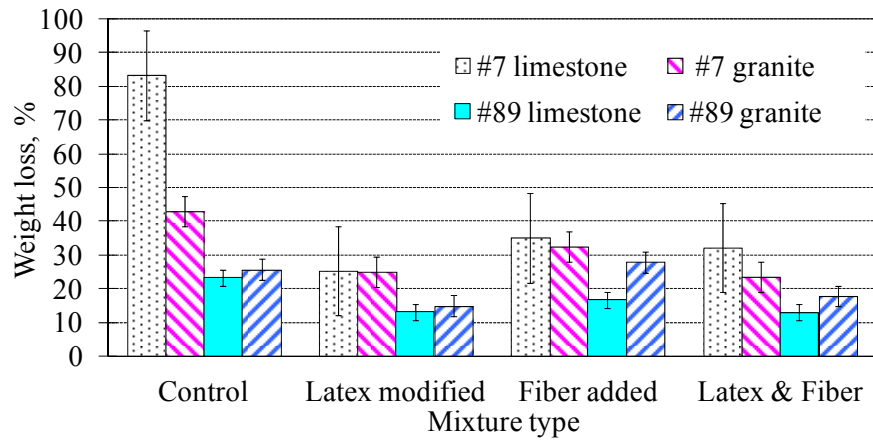
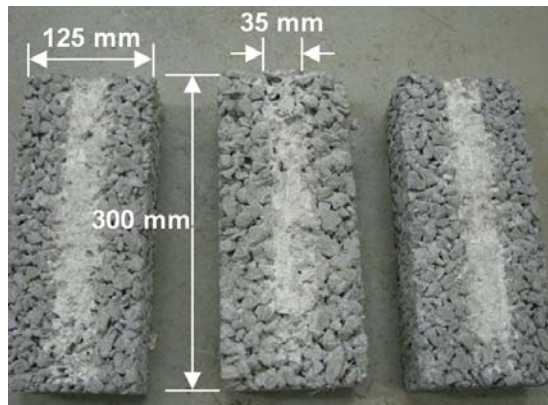


Figure 3.13 Weight loss of Cantabro test after 300 revolutions

3.6.3 LWT Abrasion Test (Loaded Wheel Abrasion Test)

The results from APA abrasion test also indicate that the PCPC mixtures made with smaller grain-size aggregate had higher resistance to abrasion and raveling than those with larger aggregate. Considering both weight loss and wear depth, the most desirable mixtures were those with latex modified, which suffered the lowest weight loss and wear depth among all the mixture groups. From Figure 3.14, the rationale for introducing LWT abrasion test can be manifested by differentiating the effects of PCPC specimens in different mixtures. It can be seen clearly from Figure 3.14b that the specimen with larger grain-size on the left had worse damage than the specimen with smaller grain-size on the

right. Moreover, by the impacting and raveling action of the testing wheels, once a single aggregate get lost on the surface the damage will develop rapidly into a pothole, and eventually a fracture cracking throughout, which also reflects the majority damage situation happened on the actual PCPC pavements.



(a) Specimen after test

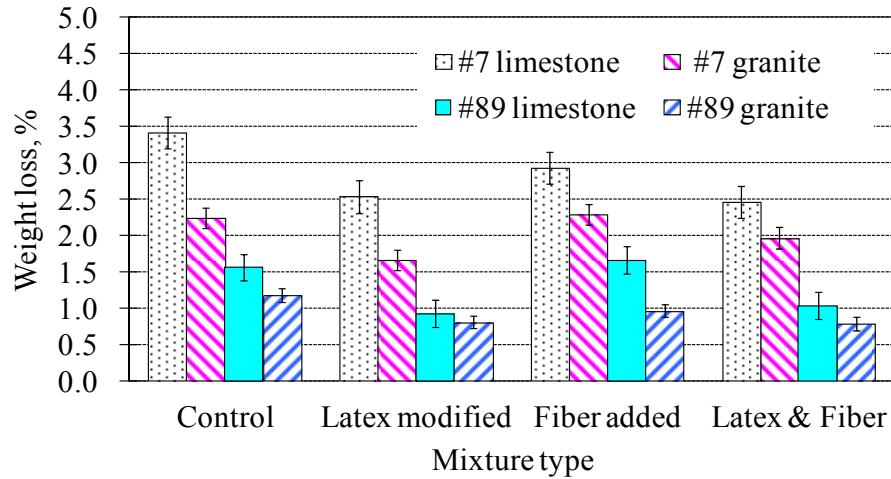


(b) The damage on specimen

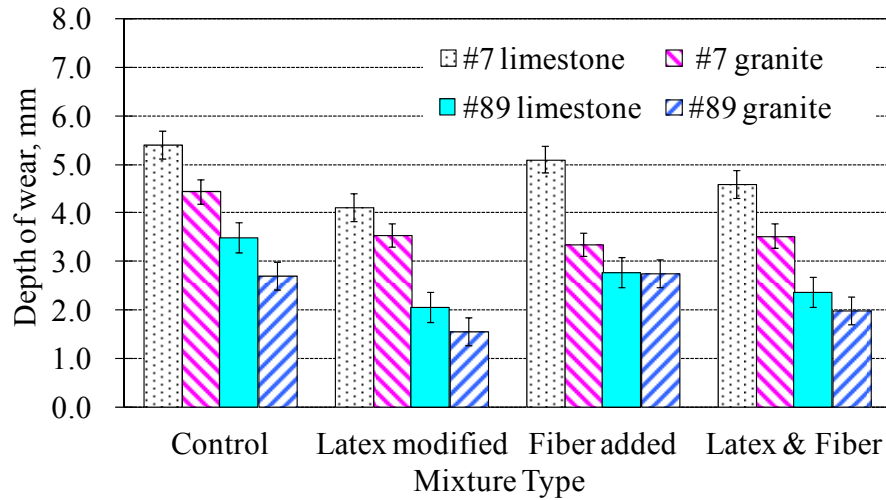
Figure 3.14 Specimens after LWT abrasion test

As shown in Figure 3.15, latex had positive effects on the abrasion resistance of PCPC, which caused a reduction of 25% of the control group based on weight loss as well as the depth of wear. However, the addition of fibers had little or no effect in improving the abrasion resistance, which is consistent with the results from Cantabro

abrasion tests. The reason is that because of the lack of the wrap of cementitious material the fibers could not fully participate in the function of bonding structure to increase abrasion resistance. Accordingly, they rather reduce the contact areas between particles than increase the bonding strength. Similar effects of latex and fiber toward different mixture types can be observed based on weight loss and depth of wear, which indicate that those two parameters are both appropriate for the evaluation of the abrasions resistance of PCPC mixtures and they were exchangeable and mutually authenticated.



(a) Weight loss



(b) Depth of wear

Figure 3.15 Results of LWT abrasion test

3.6.4 Loaded Sweep Abrasion Test

After several trial tests, it was found that in this loaded sweep abrasion tests the rubber hose is too weak to damage the concrete surfaces; only some slight “scratches” could be observed on the surface of the specimen. Unfortunately, no results could be obtained through this test; however the idea is still appropriate and better designs can be done to make it work through more modifications. Compared to the Surface Abrasion Test in ASTM C944, loaded sweep test has bigger abrading area and heavier loads which is adjustable as well. Therefore, for this test only the concept is proposed herein, but for the future study the rubber based hosed could be substituted by some steel rollers or balls.

3.7 Sensitivity and Repeatability

The sensitivities of the three tests were compared in this study. The result ranges and the ratios of the lowest result to that of the control mixture were calculated and compared

for reflecting the sensitivity. Low ratio value and high result range indicate that a test is capable of differentiating samples with different properties; thus, the test method can be considered as a feasible and effective one. The coefficient of variation (COV), the ratio of standard deviation to the mean, is a normalized measure of variation and can be used to compare the variation of data sets with different units or mean values. COVs of the test results were calculated in this study to evaluate the repeatability of these tests. A low COV value represents high repeatability. Table 3.4 presents the comparison of test efficiency, test results value, ratio of lowest result to that of the control mixture and COVs (coefficient of variance) of the three tests.

Table 3.4 Comparison of investigated abrasion tests

Items	Cantabro Abrasion Test	LWT Abrasion Test (Loaded Wheel Abrasion Test)	Loaded Sweep Abrasion Test
Test Efficiency	10min/sample	1.5h/3samples	30min/sample
Load	The weight of specimen (3200-3500g)	890N	98N
Weight Loss	10~80%	0.8~3.5%	0
Overall COV	15%	6%	N/A
Sensitivity	60%	37%	N/A

The average weight loss in Cantabro abrasion tests was much higher than that in LWT abrasion tests because of the relative strong impacting effect on the specimens in Cantabro abrasion tests. From the results of the ratios of the lowest result to that of the

control mixture, Cantabro abrasion test had higher sensitivity than that of LWT abrasion test for measuring weight loss, which indicates weight loss is more efficient to address the abrasion resistance for Cantabro abrasion test. However, the variability of data of Cantabro abrasion tests was relatively large, and while the reliability of LWT abrasion test was better. In LWT abrasion test the three specimens are in the exactly same loading conditions, but it is impossible to control the consistency for each specimen in Cantabro abrasion test.

3.8 Summary and Conclusions

A simple test was introduced to investigate the abrasion resistance of PCPC mixtures utilizing LWT. Two other potential abrasion tests for PCPC, the Cantabro abrasion and the loaded sweep abrasion tests, were also performed. Several PCPC mixtures with different mechanical characteristics were tested to verify the validity of this test. Based on the results, the following can be summarized:

1. LWT abrasion test was effective in evaluating the abrasion resistance of pervious concrete. The results from LWT abrasion tests were in good agreement with those from Cantabro abrasion tests. The results showed that using small grain-size aggregate and/or adding latex and fiber could improve the compressive strength and abrasion resistance of PCPC.
2. LWT abrasion tests were able to identify the differences of abrasion resistance for various PCPC mixtures. The LWT abrasion test had the best repeatability and sufficient sensitivity among all the abrasion tests conducted in the study.

3. Studded steel wheel and high wheel pressure were designed or modified for this test. The two parameters, weight loss and wear depth, could both be used for evaluating the abrasion resistance in the LWT abrasion test.

3.9 Acknowledgement

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CHAPTER 4 CHARACTERIZING VISCOELASTIC PROPERTIES OF ASPHALT MIXTURES UTILIZING LWT

4.1 Introduction

The viscoelastic properties of asphalt mixtures have been the subject of many studies for several decades. Many methods and analysis models have been developed to characterize the viscoelastic response of asphalt mixtures in the pavement (Schapery 1969; Kim 1995). Due to the inherent nature of viscoelastic materials, the fundamental property that determines the response caused by external influence is a function of time or frequency. Commonly, the linear viscoelastic behavior of asphalt mixtures could be investigated through experimental tests within linear viscoelastic region. The common tests currently used include creep test, relaxation test, and complex modulus test.

4.1.1 Creep Test

Due to the challenge of controlling a relaxation test, creep test is more accepted by researchers based on the interchangeability of the results from both tests. The creep test consists of measuring the time dependent strain induced from the application of a constant stress, σ_0 , and the ratio between the strain varies with time and the constant stress, $J(t) = \varepsilon(t)/\sigma_0$, is defined as creep compliance. The creep compliance is a crucial factor for determining the suitability of asphalt mixture under various loading and environmental conditions. Figure 4.1 shows the typical results from creep tests at different temperatures.

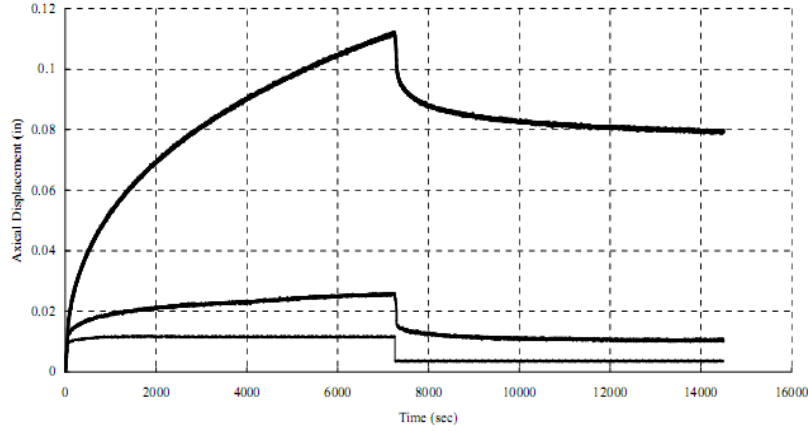


Figure 4.1 Creep test at different temperatures

Moreover, once the creep compliance is determined, the stress-strain relationship can be established according to the hereditary integral (Flügge 1975).

$$\varepsilon(t) = \sigma_i J(t) + \int_0^t J(t-t') \frac{\partial \sigma'}{\partial t'} dt' \quad (4.1)$$

where, $\varepsilon(t)$ =strain varies with time; σ_i =initial stress; t' =integration variable related to time.

4.1.2 Complex Modulus Test

Complex modulus test is a fundamental test that characterizes the dynamic viscoelastic properties of asphalt mixtures. It is also considered a mechanistically based laboratory test to characterize the stiffness and loading resistance of asphalt mixtures. Stress-strain relationship under a continuous sinusoidal loading for linear viscoelastic materials is defined by a complex number called complex modulus, E^* , and the absolute value of the complex modulus, $|E^*|$, is defined as the dynamic modulus. Dynamic modulus which is related to the stiffness is a fundamental property for describing the deformation response of asphalt mixtures under cyclic loading. Most of the research

findings indicate that any process results in the use of asphalt mixtures with better performed dynamic modulus will improve the overall performance of the pavement. The phase angle, φ , which related to the time lag between stress and strain is a major factor reflecting the viscous behavior of asphalt mixtures which indicates whether the asphalt material is predominantly elastic or viscous.

$$E^* = \frac{\sigma_{amp} e^{i\omega t}}{\varepsilon_{amp} e^{i(\omega t - \delta)}} \quad (4.2)$$

$$|E^*| = \frac{\sigma_{amp}}{\varepsilon_{amp}} \quad (4.3)$$

$$\varphi = 2\pi \cdot f \cdot \Delta t \quad (4.4)$$

where, σ_{amp} =the amplitude of sinusoidal stress; ε_{amp} =the amplitude of sinusoidal strain; ω =angular velocity; i =imaginary component; f =loading frequency; Δt =the time lag between stress and strain.

Dynamic modulus values measured over a range of temperatures and frequencies of loading can be shifted into a master curve based on time-temperature principle. The master curve of an asphalt mixture allows comparisons to be made over extended ranges of frequencies and temperatures, and thus dynamic modulus can be used as viscoelastic parameters for the performance analysis of asphalt mixtures through constitutive models (Schapery 1984; Kim and Little 1990; Park et al. 1996). A typical mater curve for dynamic modulus at three different temperatures is shown in Figure 4.2.

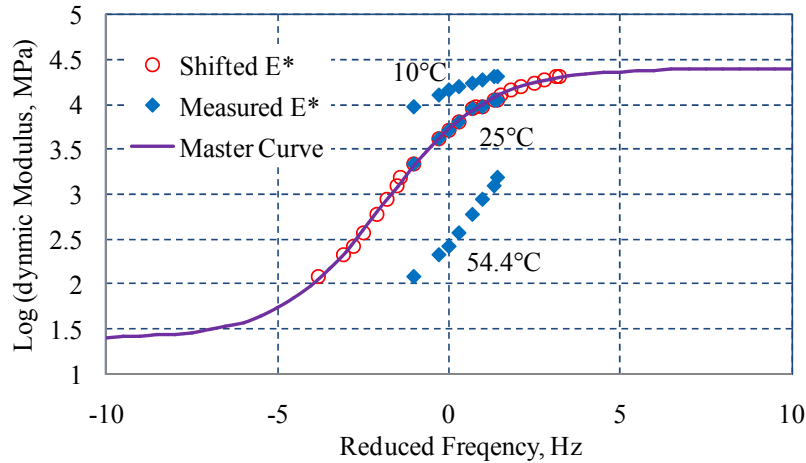


Figure 4.2 Typical master curve for dynamic modulus

4.2 Viscoelastic Experimental methods for Asphalt mixtures

According to the stress and strain situation in the real pavement, the fundamental behavior of asphalt mixtures in the surface layer of the pavement system is primarily determined by its tensile properties. When a vehicle load is acting on the pavement, the load-associated fatigue cracking of the asphalt mixture under the wheel path always initiates at the bottom of the asphalt layer and propagate to the surface. The load distribution in the surface layer of the pavement system is shown in Figure 4.3.

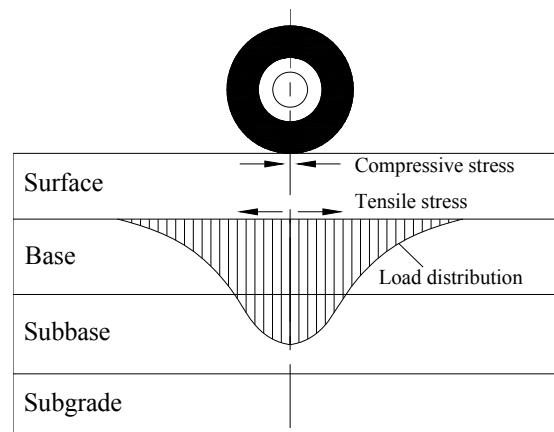


Figure 4.3 Loading distribution of the surface layer in pavement system

Although many factors have been proved that influence the viscoelastic behaviors of asphalt material, such as loading magnitude, rate of loading (loading frequency), and temperature variations, there are only a few direct evidences or relative works for studying the effect of the loading mode. The dynamic modulus and phase angle of asphalt mixtures are investigated by Kallas (1970) in three different loading modes, which are tension, tension-compression and compression. The results show that the differences of dynamic modulus caused by the type of loading conditions are insignificant when the testing temperatures are in the range from 4.4 to 21.1°C (40-70°F). However, they are remarkable when the testing temperatures are higher than 21.1°C (70°F). The phase angle results in tension are 20% higher than those in tension-compression mode, and are 50% higher than those in compression mode. In compression mode, the loads are more likely to be supported by the aggregate structures of asphalt mixtures, while in the tension situation the asphalt bonding strength plays more roles in resisting loads. Through test conditions differing from the actual states errors and unreasonable design may be caused. Therefore, it is more appropriate for testing the viscoelastic properties of asphalt mixtures in tension condition.

Although the specimen under all kinds of stress states such as uniaxial, biaxial, and triaxial can be tested in the laboratory, the real stress state that asphalt mixtures suffered in the pavement is too difficult to be simulated. Some studies have been successfully completed through full-scale or large-scale experiments, but either they are expensive or time consuming or have low repeatability and operability. Testing method and equipment are still the bottlenecks for acquiring better and clearer understanding of the properties of asphalt mixtures.

In order to better understand the viscoelastic properties of asphalt mixtures, a simple test utilizing LWT is proposed in this study. The schematic diagrams for the viscoelastic tests in this study are shown in Figure 4.4.

