THE HONG KONG POLYTECHNIC UNIVERSITY
DEPARTMENT OF CIVIL AND STRUCTURAL ENGINEERING

STUDY THE USE OF RECLAIMED ASPHALT PAVEMENT (RAP) WITH FOAMED BITUMEN IN HONG KONG

by

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(Temporary Binding for Examination Purposes)

A thesis submitted in partial fulfillment of the requirements for the Degree of Doctor of Philosophy

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CERTIFICATE OF ORIGINALITY

I hereby declare that this dissertation is my own work and that, to the best of my knowledge and belief, it reproduces no material previously published or written nor material which has been accepted for the award of any other degree or diploma, except where due acknowledgement has been made in the text.

SIGNED

____

He Gui-ping

DEDICATION

TO

Cui-xing, Jia-wei

**for all the patience,
understanding, encouragement,
support and constant love throughout the years**

ABSTRACT

In order to fulfill highway's sustainable development, it is necessary to utilize more and more reclaimed asphalt pavement (RAP) materials in the new road construction and road maintenance. This study was launched under this circumstance. The primary objectives of this research were to investigate properties of the cold-recycled RAP mixes stabilized by foamed bitumen (also called foamed asphalt mixes, or FA mixes) in the laboratory.

There are six main parts in this dissertation. Chapter 2 introduces a literature review of the cold recycling by foamed bitumen. Chapter 3 presents the study of the bitumen foamabilities under 140 combined testing conditions for Shell 60 bitumen and Shell 100 bitumen, including the foaming properties and decay functions. Water content and bitumen's viscosity significantly affect the decay line and foaming property of the bitumen. The combination of 5 bar pressure, 170 ^{0}C and 1.7% water content is determined as the optimum foaming condition (OFC) for Shell 60 bitumen. The OFC of Shell 100 is the combination of 3% water content, 5 bar pressure and 160 ^{0}C.

Chapter 4 discusses FA mixes' mix design. Two bitumens (Shell 60 and Shell 100) and two RAP materials (RAP#1 and RAP#2) were selected, and four RAP contents were considered in mix design. It is found that the optimum moisture condition (OMC) of FA mix decreases as RAP content increases. The design bitumen contents (DBCs) of the FA mixes were determined as 3.5% for the lower RAP content (0% and 20%), and 3.0% for the higher RAP content (40% and 60%) based on the soaked indirect tensile strength (ITS) test.

In Chapter 5, permanent deformation of FA mixes was evaluated in laboratory by repeated load axial creep (dynamic creep) test. As well as the ultimate strain, the creep strain slope (CSS) and secant creep stiffness modulus (SCSM) were initially used to investigate the susceptibility of FA mixes to permanent deformation. High bitumen grade may help FA mixes to reduce their susceptibilities to permanent deformation. Ageing of RAP and RAP content insignificantly affect susceptibilities of FA mixes to permanent deformation. Susceptibilities and creep strengths of FA mixes are better than those of the hot asphalt mixes.

Fatigue properties of FA mixes were analyzed and discussed in more detail in Chapter 6. The indirect tensile fatigue test (ITFT) was firstly adopted to test FA mixes in this study. Fatigue lives between FA mixes and the hot-rolled mixes were compared, characteristic of the fatigue failure and materials' effects on fatigue properties were analyzed, nonlinear characteristic of FA mix was discussed. Fatigue lives at 100 microstrains (N_{f100}) of FA mixes are far smaller than those of the hot-rolled mixes. FA mixes show an apparently violent facture at failure in the ITFT, substantially different from the fatigue failure characteristics of the hot-rolled mixes. The higher viscosity bitumen is more advantageous to N_{f100} of FA mix with/without RAP material than the lower-viscosity bitumen.

Moisture susceptibilities of FA mixes were firstly and comprehensively studied in Chapter 7. The soaked ITS test, the soaked dynamic creep test, the soaked ITFT test, and the freeze-thaw ITFT test were conducted in order to investigate and evaluate the moisture susceptibilities of FA mixes to permanent deformation, ITS and fatigue under the moisture conditions. All FA mixes meet the requirement of the moisture susceptibility to ITS in terms of Lottman's criterion. Under the soaked condition, harder bitumen (Shell 60) is statistically superior to the softer bitumen (Shell 100) in reducing the moisture susceptibility of FA mixes to permanent deformation. For the soaked FA mixes, when RAP content is small, the high penetration-grade bitumen is advantageous to fatigue lives of FA mixes; on the contrary, the low penetration-grade bitumen is advantageous to FA mixes' fatigue lives. Under the soaked or the freeze-thaw conditions, FA mixes containing 60% RAP#2 have poor moisture susceptibility to fatigue.

From tests and evaluations of permanent deformation, fatigue and moisture susceptibility of FA mixes presented in this study, it can be found FA mixes stabilized by the harder bitumen (e.g. Shell 60) and containing the less aged RAP materials (e.g. RAP#1) have better engineering properties and can be recommended as road base layer under the rainy condition in southern China. Group D has the best overall properties, which contains 60% RAP#1 and is stabilized with 3% Shell 60.

PUBLICATIONS ARISING FROM THE DISSERTATION

A. Published technical papers in international conferences

1. He Gui-ping, Wong Wing-gun. (2002). "Methodology of cold in-situ recycling of reclaimed asphalt pavement (RAP) using foamed bitumen in Hong Kong." *The 7th International Conference of Hong Kong Society for Transportation Studies -- Transportation in the Information Age*, HKSTS, December, 2002, Hong Kong, p199-207.

2. He Gui-ping, Wong Wing-gun. (2003). "Foamability of Shell Pen 60/70 and 100 Bitumen." *The 8th International Conference of Hong Kong Society for Transportation Studies – Transportation and Logistics*, HKSTS, December, 2003, Hong Kong, p74-82.

3. He Gui-ping and Wong Wing-gun. (2005). "Comparison of Foamability between Shell 60 and Shell 100." *The 10th International Conference of Hong Kong Society for Transportation Studies – Transportation and the Economy*, HKSTS, December, 2005, Hong Kong, p236-247.

B. Published technical papers in international journals

1. He Gui-ping and Wong Wing-gun. "Decay properties of the foamed bitumens." *Construction and Building Materials* (in press for publication).

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LIST OF ABBREVIATIONS

AADT	annual average daily traffic
AASHTO	American Association of State Highway and Transportation Officials
ALF	accelerated loading facility
ANVOA	analysis of variance
AP	air pressure
APA	asphalt pavement analyzer
APT	accelerated Pavement Tester
ARRA	Asphalt Recycling and Reclaiming Association
ASG	apparent specific gravities
ASTM	American Society for Testing and Materials
BC	bitumen content
BSI	British Standard Institute
C&D	construction and demolition
CIPR	cold in-place recycling
CSS	creep strain slope
CSSR	creep strain slope ratio
DBC	design bitumen content
DBM	dense base-coarse macadam
DOTs	Departments of Transportation
DSR	dynamic shear rheometer
EPD	Environmental Protection Department
ERMax	maximum expansion ratio
FA	foamed asphalt
FDR	full depth reclamation
FHWA	Federal Highway Administration
FL	fatigue life
FLR	fatigue life ratio
HIPR	hot in-place recycling
HKRRL	Hong Kong Road Research Laboratory
HKSAR	Hong Kong Special Administrative Region
HL	half-life
HMA	hot-mixed asphalt
HRA	hot-rolled asphalt
HyD	and Highway Department
ITFT	Indirect Tensile Fatigue Test
ITS	indirect tensile strength
ITSM	indirect tensile stiffness modulus
ITST	indirect tensile stiffness test
ITSR	indirect tensile strength ratio
LTT	Shell's Laboratory Test Track
LVDT	linear variable differential transformer
LWT	loaded wheel tester
MANVOA	multiple analysis of variance
MSA	million standard axle

MTSG	maximum theoretical specific gravity
NAT	Nottingham asphalt tester
NCHRP	National Cooperative Highway Research Program
OFC	optimum foaming condition
OMC	optimum moisture content
PAV	pressure aging vessel
PI	plasticity index
PRT	pavement rutting tester
RAP	reclaimed asphalt pavement
RLA	repeated loading test
RTFOT	rolling thin film oven test
SCSM	secant creep stiffness modulus
SCSMR	secant creep stiffness modulus ratio
SHRP	Strategic Highway Research Program
SPDM	Shell Pavement Design Method
SST	Superpave shear tester
SW	Scott Wilson
Temp	bitumen temperature
UCS	unconfined compressive strength
US	ultimate strain
USR	ultimate strain ratio
VFA	percent of voids in mineral aggregate that are filled with asphalt
VMA	percent of voids in mineral aggregate
WC	water content
WC20	a kind of gradation of foamed asphalt mix containing RAP materials and stabilized by the foamed bitumen
WTD	wheel-tracking device

LIST OF SYMBOLS

2α	angle at the origin subtended by the width of loading strip
AP	air pressure (bar)
B	slope of permanent strain
BC	bitumen content (%)
C_{Add}	percentage of lime or cement required (% by mass)
C_{mod}	compressive modulus
C_p	a statistic of best subsets regression procedure
C_w	corrected water of combined and oversize fractions
CI	Intercept of crossover point
CS	slope of crossover point
CSSR	creep strain slope ratio
CSS_{dry}	CSS of unconditioned specimens
CSS_{soaked}	CSS of conditioned specimens
$C\delta_D$	corrected unit dry weight of the total material (combined finer and oversize fractions)
D	dry density (kg/m^3)
DERMax	difference ERMax between 100 ^{0}C and 60 ^{0}C ($ERMax_{100}$-$ERMax_{60}$)
DHL	difference half-lives between 100 ^{0}C and 60 ^{0}C (HL_{100}-HL_{60})
ER_0	value of the asymptote
ER(t)	expansion ratio at the time *t*
$ER^{(1)}(0)$	the first order derivative of decay function at t=0
E_{mod}	extension modulus (MPa)
E_r	elastic modulus (MPa)
Factor	compensation for bitumen losses on mixing arm and bowl
FL_{dry}	fatigue life of unconditioned specimens
FL_f	fatigue live of freeze-thaw specimens
FL_{soaked}	fatigue live of soaked specimens
FL_{wet}	fatigue life of conditioned specimens
FLR	fatigue live ratio
FLR_f	fatigue live ratio of freeze-thaw specimens
FLR_s	fatigue live ratio of soaked specimens
G_M	bulk specific gravity
G_S	specific gravity of aggregate
G_{agg}	specific gravity of the aggregate
G_{asp}	specific gravity of asphalt mix
H	average height of briquette (cm)
ITS	indirect tensile strength (kPa)
ITSM	indirect tensile stiffness modulus (Mpa)
ITSR or TSR	indirect tensile strength ratio
LI	intercept of the lower line
LS	slope of lower line
Lat	latitude (degree)
MMC	moisture content of mixing and compaction (%)
M_{Add}	foamed bitumen content (% by mass)

$M_{Bitumen}$	mass of foamed bitumen to be added (g)
M_{Briq}	mass of briquette immediately after compaction (g)
M_{Cement}	mass of lime or cement to be added (g)
M_D	mass of the dry material (finer or oversize fraction) (g)
M_{DC}	mass of dry oversize fraction
M_{DF}	mass of dry finer fraction
M_M	mass of the moist material (finer or oversize fraction) (g)
M_{md}	mass of compaction mold (kg)
M_r	resilient modulus
MrR	resilient modulus ratio
M_{Sample}	dry mass of the sample (g)
M_t	mass of moist specimen and mold (kg)
M_{Water}	mass of water to be added (g)
N_f	cycles to failure
OMC	modified AASHTO OMC (%)
P	applied load (kN)
PF	percentage of fine (%)
P_C	percent of oversize fraction by weight
P_F	percent of finer fraction by weight (%)
P_L	vertically applied line loading, kN
P_{asp}	percent by weight of asphalt mix (aggregate basis)
$Q_{Bitumen}$ or Q_{Bit}	bitumen flow rate for the foamed bitumen plant (g/s)
Q_{Water}	water flow rate (l/h)
R	radius of specimen
$SCSMR$	secant creep stiffness modulus (SCSM) ratio
$SCSM_{dry}$	SCSM of unconditioned specimens (MPa)
$SCSM_{soaked}$	SCSM of conditioned specimens (MPa)
$Strain$	the accumulated permanent strain in micro-strains
$S_{\sec(35^0 C)}$	SCSM at 35 ^{0}C (MPa)
S_m	indirect tensile stiffness modulus at $\sigma_{x,mas}$ (kPa)
T	time to be set in the foaming testing (s)
T_{air}	air temperature (^{0}C)
T_d	temperature of layer located at d inches under pavement surface (^{0}C)
T_{surf}	temperature of pavement surface (^{0}C)
$Temp$	temperature (^{0}C)
UI	intercept of upper slope line
US	slope of the upper line
USR	ultimate strain ratio
US_{dry}	ultimate strain of unconditioned (dry) specimens (%)
US_{soaked}	ultimate strain of conditioned (soaked) specimens (%)
V	volume of compaction mold (m^3)
V_B	bitumen volume (%)
V_{Def}	vertical deformation at failure in the ITFT (mm)
V_V	air void volume (%)
W	water content of the respective finer or oversize fractions expressed as a decimal (%)
W_{add}	water content to be added to sample (% by mass)
W_{Moist}	moisture content of sample during compaction (% by mass)

W_{OMC}	optimum moisture content (% by mass)
$W_{Reduc.}$	reduction in water content (% by mass)
WC	added water content into the foamed bitumen (%)
a	width of loading strip (mm)
d	diameter of briquette (cm)
h	height of briquette (cm)
k	inverse of the slope of log strain (ε) vs. log repetitions of fatigue loading
$k1, k2$	material parameters of strain-fatigue equation
$k3, k4$	material parameters of stress-fatigue equation
$k7, k8$	coefficients of stress-strain equation
$k7', k8'$	coefficients of stress-strain equation (semi-log scale)
$k9, k10$	coefficients describing the nonlinear characteristics of the granular material
$k9', k10'$	coefficients describing the nonlinear characteristics of the foamed asphalt material
$logk1$	intercept of the fitted fatigue line in the log-log plot
p_m	average normal stress
t	time
t_0	time at which the maximum occurs
w	water content (%)
w_C	water content of oversize fraction expressed as a decimal
w_F	water content of finer fraction expressed as a decimal
w_{sat}	water content for complete saturation (%)
θ_1	parameters' vector of decay function of Shell-60
θ_2	parameters' vector of decay function of Shell-100
$\alpha_1, \beta_1, \gamma_1, \delta_1$	parameters of decay function of Shell 60, and $-1< \gamma_1<1$
α_2, β_2	parameters of decay function of Shell 100, and $\beta_1>1$
γ_d	dry unit weight of compacted specimen or of aggregate, (kN/m^3)
γ_w	unit weight of water, 9.789 kN/m^3 at 20 ^{0}C
δ_F	dry unit weight of the finer fraction
δ_w	unit weight of water (9.802 kN/m^3)
ε	tensile bending strain due to wheel loads
$\varepsilon_{(35^0 C,600)}$	axial strains at 35 ^{0}C caused to the specimen after 600 load cycles
$\varepsilon_{(35^0 C,1800)}$	axial strains at 35 ^{0}C caused to the specimen after 1,800 load cycles
ε_t	initial strain (microstrain)
$\varepsilon_{x,\max}$	maximum tensile strain at the center of the specimen
θ	principal stress
ρ_d	dry density of compacted specimen (Mg/m^3)
ρ_m	moist density of compacted specimen (Mg/m^3)
σ_0	applied stress (kPa)
σ_x	indirect tensile stress (horizontal) at the center of the specimen
$\sigma_{x,mas}$	maximum tensile stress at the center of the specimen (kPa)
σ_t	magnitude of stress repeatedly applied at the centre of specimen

σ_y	indirect compressive stress (vertical) at the center of the specimen
ν	Poisson's ratio

CHAPTER ONE INTRODUCTION

1.1 BACKGROUND

Transport, both of people and of goods, is a vital essence of any modern economy. Not only does it directly provide employment, it also contributes to the Gross National Product. More important, it enables the high degree of specialization and economy-of-scale that characterizes modern society, as opposed to the local economy of craftsmanship of past centuries. Road system is a key part of the infrastructure. It plays a significant role in the transportation system, and is very important for a nation's economy.

To keep Hong Kong moving in the past ten years, Hong Kong has been spending huge sums of money every year in expanding the transportation network and maintaining this valuable asset of Hong Kong people. The financial provisions for capital road projects and road maintenance of the Highways Department are presented in Table 1.1 (Wong and Hung 2001). It is clear that the expenditure for both capital project and maintenance on highways has been increasing all the time.

Table 1. 1 Financial provisions for capital road projects and road maintenance

Financial year	1997-98	1998-99	1999-2000
Capital projects ($m)	267.5	307.5	318.2
Maintenance ($m)	874.8	922.1	1007.6

There are two basic types of roads in Hong Kong, i.e., the concrete and bituminous roads. Bituminous roads have been widely used in Hong Kong because of the fast construction and ease of maintenance. The aggregates used for road construction mainly come from local quarries but these quarries have been gradually exhausted and will be closed in the next decade or two due to environmental protection. Unfortunately, the cement and bitumen are mainly imported from overseas.

Road construction and roadwork are part of daily life of the Hong Kong people. Roads keep Hong Kong moving but also generate numerous growing problems, which include resource depletion, waste generation, accident, noise pollution and traffic disruption leading to economic losses to the community.

In Hong Kong, huge solid waste and reusable materials are generated every year. There is an increasing trend in the waste generation. In 2001, 83 percent of Construction and Demolition (C&D) material (9.3 tonnes per day) was reused each day as fill material for reclamation and earth filling projects, while the rest (6.41 tonnes per day) was disposed of at landfills (HKSAR 2001). Fig. 1.1 demonstrates the quantities and percentages of inert C&D material delivered to public filling areas and C&D waste disposed of at landfills.

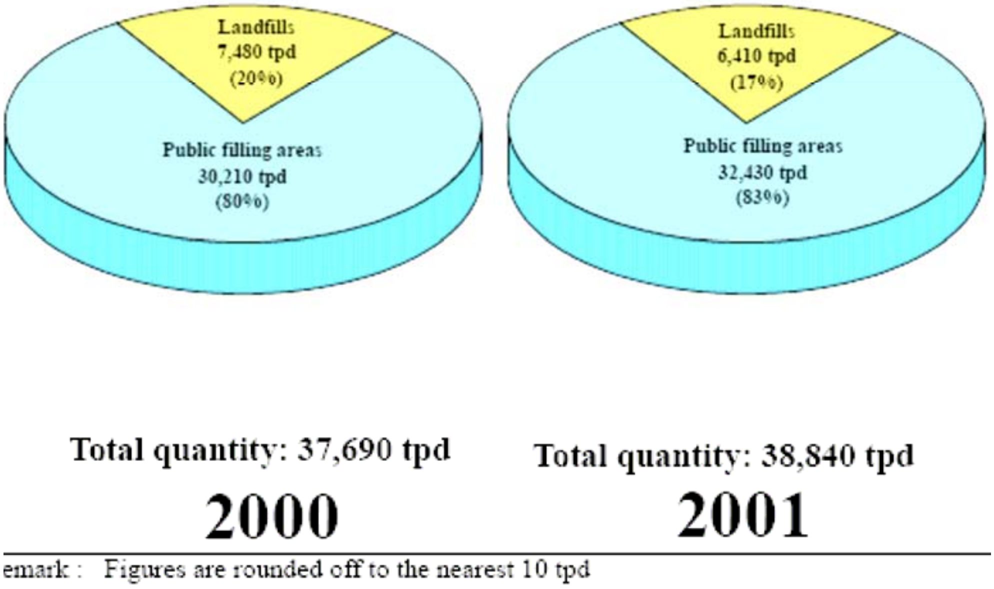


Fig.1. 1 Quantities and percentage of inert C&D material delivered to public filling areas and C&D waste disposed of at landfills

In 1999, Hong Kong Federation of Trade Unions (1999) organized a survey of public opinion on waste recycling/reuse. The result showed that 78.9% of people regarded the best way to handle waste was to recycle or reuse. Majority of people disagreed to use cremation and landfills.

In order to have a effective operation of transport, Hong Kong spends large amount of money on road maintenance every year. As of 1999, the total road area maintained by the Highway Department is 21.7 million square meters. 200,000 tonnes of asphalt mixture have been milled from the maintenance work of the existing highway every year. It is believed that road wastes will increase every year in the future. Wastes generated from road construction and maintenance constitute a significant component of C&D in Hong Kong. And this large volume of wastes has exerted huge pressure on the limited landfill spaces.

Considering the enormous expenditure of the road network, and pressure from the environmental protection and shortage of raw materials in Hong Kong, it is time to handle the problem of road C&D wastes.

Recycling or reuse of these road wastes is an economic and technically feasible method as there is considerable retaining value for the wastes. Pavement recycling and reuse of reclaimed asphalt pavement (RAP) materials as a component in new HMA pavements have greatly increased since the mid-1970’s largely due to the oil embargo as well as decrease in the availability of good quality aggregates (Lee et al 1999). These factors led highway agencies to look for ways to reduce pavement rehabilitation costs by recycling. There are five primary types of recycling method for bituminous RAP materials: (1) cold planing; (2) hot recycling; (3) hot in-place recycling (HIR); (4) cold in-place recycling (CIR); and (5) full depth reclamation (Kandhal and Mallick 1997).

Encouraged by the Environmental Protection Department (EPD) and Highway Department (HyD) of Hong Kong Special Administrative Region (HKSAR) and funded by HyD of Hong Kong Special Administrative Region (HKSAR), a research was launched to study the use of RAP materials in Hong Kong (Wong 2001, Elliott 2002, Chan 2002). Scott Wilson (SW) (Hong Kong) Ltd was designated to carry out a preliminary investigation into asphalt pavement recycling for Hong Kong' paving industry.

Compared with HIR, the advantages of cold in-place recycling include significant structural treatment of most pavement distress, improvement of ride quality, minimum hauling and air quality problems, and capability of pavement widening (OECD 1997).

Foamed bitumen is produced by a process in which water is injected into the hot bitumen, resulting in spontaneous foaming. It can be used as a stabilizing agent with a variety of materials ranging form good quality crushed stone, marginal gravels with relatively high plasticity to RAP material. Comparing with other stabilizing agent, foamed bitumen has many advantages. Among commonly used stabilizing agents, many studies confirm that foamed bitumen plus cement has best effect for cold in-place recycling according to the pavement performance and economy (Loudon and Partner 1999, Vorobieff and Preston 2004, Ramanujam and Jones 2000).

Bituminous mixture stabilized by foamed bitumen (also call foamed asphalt) has many advantages: including construction in adverse weather condition, stockpile for much extended periods, saving time in construction, strength characteristics between cemented materials and flexible materials, etc. Cold recycling by foamed bitumen is a sustainable and economic way. (Muthen 1998, AustStab 1998)

Hence some of paving contractors is interested in this cold recycling method, and looks forward to applying recycling RAP material in the local industry. This research was launched under this situation.

In Hong Kong, However, cold recycling is very new. Before using the RAP material in the paving project, it is necessary to investigate the current design procedures and performance of the mixtures which contain RAP material stabilized by the foamed bitumen.

1.2 OBJECTIVES AND SCOPE OF THIS STUDY

Foamed asphalt (FA) mix has never been studied in Hong Kong. Hence this research is firstly carried out, and there was no local experience.

This research focuses on providing a FA mix design for application of RAP recycled by foamed bitumen and studying the properties of FA mixes. There are three main tasks in this study: (1) systematic investigation of the bitumen foamability, which is the basis of FA mix design; (2) a FA mix design catered to Hong Kong's situation (weather, traffic, bitumen and RAP source); (3) systematic testing and analysis of the FA mixes' properties, including resistance to permanent deformation, material strength, fatigue resistance, and moisture susceptibility.

As a preliminary study of the use of RAP material, the mix design and testing of mix properties are limited to in the lab of Hong Kong Road Research Laboratory (HKRRL), The Hong Kong Polytechnic University. A field study is not within the scope of the research.

1.3 ENVIRONMENTAL ISSUES AND SIGNIFICANCE

If RAP is successfully applied for road paving in Hong Kong, benefits as following will be provided:

- It can reduce the demand for primary aggregates; therefore conserve resource and money;
- It can reduce energy costs related to the extraction and transport of conventional aggregates;
- It can reduce environmental costs associated with protecting and recovering the conventional aggregate quarrying, diminish negative effects of noise and dust on residents and air;
- It can reduce environmental and economic problems associated with wastes storage and dumping, and save precious land resources in Hong Kong;
- It can provide probable commercial benefit from the use of waste materials, since they are already financed by other industrial processes that generate them and, if not used, they have to be stocked or disposed of, thereby incurring extra costs.

It is undoubted that recycling of RAP will take an important role in reducing land occupation, protecting environment, saving materials and money for pavement construction, and conserving energy.

This research is a systematic study on the use of RAP recycled by foamed bitumen in Hong Kong. It is not only of theoretical value, but also of practical value for local paving industry. It is also compatible with Environment Policy of EPD. It will provide a foundation for use of the RAP material by cold-recycled method.

1.4 ORGANIZATION OF THIS DESSERTATION

This dissertation is divided into four main parts:

- The general part, consisting of the Chapters 1, 2 and 8, outlining the literature and framework of this study and its conclusion;
- The second part (Chapter 3), describing the foamability study;

- The third part (Chapter 4), presenting the FA mix design;
- The fourth part (Chapters 5 to 7), concentrating on the property study of FA mix, including permanent deformation properties, fatigue properties and moisture susceptibility;

Chapter 2 will introduce the state-of-the -art of the cold recycling pavement, including research and application of FA mix. This chapter will also outline the framework and approach of the study.

Chapter 3 describes the foamability study of foamed bitumen, including the foaming properties and decay property of the bitumens which were used in this study. The optimum foaming condition of bitumens, which is used to guide the production of the FA mixes, was determined based on the analysis for test result.

Chapter 4 presents the consideration of FA mix design, gives the basic properties tested of the raw and RAP materials, describes the method of FA mix design in detail, discusses the FA mixes' volumetric and mechanics properties. The laboratory procedures for FA mix design and the optimum moisture content (OMC) testing are enclosed in this chapter.

Chapter 5 discusses the experimental and evaluation methods for properties of permanent deformation, determines the testing methodology for evaluating the FA mixes' permanent deformation, analyzes the testing results in terms of ultimate axial strain, creep strain slope, secant creep stiffness modulus, and intercept. Comparison of permanent deformation between the FA mixes and the hot asphalt mixes is also carried out in this chapter.

Chapter 6 presents a literature review of fatigue property study, determines the fatigue testing for the FA mixes, describes in detail the analysis of the testing results, including FA mixes' fatigue laws, effects of materials on fatigue properties, characteristics of deformation, fatigue failure and nonlinearity of the FA mixes in the indirect tensile fatigue testing,

Chapter 7 firstly introduces the literature review for evaluation of the moisture susceptibility, then describes the experimental program for moisture susceptibility, finally presents results of analysis and evaluation, including moisture effects on the indirect tensile strength, permanent deformation and fatigue properties.

Chapter 8 presents the summary, conclusions and recommendations of this study.

CHAPTER TWO COLD RECYCLING BY FOAMED BITUMEN: THE STATE-OF-ART AND APPROACHES OF THE STUDY

2.1 INTRODUCTION

Recycling in road paving is the reuse of materials removed from previous pavement structures, such as recycling of asphalt pavement, Portland cement concrete, and various base course materials. Recycling may also involve using materials that do not originate from the existing pavements; examples are slags, ashes, various polymer materials, and rubber from scrap tyres (Shoenberger 2000, Delwar et al 1997, Taha et al 1998, Kandhal and Mallick 1999).

Our existing highway system is an extremely valuable resource containing many millions of tones of high quality bitumen and aggregates. When hot-mixed asphalt (HMA) pavements reach the end of their usable service lives, the materials in them retain considerable value.