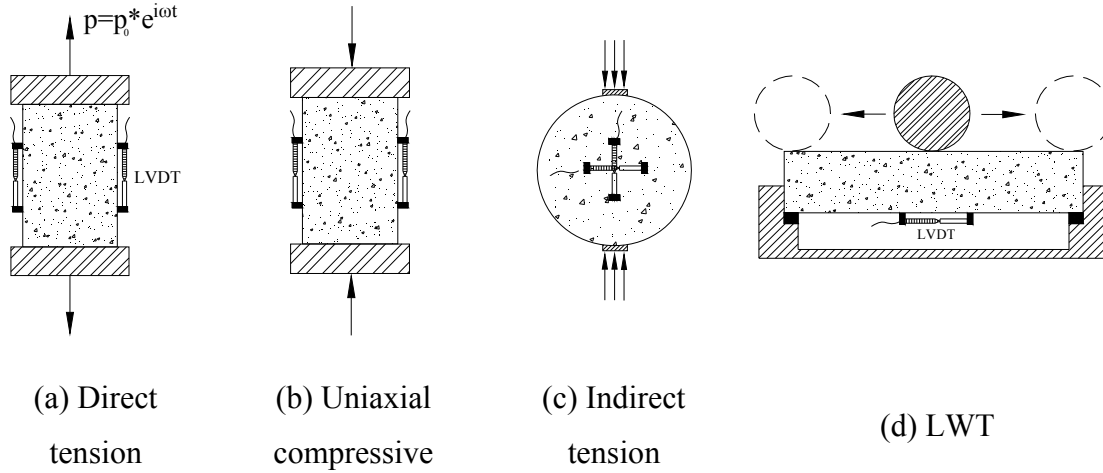


Figure 4.4 Viscoelastic tests for asphalt mixtures in this study

4.2.1 Direct Tension Test

Direct tension test (DTT) as shown in Figure 4.4a has been created for testing viscoelastic properties of asphalt mixtures for many years (Epps and Monismith 1970; Pavlovich and Goetz 1976; Bolzan and Huber 1993). The unique benefit for DTT test is that in the test the stress state of the specimen is in uniaxial tension condition, which makes the stress-strain relationship much simpler and clearer to be analyzed. Commonly, in the direct tension test, two ends of the specimen need to be adhered to the loading grips by epoxy resin. Although some successful tests have been achieved, uniformed tensile stress in the specimen is still difficult to be guaranteed, and the additional stresses induced by the loading grips on both ends are impossible to be avoided. In addition, firm and full contacts between specimen and loading grips are critical for a successful test (failures near grips may happen due to stress concentrations), otherwise big errors will be

aroused. The operability and repeatability are two critical issues need to be improved for direct tension test (Bolzan and Huber 1993).

4.2.2 Uniaxial Compressive Test

Due to the complexity of conducting tests in tension condition, compressive tests (as shown in Figure 4.4b) have been widely used to substitute direct tension tests. Extensive studies for investigating the deformation resistance of asphalt mixtures have been conducted through uniaxial compressive tests (Hills 1973; Van de Loo 1974, 1976). Currently, the most reliable and extensively used uniaxial compressive test for characterizing viscoelastic behaviors of asphalt mixtures is the simple performance test (SPT) which is also called asphalt mixture performance test (AMPT). The testing device developed by IPC Global Company is shown in the Figure 4.5.



Figure 4.5 Simple performance test (SPT)

In the SPT test, a cylindrical specimen is subjected to a controlled sinusoidal compressive stress loading with various frequencies at different testing temperatures. The vertical deformations can be measured through the LVDTs mounted on the specimen (NCHRP Report 465 2002; ASTM D 3496; Witczak et al., 2002). In terms of the applied stress and measured strain, dynamic modulus and phase angle can be calculated.

4.2.3 Indirect Tension Test

Although compressive test is considered a practical substitution for direct tension test, the viscoelastic behaviors of asphalt mixtures in tension are still pursued by researchers. Indirect tension test (IDT) has been developed during the Strategic Highway Research Program (SHRP) to characterize the Poisson's ratio, creep compliance, resilient modulus and splitting tensile strength of asphalt mixtures. As shown in Figure 4.4c, by subjecting a compressive stress on diametrical direction of the cylindrical specimen, a closely uniform tensile stress is created in the center of the specimen. Based on the benefit of operability, IDT test is currently the most accepted test in evaluating the properties of asphalt mixtures in tension condition (Christensen and Bonaquist 2004). A test protocol was developed for evaluating creep and strength properties of asphalt mixtures in indirect tension mode by Buttlar and Roque (1992). Kim et al. (2004) presents an analysis by carrying out complex modulus test in IDT mode. The results indicate that the dynamic modulus and phase angle results obtained from IDT tests are generally in accordance with the results from uniaxial compressive tests.

However, compared to the simple uniaxial stress that the specimens suffered in DTT and SPT tests, the specimen in IDT test is in a much more complex biaxial stress state, thus the calculations for post-processing are troublesome. The stress solutions commonly used for IDT tests were developed based on three dimensional finite element analyses by Roque and Buttlar (1992).

4.2.4 LWT Viscoelastic test (Loaded Wheel Viscoelastic Test)

According to the actual stress and strain situation in the real pavement, a flexural tension viscoelastic test, which is named as LWT viscoelastic test herein, is proposed in

this study (Figure 4.4d). LWT is the most common laboratory equipment to test the permanent deformation and moisture damage of asphalt mixtures which characterizes movable wheel loads corresponding to the vehicle loads in the actual field, and thus it has the potential of providing cyclic loading to the specimens. In this study, a LWT based on Asphalt Pavement Analyzer (APA) platform was employed as the loading system.

Compared to DTT and IDT tests, the loading mode, stress situation and strain response of the specimen in LWT viscoelastic test are more similar to those in the actual pavement. The cylindrical specimen for DTT and IDT tests need to be cored or trimmed from the original specimen compacted by gyratory compactor. Those processes are practically skillful, and the change of the air voids in specimens caused by those processes is also too difficult to be controlled.

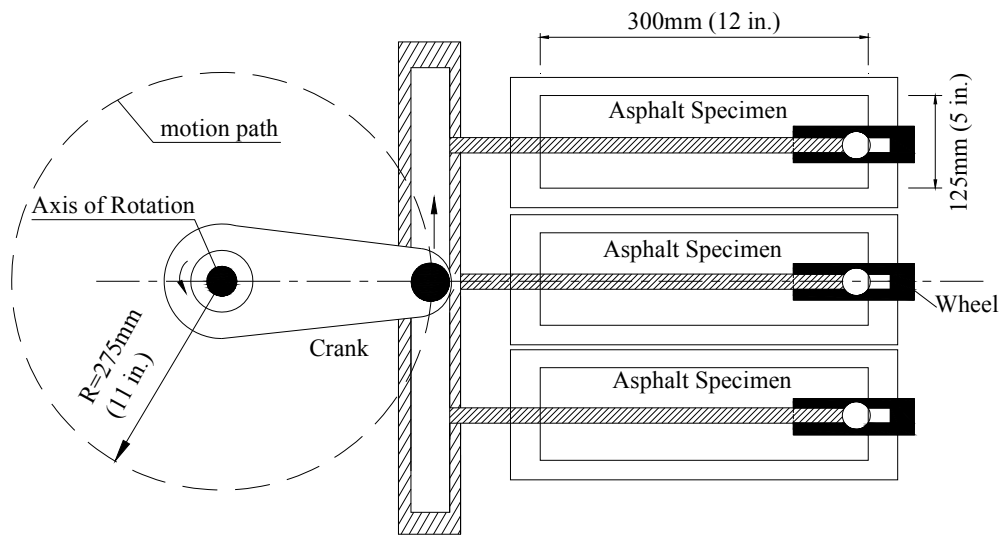
4.3 Objectives and Scope

The main purpose of this study is to develop an effective test to characterize the viscoelastic properties of asphalt mixtures utilizing LWT. In addition to the LWT viscoelastic test, uniaxial tests in compression, tension-compression and tension modes, and the indirect tension creep test were conducted to the same asphalt mixtures in this study. Four different asphalt mixtures with two types of aggregates (limestone and granite), three grades of asphalt binder (PG 64-22, PG 70-22 and PG 76-22) were considered.

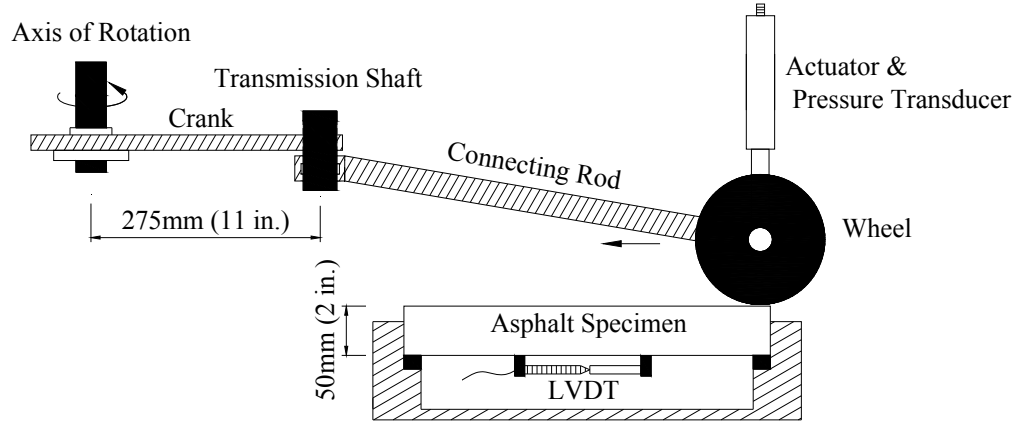
4.4 Loaded Wheel Testing System of APA

4.4.1 Mechanical Analysis

In order to understand the loading pattern on the specimen provided by APA loading system, the mechanical analysis was carried out. In APA system, the velocity of the loading wheels or the frequency of the cyclic loading can be adjusted by specifying the angular frequency of the axis of rotation. The axis of rotation drives the crank to do a circular motion, and the transmission shaft will be forced to move back and forth with the loading wheels attached. The mechanical structure of APA loading system is shown in Figure 4.6.



(a) Top view



(b) Side view

Figure 4.6 Schematic diagram of APA loading system

The movement equation for the loading wheels can be expressed as the following formula according to Figure 4.7.

$$S = R \cdot (1 - \cos \omega t), \quad \omega = \frac{2\pi}{T_r} \quad (4.5)$$

where, S =the distance of movement of the wheels; R =the radius of the rotation of crank (also equals to 1/2 path length of the loading wheels); ω =angular velocity of the rotation axis; T_r =the rotation period of the circular motion of the crank.

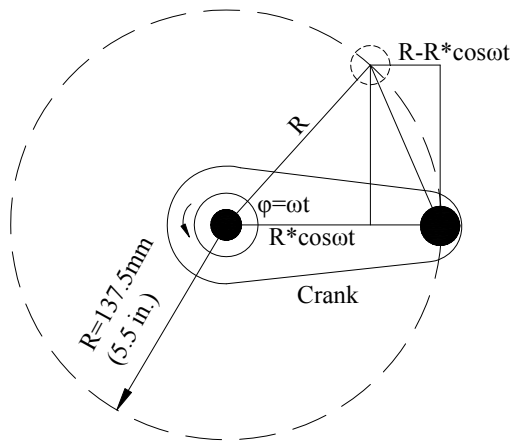


Figure 4.7 Establishment of movement equation

The typical movement pattern of the loading wheels when $T_r = 1$ is shown in Figure 4.8.

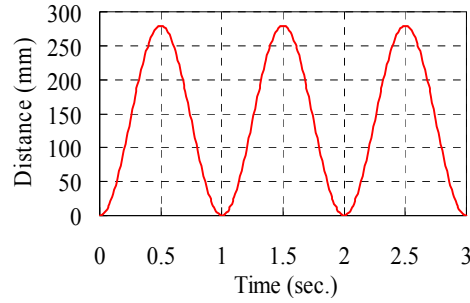


Figure 4.8 Movement function of APA Loading System ($T_r = 1$)

Based on the movement equation, the movement of the load on the specimen can be regards as a simple harmonic motion. According to theory of plane-stress, the calculation model can be simplified as below.

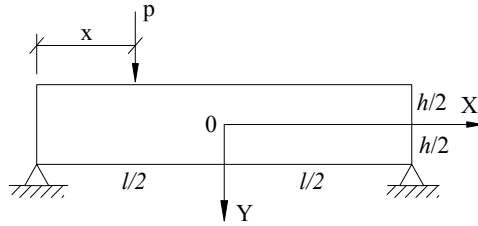


Figure 4.9 Simplified calculation model for stress analysis

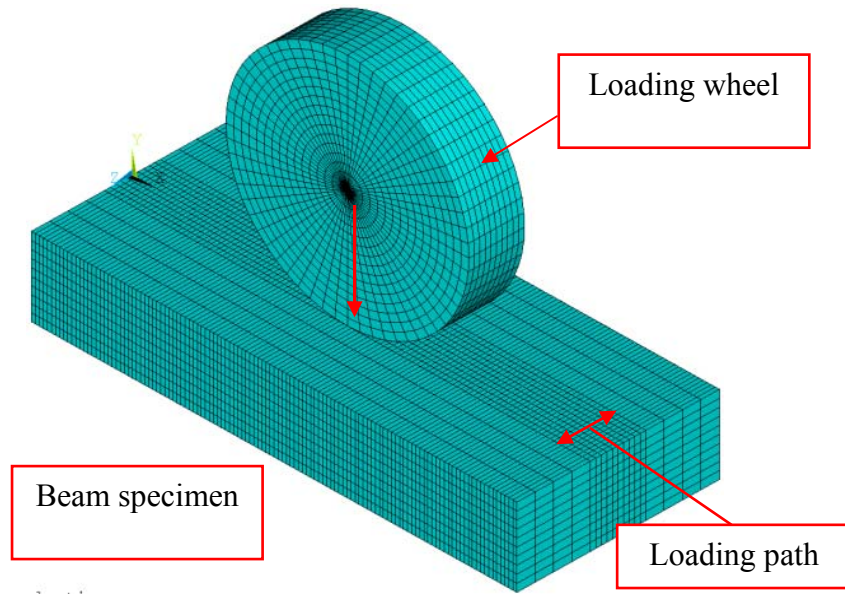
For a beam specimen subjected to a continuous sinusoidal load in the LWT test, the stress distribution along its bottom surface can be expressed as the following formulas with respect to the distance and time.

$$\left\{ \begin{array}{ll} \sigma(x,t) = \frac{3P \cdot \sin^2\left(\frac{2\pi}{T} \cdot t\right) \cdot x}{bh^2}, & x \leq \frac{l}{2} \\ \sigma(x,t) = \frac{3P \cdot \sin^2\left(\frac{2\pi}{T} \cdot t\right) \cdot (l-x)}{bh^2}, & \frac{l}{2} < x \leq l \end{array} \right. \quad (4.6)$$

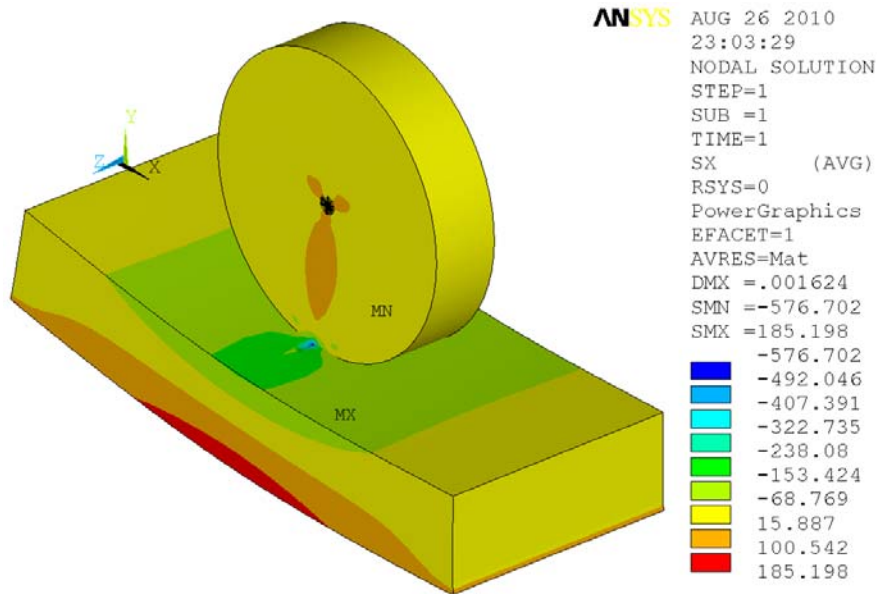
where, P = the wheel load; l =the length of the wheel loading path; b =the width of specimen; h =the height of specimen; T =testing period; t =elapsed testing time.

4.4.2 Stress Analysis

In terms of the theory of mechanics, the error caused by the assumption of plane-stress could be negligible if the width of the wheel equals to the width of the beam specimen. However, the width of the wheel of APA is only about 1/3 of the width of the beam specimen, and some error must be aroused through the simplified calculation. In order to appraise the errors caused by the simplification from 3-Dimension (3-D) to 2-Dimension (2-D) plane-stress, Finite Element Method (FEM) was adopted to analyze the stress of the beam specimen under cyclic wheel loading. The FE model of the specimen with loading wheel is shown in Figure 4.10. The element meshes on the wheel loading path area have been fined to improve the calculation accuracy.



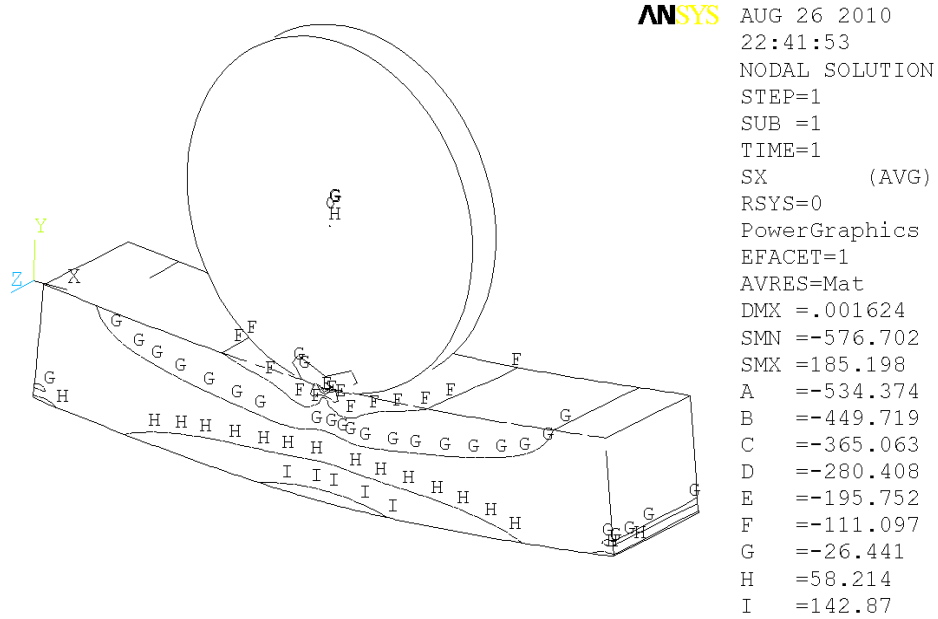
(a) 3-D FE structural model



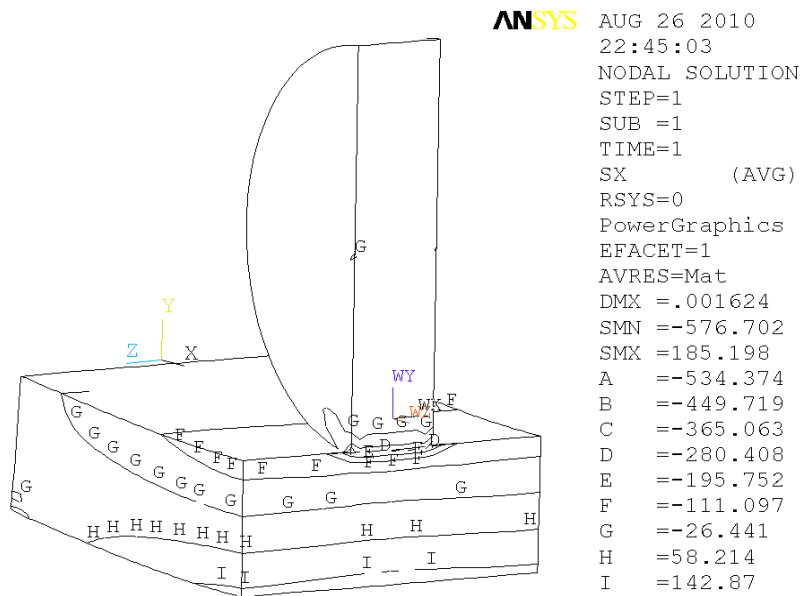
(b) Stress contour (unit: psi)

Figure 4.10 Finite element model and stress distribution

During the calculation, vertical nodal forces were placed on the nodes along the central axis of the wheel. Figure 4.10b shows the stress contour when the beam specimen is subjected to a wheel load in the midspan. The ISO-stress lines on the cross sections of the beam specimen under the wheel load are illustrated in Figure 4.11, the wheel load used for this calculation is 889 N (200 lb), and the unit of the stresses presented in the graphs is psi (1psi=6.895kPa).



(a) Longitudinal section (X-Y plane)



(b) Transverse section (Y-Z plane)

Figure 4.11 ISO-stress lines on cross sections of the specimen (unit: psi)

From the stress distributions shown in the finite element results, it is clear that the stresses distribute as uniformly as a simple supported 2-D plane-stress beam under

concentrated force, except for those on the contact area between loading wheel and beam specimen. In the case shown above, the maximum tensile stress was on the bottom and midspan of the beam. According to Figure 4.11, the value of the maximum tensile stress is 1.28MPa (185.2psi) from 3-D FEM calculation, while it is 1.24MPa (180.0psi) from the calculation of 2-D beam. The error of the stress amplitude caused by the simplification from 3-D to 2-D is about 3%, which is generally considered within the engineering tolerance.

The three normal stresses on the bottom and midspan of the specimen within three loading cycles are shown in Figure 4.12. The major stress the beam suffered from the bending moment is the normal stress in X direction (longitudinal direction), SX, which is four times more than the normal stress in Z direction (transverse direction), SZ. By comparing to SX and SZ, the normal stress in y direction (vertical direction), SY, is negligible. In addition, it can be seen obviously that only small error exists between the SX results of FEM and 2-D plane-stress calculations.

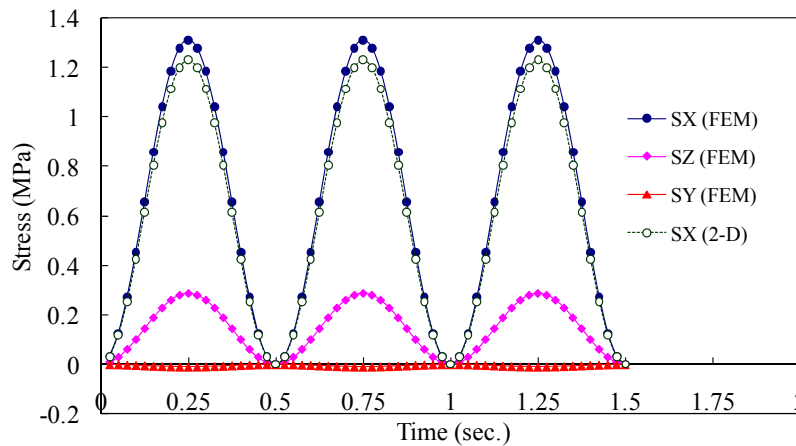


Figure 4.12 Normal stresses on the bottom and midspan of the specimen (3 cycles)

4.4.3 Modeling Viscoelastic Properties

There are two general methods to characterize the viscoelastic behavior of asphalt mixtures, one is through mechanical models and the other one is through creep compliance or relaxation modulus (Huang 1994). In terms of viscoelasticity theory, it is known that all viscoelastic response functions are interrelated, which means they can be determined either from experimental testing conducted in viscoelastic range or through formula transformations from other known response functions. From the static creep test, creep compliance can be determined as a function of time, while from the complex modulus test, dynamic modulus and phase angle can be determined as a function of frequency. Compared to obtaining relaxation modulus $E(t)$ from relaxation tests, creep compliance, $D(t)$, and dynamic modulus, $|E^*|$, from creep tests and dynamic modulus tests are much easier to be achieved (Kim 2009).

Various function forms have been used to represent the viscoelastic response obtained from experiments. Although those representations are rough and simplistic, most of them are efficient for characterizing viscoelastic parameters (Park et al. 1996). Power Law and Prony Series are generally used ones in fitting the functions of viscoelastic response.

1. Generalized Power Law (GPL):

$$D(t) = D_g + D_1 t^n \quad (4.7)$$

where, D_g is the glassy compliance, $D_g = \lim_{t \rightarrow 0} D(t)$.

2. Modified Power Law (MPL) (Williams 1964):

$$D(t) = D_g + \frac{D_e - D_g}{\left(1 + \frac{\tau}{t}\right)^n} \quad (4.8)$$

where, the constant D_e is the long-time equilibrium or rubbery compliance which is defined by $D_e = \lim_{t \rightarrow \infty} D(t)$.

3. Prony Series

Prony series consisting of a sequence of decaying exponentials has been widely used to represent viscoelastic response (Schapery 1961; Tschoegl 1989). The popularity of this series representation is attributed mainly to its ability of describing a wide range of viscoelastic response. The linear viscoelastic (LVE) response expressed by Prony series has a physical basis in the theory of mechanical models containing linear springs and dashpots (Park et al. 1996). Creep compliance and relaxation modulus can be expressed as Prony series through following forms, respectively:

$$\text{Creep compliance: } D(t) = D_0 + \sum_{m=1}^M D_m \left(1 - e^{-\frac{t}{\tau_m}}\right) \quad (4.9)$$

$$\text{Relaxation modulus: } E(t) = E_\infty + \sum_{m=1}^M E_m e^{-\frac{t}{\rho_m}} \quad (4.10)$$

where, τ_m =retardation time; D_m =regression coefficient; D_0 =glassy compliance; E_∞ =long time equilibrium modulus; E_m =regression coefficient, and ρ_m =Prony Series regression coefficients.

Based on the expression of creep compliance and relaxation modulus by Prony series, the viscoelastic numerical solutions for the loading system in this study can be derived as follows.

According to the following hereditary integral equations and the expression of creep compliance as a Prony series:

$$\varepsilon(t) = \sigma_0 D(t) + \Delta \sigma' \cdot D(t - t') \quad (4.11)$$

$$\varepsilon(t) = \sigma_0 D(t) + \int_0^t D(t - t') \frac{\partial \sigma'}{\partial t'} dt' \quad (4.12)$$

$$D(t) = D_0 + \sum_{i=1}^I D_i \left(1 - e^{-t/\tau_i}\right) \quad (4.13)$$

In dynamic modulus test, continuous sinusoidal loads applied by APA loading system induce sinusoidal strains with a time lag in terms of the phase angle. The cyclic loading can be expressed as: $\sigma(t) = \sigma_m \cdot \sin^2(\omega t)$; the strain response can be achieved through the following derivations:

$$\begin{aligned} \varepsilon(t) &= \sigma_0 D(t) + \int_0^t \left(D_0 + \sum_{i=1}^I D_i \left(1 - e^{-(t-t')/\tau_i}\right) \right) \frac{\partial \sigma'}{\partial t'} dt' \\ &= \sigma_0 D(t) + \int_0^t \left(D_0 + \sum_{i=1}^I D_i \left(1 - e^{-(t-t')/\tau_i}\right) \right) [2\sigma_m \sin(\omega t') \cos(\omega t') \omega] dt' \\ &= \sigma_0 D(t) + \frac{1}{2} D_0 \sigma_m (1 - \cos(2\omega t)) + \sigma_m \cdot \int_0^t \left(\sum_{i=1}^I D_i \left(1 - e^{-(t-t')/\tau_i}\right) \cdot \sin(2\omega t') \cdot \omega \right) dt' \\ &= \sigma_0 D(t) + \frac{1}{2} D_0 \sigma_m (1 - \cos(2\omega t)) + \frac{1}{2} \sum_{i=1}^I D_i \cdot \sigma_m (1 - \cos(2\omega t)) \\ &\quad - \sigma_m \int_0^t \left(\sum_{i=1}^I D_i \left(e^{-(t-t')/\tau_i}\right) \cdot \sin(2\omega t') \cdot \omega \right) dt' \\ &= \sigma_0 D(t) + \frac{1}{2} \sum_{i=0}^I D_i \cdot \sigma_m (1 - \cos(2\omega t)) - \sigma_m \int_0^t \left(\sum_{i=1}^I D_i \left(e^{-(t-t')/\tau_i}\right) \cdot \sin(2\omega t') \cdot \omega \right) dt' \end{aligned} \quad (4.14)$$

Assume that:

$$\begin{aligned}
\Phi &= \int_0^t \left(e^{-(t-t')/\tau_i} \cdot \sin(2\omega t') \omega \right) dt' \\
&= \frac{1}{2} \left(e^{-t/\tau_i} - \cos(2\omega t) \right) + \frac{1}{2\tau_i} \int_0^t \cos(2\omega t') \cdot e^{-(t-t')/\tau_i} dt' \\
&= \frac{1}{2} \left(e^{-t/\tau_i} - \cos(2\omega t) \right) + \frac{1}{4\omega\tau_i} \sin(2\omega t) - \frac{1}{4\omega\tau_i^2} \int_0^t \sin(2\omega t') \cdot e^{-(t-t')/\tau_i} dt' \\
&= \frac{1}{2} \left(e^{-t/\tau_i} - \cos(2\omega t) \right) + \frac{1}{4\omega\tau_i} \sin(2\omega t) - \frac{1}{4\omega\tau_i^2} \cdot \frac{\Phi}{\omega}
\end{aligned} \tag{4.15}$$

The viscoelastic solution for the strain response in the LWT test can be written as the following formulas:

$$\begin{cases} \Phi = \frac{2\omega^2\tau_i^2(e^{-t/\tau_i} - \cos(2\omega t)) + \omega\tau_i \sin(2\omega t)}{4\omega^2\tau_i^2 + 1} \\ \varepsilon(t) = \sigma_0 D(t) + \frac{1}{2} D_0 \sigma_m (1 - \cos(2\omega t)) + \frac{1}{2} \sum_{i=1}^l D_i \cdot \sigma_m (1 - \cos(2\omega t)) - \sigma_m \cdot \sum_{i=1}^l (D_i \cdot \Phi) \end{cases} \tag{4.16}$$

Based on the viscoelastic solutions derived above, the strain response induced by the cyclic stress could be regarded as the deformation from loading and unloading accompanied by a continuously increased creep deformation. The tensile stress on the bottom surface at midspan of the specimen, σ_0 , with respect to the elapsed-time of cyclic loading can be written as:

$$\sigma_0 = \sigma_{amp} \cdot \sin^2\left(\frac{2\pi}{T} \cdot t\right) \tag{4.17}$$

where, σ_{amp} = the amplitude of sinusoidal stress; T = the testing period (cycle/sec.).

Because one cycle of the loading wheels from one end of the beam specimen to the other end leads to two identical cycles for the stress, thus the actual loading frequency of

stress is as twice as the frequency of the movement of loading wheels ($T=1/2Tr$). The typical sinusoidal stress provided by the APA loading wheels is shown in Figure 4.13.

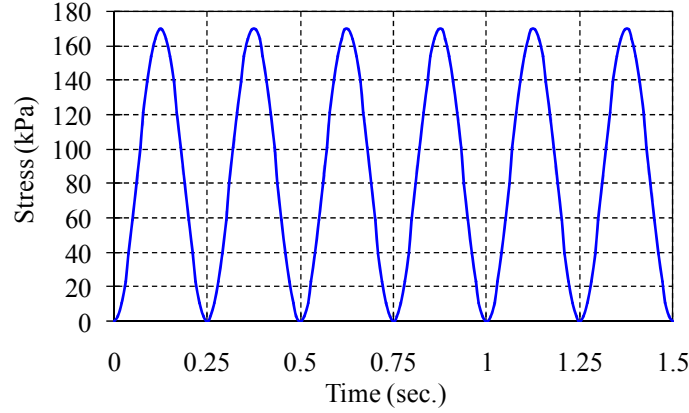


Figure 4.13 Typical sinusoidal stress induced by loading wheel in APA ($T=0.5s$)

According to the stress solution, the dynamic modulus can be calculated as:

$$|E^*| = \frac{\sigma_{amp}}{\varepsilon_{amp}} = \frac{3l \cdot P}{2bh^2 \cdot \varepsilon_{amp}} \quad (4.18)$$

where, σ_{amp} = the amplitude of sinusoidal stress; ε_{amp} = the amplitude of measured strain.

For the viscoelastic materials subjected to continuous sinusoidal stress, the relationship between stress and strain in complex form can be written as:

$$\varepsilon^*(t) = \frac{\sigma^*(t)}{E^*} = \frac{3l \cdot p_0 \cdot e^{i\omega t}}{2bh^2 \cdot |E^*| e^{i\varphi}} = \frac{3p_0 l}{2bh^2 |E^*|} \cdot e^{i(\omega t - \varphi)} \quad (4.19)$$

In creep test, when a constant load is applied in the mid-span of the beam specimen, the stress on the bottom surface is: $\sigma_0 = \frac{3Pl}{2bh^2}$. Thus, the creep compliance can be expressed as:

$$D(t) = \frac{\varepsilon(t)}{\sigma_0} = \frac{2 \cdot bh^2 \cdot \Delta H(t)}{3 \cdot P \cdot l \cdot GL} \quad (4.20)$$

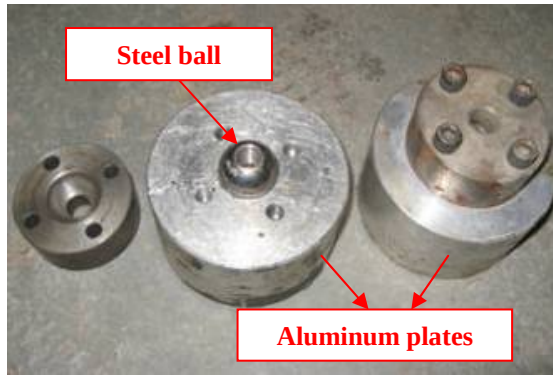
where, $\varepsilon(t)$ =the strain with time change, σ_0 =the constant stress; $\Delta H(t)$ =horizontal deformation with time change; GL =gage length of the extensometer or strain gage; P =the wheel loading; l =the length of the loading path; b =the width of specimen; h =the height of specimen.

4.5 Laboratory Experiments

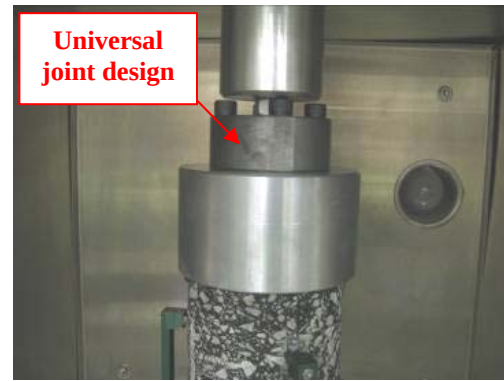
4.5.1 Uniaxial Viscoelastic Test

Three types of loading conditions, compression, compression-tension, and tension were considered in uniaxial viscoelastic tests. The typical testing setups for the uniaxial test are presented in Figure 4.14. During the test, constant loads for creep tests and sinusoidal loads for dynamic modulus tests were supplied by a MTS loading system, while the testing temperature can be controlled through an environmental chamber. Extensometers were mounted on the specimens to measure the axial deformations. Dynamic modulus tests were conducted at three temperatures, 10, 25, and 40°C, with nine frequencies, 25, 20, 10, 5, 2, 1, 0.5, 0.2 and 0.1 Hz. Creep tests were conducted at 10 and 40°C.

Prior to testing, two aluminum plates were adhered to the ends of the specimen by epoxy putty. Universal joints were designed for reducing the effect of eccentricity existed in aligning the position of loading rods (Figure 4.14a, b). Mounting frame shown in Figure 4.14c was used to ensure that the specimen and the aluminum plates were installed straightly in vertical direction. For the purpose of providing full contacts on the specimen, firm bonding between the epoxy putty, aluminum plates and specimen must be guaranteed.



(a) Universal joint



(b) Loading connection



(c) Specimen and loading plates alignment



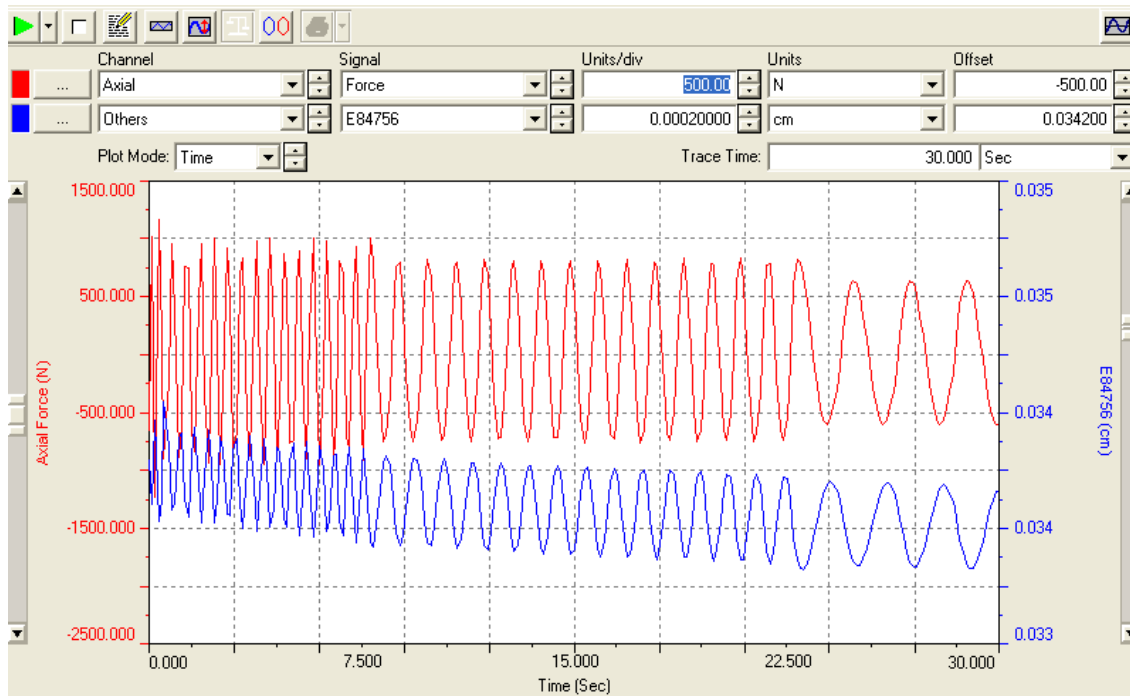
(d) Specimen in the test

Figure 4.14 Uniaxial viscoelastic test setup

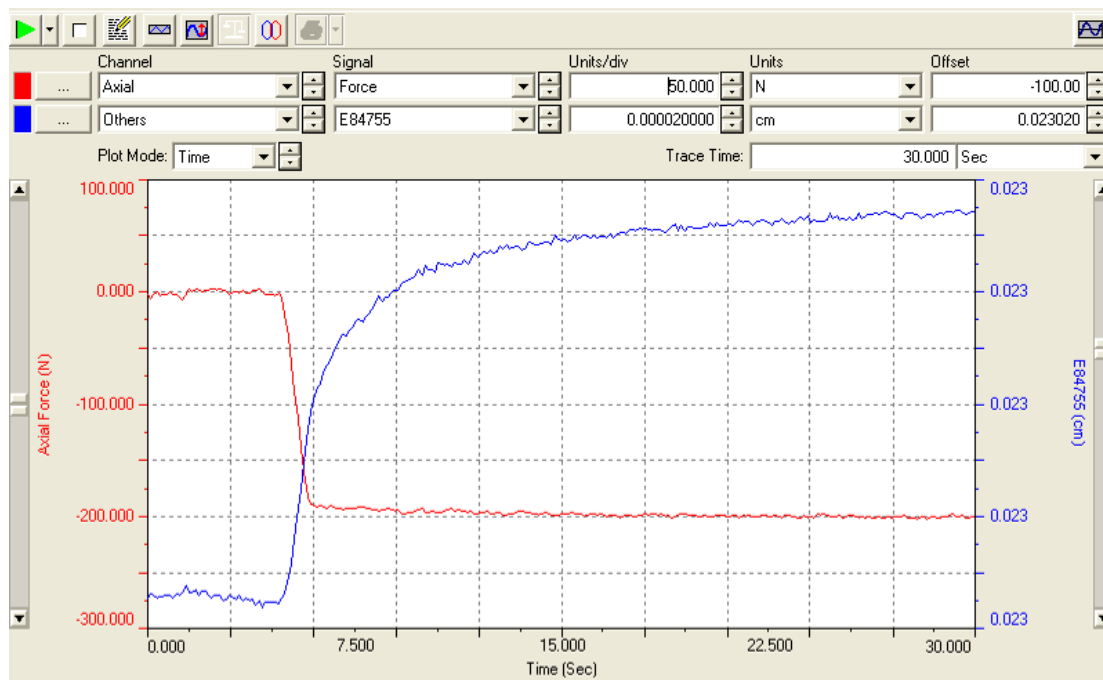


Figure 4.15 MTS loading system with environmental chamber

In the uniaxial dynamic modulus test, in order to maintain the strain response within the range of linear viscoelastic, different stress amplitudes were chosen in terms of temperature, frequency, as well as loading mode. Asphalt mixtures with a micro-strain level lower than 200 were considered in linear viscoelastic situation. During the test, the stress was applied at each frequency until steady strain response has achieved. The data of 10 cycles were collected after 10 pre-loading cycles. Between each test, 30 minutes rest period was allowed for the specimen's recovery. For the uniaxial creep test, the specimen was subjected to a static axial load in both tension and compression modes. The typical raw data from uniaxial viscoelastic tests are shown in Figure 4.16.