In the field of recycling bituminous concrete pavement, Neville, C., Lombard, E. L., and Roster F. S. were among the pioneers. The first printed mention of recycling was in Warren Brother's portable asphalt plant sales brochure of 1915 (Gannon et al 1980). Reclaimed asphalt pavement (RAP) material is old pavement material that is milled up or ripped off the roadway. This material removed during most road resurfacing and reconstruction projects can be reused in new asphalt mix. Using RAP in new mix can conserve the amount of new material that has to be added, save money, eliminate disposal problems, and lower fuel consumption. However, the total quantity of pavement materials recycled by all methods from 1915 to 1975 was small in comparison to the amount that has been recycled since 1975. Pavement recycling and reuse of RAP materials as a component in new HMA pavements have greatly increased since the mid-1970's largely due to the oil embargo as well as decrease in the availability of good quality aggregates (Lee et al 1999). These factors led highway agencies to look for ways to reduce pavement rehabilitation costs by recycling. In 1981 the Federal Highway Administration (FHWA) issued its policy statement that "Recycling should be one of the options considered at the design state on all rehabilitation projects." (Kearney 1997).

In Japan, research on the recycling of materials used for pavement dates back to about 1950, and full-scale technical development began in the 1970's. In 1991, half of cement concrete lumps and asphalt concrete lumps, about 21.1 million tonnes, were recycled (Nakamura 1995). In Canada, old asphalt material has been used to recycle in more than 10 provinces, 3 provinces have their own specifications of asphalt pavement recycling; other wastes, such as blast furnace slag, fly ash, steel slag, scrap tyres and etc have been widely used in pavement industry (Emery 1993). Compared with Canada, there are 42 states that used reclaimed asphalt pavement as road material and 38 states that recycle scrap tyres (Tarricone 1993). At least half the state legislatures in USA have instructed their Departments of Transportation (DOTs) to investigate recycling opportunities in transportation projects. Some state environmental agencies have encouraged or offered incentives to their counterparts at DOT to reuse wastes. In Europe, environmental laws prohibit the unregulated dumping of RAP, many European countries use RAP as material for road rehabilitation. In the United Kingdom, for example, reuse of RAP has been prompted by the introduction of the landfill tax in October 1996. The standard rate of £10/t was subject to a landfill tax escalator of £1/t/year for at least another five years, reaching £15/t in 2004 (Woodside et al 2000). In order to guide design and construction of pavement, Highways Agency provides the specification, "Design Guide and Specification for Structural Maintenance of Highway Pavements by Cold In-situ Recycling." (Milton and Earland 1999). Recycling of existing pavement materials has become an increasingly important feature of the maintenance of highway in UK.

Recycling of existing asphalt pavements materials to produce new pavement materials results in considerable savings of material, money, and energy. The specific benefits of recycling can be summarized as follows (Epps 1976):

- When properly used, recycling can result in substantial savings over the use of new material. Also, the cost of haulage can be avoided if recycling is performed in-place. The need for economic consideration is felt now more than ever, because of tightening budgets and ever increasing cost of material.
- Recycling can help in conservation of natural resources by reducing the need for new materials. This translates to substantial savings in aggregate resources and demand for asphalt binder, especially during supply interruptions.
- Recycled materials have proven to be equal or even better than new materials in quality. A hot-mixed asphalt (HMA) overlay on recycled base is expected to perform better than an HMA overlay on the existing surface, even though they have the same thickness, because the former can substantially reduce the potential of reflective cracking.
- Recycling can maintain pavement geometrics as well as pavement thickness. The existing pavement structure can be strengthened by recycling without adding substantial overlays.
- Recycling can save a considerable amount of energy compared to conventional construction techniques.

Over the years, recycling has become one of the most effective and attractive pavement rehabilitation alternatives.

2.2 RECYCLING METHODS AND STRATEGIES

2.2.1 Recycling Methods

Different recycling methods are now available to address specific pavement distress and structural needs.

There are five primary recycling methods for bituminous RAP materials identified by the Asphalt Recycling and Reclaiming Association (ARRA): (1) cold planing; (2) hot recycling; (3) hot in-place recycling; (4) cold in-place recycling; and (5) full depth reclamation. These methods are briefly introduced as follows (Kandhal and Mallick 1997):

1. Cold planing

Cold planing is described as an automatic method of removing asphalt pavement to a desired depth and restoration of the surface to a desired grade and slope and free of humps, ruts and other distresses. This method can be used for the roughening or texturing of a pavement to improve frictional resistance. Cold planing is performed with a self-propelled rotary drum of cold planing machine with the RAP transferred to trucks for removal from the job site. The resulting pavement can be used immediately by regular traffic and overlaid at some future time or left as a textured surface.

2. Hot recycling

Hot recycling or hot mix recycling is the process in which RAP material is combined with new materials, sometimes along with a recycling agent, to produce HMA mixtures. Both batch and drum type hot mix plants are used to produce recycled mix. The RAP material can be obtained by milling or ripping and crushing operation. The mix placement and compaction equipment and procedures are the same as those for regular HMA. Typical RAP to new aggregate ratio varies between 10:90 and 30:70 with a maximum of 50:50 (drum plant). The advantages of hot mix recycling include significant structural improvement, equal or better performance compared to conventional HMA, and capability to correct most surface defects, deformation, and cracking.

3. Hot in-place recycling (HIPR)

HIPR consists of a method in which the existing pavement is heated and softened, and then scarified/milled to a specified depth. New HMA (with/without RAP) and/or recycling agent may be added to the scarified RAP material during the recycling process. HIPR can be performed either as a single pass or as a multiple pass operation. In single pass operation, the scarified in-place material can be combined with new material if needed or desired. In multiple pass operation, the restored RAP material is recompacted first, and a new wearing surface is applied later. The depth of treatment varies between 20 to 50 mm. ARRA has identified three HIPR processes: (1) surface recycling, (2) repaving, and (3) remixing. The advantages of hot in-place recycling are that surface cracks can be eliminated, ruts and shoves and bumps can be corrected, aged asphalt is rejuvenated, aggregate gradation and asphalt content can be modified, traffic interruption is minimal, and hauling costs are minimized. The major disadvantage of hot in-place recycling is that it cannot make significant changes to the mix. As a result, hot in-place recycling cannot apply to the pavements that exhibit structural base failure, irregular patching or the need for major drainage or grade improvements.

4. Cold in-place recycling (CIPR)

CIPR involves reuse of the existing pavement material without the application of heat. Except for any recycling agent, no transportation of materials is usually required, and aggregate can be added, therefore hauling costs is very low. Normally, an asphalt emulsion is added as a recycling agent or binder. The emulsion is proportioned as a percentage by weight of the RAP. Fly ash or cement or quicklime may also be added. These additives are effective for over aged and low stability mixes. The process includes pulverizing the existing pavement, sizing of the RAP, and application of recycling agent, placement, and compaction. This process is performed by a train of equipment. The use of the recycling train, which consists of pulverizing, screening, crushing and mixing units, is quite common in USA. The processed material is deposited in a windrow from the mixing device, where it is picked up, placed, and compacted with conventional hot mix asphalt laydown and rolling equipment. The depth of treatment is typically from 75 to 100 mm. The finished product is generally used as a stabilized base, which would be overlaid with an asphalt surface course. The advantages of cold in-place recycling include significant structural treatment of most pavement distress, improvement of ride quality, minimum hauling and air quality problems, and capability of pavement widening.

5. Full depth reclamation (FDR)

FDR has been defined as a recycling method where all of the asphalt pavement section and a predetermined amount of underlying base material are treated to produce a stabilized base course. It is basically a cold mix recycling process in which different types of additives such as asphalt emulsions and chemical agents such as calcium chloride, Portland cement, fly ash, and lime, are added to obtain an improved base. The four main steps in this process are pulverization, introduction of additive, compaction, and application of a surface or a wearing course. If the in-place material is not sufficient to provide the desired depth of the treated base, new materials may be imported and included in the processing. New aggregates can also be added to the in-place material to obtain a particular gradation of material. This method is normally performed to a depth of 100 to 300 mm. The advantages of full depth reclamation are that most pavement distresses are treated, hauling costs are minimized, and significant structural improvements can be made.

2.2.2 Strategies

All of the different recycling techniques offer some advantages over conventional rehabilitation techniques. However, the choice of a particular recycling method should be primarily on the basis of the type of distress shown in the existing pavement. This is because all of the recycling methods are not equally suited for treating different types of distress, and here the choice must be made for the particular method which is capable of rectifying the existing distress conditions. The applicability of a particular recycling technique not only depends on the pavement defect, but also on the extent and severity of the distress. For this reason, a comprehensive evaluation of the existing pavement is necessary before attempting any recycling process. Once the type, extent and diversity of distresses have been identified, candidate-recycling procedures should be evaluated on the basis of their effectiveness and cost. The following primary types of distresses have been recognized by ARRA: (1) surface defects, (2) deformation, (3) cracking, (4) maintenance patching, (5) base/subgrade problems (ARRA 1992).

Based on these distresses, the ARRA recommends Table 2.1 as a guideline for selecting a recycling alternative. This table shows that hot mix recycling can be used to treat all types of distresses except problem base or subgrade. HIPR can be used for all but reflect cracks, maintenance patching, and problem base or subgrade. CIPR and FDR are capable of treating rutting, cracks, and maintenance patches. Only FDR can be used for rectifying problem base and/or subgrade. The use of CIPR recycling and FDR are not shown for treating surface defects. However, for these two methods, as well as for multiple pass method of HIPR, a new surfacing is generally required. The type of surfacing depends on the amount and type of traffic. An asphalt surface treatment such as a chip seal, slurry seal or cape seal may be adequate for light traffic, but a HMA overlay should be used for heavier traffic.

Table 2. 1 Guide for selection of recycling method (ARRA 1992)

Type of Pavement Distress[1]	Hot Recycling	Hot In-Place Recycling	Cold In-Place recycling	Full Depth Reclamation
Surface Defects				
Raveling	X	X[2]		
Bleeding (flushing)	X	X[5]		
Slipperiness	X	X[2]		
Deformation				
Corrugations (washboarding)	X	X[5]		
Rutting-shallow[3]	X	X[5]		
Rutting-deep[4]	X		X[6]	X[6,7]
Cracking/Load Associated				
Alligator	X		X	X
Longitudinal-wheel path	X	X[8]	X	X
Pavement edge	X		X	X
Slippage	X	X[9]		
Cracking/Non-Load Associated				
Block (shrinkage)	X		X	X
Longitudinal-joint	X	X[10]		
Transverse (thermal)	X		X	X
Reflection	X		X	X
Maintenance Patching				
Spray	X[11]		X[11]	X
Skin	X[11]		X[11]	X
Pothole	X		X	X
Deep (hot mix)	X		X	X
Problem Base/Subgrade (Soft, Wet)				X
Ride Quality/Roughness				
General unevenness	X	X		
Depressions (settlement)	X[12]	X[12]		X[13]
High spots (heaving)	X[12]	X[12]		X[14]

Notes:

[1] A pavement in which asphalt mixtures are used for all course above the subgrade or an improved subgrade having Portland cement, lime, lime-fly ash, fly ash or calcium chloride modification.
[2] Applicable if the surface course thickness does not exceed 1.5 inches.
[3] Rutting is limited to the upper portion of the pavement structure (top 1.5-2 inches).
[4] Rutting is originating from the lower portion of the pavement (below surface course and includes base and subgrade).
[5] May be a temporary correction if entire layer affected not removed or treated by the addition of special asphalt mixtures.
[6]The addition of new aggregate may be required for unstable mixes.
[7] The chemical stabilization of the subgrade may be required if the soil is soft, wet.
[8] Applicable if the cracking is limited to the surface course of the pavement.
[9] Applicable if the treatment is to a depth below the layer where the slippage is occurring.
[10] Applicable if the cracking is limited to the surface course of the pavement.
[11] In some instances, spray and skin patches may be removed by cold planning prior to these treatments (considered if very asphalt rich, bleeding).
[12] May be a temporary correction if the distress related to a subgrade problem.
[13] Used if depressions due to a soft, wet subgrade condition.
[14] Used if the high spots caused by frost heave or swelling of an expansive subgrade soil.

2.3 COLD RECYCLING

Cold in-place recycling processes, typically, produce the structural course (e.g. the roadbase) and the foundation platform (also known as the sub-base) in one operation. It is apparent that cold in-place recycled asphalt is effective in mitigating reflection cracks, and few problems with rutting were observed. Subgrade instability is found to be the only cause of complete failure in a recycling project. Jahren et al (1999) pointed out that the cold in-place recycled asphalt roads in Iowa were performing well. The expected service lives of these investigated roads, based on the first 10 years of performance, were predicted to be 15 to 26 years.

2.3.1 Cold Recycling Process

Recycling machines have evolved over the years from modified milling machines and soil stabilizers to the specialized recyclers of today. As they are specifically designed to have the capability of recycling a thick pavement layer in a single pass, modern recyclers tend to be large, powerful machines, which may either be track mounted, or mounted on high flotation pneumatic tyres.

The heart of these machines is a milling/mixing drum equipped with a large number of special cutting tools. The drum rotates, milling the material in the existing road pavement, as illustrated in Fig. 2.1 (Wirtgen GmbH 2001).

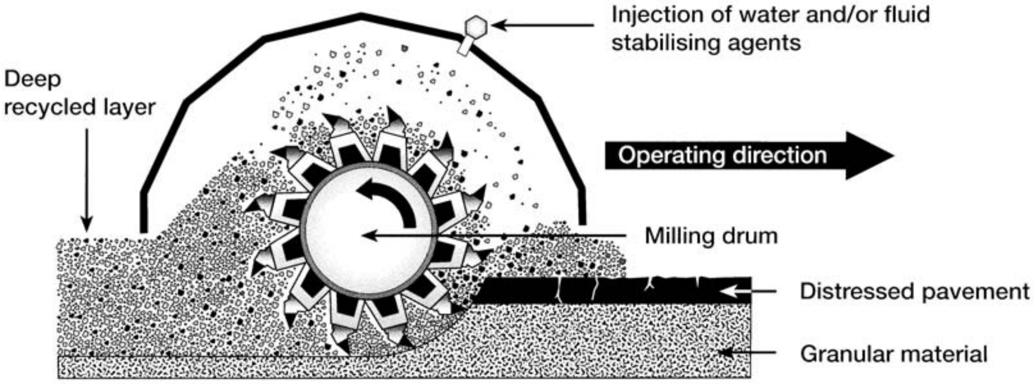


Fig.2. 1 Configuration of the milling/mixing drum and spray bar systems

As the milling process is taking place, water from a tanker coupled to the recycler is delivered through a flexible hose and is sprayed into the recycler's mixing chamber. The water, which is metered accurately through a microprocessor controlled pumping system, is mixed thoroughly together with the milled material to achieve optimum compaction moisture content.