(a) Dynamic modulus test

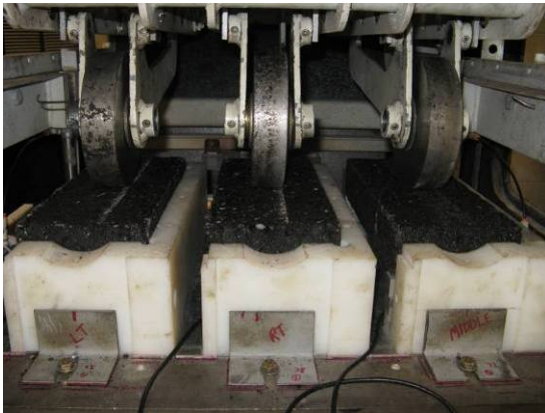


(b) Static creep test

Figure 4.16 Typical recording in uniaxial viscoelastic test

4.5.2 LWT Viscoelastic Test

In LWT viscoelastic test, triplicate beam specimens were subjected to the cyclic loads supplied by the moving wheels. The tensile deformation of the beams are measured by LVDTs mounted on the bottom surface and the midspan of the specimens, as show in Figure 4.17a. During the test, a high range LVDT was connected to the loading arm to record the movement of wheels (Figure 4.17d).



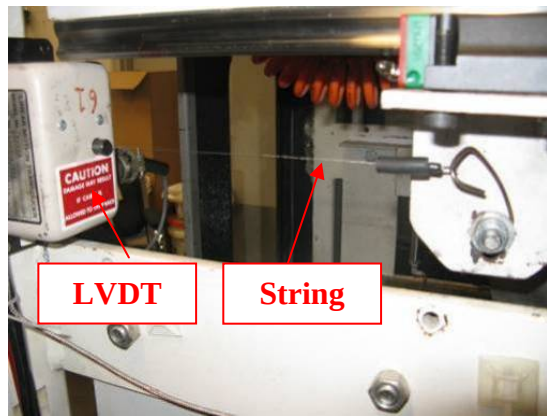
(a) Movable wheel loading system



(b) Specimen with LVDT



(c) Loaded wheel viscoelastic test setup

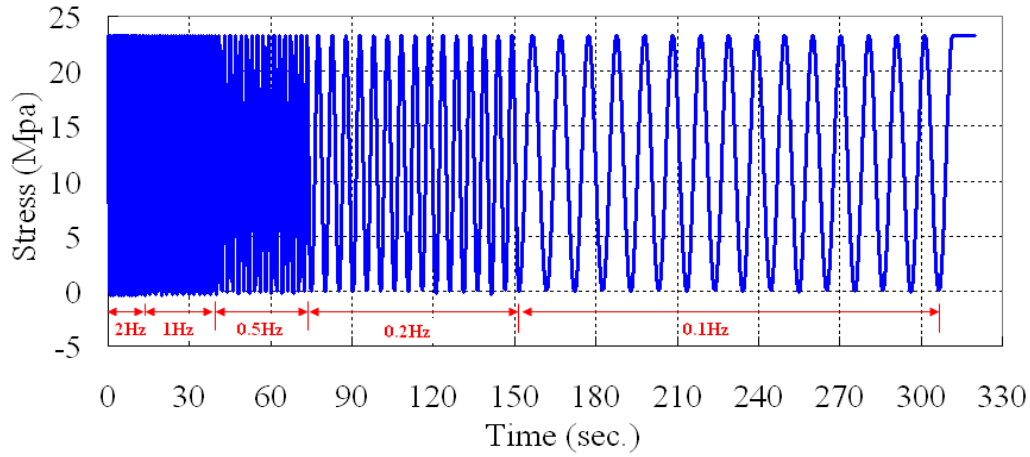


(d) Wheel movement measuring

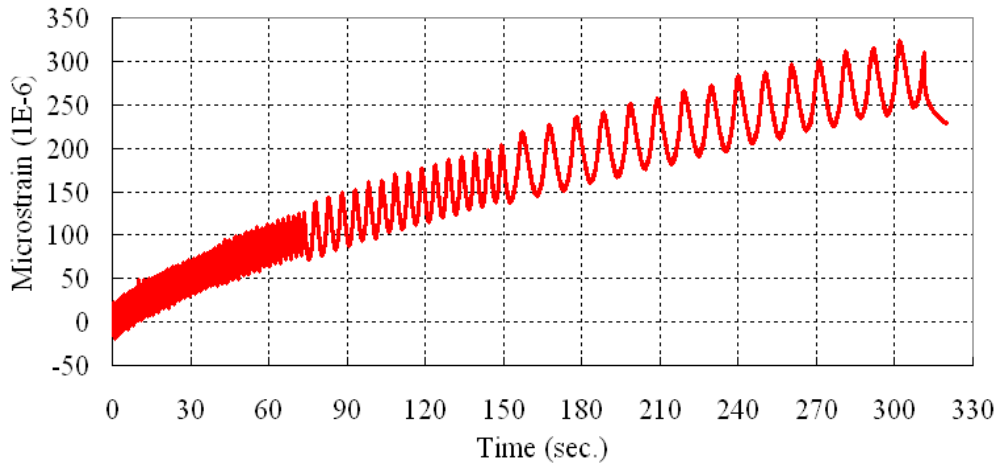
Figure 4.17 LWT viscoelastic test

In LWT dynamic modulus tests, a serial of frequencies, 2, 1, 0.5, 0.2 and 0.1 Hz were tested by specifying the angular frequency of the rotation axis. All the asphalt

mixtures were tested at three different temperatures, 10, 25 and 40°C. The typical patterns of the stress and measured tensile strain in the test are presented in Figure 4.18.



(a) Sinusoidal stress



(b) Measured tensile strain

Figure 4.18 Typical stress and strain in LWT dynamic modulus test

As long as the asphalt mixture behaves linear viscoelastically under loading and unloading, it can be assumed to remain undamaged, and also there is no energy dissipated in the testing process. Therefore, the area of hysteresis loop under loading and unloading

will not change with the cyclic loadings (Lytton 2000). The typical hysteresis loops in the LWT dynamic modulus test are shown in Figure 4.19.

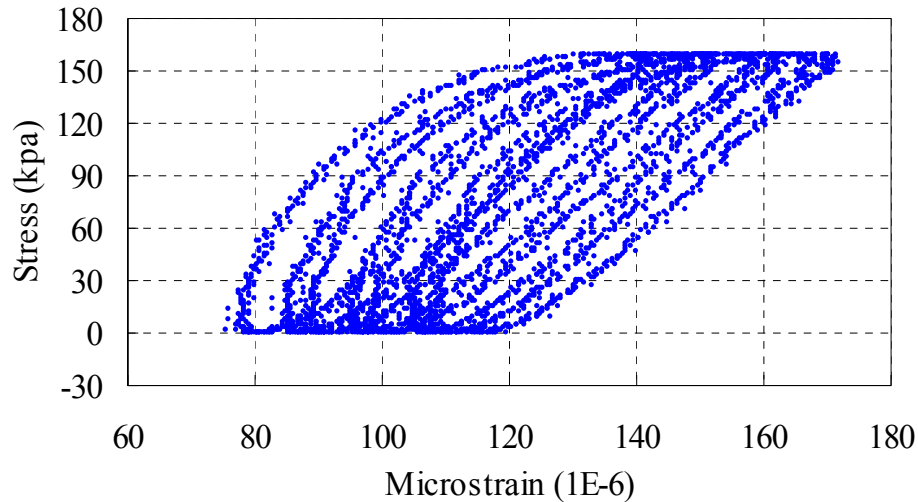


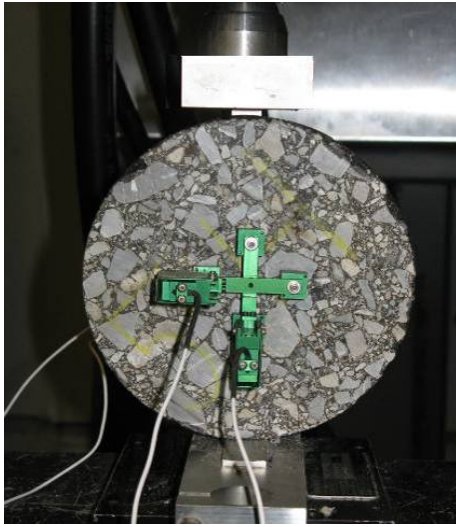
Figure 4.19 Typical hysteresis loop in LWT dynamic modulus test

During the LWT creep test, the beam specimen was subjected to a constant wheel load at midspan and the tensile strain on the bottom of the beam was measured with respect to loading time. Referring to AASHTO T322, the loading time for creep test was chosen as 100 seconds, and every specimen was tested three times with 30 minutes' relaxation interval. Creep tests were carried out at 10 and 25°C, because the damage from a large deformation is easily to be generated in creep test at high temperature (greater than 40°C). In the test, stress levels were designed to ensure that the induced strains would not exceed 500 micro-strains within 100 seconds loading period.

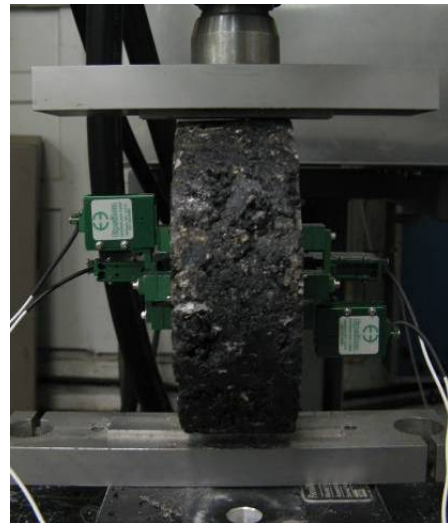
4.5.3 Indirect Tension Creep Test

Creep test was also conducted in indirect tension mode for various asphalt mixtures. During the test, constant axial stress was applied on specimens, and the extensometers installed on two directions can record the vertical and horizontal deformations while

loading and the recovered deformations after unloading. The test setup is shown in Figure 4.20. Specimens with 150mm (6 in.) in diameter and 50mm (2 in.) in height were fabricated through Superpave compaction method and then cut to thinner pills with the same diameter but the thickness of 50mm (2 in.). The tests were conducted following the procedures specified in AASHTO T322, hence the detailed description for this test is not provided herein.



(a) Front view



(b) Side view

Figure 4.20 Indirect tension test (IDT)

4.6 Data Processing

Because of the noise influence for the measurement, the data obtained from data acquisition system might not be accurate enough, especially for the valley and peak values. Generally, this problem can be solved by using Savitzky-Golay and Fast Fourier Transform (FFT) filter smoothing method. Savitzky-Golay filter method essentially performs a local polynomial regression to determine the smoothed value for each data

point. This method is superior to adjacent averaging because it tends to preserve features of the data such as peak height and width. While, FFT smoothing can be used to eliminate noise above a specified frequency using a sum of weighted sine and cosine terms of increasing frequency. The data must be equally spaced and discrete smoothed data points will be returned (Orfandis 1996). Based on the essential of those two methods, FFT filter smoothing method was used in this study. This process can be accomplished by removing Fourier components with frequencies higher than a cut-off frequency expressed below:

$$F_{cutoff} = \frac{1}{n \cdot \Delta t} \quad (4.21)$$

where, n is the number of data points specified by the user. Larger values of n result in lower cut-off frequencies, and thus a greater degree of smoothing. Δt is the time spacing between two adjacent data points.

The example of a stress-strain curve before and after FFT smoothing process is shown in Figure 4.21.

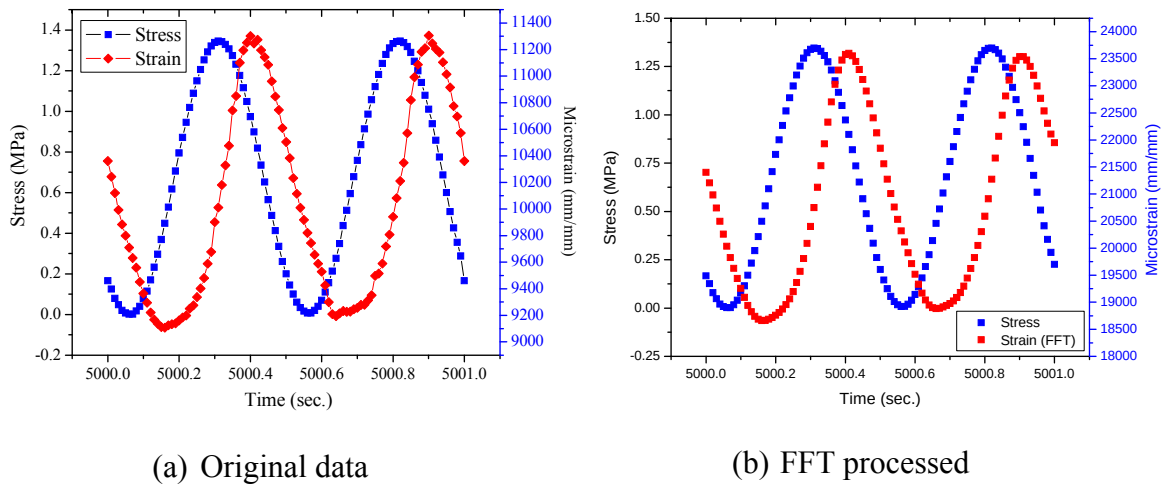


Figure 4.21 Stress-strain curves before and after FFT smoothing process

4.7 Materials and Specimen Preparation

Four asphalt mixtures typically used in Tennessee State were tested in this study. Two types of aggregates (limestone and granite) and three types of asphalt binder (PG 64-22, PG 70-22 and PG 76-22) were considered in the mixtures. An aggregate structure meeting Tennessee Department of Transportation (TDOT) specifications for 411-D mixtures was used as the design basis, and both limestone and granite were with a maximum aggregate size of 19 mm (3/4 in). The fine aggregates consisted of No.10 screenings, natural sand, manufactured sand, agricultural lime and screened RAP material. 10% RAP materials used in this study were obtained from limestone or granite sources and were used as substitutes for the fine aggregates in equal proportions for corresponding mixtures. 5.0 percent of asphalt content was designed for the mixtures with limestone aggregates, while 5.8 percent of asphalt content was designed for the mixtures with granite aggregates. In this study, attentions were not focused on the effect of RAP in the asphalt mixtures but on their viscoelastic properties. The information of the asphalt mixtures tested in this study is presented in Table 4.1.

Table 4.1 Asphalt mixtures tested in this study

Mixture	Aggregate	Asphalt Binder	Asphalt Content
GN-1	Granite	PG64-22	5.8%
LS-1	Limestone	PG64-22	5.0%
LS-2	Limestone	PG70-22	5.0%
LS-3	Limestone	PG76-22	5.0%

The detailed information of specimens for the viscoelastic tests is presented in Table 4.2.

Table 4.2 Specimen designed for the tests

Test	Specimen Type	Compaction Method	Air Voids (%)
IDT	150*50mm cylindrical pill	Superpave Gyratory Compactor (SGC)	4±1
Uniaxial	100*150mm cylinder	Superpave Gyratory Compactor (SGC)	4±1
LWT	300*125*50mm beam	Asphalt Vibratory Compactor (AVC)	5±1

4.8 Results and Discussions

4.8.1 Creep Compliance

As an example, the typical creep compliance results obtained from LWT creep tests at 10°C is shown in Figure 4.22. The deformations increased rapidly in the beginning of the test, and then tending to be stabilized with the time increasing. From those curves, the discrepancies of creep behavior among those mixtures can be visualized clearly, and the deformation resistance of the mixtures under a constant load (stress) can be reflected. Higher creep compliance usually represents lower creep deformation resistance of the asphalt mixture.

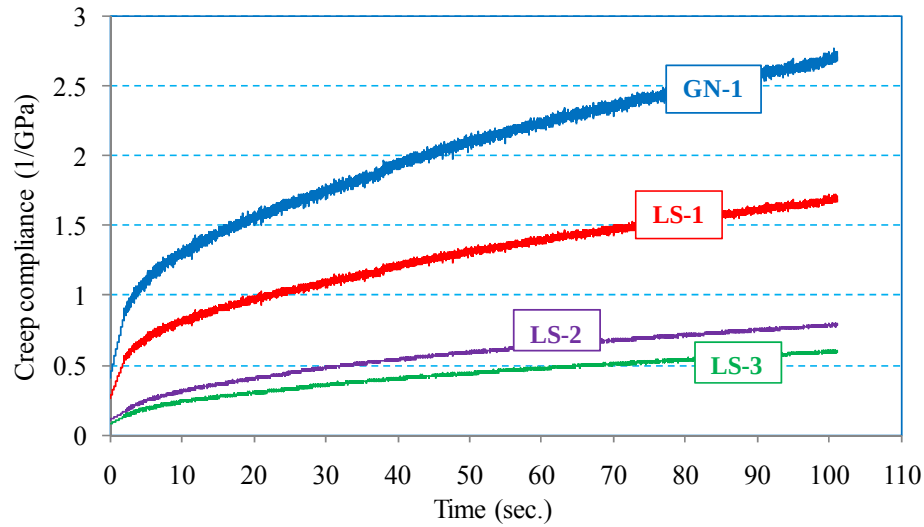
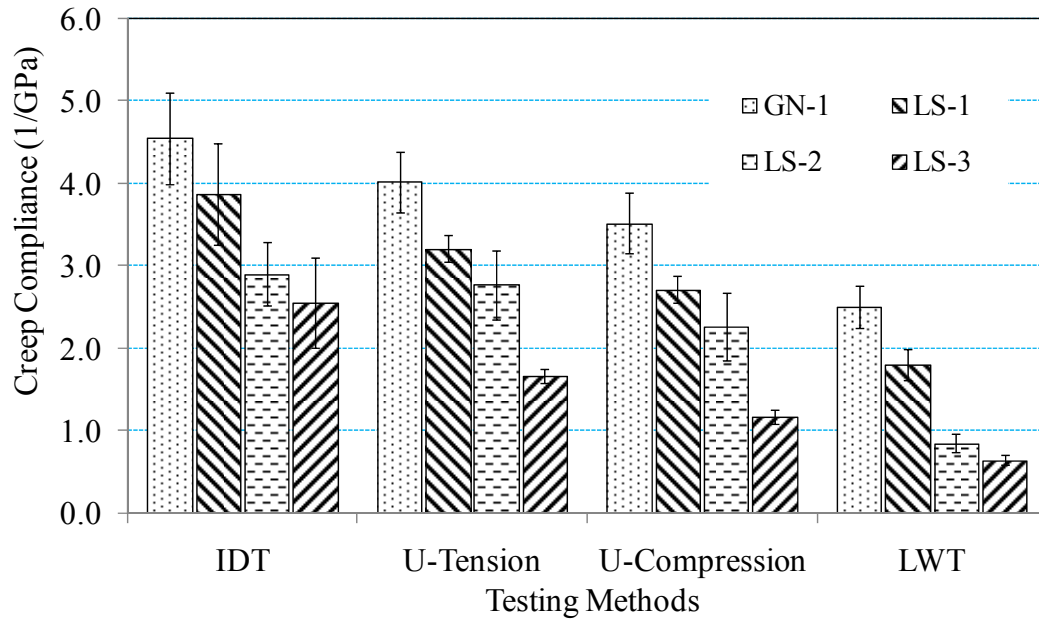
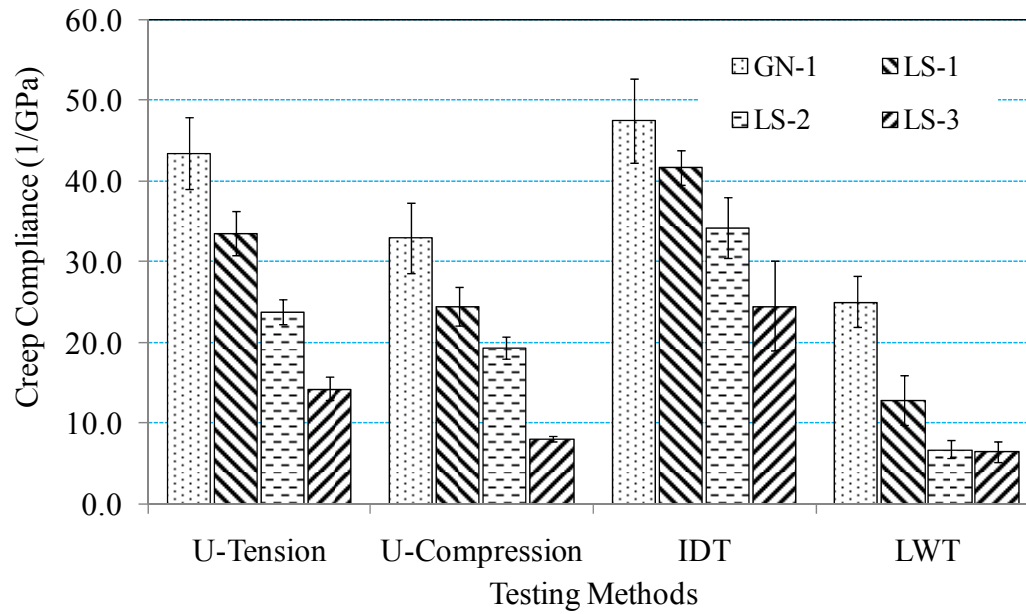


Figure 4.22 Creep test results from LWT test (10°C)

Figure 4.23 shows the results of creep test at 10°C and 25°C with respect to different testing methods. The creep compliance values shown in the figures were calculated according to stress and deformation at 100 second. The relationship of the results between each test was unable to be compared because of the combined effects of the various loading condition and air voids content. However, all the tests showed the identical contrast results toward different asphalt mixtures. In which, the mixture LS-3 with the highest grade of asphalt binder (PG 76-22) presented the lowest creep compliance, which reflects highest ability of resisting creep deformation. On the other hand, GN-1 with the higher asphalt binder content showed higher creep compliance, which indicates that higher asphalt binder content had no benefits on improving the deformation resistance of asphalt mixtures. Furthermore, taking the loading condition into account, the asphalt mixtures exhibited higher creep deformation resistances in compression condition than in tension condition.



(a) 10°C

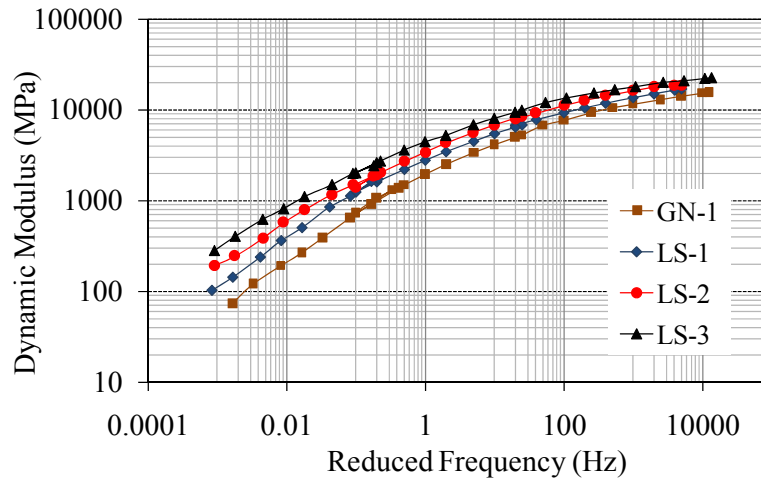


(b) 25°C

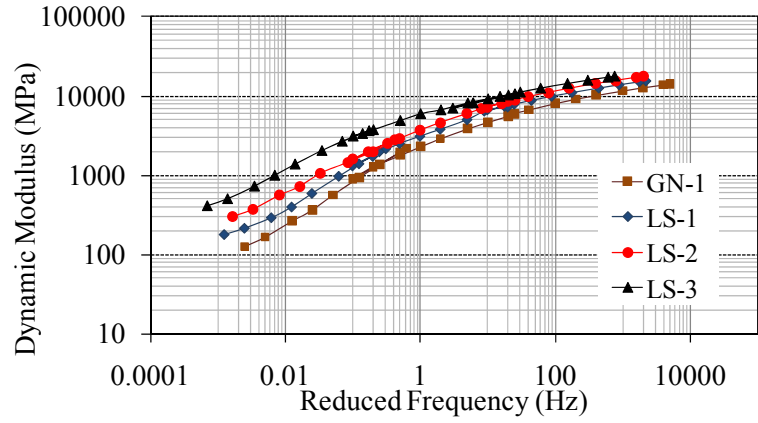
Figure 4.23 Creep compliance results

4.8.2 Dynamic Modulus

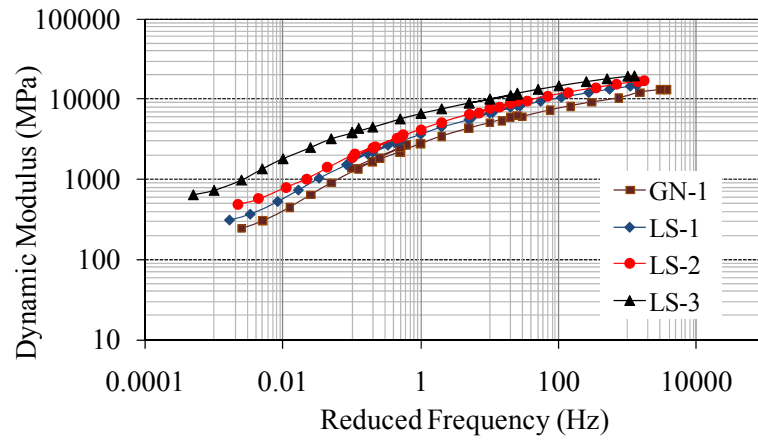
Regarding to the results shown in Figure 4.24, the dynamic modulus results from LWT tests were in good agreement with those from uniaxial tests for various asphalt mixtures. The asphalt mixtures contain higher grade of asphalt binder exhibited greater dynamic moduli which represents higher capability in resisting repetitive traffic loads. The mixture LS-3 with polymer modified asphalt binder (PG 76-22) showed the highest dynamic moduli among all the mixtures. Considering the mixtures with different aggregate types, LS-1 with limestone aggregate base exhibited higher dynamic moduli than those of GN-1 with granite aggregate base. It indicates that better interlocking actions among aggregate particles were achieved due to the higher stiffness of limestone aggregates as well as the lower asphalt content in the mixture.



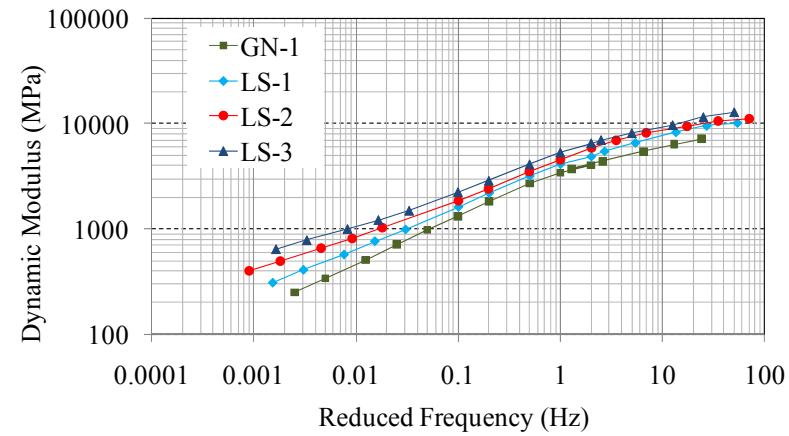
(a) Uniaxial tension



(b) Uniaxial tension-compression



(c) Uniaxial compression



(d) LWT

Figure 4.24 Dynamic modulus master curves for different tests

As an example, dynamic modulus master curves for the mixture LS-3 from different tests are shown in Figure 4.25. It is found that the dynamic modulus decreased more rapidly at high temperature in tension mode than that in tension-compression and compression modes. This phenomenon can be observed from the tail part of the master curves at low frequencies. The dynamic modulus results obtained from LWT tests mainly lie between the results from uniaxial tension and uniaxial tension-compression tests. Only the results of the mixture LS-3 were discussed herein, but similar characteristic could be found for all the other mixtures.

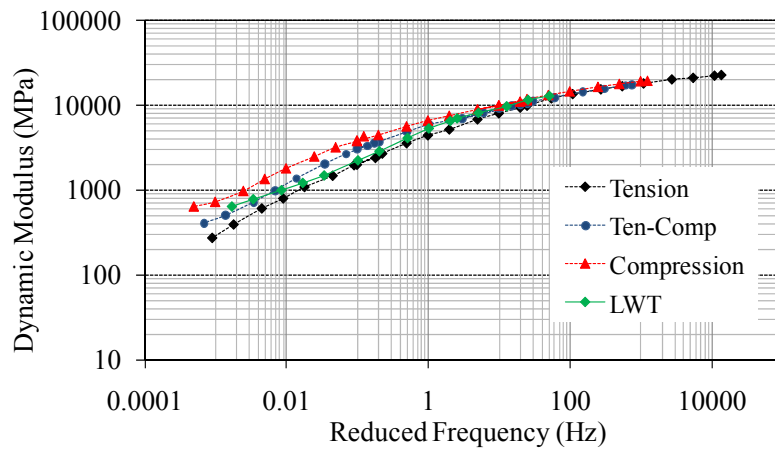
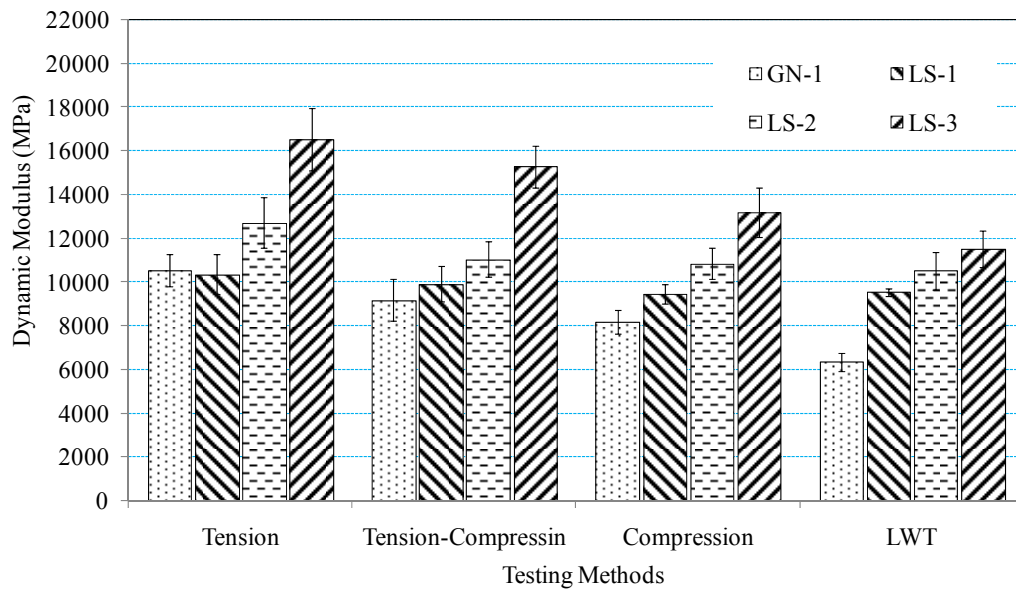


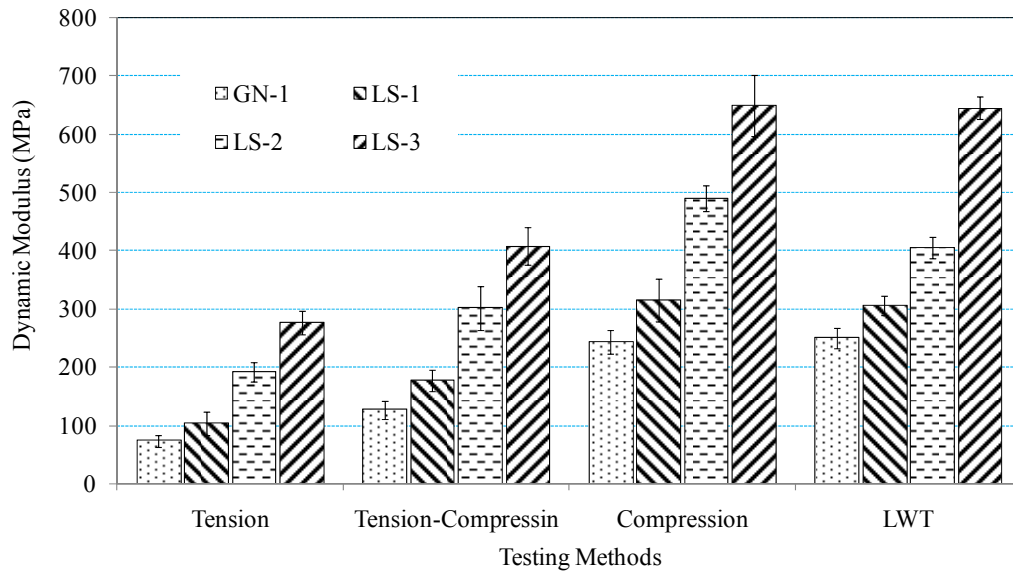
Figure 4.25 Dynamic modulus results from different tests (LS-3)

As shown in Figure 4.26, the dynamic modulus results at 40°C, 0.1Hz and 10°C, 2Hz. The ranking order of results from LWT tests was identical to that from uniaxial tests. According to the results from uniaxial tests, the dynamic moduli obtained from tension tests at high temperature (40°C) were only 40-60% of those from compression tests, and 65%-85% of those from tension-compression tests. However, the dynamic moduli obtained from tension tests at low temperature (10°C) were 7%-10% greater than those obtained from tension-compression tests, and 15%-30% of those from compression tests.

This phenomenon is based on the fact that for the specimen tested under tension condition the tensile loads and deformations were principally borne or resisted by asphalt bonding strength, while under compression condition the skeleton structure of aggregates could give more support to the whole mixture through interlocking actions. Additionally, distinct from aggregates, asphalt binder becomes softer as the temperature increases and its viscous property will be more dominant in the material.



(a) 10°C, 2Hz



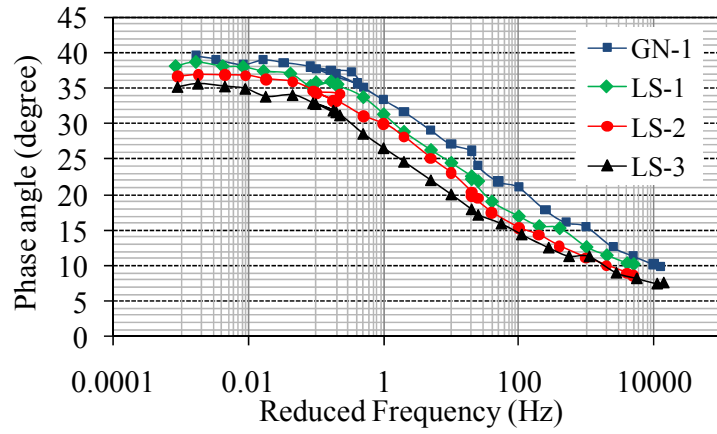
(b) 40°C, 0.01Hz

Figure 4.26 Dynamic modulus results at 10°C, 2Hz and 40°C, 0.1Hz

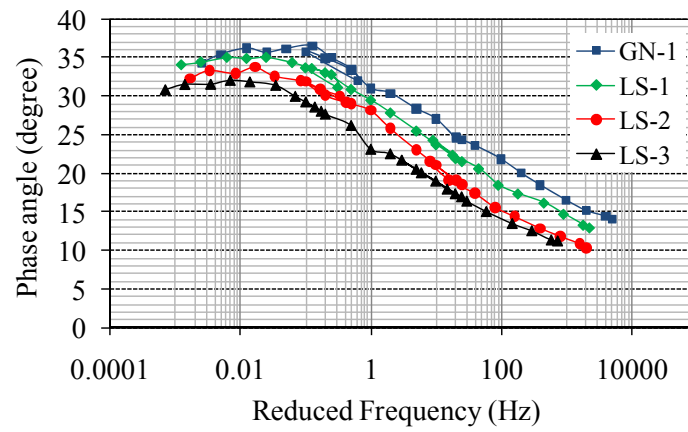
4.8.3 Phase Angle

The phase angle results in Figure 4.27 shows that the mixtures with higher asphalt grade exhibited smaller phase angles, and the mixture with higher asphalt content presented higher phase angles. The results from LWT tests were generally in agreement with those from uniaxial tests, and the results of phase angle for various mixtures were also coincident with the corresponding results of dynamic modulus.

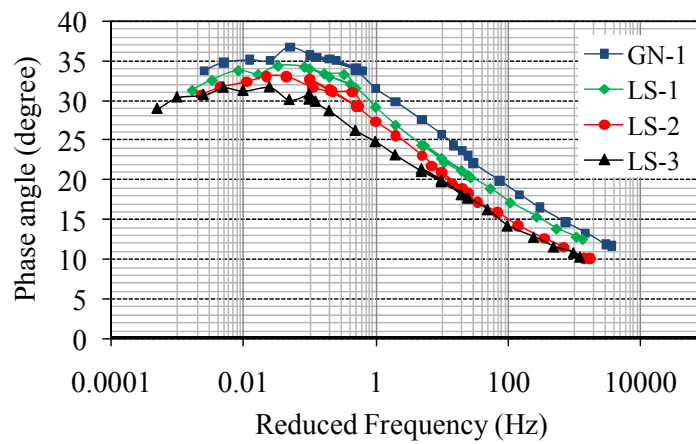
It can also be seen from Figure 4.27, the tests in tension condition showed relatively high phase angles at high temperature (low frequencies) than those from tension-compression and compression tests, because more viscous properties were exhibited in the mixtures when they were tested in tension condition at high temperature. This is also similar to the dynamic modulus results in tension condition at high temperature.