Fluid stabilizing agents, like cement/water slurry or bitumen emulsion, either individually or in combination, can also be introduced directly into the mixing chamber in a similar manner. In addition, foamed bitumen may be injected into the mixing chamber, through a separate specially designed spray bar.

Powdered stabilizing agents, such as Portland cement, are normally spread onto the existing road surface ahead of the recycler. The recycler passes over the powder, mixing it, together with the water, into the underlying material in a single operation.

Recycling trains may be configured differently, depending upon the recycling application and the type of stabilizing agent that is used. In each case the recycling machine acts as the locomotive, and either pushes or pulls the equipment that is coupled to it, by means of pushbars or drawbars. Typical recycling trains are illustrated in Fig. 2.2 and 2.3. When bitumen emulsion is used together with cement a recycling train can be configured as illustrated in Fig. 2.2. Two typical recycling trains that are used when recycling with foamed bitumen are shown in Fig. 2.3.

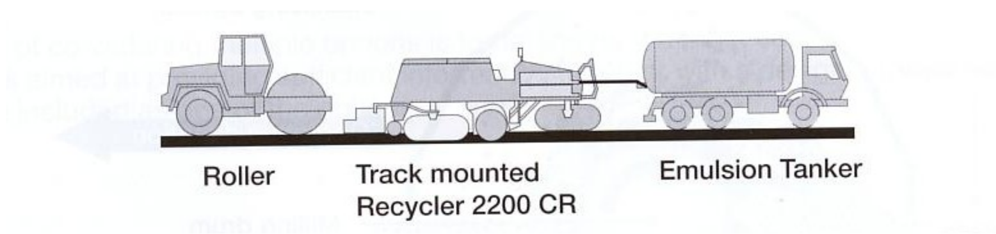


Fig.2. 2 Recycling train with bitumen emulsion using track mounted recycler

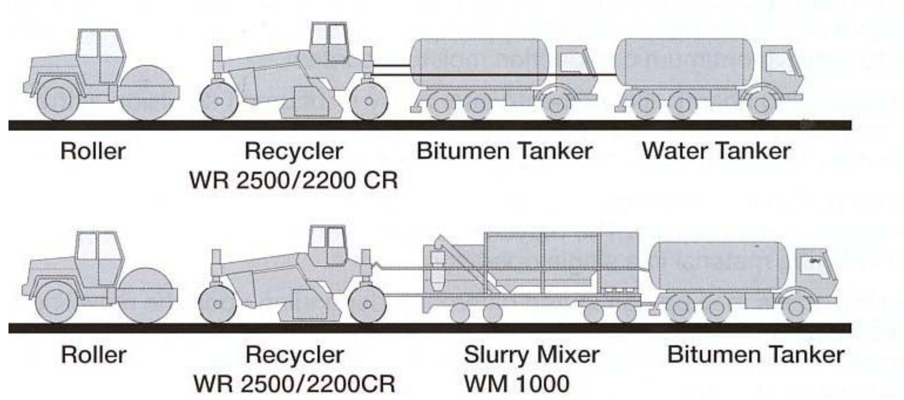


Fig.2. 3 Typical recycling trains using foamed bitumen, along and in combination with cement

When foamed bitumen is used on its own, as may be done when recycling a pavement that includes asphalt and/or good-quality crushed stone, the recycler pushes two tankers ahead of it, firstly a tanker filled with hot bitumen and then a water tanker.

When foamed bitumen and cement are used in combination, the cement can be added either in slurry form, using a slurry mixer, or spread as a powder in the existing road surface ahead of the recycling train.

2.3.2 Structural Design Of Cold Recycled Pavement

The method of structural design, which determines the required strength for a pavement structure, has evolved from an empirical to a semi-mechanistic procedure. There are some typical design methods for cold recycled pavement, such as AASHTO method, Asphalt Institute method, and UK method.

In hot-mix recycled asphalt pavement design, the AASHTO method is based on the derivation of the structural number required for the pavement with the help of design traffic, reliability level of prediction of traffic and performance, performance period, and the pavement condition rating. The structural number can be expressed as the sum of the product of the depth, layer coefficient, and drainage coefficient of each of the layers. The AASHTO design method for cold recycled mixes is similar to the design method for the hot-mix asphalt (AASHTO 1986). However, layer coefficients for cold-recycled mixes are dependent on construction methods, and should be determined on the basis of engineering judgment.

The Asphalt Institute method (Asphalt Institute 1986) assumes the pavement as a multi-layered elastic structure, and determines the required thickness on the basis of design traffic and subgrade strength. The combined thickness of cold-recycled base and surface course is obtained from charts. The thickness of cold-recycled bases can be obtained by taking into consideration the recommended thickness of hot mix asphalt overlay on the cold-recycled base.

In the UK pavement structure is designed according to the road to be recycled. This depends on the amount of commercial traffic to be carried in one lane, or one direction, over a 20 year design life. The categories are given in Table 2.2 (Milton and Earland 1999).

Table 2. 2 Road type categories (Milton and Earland 1999)

Road type category	Traffic design standard (million standard axles)
1	More than 10 up to 30
2	More than 2.5 up to 10
3	More than 0.5 up to 2.5
4	Up to 0.5

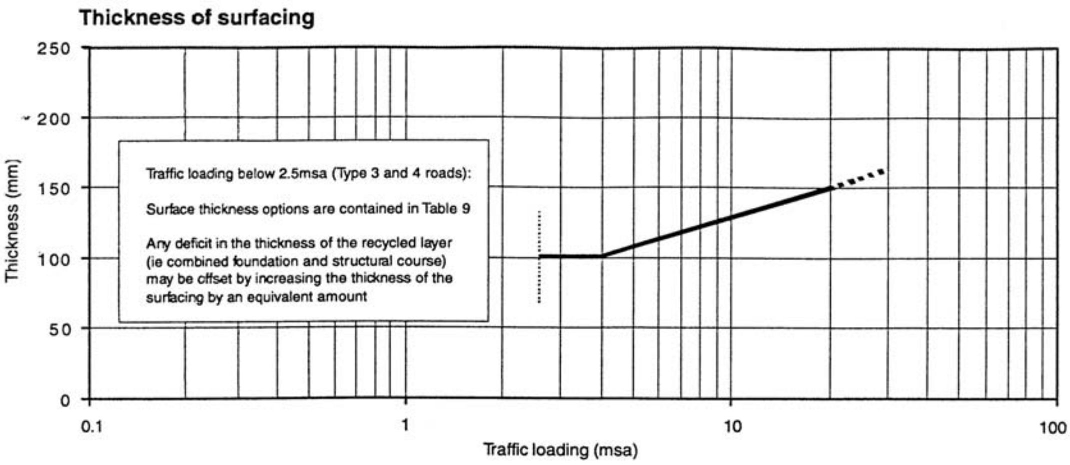
The modulus tested by indirect tensile stiffness modulus test of the recycled structural pavement layer must meet the requirement of 2000 MPa for Type 4 road and 2500 MPa for Type 1, 2, and 3 roads. Thickness of pavement layer can be obtained from design charts. The recommended depth of cold in-place recycling for the structural course for road category Types 1 and 2 are given in Fig. 2.4 and Fig. 2.5 for cement bound and foamed bitumen bound constructions respectively (Milton and Earland 1999).

2.3.3 Design Of Cold In-Place Recycled Material

The nature and grading of the aggregate produced by pulverization using any of the recycling machines currently available will depend largely on the nature, thickness and proportions of the existing road material to be recycled.

2.3.3.1 Grading

Ideally, the particle size distribution of the pulverized aggregate, immediately prior to stabilization, including any supplementary aggregate and/or filler, should be equivalent to a uniformly graded aggregate of 50 mm nominal size, complying with the Zone A grading envelope described in Table 2.3. In certain circumstances, pulverized aggregate falling within Zone B grading envelope may be acceptable, provided the results of mixture design tests demonstrate that an acceptable recycled material can be produced consistently using this aggregate.



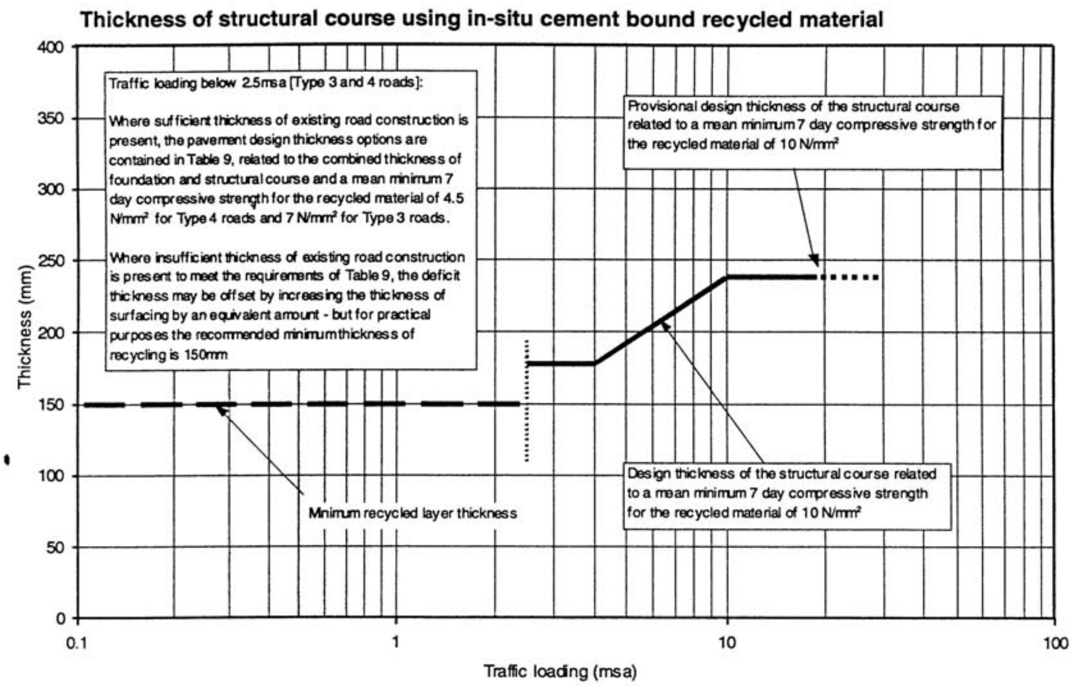


Fig.2. 4 Pavement layer thickness using in-place cement bound recycling (Milton and Earland 1999)

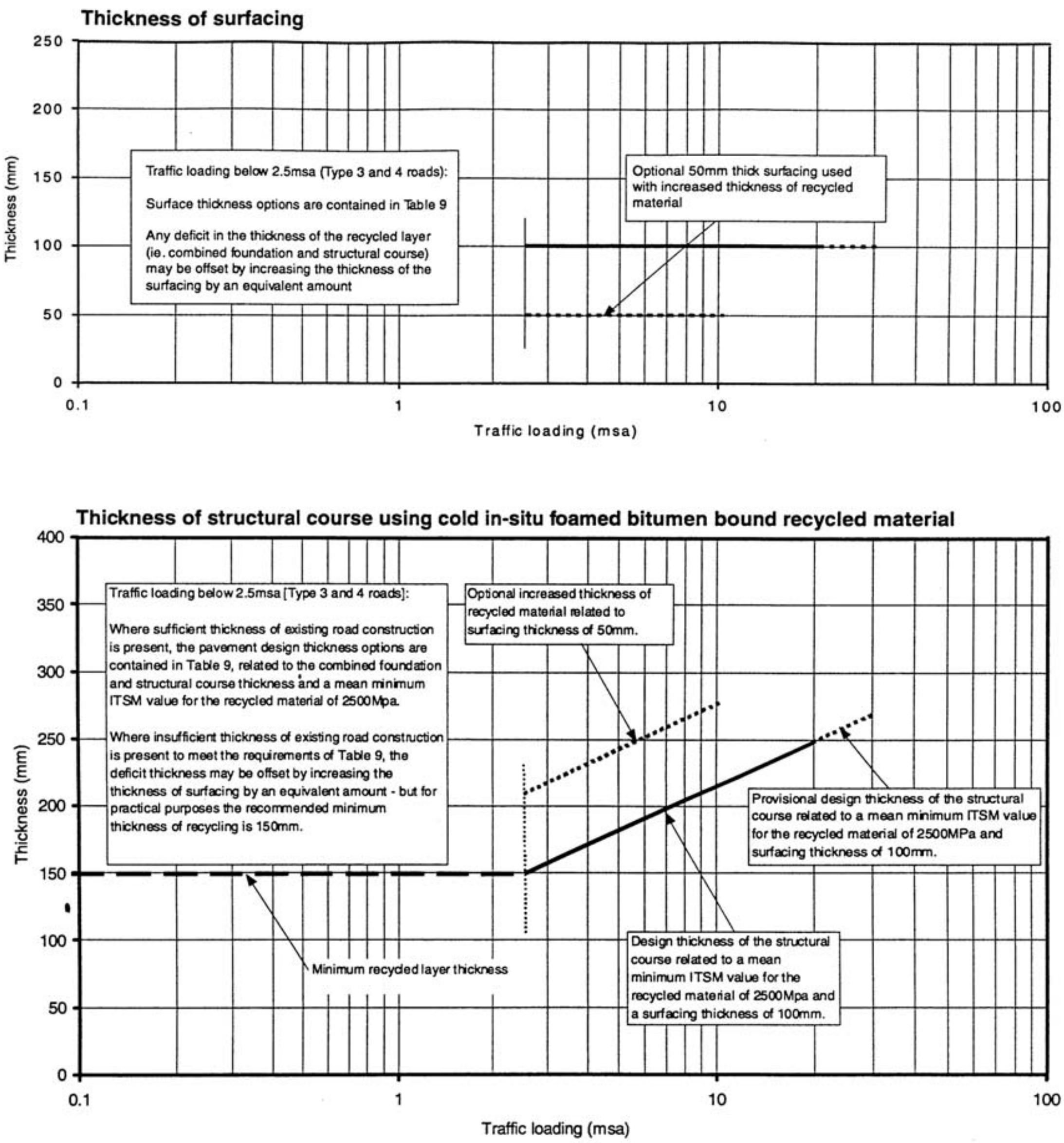


Fig.2. 5 Pavement layer thickness using cold in-place foamed bitumen bound recycling (Milton and Earland 1999)

Table 2. 3 Particle size distribution of granular material for cold in-place recycling (Milton and Earland 1999)

Sieve size (mm)	Percentage by mass passing	
	Zone A	Zone B
50	100	-
37.5	94-100	-
20	66-100	100
10	48-75	75-100
5	35-57	57-95
2.36	25-42	42-77
0.6	13-28	28-52
0.3	10-24	24-45
0.075	5-20	20-35

Notes: [1]. Zone A material - ideal material. [2]. Zone B material – suitability of material form this zone is subject to the results of material performance tests on trial mixtures. [3]. Granular material to be stabilized should contain not more than 2% of organic matter determined in accordance with BS 1377: Part 3: Clause 3. [4]. Aggregate grading should have a coefficient of uniformity exceeding 10.

In comparison, bitumen bound recycled material is often highly sensitive to the pulverized aggregate grading, particularly the fines component. It has been found that the amount passing the 75 micron BS sieve should ideally be restricted to the range of not less than 5 percent and not more that 20 percent, none of which should be clay.

2.3.3.2 Moisture content

The moisture content of the pulverized aggregate prior to stabilization is equally important as grading. The optimum moisture content would normally be considered as the target moisture content, but in practice for recycled mixtures, the specified moisture content depends to a certain extent, on the binder used and whether a filler has been added in any great quantity.

For the pulverized aggregate stabilized using foamed bitumen binder, the "fluid" component of the binder will enhance the workability of the mixture and aid the thick lift compaction of the material. Hence, a target moisture content in the range $\pm 2\%$ of optimum moisture content (Milton and Earland 1999).

2.3.3.3 Stabilizing Agents

The principal reasons for the treatment of recycled material with stabilizing agents are as follows:

- To improve strength and hence the structural capacity of the pavement without the need to import additional material to increase pavement thickness;
- To enhance durability so as to ensure the long-term performance of the pavement and;
- To improve resistance to moisture.

The first recorded use of stabilizing agents for road construction was by the Romans, some two thousand years ago. In addition to their advanced segmented block paving systems (cobble stones), they also used a form of lime treatment to improve pavement strength for heavily loaded transport wagons.

A wide range of stabilizing agents is currently in use around the world. These include chemical compounds, such as calcium chloride, long-chain polymers and petroleum products, other proprietary produces and the more conventional agents like cement. All aim to achieve the same objective of binding the individual particles together to increase strength and/or make the material more water resistant. Some are more effective than others on specific materials, some have clear cost advantages, but all have a place in the market and most are best applied using modern recycling machines.

Cement and bitumen as stabilizing agents have been thoroughly researched. They are used extensively and standard test methods are available for determining optimum mix designs and quality assurance requirements.

Cement has been in use for the longest period of time, the first recorded used as a formal stabilizing agent being in the USA in 1917 (Wirtgen Gmgh 2001). Due largely to technological advances, the use of bitumen as a stabilizing agent is becoming increasing popular, applied both in an emulsified form and as a foam.

The choice of the most effective stabilizing agent for a particular application depends on several factors, the principals of which are: price, availability, acceptability, material type. In Table 2.4, the advantages and disadvantages of the various stabilizing agents are compared (Lewis and Collings 1999).

In the Austroads Guide to Stabilization in Roadworks (Vorobieff and Wilmot 2001), guidelines are provided to help engineers select appropriate binders for initial laboratory testing (Table 2.5)

Table 2. 4 Advantages and disadvantages of stabilizing agent

Type	Advantage	Disadvantage
Cement	Easy to apply as a powder or slurry. Normally less expensive than bitumen or emulsion. Improves material's resistance to moisture.	Shrinkage cracking can be a problem. Cracking can, however, be substantially reduced by careful mix design – Keeping cement content as low as possible and keeping the moisture cement on the low side.
Bitumen Emulsion	Easy to apply – the emulsion is sprayed directly into the recycler's mixing chamber. Emulsion treatment produces a flexible, fatigue resistant layer that is not prone to cracking. Once fully cured, emulsion treated material is resistant to the ingress of moisture.	Usually more expensive than cement of foamed bitumen. Emulsion must be formulated to be compatible with the recycled material, with a suitable "break" time to enable proper mixing and compaction. Emulsion treatment can be a problem when in-place moisture contents are high – the addition of the emulsion will push the moisture content well above optimum resulting in heaving of the recycled layer.
Combination of bitumen emulsion and cement	The cement can be injected as a slurry together with the emulsion into the recycler's mixing chamber. Alternatively it can be applied as a powder, and the emulsion sprayed by itself into the mixing chamber. The cement/emulsion combination procedures higher strengths, cures quicker, and is more resistant to water than emulsion alone. If properly designed it is not prone to shrinkage cracking.	More expensive than either cement or emulsion alone. Also more expensive than foamed bitumen. The emulsion must be formulated to have a suitable "break" time when it is mixed together with the recycled material and the cement. Premature breaking of the emulsion when it comes into contact with the cement will cause problems with unsatisfactory mixing and "balling"- correct formulation of the emulsion is essential.
Foamed bitumen	Easy application – the foamed bitumen is sprayed directly into the recycler's mixing chamber. Foamed bitumen treated material forms a flexible layer with good fatigue properties that is not prone to shrinkage cracking. It is resistant to the ingress of water. Usually less expensive than bitumen emulsion or a combination of emulsion and cement. Additional water is not added to the recycled material, as it the case when emulsion is used. Rapid strength gain – the road can be trafficked immediately after compaction is complete.	Requires a supply of hot (above 150 ^{0}C) bitumen. For foamed bitumen treatment, the material should have between 5% and 15% passing the 75 micron sieve size. If this is not the case, the grading should be rectified by importing and spreading a layer of suitably graded aggregate over the layer to be recycled.
Foamed bitumen and cement	When a low percentage of cement (1% to 2%) is used in combination with foamed bitumen, it improves the strength of the recycled material significantly. In addition, material treated with this combination will have an even higher resistance to water compared to foamed bitumen alone. Less expensive than a combination of bitumen emulsion and cement.	It has the same bitumen temperature and aggregate grading requirements as foamed bitumen. It is more expensive than foamed bitumen alone.

Table 2. 5 A guide to selecting a binder for stabilization

%age of filler	More than 25% passing 0.075mm			Less than 25% passing 0.075mm		
Plasticity Index	PI ≤ 10	10<PI<20	PI ≥ 20	PI ≤ 6 PI × % passing 0.075mm	PI ≤ 10	PI>10
Form of Stabilization						
Cement and Cementitious Blends						
Lime						
Bitumen						
Bitumen/Cement Blends						
Granular						
Dry powdered polymers						
Meanings of colours	Usually Suitable		Doubtful		Usually not Suitable	

1. Cement

It is the most commonly used stabilizing agent, its use worldwide is far exceeding all other stabilizing agents combined. The main reason for this is its availability. Cement is manufactured in most countries and is readily available throughout the world. Another reason is its acceptance as a construction material.

Cement stabilization, however, requires careful attention. All cement-treated material, including concrete, is prone to cracking. Such cracks can be controlled and are not necessarily detrimental. Cracks may be caused by traffic and not caused by traffic.

2. Bitumen emulsion

Bitumen emulsions were originally developed as a means of obviating the difficulties of working with hot bitumen and, of particular relevance for stabilizing, for mixing with damp material at ambient temperatures. Many recycled projects of RAP materials in 1970’s to 1990’s were carried out using bitumen emulsion as stabilizing agent (Kandhal and Mallick 1999, Huffman 1999).

An emulsion consists of two immiscible liquids, the one dispersed in the other in the form of small globules or droplets. Standard bitumen emulsions consist of bitumen dispersed as droplets in a continuous water phase, the bitumen particles being prevented from rejoining by a surface-active agent that forms a protective film around the particles. Most emulsions used as stabilizing agents have a “residual bitumen” component of 60%, which means that 60% of the volume of the emulsion is made up of bitumen dispersed in 40% of the volume that is water.

There are two types of bitumen emulsion, anionic and cationic. The basic difference between them is the charge on the bitumen and the suspension "phase". Bitumen particles in an anionic emulsion are negatively charged in an alkaline phase, whereas cationic emulsions have positively charged bitumen particles in an acidic phase.

High float medium set (HFMS-2), cationic medium set (CMS) and occasionally slow setting (CSS-1 and CSS-1h) are the most popular emulsion types. A few states are using Class C (self cementing) fly ash or Portland cement to increase early strength and resistance to rutting and water damage in cold in-situ recycling. However curing time for mixture rejuvenated by emulsion is long, and construction depends on the weather (minimum temperature of construction is 10~16℃, not in rainy day) (Lee et al 1999).

3. Foamed bitumen

Foaming occurs when small amounts of water are added to hot bitumen, thereby increasing the surface area and significantly reducing the viscosity of the bitumen. In this form it is well suited for mixing with cold damp aggregate. The potential of using foamed bitumen as a cold-mix binder was first realized by Prof. Ladis Csanyi at the Engineering Experiment Station of the Iowa State University in 1956 where a stream injection process was used to make foam (Csanyi 1957).

Foamed bitumen can be used as a stabilizing agent with a variety of materials ranging form good quality crushed stone to marginal gravels with relatively high plasticity. Comparing with other stabilizing agent, foamed bitumen has many advantages. Loudon and Partner compared the stabilizing agents using the deep cold in-situ recycling process. Among five stabilizing agents, namely: cement, bitumen emulsion, combination of bitumen emulsion and cement, foamed bitumen, foamed bitumen and cement, it was found that 3.5% foamed bitumen + 1% cement had best effect for cold in-place recycling according to the pavement performance and economy (Loudon and Partner 1999).

Compared to bitumen emulsion, foamed bitumen treated material can be placed, compacted and opened to traffic immediately after mixing; and this kind of material remains workable for extended periods and can be worked in adverse weather conditions without the bitumen washing out of the aggregate (Wirtgen GmbH 2001).

For mixture design of cold recycling using foamed bitumen, the method base on binder film thickness would be inappropriate for a recycled material, since the distribution and coating characteristic of the foamed bitumen is significantly different from that of the plant hot-mix. In general, the larger particles remain substantially uncoated with the bitumen adhering mostly to the finer particles, forming a fines/bitumen mortar matrix. The design principle for such material is to waterproof the fines and provide enough adhesion within the matrix to hold the material together and at the same time, to achieve efficient aggregate packing for low air voids content.

2.3.3.4 Mixture design method

There are several mixture design methods for cold-recycled mixture. The Marshall mix design is the most common method used, but other mix designs are also used including Hveem, Gyratory compaction and the Oregon method. Lee found that the maximum size of the RAP allowed in the mixtures ranges from 19 to 50 mm, and two state agencies did not have a maximum size limit (Lee et al 1999). The tests performed on the RAP include bitumen content, extracted gradation, RAP gradation, viscosity, penetration, and visual inspection.

The conventional Marshall design method for hot-mixes asphalt can be used to design for cold-recycled RAP material. The AASHTO expert task group developed a modified Marshall mix design for the emulsion bitumen mix. This procedure is comprised two parts. The first part is the determination of the optimum emulsion content and the second part is the determination of the optimum water content.

Because Marshall method is simple, it is used for bitumen mixture design in many countries. Ordinarily, Marshall mix design method is used to guide the design of RAP recycling and practical foamed bitumen projects. However Marshall method is an empirical method, not performance-related/performance-based method. This method cannot answer the following questions when evaluating the cold in-place recycling mixture (Lee et al 1999):

- The procedure does not give any specifications for when new aggregate should be added to the mixture.
- The procedure does not mention how long to cure the specimen.
- The procedure does not state how long to heat the rejuvenators in the oven.
- The procedure does not clearly state how to determine the optimum values for the rejuvenators and water contents.
- The design has no bearing on how well the mix will perform.

From 1987 through 1993, the Strategic Highway Research Program (SHRP) carried out several major research projects to develop the Superpave method for performance-based HMA design. SHRP completed in 1993 developed Superpave system for designing hot mix asphalt.

The Superpave mix design system is designed as a comprehensive method tailored to the unique performance requirements dictated by the traffic, environment (climate) and structural section at a particular pavement. It facilitates selecting and combining asphalt binder, aggregate, and any necessary modifier to achieve the required level of pavement performance (Cominsky 1994). While the Level 1 procedures are proving acceptable to the U.S. paving industry, the more complex testing required for Level 2 and 3 is still the subject of debate (Brown and Gibb 1996). Based on recent research, the restricted zone is expected to be deleted entirely from Superpave (Kandhal and Cooley Jr. 2002).

Although Superpave method can be applied to either new construction or overlays, it does not provide any guidelines to select the PG grade of virgin asphalt binder to be used in the recycled mix or to conduct a Superpave volumetric mix design and analysis of a recycled HMA mixtures. So a National Cooperative Highway Research Program project, "Incorporation of reclaimed asphalt pavement in the Superpave system (NCHRP 9-12)", was conducted by North Central Superpave Center (Mcdaniel and Anderson 2000). This is the first systematic research on RAP recycling using Superpave method. Up to now, a detail procedure has been determined. Foo and Kandhal (1998) also carried out a study on adapting Superpave technology to design of hot recycled mixes.

However, cold recycled mixture designed using volumetric mix design of Superpave technology is only a pilot performed for the Kansas and Ontario RAP projects (Lee et al 1999). Lee et al (2003) also performed a further development of a procedure to evaluate the performance of CIPR mixtures that have been prepared in accordance with volumetric design method of Superpave. Mallick et al (2001) carried out a project to develop a rational mix design system for full depth reclamation and evaluate different additives used in FDR. Up to now, the application of Superpave technology in cold-recycled mixture design has being limited to laboratory evaluation. In order to make a good application of the cold-recycled mix, it is necessary to solve some problems, e.g. the lack of a uniform, raveling, thermal cracking, compaction problems, low early strength and extended curing time. The design procedure is needed to further develop, performance-related tests and specification for cold in-place recycling are needed to provide. Thomas and Kadrmas (2003) summarized the application of the Superpave technology in the cold in-place recycling in the USA, including development of the performance-related test methods to improve the reliability of the process for emulsion cold in-place recycling, laboratory raveling test run on Superpave Gyratory Compactor, indirect tensile test of Superpave design and test method for CIPR resistance to thermal cracking.