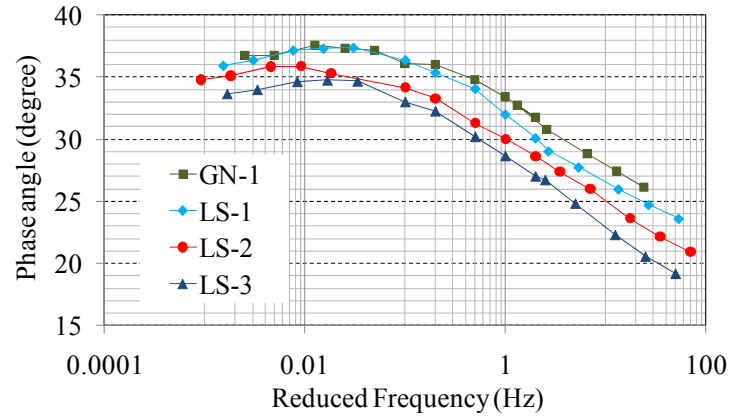
(a) Uniaxial tension



(b) Uniaxial tension-compression



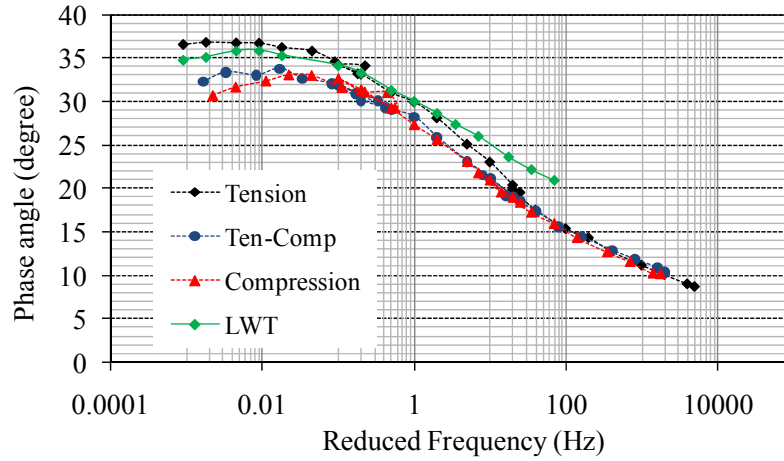
(c) Uniaxial compression



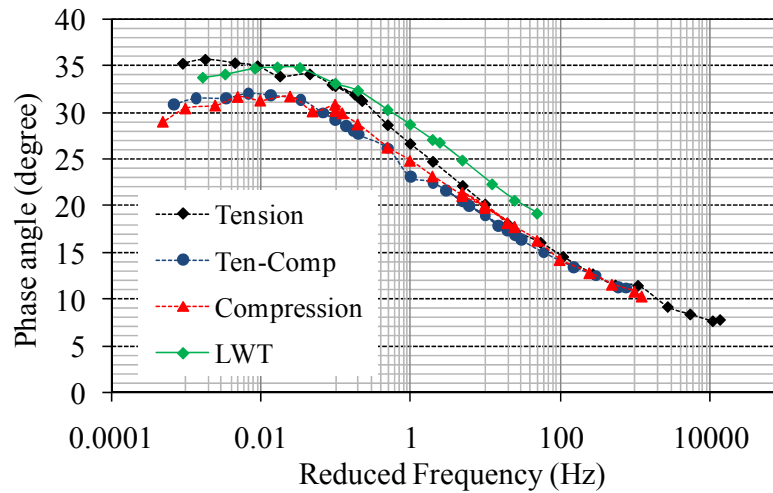
(d) LWT

Figure 4.27 Phase angle results for different testes

In Figure 4.28, the mixtures of LS-2 and LS-3 were chosen to demonstrate the effect of loading condition in phase angle results. The phase angles at high temperature obtained from tension tests were considerably greater than those obtained from tension-compression and compression tests, but the discrepancies were less remarkable as the temperature decreasing. The results indicate that asphalt mixtures showed more obvious viscous property in tension tests than that in tension-compression and compression tests, especially at the high temperature. According to the results from different tests, the phase angle results obtained from LWT tests were intermediate between the results from uniaxial tension and uniaxial tension-compression tests at high temperature (low frequencies). However, there was no consistent relationship for the results from those tests at low temperature (high frequencies).



(a) LS-2



(b) LS-3

Figure 4.28 Phase angle results from different tests (LS-2 & LS-3)

4.9 Summary and Conclusions

An innovative tension test for characterizing the viscoelastic behaviour of asphalt mixtures utilizing LWT is proposed in this study. The detailed analysis for the mechanical system and the procedures to perform the test are presented. In order to validate the feasibility of LWT viscoelastic tests, uniaxial tests in tension, tension-

compression and compression, and indirect tension creep tests were also conducted to the same asphalt mixtures. Based on the results from this study, the following can be summarized:

1. The results from LWT viscoelastic tests were in consistent with the results from the other viscoelastic tests. LWT viscoelastic tests were able to characterize the viscoelastic properties of asphalt mixtures with different asphalt contents, aggregates, and asphalt binders.
2. As discussed in the study, some differences were observed in the results from the tests in different loading conditions, especially when the test was conducted at high temperature, thus it is more appropriate to test the viscoelastic properties of asphalt mixtures under the loading condition they are suffered in the real pavement.
3. Compared to the other viscoelastic tests in the study, the efficiency of LWT tests was relatively high. Three specimens can be tested simultaneously. The fabrication method for specimens is also relatively simple and convenient.

Although the testing method proposed in this study is still on experimental phase, and some improvements need to be accomplished to make the test more efficient and convenient, the concept of using LWT for the viscoelastic tests was validated to be reasonable and feasible.

CHAPTER 5 CHARACTERIZING FATIGUE BEHAVIORS OF ASPHALT MIXTURES UTILIZING LWT

5.1 Introduction

Fatigue cracking is a primary concern for evaluating the service life of an asphalt pavement. Fatigue strength (resistance) is one of the major factors that engineers need to consider for designing an asphalt pavement. The material or structure expected to suffer fatigue could be very complicated and it can be studied in many ways. The testing methods currently contain from full-scaled or large-scaled structural tests to small-scale laboratory tests.

5.2 Fatigue Tests for Asphalt Mixtures

The fatigue properties of asphalt mixtures can be characterized by repeated loading tests either using a controlled stress or controlled strain. Currently, the mainly used fatigue tests for asphalt mixtures include two-point bending (trapezoidal beam) fatigue test, three-point bending beam fatigue test, flexural beam (four-point beam) fatigue test, semi-circular bending fatigue test, and direct or indirect tension fatigue test. All those tests are mainly performed by subjecting an asphalt specimen to a sinusoidal stress or strain under a tension condition.

In this study, a tension fatigue test was proposed to investigate fatigue properties of asphalt mixtures utilizing LWT. In order to verify the validity of LWT fatigue test, two conventional fatigue tests, direct tension fatigue test and flexural beam fatigue test, were

conducted to the same asphalt mixtures as well. The sketches of the test setups for those fatigue tests are shown in Figure 5.1.

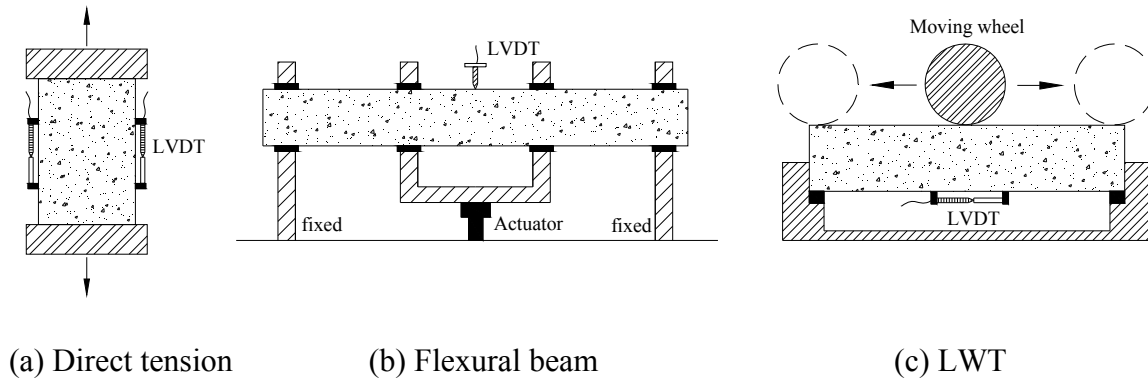


Figure 5.1 Test setups for the fatigue tests

5.2.1 Flexural Beam Fatigue Test (Four-Point Beam Fatigue Test)

The flexural beam fatigue test, also called four-point beam bending test, is a standard test method for determining the fatigue life of compacted Hot Mix Asphalt (HMA) subjected to repeated flexural bending (AASHTO T321; ASTM D 7460). It has been widely used for testing and evaluating the fatigue resistance of asphalt mixtures in the Strategic Highway Research Program (SHRP) Project A-003A. In the test, a pneumatic beam specimen is subjected to a repeated stress-controlled or strain-controlled load which is applied at the center of the beam until failure occurs. The beam specimen is placed in the fixture as shown in Figure 5.2, which allows four-point bending with free rotation and horizontal translation at all loading and reaction points. Haversine loading is applied to the beam through the built-in digital servo-controlled pneumatic actuator, and the bending deflections can be measured by the LVDT mounted on the specimen.

For the flexural beam fatigue tests in this study, a constant strain level was applied to the beam specimen at a frequency of 10 Hz with 600 microstrains until a fracture

failure occurs, and all the specimens were tested at 10 °C. The magnitudes of tensile stress, σ_t , tensile strain, ε_t , stiffness, S , and phase angle, φ , can be determined through the following formulas:

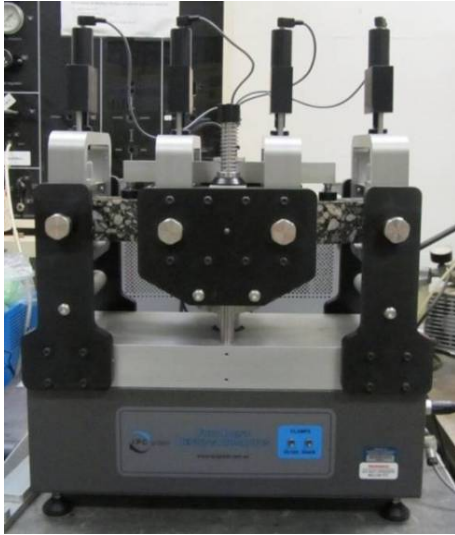
$$\sigma_t = \frac{3aP}{wh^2} \quad (5.1)$$

$$\varepsilon_t = \frac{12h\delta}{3L^2 - 4a^2} \quad (5.2)$$

$$S = \frac{\sigma_t}{\varepsilon_t} \quad (5.3)$$

$$\varphi = 360 \cdot f \cdot s \quad (5.4)$$

where, σ_t =peak-to-peak tensile stress; ε_t =peak-to-peak tensile strain; P =applied peak-to-peak load; S =stiffness; L =beam span; w =beam width; h =beam height; δ =beam deflection at neutral axis, and $a = L/3$.



(a) Testing device



(b) Specimen after test

Figure 5.2 Flexural beam fatigue test

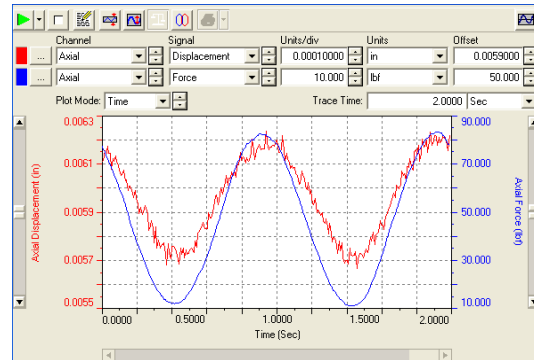
Flexural beam fatigue test is currently the most accepted fatigue test in the United States. It has the following advantages: (1) the mechanical condition is very clear and reliable, the stress between two inner clamps is uniform and no shear stress exists; (2) well-designed and friendly interfaced software has already been developed for data processing. However, it also has some disadvantages: (1) the fabrication of the specimen is troublesome and the requirements for the specimen are rigorous; (2) a supplemental environment chamber is required for running the test at various temperatures; (3) the specimen is relatively small, the non-uniformity in the specimen produced during fabrication may influence the test results.

5.2.2 Direct Tension Fatigue Test

The direct tension fatigue test provides a direct measurement of the fatigue behavior of asphalt mixtures under cyclic tensile loading. Before testing, specimens were placed in the environmental chamber at 10°C for at least two hours. During the test, specimen is suffered to a uniaxial repeated load which gives the specimen a relatively uniform tensile strain in its central section. Deformation over the central part of the specimen was monitored by means of three LVDTs attached to the glued-on studs (Figure 5.3a). Once the stress and strain data have been obtained, the theoretical analysis for characterizing the fatigue behavior can be carried out. Figure 5.3b illustrates the typical repeated sinusoidal loading and the corresponding response of the axial deformation recorded in the test.



(a) Test setup

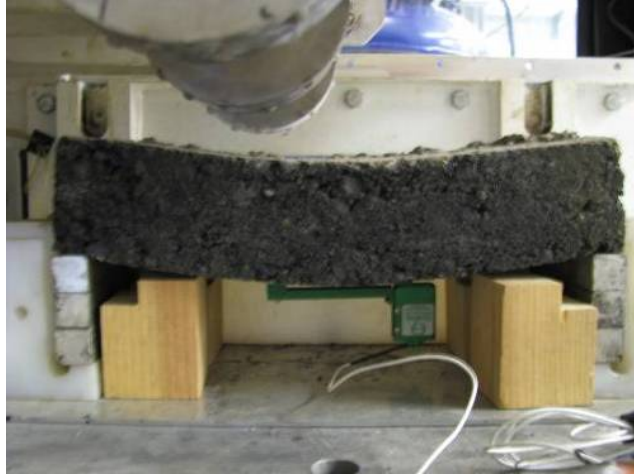


(b) Raw data from the test

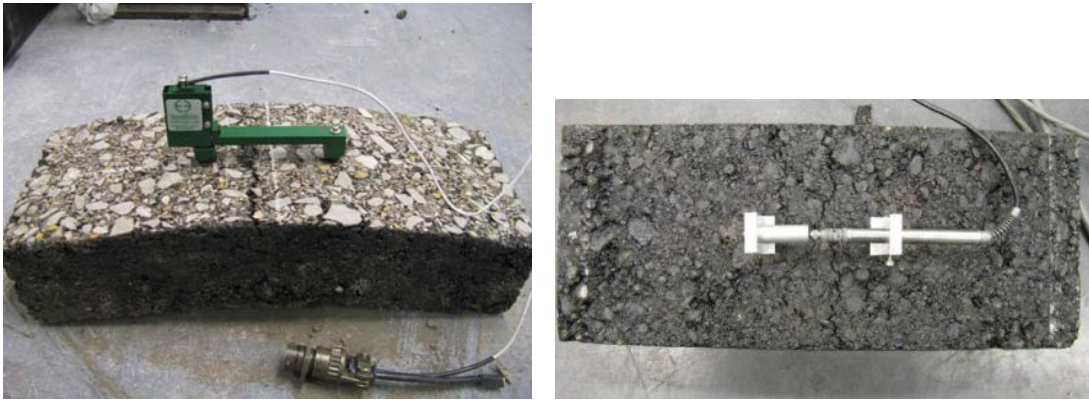
Figure 5.3 Direct tension fatigue test

5.2.3 LWT Fatigue Test (*Loaded Wheel Fatigue Test*)

The LWT fatigue test carried out in this study is intended to simulate the realistic conditions experienced by an asphalt layer in the pavement. Its principal distinctive feature is that the cyclic loads are applied by means of the moving wheels. In the test, beam specimens were subjected to cyclic loads supplied by APA loading system, and the stress on bottom of the specimen was calculated according to the stress solutions in CHAPTER 4 . Extensometers or LVDTs were installed on the specimens for measuring the tensile strains induced by the cyclic stresses (Figure 5.4). Differing from the old version of APA fatigue test which defines the fracture moment of the specimen as the fatigue life of the asphalt mixture, theoretical approaches can be adopted in this new test to analyze the fatigue behavior of the asphalt mixture based on the stress and measured strain.



(a) Test setup



(b) Specimen after test

Figure 5.4 LWT fatigue test

All the LWT fatigue tests were performed at 10°C with 2Hz loading frequency in this study. The typical hysteresis loops with the change of loading cycles obtained from the tests are shown in Figure 5.5. It can be seen clearly that the areas of the hysteresis loops were increasing with the increase of loading cycles.

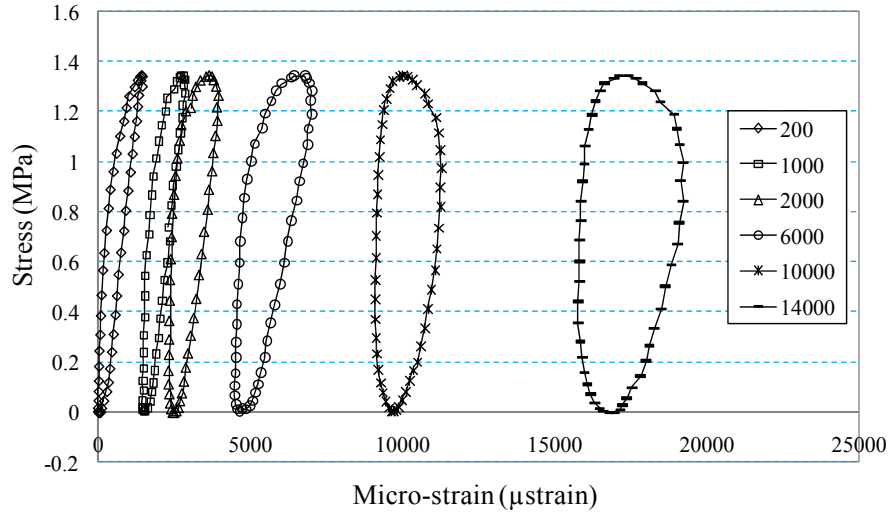


Figure 5.5 Typical hysteresis loops with the change of load cycles (LWT fatigue test)

Compared to the direct tension fatigue test and flexural beam fatigue test, the LWT fatigue test has the following potential benefits: (1) the loading condition of specimens is more in consistent with the actual situation in real pavements (repetitive moving loads); (2) the fabrication of specimens and test preparations are relatively simple and convenient; (3) three specimens can be tested simultaneously in both dry and water submerged conditions, and the testing temperature can be controlled intelligently. However, there are some disadvantages in LWT fatigue tests as well: (1) LWT is only able to provide stress-controlled mode for testing; (2) the accuracy of the stress calculated from a 2-D simple supported beam is hard to be guaranteed, even it has been proved within an acceptable engineering tolerance by FEM; (3) during the test the deformation measured by the extensometer or LVDT is not exactly on deforming direction (longitudinal direction of the beam specimen) when the specimen is suffering a large bending deformation. Accordingly, the LWT fatigue test proposed in this study is more like a structure fatigue test rather than an accurately controlled material test.

All the specimens in this study were tested until fracture or observable cracks occurred. The stress and strain were recorded throughout the whole experimental process for theoretical analysis. The fundamental information for the fatigue tests in this study is presented in Table 5.1.

Table 5.1 Fatigue test summary

Type	Test name	Temperature/ Frequency	Loading mode	Equipment
Conventional	Direct tension	10°C/2Hz	Controlled - stress	Material Testing System (MTS)
	Flexural beam	10°C/10Hz	Controlled - strain	Beam Fatigue Apparatus (BFA)
Modified	LWT (Loaded wheel)	10°C/2Hz	Controlled - stress	Asphalt Pavement Analyzer (APA)

5.3 Interpreting Fatigue Behavior of Asphalt Mixtures

One of the big challenges that asphalt fatigue tests concerning is the failure criterion. From the point of view on engineering, the approaches based on the analysis of stress and strain are widely used to describe the material's behavior during cyclic loading. The principle by using the tensile strain on the bottom of the asphalt layer to evaluate the fatigue life of pavements has been extensively accepted. Considerable researches have been devoted to define the failure life of asphalt mixtures based on this principle. Although diverse concepts have been developed, the stiffness and dissipated energy are the most considered and related ones.

5.3.1 Stiffness Approach

The stiffness of the asphalt mixture decreases throughout the crack developing process in the pavement. Basically, it follows three regimes of evolution, as shown in Figure 5.6. In phase I, rapid decreases in stiffness can be observed, which followed by phase II which corresponds to a linear decrease in stiffness. While in phase III, fracture cracking will occur due to the damage acceleration of micro-cracks and ultimately turn to observable macro-cracks which will cause the failure of the mixture.