2. 4 FOAMED ASPHALT PAVEMENT

The use of foamed bitumen as a binding agent for stabilization has recently generated a lively interest. Foamed bitumen and foamed asphalt mix epitomize the asphalt industry's drive towards energy efficient, environmentally friendly and cost-effective solutions for road building (Wirtgen GmbH 2003).

2.4.1 Definition Of Foamed Bitumen And Foamed Asphalt

The term 'foamed asphalt' refers to a mixture of pavement construction aggregates and foamed bitumen. The foamed bitumen, or expanded bitumen, is produced by a process in which water is injected into the hot bitumen, resulting in spontaneous foaming (Wirtgen GmbH 2003). The physical properties of the bitumen are temporarily altered when the injected water, on contact with the hot bitumen, is turned into vapor which is trapped in thousands of tiny bitumen bubbles. However the foam dissipates in less than a minute and the bitumen resumes its original properties. In order to produce foamed asphalt, the foamed state bitumen has to be sprayed into the aggregates.

Foaming increases the surface area of the bitumen and considerably reduces its viscosity, making it well suited for mixing with cold and moist aggregates. Foamed bitumen can be used with a variety of materials, ranging from conventional high-quality graded materials and recycled pavement materials to marginal materials such as those having a high plasticity index. Foamed asphalt can be manufactured in-place or in a central plant.

The foamed bitumen is characterized in terms of expansion ratio and half-life. The **expansion ratio** of the foam is defined as the ratio between the maximum volume achieved in the foam state and the final volume of the binder once the foam has dissipated. The **half-life** is the time, in seconds, between the moment the foam achieves maximum volume and the time it dissipates to half of the maximum volume.

2.4.2 Historical Perspective

The bitumen foaming process was first proposed by Prof. Csanyi in the mid 1950's (Csanyi 1957). The original process consisted of injecting steam into hot bitumen. The steam foaming system was very convenient for asphalt plants where steam was readily available but it proved to be impractical for in situ foaming operations, because of the need for special equipment such as steam boilers. In 1968, Mobil Oil Australia, which had acquired the patent rights for Csanyi's invention, modified the original process by adding cold water rather than steam into the hot bitumen. The bitumen foaming process thus became much more practical and economical for general use. Continental Oil Company had further developed the process and had been licensed by Mobil Oil Australia to market the process in the US (AustStab 1998).

From 1970's to 1980's, the bitumen foaming process has ever been successfully tried and tested in Australia and the United States (Bowering and Martin 1976, Little et al. 1983, Castedo-Franco and Wood 1983, Ruckel et al. 1983, Brennen et al. 1983).

Since the expiration of Mobil's patent on the foamed bitumen process in 1991 and subsequent intensive research, many road authorities worldwide have carried out successful trials using foamed bitumen stabilization technology (Kendall et al. 1999, Nataatmadja 2001, Asi 2001). After 1997, Main Roads in Australia launched a series of systematic trials of foamed bitumen application based on previous small trials in Queensland State for the purpose of producing flexible and fatigue-resistant recycled base. These trials include projects in Gladfield, Rainbow Beach, Inglewood, Allora and Redland shore. Although several foamed bitumen stabilization trials had been carried out and were performing satisfactorily, long term monitoring was required to confirm this observation (Ramanujam and Jones 2000).

In the United Kingdom, as with some other European countries, foamed bitumen is incorporated into good quality pavement materials with the stabilized material used as a subbase material with a minimum asphalt base thickness of 100 mm. In these situations, cement is usually the preferred supplementary binder (Milton and Earland 1999).

In South Africa, CSIR Transportek developed a standard design procedure based on the in-place method of construction in KwaZulu-Natal and previous research in order to specify asphalt mix design (Muthen 1998). However, no efforts of moisture study and few fatigue study were included in the CSIR project.

The first Ministry of Transportation Ontario (MTO) contract using foamed bitumen stabilization was constructed in 2001. This flexible pavement recycling technique was selected mainly because of a lack of aggregate availability in this area (Lane and Kazmierowski 2003). Stabilization of base material using foamed bitumen is also being tested and tried in USA. An investigation on the potential use of foamed bitumen treated RAP as a base course material in lieu of a crushed lime stone base underneath a concrete pavement layer was carried out in Louisiana (Mohammad et al 2003). In order to solve the inadequate support of base of Mid-Western Roads in Iowa, foamed bitumen was used to treat the full depth reclamation materials (Romanoschi et al 2003).

In Norway, Netherlands and Baltic Republics, the "mix-in-place" recycling method with foamed bitumen has become very popular mainly due to lack of raw construction materials. In Norway, for example, where cold recycling with foamed bitumen has been in use since 1983, as much as approximately 1.8 million m^2 of roads were rehabilitated with this process in 1997 (Wirtgen GmbH 2003). However, cold recycling with foamed bitumen is a fairly young discipline in other Europe countries for which generally accepted rules do not yet exist.

Stabilization technology using foamed bitumen is even newer in China. In 1998, a type of cold-recycled machine made in Germany, WR2500, was imported from Wirtgen Co. and was used to rehabilitate a asphalt pavement section in Hubei Province (Cao et al 2003). This is the first trial of cold recycling with foamed bitumen in China. However, this trial was not presented in detail. In Taiwan Province, a study was performed to investigate the engineering properties of foamed bitumen treated base which contained RAP materials. Marshal method was used in the mix design. The properties of foamed asphalt mixes were compared with those of hot recycled mixes. Test results showed benefits of utilizing the milled asphalt in foamed asphalt mixes (Chiu and Huang 2003).

2.4.3 Advantages And Disadvantages Of Foamed Asphalt Mixes

The following advantages of foamed asphalt are well documented (Muthen 1998, AustStab 1998):

- The foamed binder increases the shear strength and reduces the moisture susceptibility of granular materials. The strength characteristics of foamed asphalt approach those of cemented materials, but foamed asphalt is flexible and fatigue resistant.

- Foam treatment can be used with a wider range of aggregate types than other cold mix processes.
- Reduced binder and transportation costs, as foamed asphalt requires less binder and water than other types of cold mixing.
- Saving in time, because foamed asphalt can be compacted immediately and can carry lignt traffic almost immediately after compaction is completed.
- Energy conservation, because only the bitumen needs to be heated while cold and damp aggregates can be directly mixed without drying.
- Environmental side-effects resulting from the evaporation of volatiles from the mix are avoided since curing does not result in the release of volatiles. While volatiles will raised in the application of the cut back asphalt.
- Foamed asphalt can be stockpiled with no risk of binder runoff or leeching. Since foamed asphalt remains workable for long extended periods, the usual time constraints for achieving compaction, shaping and finishing of the layer are avoided.
- Foamed asphalt layers can be constructed in adverse weather conditions, such as in cold weather or light rain, without affecting the workability or the quality of the finished layer.

Disadvantages of foamed asphalt stabilization include the following:

- more expensive than lime/flyash stabilization;
- not suitable for all pavement types (requires a full particle size distribution);
- design methodologies for foamed bitumen are relatively new, as a rapid evolution of the technology associated with foamed bitumen stabilization has only recently occurred;
- the process requires hot bitumen (180 ⁰C) for the foaming action to be successful, and thus there is a risk of burning (common to all road construction operations involving bitumen); and
- purpose built foamed bitumen stabilizing equipment is required.

2.4.4 State-Of-Art Of Foamed Asphalt Pavement

2.4.4.1 Bitumen properties

1. Foaming characteristic

The foaming characteristics of bitumen play an important role during the mixing stage of foamed asphalt production. It can be expected that maximized expansion ratios and half-lives will promote binder dispersion within the mix. Castedo-Franco and Wood (1983) found that any bitumen, irrespective of grade or origin, could be foamed with an appropriate combination of nozzle type, water, air and bitumen injection pressure. However, Abel (1978) found that some properties of bitumen and testing condition as following affected the foaming characteristics:

- bitumen which contained silicones could have reduced foaming abilities;
- bitumens with lower viscosities foamed more readily and had higher foam ratios and half-lives than bitumens with higher viscosities, but the use of high viscosity bitumens resulted in superior aggregate coating;
- anti-stripping agents intensified the foaming ability of bitumens, and
- acceptable foaming was only achieved at temperatures above 149° C.

Brennen et al (1983) found that the half-life and expansion ratio of the foam produced from a particular bitumen was affected by the volume of foam produced, the quantity of water used and the temperature at which the foam was produced. Higher foaming temperatures and increased quantities of water both resulted in increased expansion ratios, but resulted in decreased half-lives. In laboratory tests, the size of the container was found to affect the foam parameters. Kendall et al (1999) found that silicones in Australian bitumen reduced its ability to foam, in order to obtain high quality foamed bitumen, foaming agent should be added.

2. Foamed bitumen content

In foamed-asphalt mixes the optimum bitumen content often cannot be clearly determined as it can in the case of hot-mix asphalt. The range of bitumen content (BC) that can be used is limited by the loss in stability of the mix at the upper end of the range and by water susceptibility at the lower end. It appears that one significant parameter is the ratio of bitumen content to fines content, i.e. the viscosity of the bitumen-fines mortar plays a significant role in mix stability. Table 2.6 may be used as a guide to select the appropriate binder content based on the fines content of the mix. Akeroyd and Hicks (1988) also proposed the use of a proportional bitumen-fines relationship to select the bitumen content, ranging from a bitumen content of 3.5 percent bitumen for 5 percent fines content to a bitumen content of 5 percent for 20 percent fines content. However this approach may not be applicable for all types of material, because of the varying bitumen-absorption characteristics of fines which, in turn, depend on the source (parent) material.

Table 2. 6 Foamed bitumen content (Ruckel et al,1983)

% passing 4,75 mm sieve	% passing 0,075 mm sieve	% Foamed bitumen
< 50 (gravels)	3 – 5	3
	5 – 7.5	3.5
	7.5-10	4
	>10	4.5
> 50 (sands)	3-5	3.5
	5-7.5	4
	7.5-10	4.5
	>10	5

2.4.4.2 Aggregate properties

Research has shown that a wide range of aggregates may be used with foamed bitumen, ranging from crushed stone to silty sands and even to ore tailings, as shown in Table 2.7. Certain types of soil may require lime treatment and grading adjustments to enable them to perform satisfactorily.

Table 2. 7 Foam treated materials (Bowering and Martin 1976)

Soil type	Optimum range of binder contents (%)	Additional requirements
Well graded clean gravel	2-2.5	
Well graded marginally clayey/silty gravel	2-4.5	
Poorly graded marginally clayer gravel	2.5-3	
Clayey gravel	4-6	Lime modification
Well graded clean sand	4-5	Filler
Well graded marginally silty sand	2.5-4	
Poorly graded marginally silty sand	3-4.5	Low penetration bitumen; filler
Poorly graded clean sand	2.5-5	Filler
Silty sand	2.5-4.5	
Silty clayey sand	4	Possibly lime
Clayer sand	3-4	Lime modification

Fig. 2.6 shows the Mobil foam stabilization grading chart (Akeroyd and Hicks, 1988). Materials conforming to Zone A of the chart have been found to be suitable for foam treatment for heavily trafficked roads. Materials conforming to Zone B are suitable for lightly trafficked roads, but could be adjusted to Zone A materials by the addition of coarse fractions. Materials in Zone C are deficient in fines and are not appropriate for foam stabilization unless fines are added.

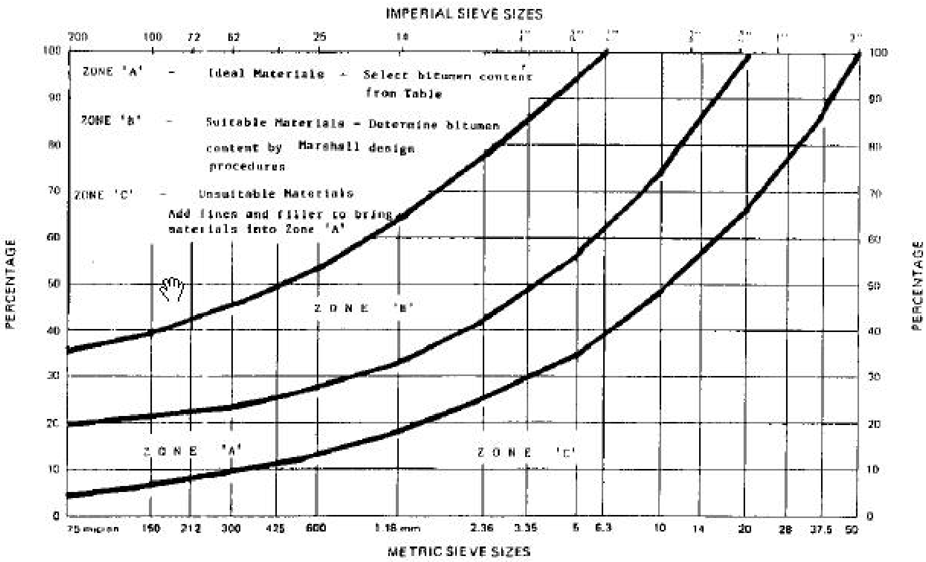


Fig.2. 6 Aggregate grading zones for foamed asphalt pavement (Akeroyd and Hicks, 1988)

Fig. 2.7 illustrates the preferred gradation recommended by Austroads for pavement materials stabilized by foamed bitumen. The only major requirement for the pavement material was the grading curve and plasticity index (PI). In order to assure the quality of bitumen stabilization, the following requirements should be met (Vorobieff and Preston 2004):

- Less then 25% passing the 75 micron, and PI $\leq$ 10

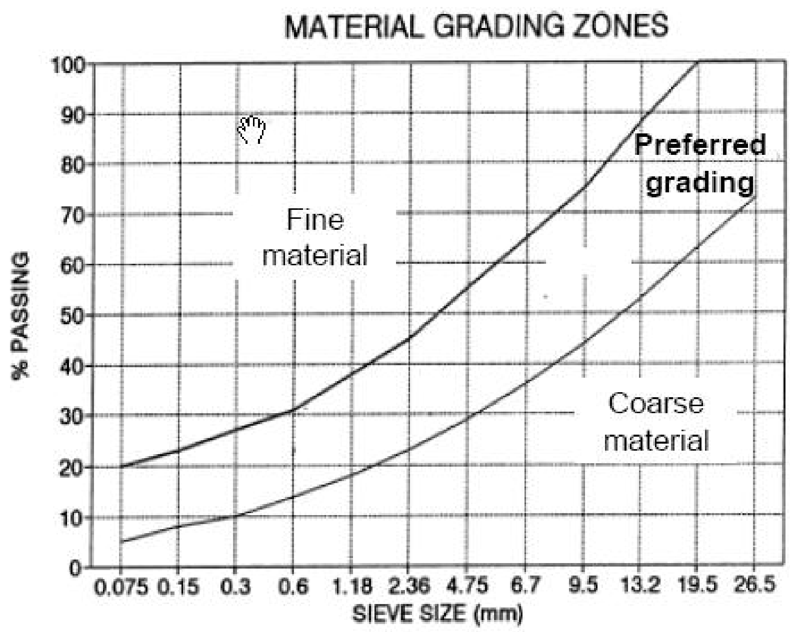


Fig.2. 7 The preferred gradation for foamed asphalt mixture (Vorobieff and Preston 2004)

The fines content of the aggregate is an important consideration and should preferably be above 5 percent (Ruckel et al (1983). The ability of foamed bitumen to selectively mix with and coat the fines (minus 0,075 mm particles) has been well documented. Mix of bitumen and fines acts as a mortar between the coarse aggregates and hence increases the strength of the mix. However, the relationship between the fines content and bitumen content is critical because excess bitumen in the mortar will tend to act as a lubricant and result in loss of strength and stability. Sakr and Manke (1985) showed that foamed asphalt mix with higher percentages of fines had higher stabilities. Bissada (1987) showed a similar trend for tensile strength.

In a limited study, Sakr and Manke (1985) showed that the stability of foamed asphalt mixes was affected to a greater extent by the aggregate interlock than by the viscosity of the bitumen, its behaviour thus differing from that of hot-mix asphalt. This implies that foamed asphalt mixes are not as temperature susceptible as hot-mix asphalt, and supports the finding that the viscosity (grade) of the bitumen used is not very critical for foamed asphalt mixes. Sakr and Manke (1985) also found that the angularity of fine aggregates is an excellent indicator of suitability for foam stabilization. A minimum particle index of 10 was suggested in order to achieve good stabilities. The particle index can quantify the shape and the texture of fine aggregates, it can be determined by ASTM D 3398.

2.4.4.3 Moisture conditions

The moisture content during mixing and compaction is considered by many researchers to be the most important mix design criteria for foamed asphalt mixes. Moisture is required to soften and breakdown agglomerations in the aggregates, to aid in bitumen dispersion during mixing and for field compaction.

Insufficient water reduces the workability of the mix and results in inadequate dispersion of the binder, while too much water lengthens the curing time, reduces the strength and density of the compacted mix and may reduce the coating of the aggregates. The optimum moisture content (OMC) varies, depending on the mix property that is being optimized (strength, density, water absorption, swelling). However, since moisture is critical for mixing and compaction, these operations should be considered when optimizing the moisture content (Muthen 1998).

Extensive trials and studies suggest that the optimum moisture content for mixing lies at about the "fluff point" of the aggregate, i.e. the moisture content at which the aggregates have a maximum loose bulk volume (70 % - 80 % mod AASHTO OMC) (Lee (1981) and Bissada (1987)).

The concept of optimum fluid content as used in granular emulsion mixes may also be relevant to foamed asphalt. This concept considers the lubricating action of the bitumen in addition to that of the moisture. Thus the actual moisture content of the mix for optimum compaction is reduced in proportion to the amount of bitumen incorporated. Muthen (1998) recommended that the moisture content for mixing and compaction be selected at OMC minus the BC used for the Marshall method of compaction.

In recent years the gyratory compaction method has gained popularity for the preparation of samples in the laboratory. Because the gyratory compaction effort is usually higher than the Modified compaction effort, the OMC obtained is lower than Modified AASHTO OMC. Hence, when gyratory compaction is used for the preparation of foamed asphalt, the OMC is advocated for mixing and compaction (Muthen 1998).

2.4.4.4 Curing conditions

Studies have shown that foamed asphalt mixes do not develop their full strength after compaction until a large percentage of the mixing moisture is lost. This process is termed curing. Curing is the process whereby the foamed asphalt gradually gains strength over time accompanied by a reduction in the moisture content. Ruckel et al (1983) concluded that the moisture content during the curing period had a major effect on the ultimate strength of the mix. The laboratory mix design procedure would need to simulate the field curing process in order to correlate the properties of laboratory-prepared mixes with those of field mixes. Since the moisture in the mixture takes several months to completely evaporate, it is impractical to reproduce actual field curing conditions in the laboratory. An accelerated laboratory curing procedure is required, in which the strength-gaining characteristics can be correlated with field behaviour, especially with the early, intermediate and ultimate strengths attained. This characterization is especially important when structural capacity analysis, based on laboratory-measured strength values, is required.

Most of the previous investigations have adopted the laboratory curing procedure proposed by Bowering (1970), i.e. 3 days oven curing at a temperature of 60 ^{0}C. This procedure results in the moisture content stabilizing at about 0 to 4 percent, which represents the driest state achievable in the field. The strength characteristics of samples cured in this manner are representative of the in-service state approximately a year after construction (Maccarrone et al, 1995). Concerns have been expressed over the bitumen ageing which may occur at a curing temperature of 60 ^{0}C. Also, since this temperature is above that of the softening point of common road-grade bitumens, changes in bitumen dispersion within the mix are possible during curing. Further, Nataatmadja (2001) found that an accelerated curing temperature of 60 ^{0}C was found to be too high, resulting in an over-estimation of the resilient modulus compared with specimens cured at an ambient temperature. An alternative approach would be to oven-dry the foamed asphalt to a constant mass, at a lower temperature (40 ^{0}C). After comparing effects of different curing method, Nataatmadja suggested that the 3 days curing at 40 ^{0}C would probably be more suitable for curing of the foamed bitumen treated mixes.

The amount of filler affects the strength of foam asphalt mixes. If amount of filler smaller than 75 micron is smaller than 5%, strength will decrease. Curing has a significant influence on the strength of foamed asphalt mixes (Van Wijk and Wood, 1983).

2.4.4.5 Temperature conditions

The optimum mixing temperature of the aggregates for foamed asphalt mixes lies in the range of 13^{0}C to 23 ^{0}C, depending on the type of aggregate. Foamed asphalt mixes may also be prepared with heated aggregates which will increase the bitumen dispersion within the mix and aid in the coating of the larger aggregates (Muthen 1998).

Jenkins et al (2000) found that the heating of aggregates above ambient temperatures but below 100 ^{0}C (half-warm) prior to mixing with foamed bitumen had an impact on the mixture produced. The distribution of the bitumen is improved in the foamed bitumen process through the heating of aggregates prior to mixing and cohesion increase with higher mixing temperatures (measured through vane-shear device). RAP materials show great potential for treatment with the half-warm foamed bitumen process. In particular, RAP that is heated to between 80 ^{0}C and 95 ^{0}C and stabilized with foamed bitumen without the addition of filler or moisture, produces asphalt comparable to HMA. The addition of filler and moisture to the RAP in the half-warm process does enhance cold-mix qualities, but forfeits tensile strength and performance in the process. However resistance to moisture susceptibility was not investigated in this limited feasibility study. Koenders et al (2002) also confirmed that mixture produced at 80 ^{0}C to 120 ^{0}C would have good engineering properties as hot-mixed mixture produced at 160 ^{0}C to 170 ^{0}C.

2.4.4.6 Engineering properties

All studies confirm that strength parameters such as resilient modulus and stability are optimized at a particular level of appropriate bitumen content. The most common method used in the selection of the design bitumen content was to optimize the Marshall stability and minimize the loss in stability under soaked moisture conditions. The major functions of foamed bitumen treatment are to reduce the moisture susceptibility, to increase fatigue resistance and to increase the adhesion of the untreated aggregate to acceptable levels. The design foamed bitumen content could also be selected as the minimum (not necessarily optimum) amount of bitumen which would result in a suitable mix (Muthen 1998).

1. Moisture susceptibility

The strength characteristics of foamed asphalt mixes are highly moisture-dependent. This is because of the relatively low bitumen contents and high void content of foamed asphalt mixes. Castedo-Franco et al (1984) reported that vacuum saturation weakened the foamed asphalt mixes, and durability was generally better with higher levels of bitumen content in the water sensitivity test. Marquis et al (2003) conducted a study by submerging the specimens in 60 ^{0}C water for a period and then bring back to 25 ^{0}C at the end of conditioning and then testing for retained resilient modulus. Moisture susceptibility under cyclic loading in water showed that foamed asphalt mixes were not inferior to emulsion plus lime mixes. They also indicated that compaction effect (air void of FA mixes) was a key to achieve good performing FA pavements.

Cement was also found to be as effective as lime, and cheaper (Lewis, 1998). In the project of stabilization of Sebkha soil using foamed bitumen, the mixture stabilized by foamed bitumen plus 2% cement exhibited a superior rating as compared with the other stabilizing agents (Asi 2001).

Higher bitumen content also reduces moisture susceptibility because higher densities are achievable, leading to lower permeability (lower void contents), and to increased coating of the moisture-sensitive fines with bitumen.

2. Temperature susceptibility

Foamed asphalt mixes are not as temperature-susceptible as hot-mix asphalt, although both the tensile strength and modulus of the former decrease with increasing temperature. Bissada (1987) found that, at temperatures above 30 ^{0}C, foamed asphalt mixes had higher moduli than equivalent hot-mix asphalt mixes after 21 days' curing at ambient temperatures. In foamed asphalt, since the large aggregates are not coated with bitumen, the friction between the aggregates is maintained at higher temperatures.

However the stability and viscosity of the bitumen-fines mortar will decrease at high temperatures, thus accounting for the loss in strength. Nataatmadja (2001) found that an accelerated curing temperature of 60 ^{0}C was too high, resulting in an over-estimation of the resilient modulus compared with samples cured at ambient temperature.

3. Unconfined compressive strength (UCS)

Bowering (1970) suggested the following UCS criteria for foamed asphalt mixes used as a base courses under thin surface treatments (seals): 0.5 MPa (4 day soaked) and 0.7 MPa (3 day cured at 60° C). Bowering and Martin (1976) suggested that in practice the UCS of foamed asphalt materials usually lie in the range 1.8 MPa to 5.4 MPa. They also found that foamed asphalt had strength characteristics superior to those of emulsion-treated materials at bitumen contents above 1.5 percent.

Asi (2001) found that properly-designed Sebhka soil treated by foamed bitumen with as little as 2% cement displayed significant improvement in the UCS of mixture.

4. Tensile strength

Bowering and Martin (1976) estimated that the tensile strengths of foamed asphalt materials lay in the range 0.2 MPa to 0.55 MPa, depending on moisture condition.

Maccarrone (1995) recommended that, for good performance, cured foamed asphalt samples should have minimum indirect tensile strength (ITS) of 100 kPa when tested in a soaked state and 200 kPa when tested dry.

Wirtgen's Cold Recycling Manual suggests that bitumen stabilized material is normally evaluated using ITS in preference to Marshall testing. The test is conducted on Marshal briquette specimens at one temperature only (25 ^{0}C). ITS results of RAP/crushed stone (50/50 blend), crushed stone and natural gravel (PI<10, CBR>30) should be higher than 350, 400 and 250 MPa respectively (Wirtgen GmbH 2001).

In terms of strength measurement, the fact that a mixture is stabilized rather than being "bound" has led to the use of a partially confined tensile strength measurement being made (Wirtgen GmbH 2001). From the literature review, it can be found that most of the foamed asphalt in highway is designed by this method. Khweir et al (2002) also used this method design the foamed asphalt mixture in an urban high street in UK.

5. Stiffness/resilient modulus

As with all viscoelastic bituminous materials, the stiffness of foamed asphalt depends on the loading rate, stress level and temperature. Generally, stiffness has been shown to increase as the fines content increases. In many cases the resilient moduli of foamed asphalt mixes have been shown to be superior to those of equivalent hot-mix asphalt mixes at high temperatures (above 30 ^{0}C). Foamed asphalt can achieve stiffness comparable to that of cement-treated materials, with the added advantages of flexibility and fatigue resistance (Ramanujam and Fernando, 1997).

An in-place investigation in Louisiana shows that foamed bitumen treated RAP materials have higher in-place stiffness values and structural number than those of lime stone base layer, and there is no significant difference between the foamed bitumen treated base containing 100 percent RAP materials and that containing 75 percent RAP and 25 percent crushed concrete (Mohammad et al 2003).

In contrast with previous research, the test results obtained by Nataatmadja indicated that the foamed bitumen mix had good resistance to water deterioration despite the reduction in stiffness (Nataatmadja 2002).

6. Abrasion resistance

Foamed asphalt mixes usually lack resistance to abrasion and raveling and are not suitable for wearing/friction course applications. This is because the mixture treated with foamed bitumen is stabilized by the mortar matrix among the aggregates rather than being “bound”.

7. Density and volumetrics

Generally density increases to a maximum and voids in the aggregate decreases to a minimum as the bitumen content of a foamed asphalt mix increases. Many studies have shown that the strength of foamed asphalt mixes depends to a large extent on the density of the compacted mix. Hence it is foreseeable that density and mix volumetrics could be used as criteria to determine the optimum bitumen content of a foamed asphalt mix. However, volumetrics design method is not extensively used in the foamed asphalt mixture design (Lee et al 1999, Lee et al 2003, Mallick et al 2001, Thomas and Kadrmas 2003).

8. Fatigue

Due to the foamed asphalt materials being usually paved as base layer, fatigue resistance of this mixture is an important factor in determining the structural capacity of foamed asphalt pavement layers.

So far no country mandates the fatigue test as a standard for pavement design in its specification. In North America, many states/provinces use flexural bending beam method to evaluate fatigue. In Europe, many countries use cantilever beam method to evaluate fatigue property, some use indirect tensile method to test cylindrical specimen for this purpose. In terms of operation difficulty and the extent of simulating the field performance, the flexural bending beam method is the best one followed by the indirect tensile method. (Rao et al 1990, SHRP-A-404 1994) The disadvantage of the former is time-consuming.