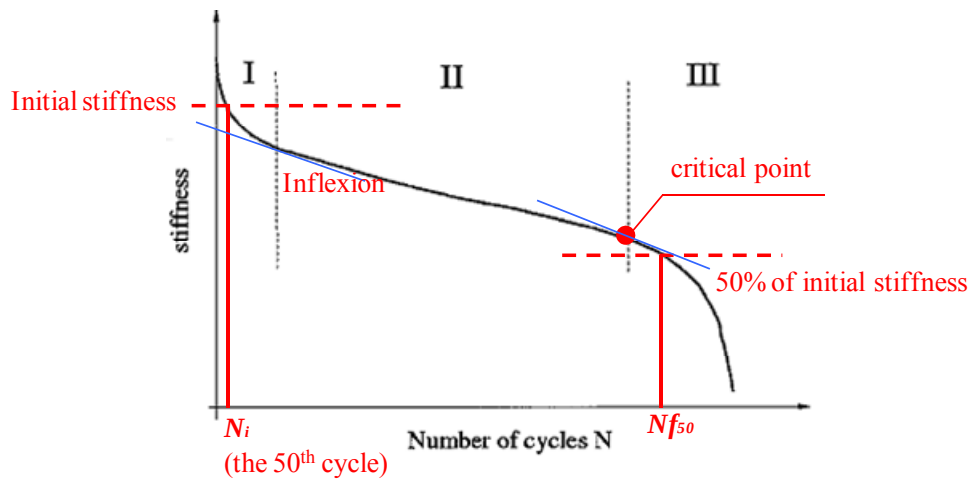


Figure 5.6 Typical developing phase of stiffness in fatigue test

Usually, the failure of the fatigue test can be determined according to the number of cycle generates a 50 percent reduction in initial stiffness (Roberts et al.1991; Hicks et al. 1993; Williams 1998). The concept of pseudostiffness was proposed by Kim et al. (1994, 1995) and Lee (1996) in evaluating fatigue life of asphalt mixtures, and they reported that 50 percent reduction in pseudostiffness is an appropriate criterion indicting failure in the material.

5.3.2 Dissipated Energy Approach

On the basis of the stress and strain, a hysteresis loop can be constructed, which is the most creative analytical tool in the study of fatigue. For viscoelastic materials, the most important property of hysteresis loops is not their ability to show cyclically varying stress and strain but their ability to reflect the plastic strain caused by the loading-unloading cycles. A typical stress-strain hysteresis loop is shown in Figure 5.7.

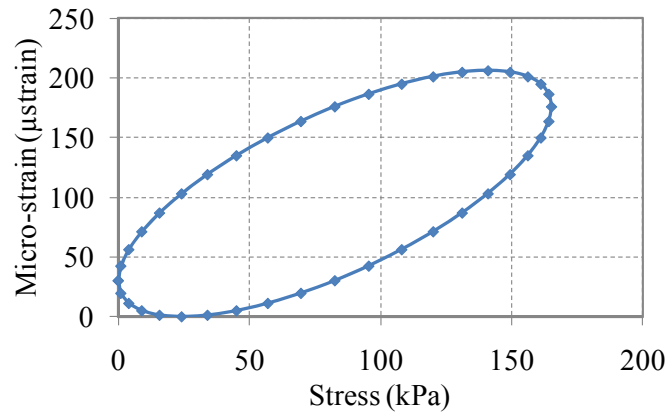


Figure 5.7 Typical stress-strain hysteresis loop

When asphalt mixture is subjected to an external load, the area of hysteresis loop represents the energy absorbed by the mixture. During the fatigue test the change of the area of hysteresis loops indicates that part of the energy in the system has been dissipated, and some plastic strain or damage have occurred to the asphalt mixture. As the increasing loading cycles and the propagation of cracks, the dissipated energy changes continuously throughout the fatigue process. Therefore, the concept of dissipated energy (DE) generated by an external work can be used as a direct and visualized way to describe the development of damage in asphalt mixtures. Dissipated Energy per cycle can be calculated by the stress, strain and phase angle, $DE = \pi \sigma_n \varepsilon_n \sin \phi_n$. But the areas of the

hysteresis loop are usually used for calculating the dissipated energy as the development of loading cycles.

Many studies have been reported in predicting the fatigue life and crack propagation of asphalt mixtures based on dissipated energy method. Chomton and Valayer (1972) presented that the cumulative dissipated energy had strong relationship with the failure life of asphalt mixtures, and it was independent of the asphalt mixtures. Through a serial experiments, Van Dijk (1975) and Van Dijk and Vesser (1977) found a strong relationship between the total amount of energy dissipation and the number of loading cycles to failure, which was not significantly affected by the loading modes, frequency, temperature, and occurrence of rest periods, but was highly material dependent. Although there are some arguments, similar relationship between cumulative dissipated energy and fatigue life were found in those studies. Baburamani and Porter (1996) showed a strong correlation between initial dissipated energy and fatigue life of asphalt mixtures. Pronk and Hopman (1991) also suggested in their study that the dissipated energy per cycle or period was responsible for the fatigue damage in the asphalt mixtures. Tayebali et al. (1992) introduced two terms: the “stiffness ratio,” which is the ratio of the stiffness at load cycle to the initial stiffness, and the “dissipated energy ratio,” which is defined as the ratio of cumulative dissipated energy to load cycles. Their work suggested that there was a unique relationship between the stiffness ratio and the dissipated energy ratio, but not necessarily between cumulative dissipated energy and fatigue life, which was also proposed by Fakhri (1997).

More recent studies suggested that more consistent results can be achieved through the Ratio of Dissipated Energy Change (RDEC) (Carpenter et al., 2003; Ghuzlan and

Carpenter, 2000; Shen and Carpenter, 2005). This concept was first introduced by Carpenter and Jansen (1997) who suggested using the change in dissipated energy to characterize the damage accumulation and fatigue life. The change in dissipated energy represents the total effect of fatigue damage without the necessity of considering material type and loading modes. The application and study of RDEC were modified and expanded by Ghuzlan and Carpenter (2000), and then applied and verified by Carpenter et al. (2003) in which the ratio of dissipated energy change was successfully used and expressed as the following formula:

$$RDEC = \frac{DE_{n+1} - DE_n}{DE_n} \quad (5.5)$$

where, $RDEC$ = the ratio of dissipated energy change;

DE_n = the dissipated energy in load cycle n ;

DE_{n+1} = the dissipated energy in load cycle $n+1$.

Based on those studies, a Plateau Value (PV) which presents the nearly constant value of RDEC was proposed by Shen et al. (2006). This PV value represents a period where there is a constant percent of input energy being turned into damage during the fatigue process. The PV value is a function of the loading inputs for any mixture, and it varies with the mixture type for similar loading inputs. The PV value is significant because it provides a unique relationship with the fatigue life even for different mixtures, loading modes and loading levels (Shen 2006).

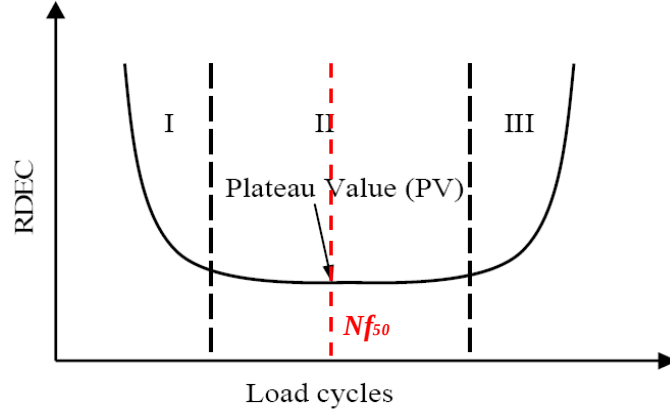


Figure 5.8 Typical RDEC vs. load cycle curve (Carpenter et al. 2003)

Figure 5.8 shows the typical pattern of RDEC vs. load cycles. The fatigue life can be characterized by a plateau value (PV) through the number of cycles at 50% reduction of initial stiffness.

$$PV = \frac{1 - \left(1 + \frac{100}{Nf_{50}}\right)^k}{100} \quad (5.6)$$

According to Shen (2006), a unique relationship can always be established between PV and N_f regardless of the asphalt mixture type, loading mode and testing condition. Thus, the PV value method associated to RDEC was employed in this study to evaluate the fatigue life of asphalt mixtures.

5.3.3 Pseudostiffness and Pseudo-Strain Energy Approach

The extended elastic-viscoelastic correspondence principle was proposed by Schapery (1984). It suggests that the constitutive equations for viscoelastic material can be expressed as elastic problems, but the pseudo variables need to be used to substitute the physical variables in the convolution integral. According to correspondence principle for viscoelastic materials, the stress-strain relationship can be expressed as:

$$\sigma = \int_0^t E(t-t') \frac{d\varepsilon}{dt'} dt' \quad (5.7)$$

where, σ =stress; ε =strain; $E(t)$ =the relaxation modulus related to time; t =time of interest; t' =integration variable related to time.

The equation above can also be written as linear elastic form as similar as the Hooker's law:

$$\sigma = E_R \cdot \varepsilon^R \quad (5.8)$$

Hence, substitute pseudo-variables for the physical variables in the equation the expression of pseudo-strain is:

$$\varepsilon^R = \frac{1}{E_R} \int_0^t E(t-t') \frac{d\varepsilon}{dt'} dt' \quad (5.9)$$

where, ε^R =pseudo-strain; E_R =reference modulus.

According to Equation 5.9, it is clear that the correspondence of stress-strain can be expressed through a linear form. It also demonstrates that the equation for stress-strain in elastic can be employed as a pseudo form to interpret the system as a linear elastic problem even the system is actually in viscoelastic situation. Based on those theoretical principles, the concepts of pseudo stiffness and dissipated pseudo-strain energy have been proposed by Kim et al. (1997) and Little et al. (1997) for evaluating the fatigue life of asphalt mixtures.

Figure 5.9 in Kim (2009) shows typical stress-pseudostrain hysteresis loops at different cycles in a controlled-stress fatigue test. The change in the slope of each $\sigma - \varepsilon^R$ cycle can be observed due to the damage incurred in the asphalt mixture.

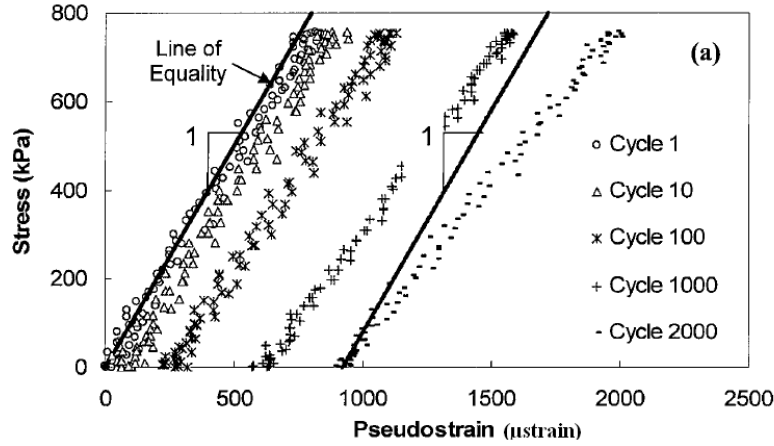


Figure 5.9 Stress-pseudostrain hysteresis loops in a fatigue test (Kim 2009)

The slope of the linear regression of the hysteresis loop is defined as pseudo stiffness, and the decrease in pseudo stiffness indicates damage growth due to cyclic loading. The pseudostiffness can be expressed as (Kim 2009):

$$S^R = \frac{\tau_m}{\gamma_m^R} \quad (5.10)$$

where, S^R =pseudo stiffness; γ_m^R =peak pseudo strain; τ_m =physical stress corresponding to the peak pseudo strain in each cycle.

Little et al. (1997) reported that dissipated pseudo strain energy can be used to describe the real damage during the fatigue test. The area within the hysteresis loop is defined as pseudo-strain energy, which can be described by the following formula:

$$DPSE = \sum \psi(t) (\epsilon_R^d(t) \sigma_m^d(t)) \quad (5.11)$$

Many other studies based on the pseudo concept have also been conducted, and most of them were proved to be appropriate to characterize the fatigue properties of asphalt mixtures. However, the calculation of pseudo-strain or pseudo-stress is too complicated

to be carried out. For analyzing the large amount of data created in fatigue tests, well-designed computer programs are required for applying those approaches and obtaining satisfactory efficiency.

5.4 Materials and Specimen Preparation

The same asphalt mixtures used for the viscoelastic test in CHAPTER 4 were also used for the fatigue tests in this study. Two different methods were used for the compaction of specimens in this study. Beam specimens for flexural beam fatigue test were cut from the original specimens compacted by asphalt vibratory compactor (AVC). Cylindrical specimens for direct tension fatigue test were cored and trimmed from the original cylindrical specimens compacted by Superpave gyratory compactor (SGC). While the beam specimens for LWT fatigue test were compacted directly through AVC, and no trimming or coring process is needed. The detailed information of specimens for the fatigue tests is provided in Table 5.2.

Table 5.2 Specimens for fatigue tests

Test	Specimen Type	Compaction Method	Air Voids (%)
Flexural beam	380*50*63mm beam	Asphalt Vibratory Compactor (AVC)	4±1
Direct tension	100*150mm cylinder	Superpave Gyratory Compactor (SGC)	4±1
LWT	300*125*50mm beam	Asphalt Vibratory Compactor (AVC)	5±1

Prior to the fatigue tests, IDT resilient modulus and creep tests were conducted for investigating the properties which are associated to the fatigue resistance of the asphalt mixtures. All of the IDT performance tests were conducted on triplicate 50mm (2.in) thick specimens compacted to $4\pm 1\%$ air voids at 25°C (77°F). The results of resilient modulus (M_r), fracture energy (FE), and dissipated creep strain energy ($DCSE_f$) are presented in Table 5.3, but the procedures for those tests are not provided in detail herein.

Table 5.3 Summary of the properties of asphalt mixtures

Mixture ID	Asphalt Binder	Asphalt Content (%)	Resilient modulus (MPa)	COV (%)	Tensile strength (MPa)	COV (%)	Fracture Energy (kJ/m ³)	COV (%)	DCSE _f (kPa)	COV (%)
GN-1	PG64-22	5.8	2.32	9.3	0.73	3.4	9.81	9.6	161.27	11.3
LS-1	PG64-22	5.0	2.83	7.1	0.88	4.5	11.80	5.6	142.80	3.6
LS-2	PG70-22	5.0	3.49	2.9	1.30	8.3	25.59	9.7	119.63	5.3
LS-3	PG76-22	5.0	3.92	6.6	1.35	7.1	27.13	6.6	94.19	6.0

* GN-granite; LS-limestone; Std.-standard deviation; COV-coefficient of variation.

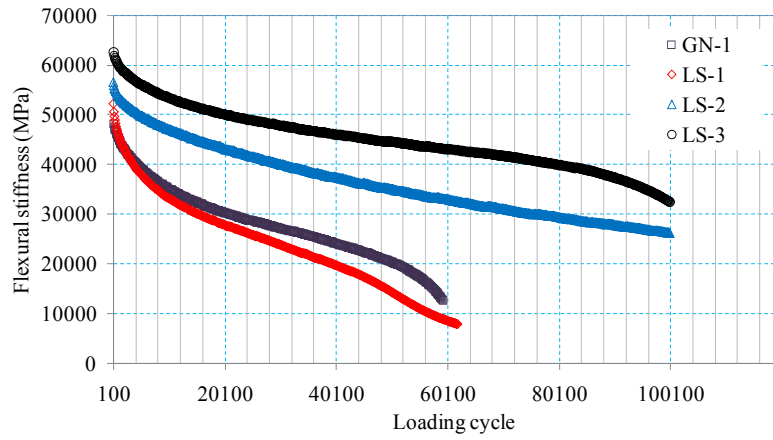
5.5 Results and Discussions

The stiffness and the ratio of dissipated energy change (RDEC) approaches were employed for evaluating fatigue life of asphalt mixtures in this study.

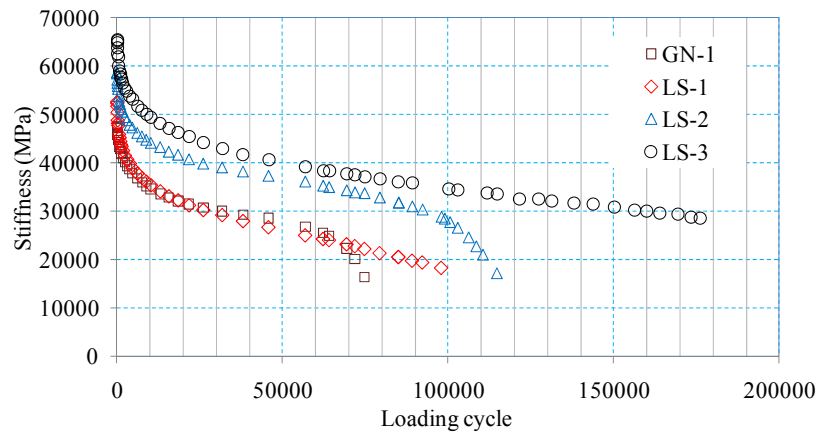
Generally, the failure criteria used for controlled-strain and controlled-stress tests are different. In controlled-strain test, it is normally considered that the material have reached the threshold of service when the stiffness has dropped to 50% of its initial value. While, in controlled-stress test, the material is considered as failure when either it has fractured or its stiffness is lower than 10% of its initial value. The differences of the failure criteria between those two testing modes always results in a longer life from a controlled-strain fatigue test than that from a controlled-stress test. In this study, in order to make comparisons for different testing methods, 50% reduction of stiffness was chosen as the failure criteria for both controlled-strain and controlled-stress fatigue tests. In addition, the number of cycles at 50% reduction of stiffness was also used for determining the PV in the RDEC approach regardless of loading modes and testing methods.

5.5.1 *Reduction of Stiffness*

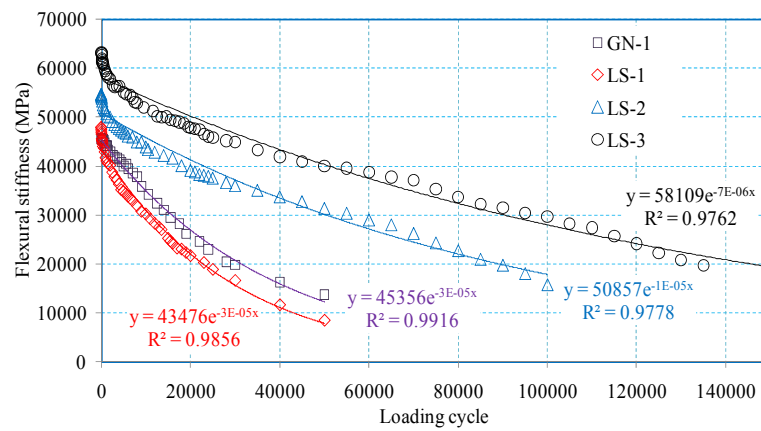
The results of the stiffness with respect to load cycles are presented in Figure 5.10.



(a) Flexural beam fatigue test



(b) Direct tension fatigue test



(c) LWT fatigue test

Figure 5.10 Stiffness vs. loading cycle for different tests

It is clear that all the three fatigue tests showed identical ranking order according to the stiffness results for various mixtures. The mixtures with higher asphalt binder grade exhibited higher initial stiffness and larger number of cycles to the rapid reduction of the stiffness. In addition, the stiffness vs. loading cycle curves from LWT fatigue tests in Figure 5.10c can be well fitted by exponential functions. Although some non-coincident fittings are shown in the initial and tail parts of the curves, all of the R^2 of the fittings were greater than 0.97.

The results of failure life (N_f) which is defined as the number of the loading cycle at 50% reduction of the initial stiffness is shown in Figure 5.11.

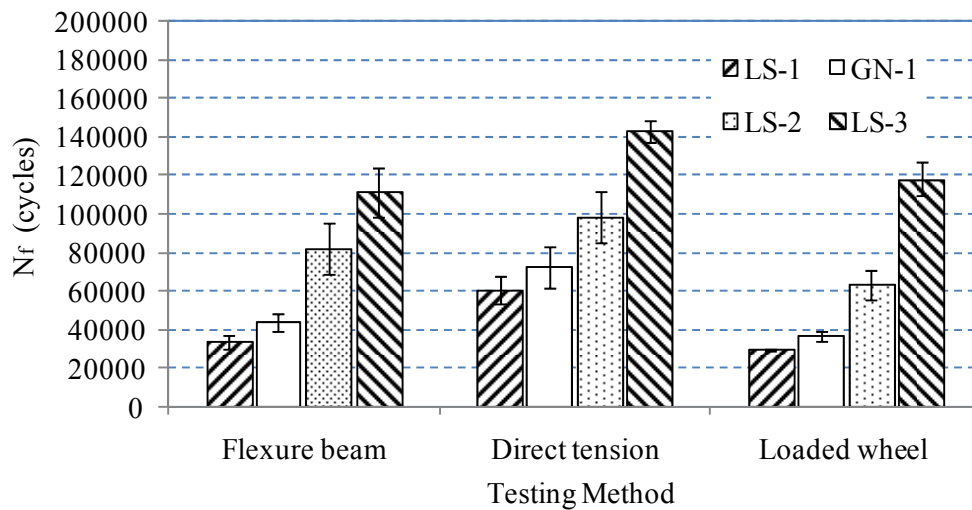


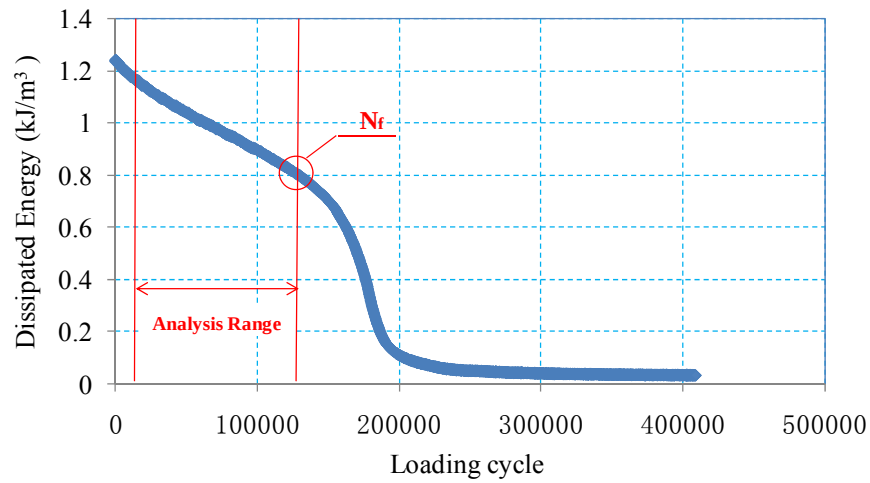
Figure 5.11 N_f results from different tests for various mixtures

According to the N_f results, LS-3 showed longest fatigue life among all the mixtures, which followed by LS-2, GN-1 and LS-1. It indicates that higher grade asphalt binder had benefits on increasing the stiffness and crack resistance of asphalt mixtures. Although the initial stiffness of GN-1 was smallest among all the mixtures, with relatively higher asphalt content (5.8%) it exhibited longer fatigue life than LS-1 (5.0%).

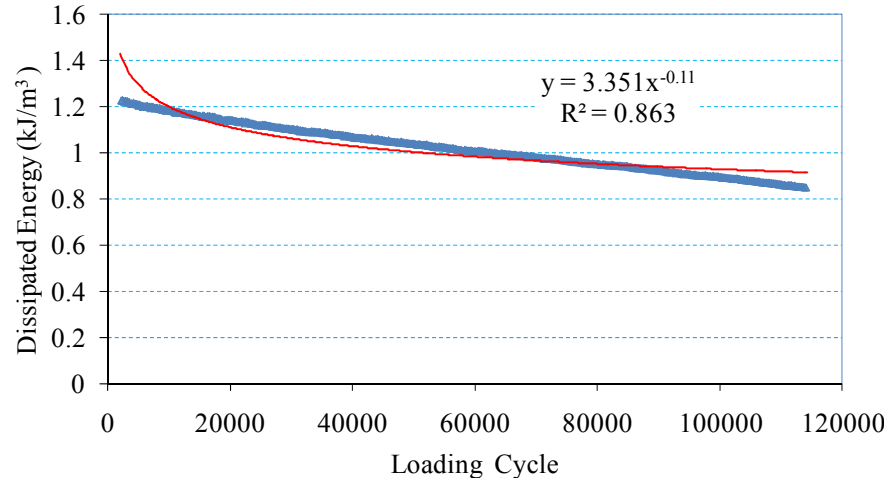
It indicates that higher asphalt binder content had positive influence on the fatigue resistance of asphalt mixtures.

5.5.2 Ratio of Dissipated Energy Change (RDEC)

Before dissipated energy results are used to calculate RDEC, the analysis range for the DE vs. loading cycle curve should be selected in order to minimize the error for the calculation, as shown in Figure 5.12. A curve fitting process for this selected analysis range is required to obtain the most accurate fitting equation for the calculation of RDEC. For selecting this analysis range, some rules given in Shen (2006) should be complied with to reduce the subjective effects. In this study, the fitting functions with highest R^2 were chosen, and the exponents of the functions were used for the further calculations.



(a) Dissipated energy vs. loading cycle



(b) Exponential curve fitting

Figure 5.12 Curve fitting process for dissipated energy vs. loading cycle

Once the exponent function from the curve fitting is obtained, RDECs can be calculated through a simplified formula proposed in Shen (2006). The typical RDEC vs. load cycle curve is shown in Figure 5.13.

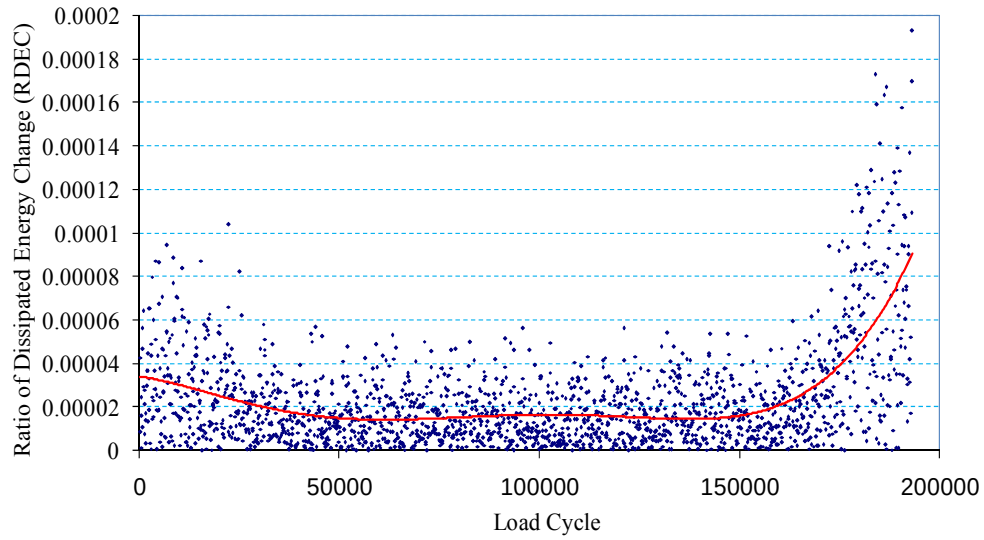


Figure 5.13 RDEC vs. load cycle

Plateau Value (PV) is defined as the value of RDEC when the number of the cycle is at the 50% reduction of stiffness (N_{f50}). It can be seen from Equation 5.6 that the PV is

depending on both the change of dissipated energy and the change of stiffness. The lower PV usually represents a longer fatigue life, and vice versa.

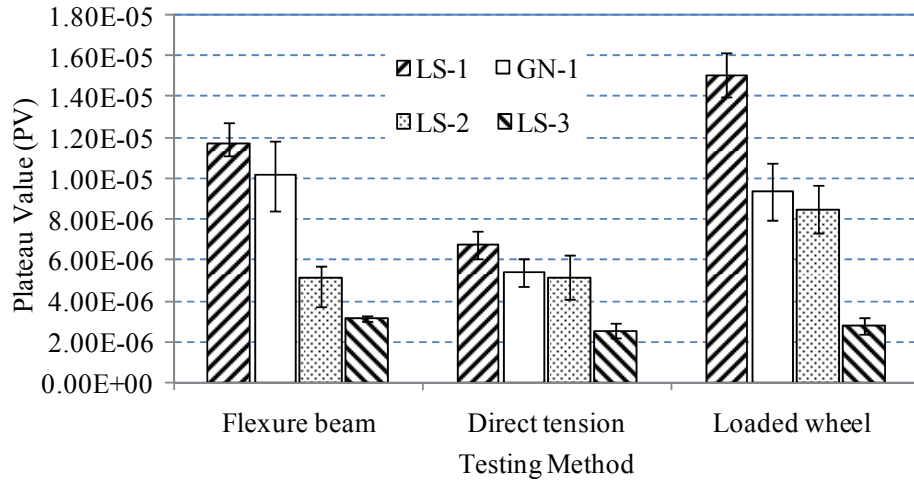


Figure 5.14 PV results from different tests for various mixtures

As shown in Figure 5.14, the results of PV from LWT fatigue tests were in good agreement with the results from flexural beam and direct tension fatigue tests. The fatigue lives of various mixtures interpreted by PV were generally in consistent with those represented by N_f . The mixtures with higher grade asphalt binder such as LS-3 and LS-2 exhibited lower PVs, which imply longer fatigue lives. The mixture with higher asphalt content such as GN-1 showed a higher PV, which indicates a shorter fatigue life.

The relationship between N_f and PV is plotted in Figure 5.15. An apparent exponential relationship can be found between the N_f s and PVs. The similar relationship in form was also reported in Shen (2008) through a series fatigue tests, and it was said that this relationship is unique independent of mixture types. According to the results from this study, it seems that this unique relationship can be extended to the tests in different loading modes (controlled-stress or controlled-strain) and testing methods.

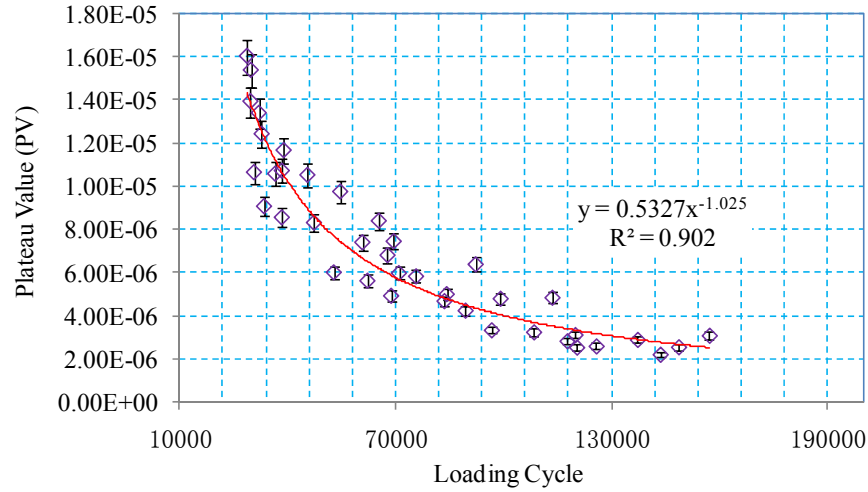


Figure 5.15 Relationship between PV and N_f

5.6 Discussions on LWT Fatigue Test

For the old version of APA fatigue test, the criteria for evaluating the fatigue life are either through the maximum vertical deflection or the rate of vertical deflection of the specimen, or the observation of appearance of fractures on the specimen. Those evaluation criteria could be accepted to some extent for structural analysis. However, they are too subjective and inaccurate for the study of materials. Generally, the fatigue life has already reached before the specimen is suffered to a large deformation or observable damage, or to say, failure.

Figure 5.16 presents an example which shows an obvious difference between fatigue life and fracture point in a fatigue test. It can be seen clearly that as the increase of deformation the 50% reduction of the stiffness for the asphalt mixture reached at around 27000 cycles, but the fracture damage was occurred after 40000 cycles.

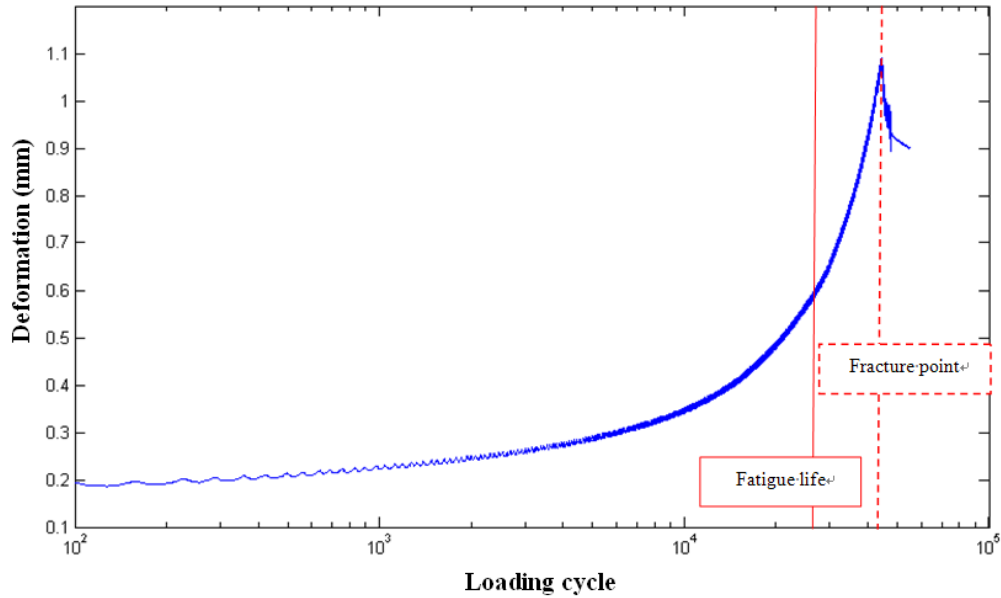


Figure 5.16 Deformation vs. loading cycle in fatigue test

5.7 Summary and Conclusions

A testing method of using LWT to investigate the fatigue properties of asphalt mixtures is proposed in this study. As discussed in the previous chapters, LWT has its unique benefits for simulating the field situation that asphalt material suffered in the pavement. Thus, the test results are more reasonable to reflect the real fatigue behavior of asphalt mixtures. Based on the results from this study, the following can be summarized:

1. LWT fatigue tests were able to differentiate the differences between the fatigue resistances of various asphalt mixtures. The results from LWT fatigue tests were in consistent with those from flexural beam and direct tension fatigue tests. The results clearly indicated that the mixtures made with higher grade of asphalt binder showed higher initial stiffness and a longer fatigue life. The mixtures made

with higher asphalt content exhibited lower initial stiffness but a longer fatigue life.

2. Compared to the old version of APA fatigue test, the modified test proposed in this study was more reasonable to characterize the fatigue behavior of asphalt mixtures. In this modified test, theoretical approaches for modeling the fatigue behavior of asphalt mixtures are able to be adopted once the stress and strain are known.
3. In the direct tension fatigue test, the direction of the pull load on the specimen is identical to the direction of the specimen's compaction. Thus, the pre-produced interlocking forces between aggregates created in the compaction process could resist a part of the load or absorb some energy during the test. However, in the LWT fatigue and flexural beam fatigue test, the direction of the tensile stress that the specimen suffered during the test is perpendicular to the direction of the compaction when the specimen was fabricated, which is similar to the situation in the field.

CHAPTER 6 CONCLUSIONS AND RECOMMENDATIONS

6.1 Conclusions

This study is composed of four innovative or modified tests for characterizing pavement materials (base courses, pervious concrete, and asphalt mixtures). Loaded wheel tester (LWT) on the platform of asphalt pavement analyzer (APA) was used as the loading system for these tests. Some other conventional or currently existing laboratory tests were also conducted for comparison purpose.

The first test is to utilize APA for evaluating the effect of geogrids in pavement base courses. River sand with one gradation and gravel with two gradations were used as base courses. Four types of geogrids were evaluated, with two for river sand base and three for gravel base. Based on the results, the following conclusion can be drawn:

1. The LWT system is appropriate to be used to evaluate the reinforcement effects of geogrids in pavement base courses. The results were repeatable and generally in agreement with the results from another independent study by the researchers at the University of Kansas with similar base materials.
2. The testing method proposed in this study is able to differentiate the effects existing among the various aggregates and geogrids combinations. All the geogrids tested in this study exhibited significant improvement in the rutting resistance for both river sand and gravel base courses.

The second test is to evaluate the abrasion resistance of portland cement pervious concrete (PCPC) using a LWT. Two other potential abrasion tests for PCPC, the

Cantabro abrasion and the loaded sweep abrasion tests, were also performed. PCPC mixtures made with two different aggregate sizes were tested with the above-mentioned tests. Based on the results, the following conclusions can be drawn:

1. LWT abrasion test was effective in evaluating the abrasion resistance of pervious concrete. The results from LWT abrasion tests were in good agreement with those from Cantabro abrasion tests. The results showed that using small grain-size aggregate and/or adding latex and fiber could improve the strength performance and abrasion resistance of PCPC.
2. LWT abrasion tests were able to identify the differences of abrasion resistance in various PCPC mixtures. The LWT abrasion test had the best repeatability and sufficient sensitivity among all the abrasion tests. Compared to LWT abrasion test, the Cantabro abrasion test showed relatively high variance, because of its excessive impact and severity.

This third test is to use LWT for characterizing the viscoelastic behavior of asphalt mixtures in tension condition. In addition to the LWT viscoelastic test, uniaxial viscoelastic tests in tension, tension-compression, and compression modes, and indirect tension creep test were conducted. Based on the results, the following conclusions can be drawn:

1. The results from different viscoelastic tests showed the identical ranking order for the asphalt mixtures used in the evaluation. The LWT viscoelastic test was validated to be able to characterize the viscoelastic properties of asphalt mixtures with different asphalt contents, aggregates, and asphalt binders.

2. Compared to other tests, the efficiency of LWT viscoelastic test was relatively high. Three specimens can be tested simultaneously. The fabrication of the specimens is relatively simple and convenient.

In the last, a LWT fatigue test was developed for characterizing the fatigue properties of asphalt mixtures. Direct tension fatigue test and flexural beam fatigue test were also performed on the same asphalt mixtures to validate the reliability of the LWT fatigue test. Based on the results, the following conclusions can be drawn:

1. LWT fatigue tests were able to differentiate the differences between the fatigue resistances of various asphalt mixtures. The results from LWT fatigue test were in consistent with those from flexural beam and direct tension fatigue tests.
2. The results from the fatigue tests clearly indicated that the mixtures made with higher grade of asphalt binder showed higher initial stiffness and a longer fatigue life. The mixtures made with higher asphalt content exhibited lower initial stiffness but a longer fatigue life.
3. Compared to the old version of APA fatigue test, the modified test proposed in this study was more reasonable to characterize the fatigue behavior of asphalt mixtures. In this modified test, theoretical approaches for modeling the fatigue behavior of asphalt mixtures are able to be adopted once stress and strain are known.

6.2 Recommendations

This study is focused on developing new or modified testing methods to investigate pavement materials. The four tests proposed in this study were validated to be feasible

and effective. However, these tests are not perfectly designed and refinements to these tests are still possible. Based on the experiences of the author, some recommendations are provided as follows:

1. In the LWT geogrids reinforcement test, the area on which the loaded wheels pass is relatively small compared to the apertures of geogrids and the grain-size of gravel aggregates. Wider wheels or pressured rubber hoses should be used to increase this area.
2. Cylindrical specimen with 150 mm in diameter and 100 mm in height is recommended for the Cantabro abrasion test. In some trial tests, smaller specimens (100 mm in diameter and 75 mm in height) were too weak to survive the collision during the testing. In order to mitigate the impact, a piece of rubber pad can be placed in the LA abrasion machine.
3. More modifications to the loaded wheels should be considered in the LWT abrasion test. In the current study, the studs soldered on the wheel were too small and the contact area was insufficient to cause obvious abrasion at low loading cycles. On the other hand, the abrading area caused by the steel wheel was too small for the specimen. However, a reduction in specimen size may cause ununiformity for the mixture. Therefore, increase in the steel wheel's width will be a better way to solve this problem.
4. In the LWT viscoelastic and fatigue tests, the following need to be considered for improvement:

- a. The data acquisition system and the data processing system are inconvenient and time consuming. Automatic data acquisition and data processing systems should be established.
- b. Accurate stresses are fundamental for the calculations of dynamic modulus, creep compliance, and stiffness. Through the simplification of the stress calculation from 3-D to 2-D, an error will arise in the computation of stress. Either wider wheels or narrower specimens should be used in the test to achieve more accurate results.
- c. Various stress amplitudes and temperatures for LWT fatigue tests should be considered. Using LWT fatigue test to investigate the fatigue behaviors of asphalt mixtures in water submerged condition could be a topic for future study. It is well known that water entering the cracks is likely to weaken the bonding strength and then accelerate the failure of asphalt mixtures.

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