Foamed asphalt mixes have mechanical characteristics that fall between those of a granular structure and those of a cemented structure. Bissada (1987) considered that the fatigue characteristics of foamed asphalt would thus be inferior to those of hot-mix asphalt materials. Little et al (1983) provided evidence of this when he showed that certain foamed asphalt mixes exhibited fatigue responses inferior to those of conventional hot-mix asphalt or high quality granular emulsion mixes. These findings are contradictory to those resulting from the approach adopted by Maccarrone et al (1993) who suggested that the fatigue characteristics of foamed asphalt are similar to those of hot-mix asphalt. Research in Taiwan Province, China shows that the fatigue life of foamed bitumen treated mixes was close to that of hot recycled mixes (Chiu and Huang 2003).

The laboratory fatigue test usually underestimates the fatigue life of mix, a shift factor can be considered in the prediction of fatigue life. Shift factor is assumed to consist of the combined effect of a strain recovery and crack recovery component. In the structure design of the foamed asphalt pavement, Khweir et al (2002) determined the fatigue life of the foamed asphalt mixes based on a laboratory generated profile and a shift factor of 3. However, the shift factor of fatigue life in the pavement structure design is difficult to be accurately determined for both hot mixed asphalt and cold-recycled mix.

Preparation of specimen of foamed asphalt mixes for fatigue test is not easy due to the poor abrasion of the mix. Hence research on fatigue of this mix is not found extensively in literature.

2.5 DISCUSSION

Foamed bitumen has been successfully applied to stabilized pavement material on the low traffic road, especially for marginal material in Australia and South Africa. As mentioned above, cold-mixed mixture is commonly used as base or subbase covered by a hot-mixed wearing course due to its poor abrasion. Up to now, foamed asphalt pavement has never been paved in Hong Kong. As a new technology, its application in Hong Kong will face some problems as follows:

1. Mix design

In Hong Kong, Scott Wilson (SW) (Hong Kong) Ltd was designated to carry out a preliminary investigation into asphalt pavement recycling for Hong Kong' paving industry. In this project, hot in-plant recycling (HIR) is recommended to manufacture mixture (Elliott 2002, Chan 2002). However, cold recycling is absolutely new in research and application fields.

The traffic in Hong Kong is very heavy. For example, Tun Mun Road is a major link between Tun Mun and Kowloon/Tsuen Wan, it was designed and constructed in the mid-1970's. The annual average daily traffic (AADT) was 17200 and 98310 standard vehicles in 1978 and 2001 respectively. From 1978 to 2001, the accumulated traffic load is 178.95 million standard axles (MSA) (Maunsell Consultant Asia Ltd. 2003). If cold mixed RAP will be applied in Hong Kong, or foamed asphalt mix will be applied to the pavement of the heavy traffic road, the mix design of cold-recycled RAP mixture stabilized by foamed bitumen should be determined based on requirement of the existing gradation, RAP material in Hong Kong, and foamed asphalt design method.

Although the foamed bitumen process was developed more than 40 years ago and lauded by researchers worldwide, it is believed that the lack of standardized design procedures has contributed to the limited implementation of this technology. There is no recommendation for RAP content used in the cold recycling.

In the future (next 10 years), the Hong Kong Government will further improve the transportation system in Hong Kong. Mileage of new construction road and rehabilitated road will reach 100km. In order to fulfill highway network's sustainable development, it is necessary to utilize more and more RAP material in the new road construction and road maintenance. Highways Department of Hong Kong Special Administrative Region (HKSAR) encourages the use of RAP in large projects (e.g. new build). Environmental Protection Department (EPD) also wishes to see an increase in the amount of recycling performed in Hong Kong Highways.

However, the unknown at this time is the appropriate content of RAP blended in the cold-recycled mix. In this research, the RAP content of the cold-recycled mix was studied. Mix design of foamed asphalt mixtures should answer mix gradation, the optimum foaming condition, the optimum moisture content and the design bitumen content in the mixing process of recycled materials.

2. Fatigue performance and permanent deformation

There are two criteria of pavement structure failure according to elastic-layered pavement structure theory. One is the strain of the bottom layer of bituminous material; the other is the permanent deformation on the top of the subgrade (Huang 2004, Claessen et al 1977, SPDM 1978). It should be noted that the second criteria is an empirical concept. Fig.2.8 illustrates these criteria of the pavement under environmental impact and traffic loading (Brown 1997).

Of the various forms of pavement distress, fatigue (which leads to cracking) and rutting (i.e. permanent deformation) are of great impact. In well-built thick bituminous-bound flexible pavements which are sufficiently strong to resist structural damage in their early life, the conventionally-accepted mechanisms of pavement deterioration (roadbase fatigue and structural deformation) are far less prevalent than surface deterioration that is associated with excessive aging of the surface courses (Nunn et al 1997). This is the reason why Superpave design method mandates bitumen should be tested by pressure aging vessel (PAV) that can simulate the long term ageing of the binder.

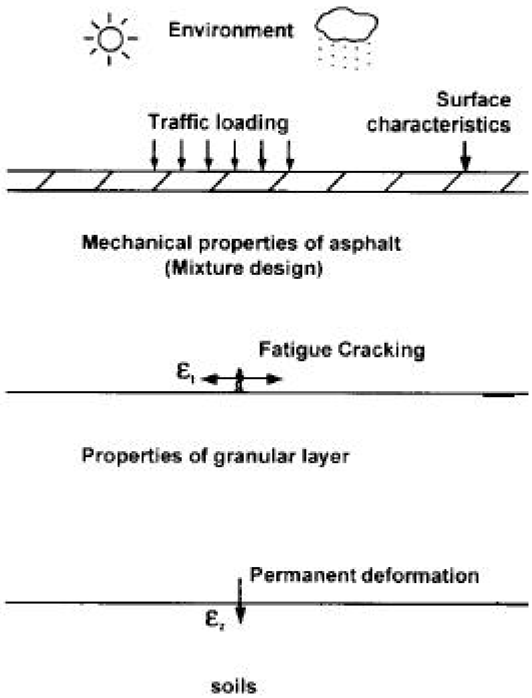


Fig.2. 8 Environmental impact and traffic loading on the pavement

However, RAP's binder is aged. The fatigue distress is prone to occur in the recycled RAP mixture. Because RAP material is often paved in the road base or base coarse in the current stage, the tensile crack also appear in the bottom of these layer. Hence fatigue property is a dominant factor of pavement performance.

On the other hand, pavement layer should be prevented from excess permanent deformation. If deformation-resistance is poor, early failure will occur in the pavement. Permanent deformation is also a dominant factor of pavement performance.

Therefore, the critical fatigue performance and rutting-resistance of the foamed bitumen treated material must be studied in the laboratory for application of cold-stabilized RAP materials.

3. Moisture susceptibility

Moisture-induced damage is another predominant distress of bituminous pavement, especially in the rainy season in hot region. Moisture-induced damage of bituminous materials (e.g. severe raveling and pothole) may occur if moisture resistance of these materials is poor (Sha 2001). Therefore the effects of water on the properties of asphalt concrete mixtures become an important research topic.

Rainfalls in southern China are frequent and heavy. The mean annual rainfall in Hong Kong ranges from around 1300 millimetres to more than 3000 millimetres; about 80 percent of the rainfalls between May and September, almost half year (Hong Kong Observatory 2004). As moisture exists commonly in the voids of pavement materials, moisture-induced damage can be often found in the rainy and hot season in Hong Kong.

Many studies on bitumen properties and aggregate properties of foamed asphalt (FA) mixes had been conducted. However, studies on moisture susceptibility of FA mixes were not yet reported extensively. Therefore, moisture susceptibility of cold-mixed RAP mixture is another factor which should be also considered.

The evaluation of mixes' moisture-resistance includes: (a) to evaluate the cohesiveness between bitumen and aggregate and (b) to evaluate moisture resistance of the mixture. There are three types of moisture susceptibility test: Lottman test, Tunnicliff-Root test and the modified Lottman test (AASHTO T 283).

All specimens in above three methods are tested for resilient modulus (MR) and/or indirect tensile strength. Indirect tensile strength ratio (ITSR) is calculated for conditioned specimens by dividing their ITSs by ITSs of specimens without conditioning (Lottman 1982 and Tunnicliff and Root 1984). A minimum ITSR of 0.7 is specified in these three tests.

However, it has been argued that Lottman procedure is too severe because the warm water soak of the vacuum saturated and frozen specimen can develop internal water pressure. No test has proven to be “superior” and can correctly identify a moisture-susceptible mix in all cases. These test methods do not always yield accurate results either because they do not simulate field conditions or because they are subjective in nature (Terrel and Al-Swaili 1994).

SHRP program analyzed a series of methods and indicated that AASHTO T283 and Tunnicliff-Root method are the best. AASHTO T 283 intends to evaluate the effects of water saturation and accelerated water conditioning with a freeze-thaw cycle of compacted mixtures in terms of modulus ratio and indirect tensile strength ratio. Disadvantage of this method is time-consuming.

4. Curing time

Studies indicate that cold-mixed mixture stabilized/rejuvenated by foamed bitumen spends less curing time than that by emulsion, but more curing time than HMA mixture (Marquis et al 2003). Curing time (approximate 1~2 days) is relatively long in the cold recycling application of the foam bitumen. This is mainly due to the water which is added into the cold mixture for obtaining the maximum density.

Long curing time is a disadvantage in the foam bitumen application under emergent maintenance work for heavy traffic road. Therefore how to shorten the curing time becomes another problem for RAP material stabilized by foamed bitumen.

2.6 APPROACHES OF THIS STUDY

2.6.1 Main Tasks

This research is aimed at studying mix design and the performance of cold-mixed RAP mixture stabilized by foamed bitumen. It is hoped that this research can provide a foundation for use of cold-recycled RAP material stabilized by foamed bitumen in the Hong Kong’s paving industry.

Considering the weather condition (high temperature and moisture) in Hong Kong and limitation of the RAP (mainly used as road base or base course), pavement performance studies were focused on the fatigue-resistance, permanent deformation and moisture-resistance. In this research, effect of different curing method on properties of foamed asphalt mix is not considered. On the other hand, average annual temperature is high, damage of bituminous pavement resulted from low temperature is insignificant, hence this research excludes performance study of this type of mixture under low temperature condition.

There are four main tasks in this research. In Tasks 2, 3 and 4, the effects of RAP content, RAP ageing, bitumen grade were studied.

- Task 1: mix design to determine the optimum foaming condition (OFC), OMC, gradation, design bitumen content (DBC), and RAP content of the cold-recycled RAP mixture;
- Task 2: to study the permanent deformation property of the foamed asphalt mix;
- Task 3: to study the fatigue property of the foamed asphalt mix;
- Task 4: to study the moisture susceptibility of the foamed asphalt mix.

2.6.2 Methodology

Fig. 2.9 illustrates the flowchart of this research. Detailed approach is briefly described as follows (He and Wong 2002):

Step1: Collection and basic properties of materials

Materials include RAP, virgin aggregate, and bitumens. All these materials were collected form contractors or sponsors. RAP must be picked up from 2~3 different sites. As well as the typical bitumen used in the road paving industry in Hong Kong, high penetration bitumen possibly used to compensate for the harding of the binder in the RAP was also picked up for the purpose of studying effects of the bitumen grade on foaming characteristics and mix properties.

Different sizes of aggregate (including sand, crushed and natural gravel, fillers, hydraulic materials e.g. cement) should be collected.

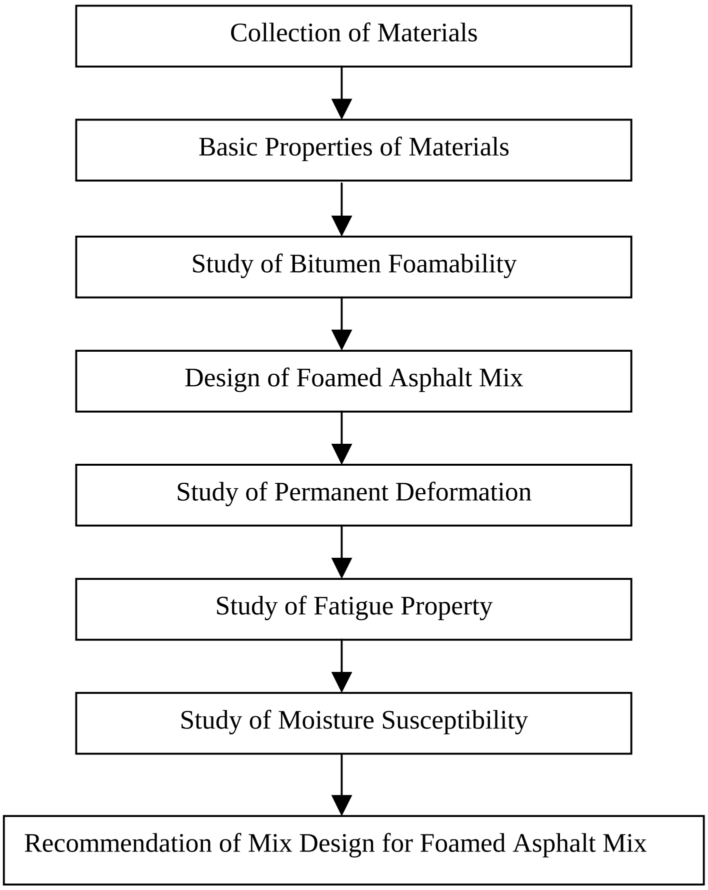


Fig. 2. 9 Flowchart of the research

Step 2: Materials' Properties

Basic properties of materials, including RAP and virgin materials, were tested in order to provide information for mix design and study of mix properties.

1. Virgin materials

Virgin binder tests include penetration, viscosity (by Bohlin high temperature rotational viscometer), softening point, and Superpave performance tests. In Superpave testing, bitumen's performance in three stages, viz. a) virgin bitumen, short-term aging after rolling thin film oven test (RTFOT), and long-term aging after PAV, were investigated. Dynamic shear rheology of bitumens was tested and analyzed through the Dynamic shear rheometer (DSR).

Virgin aggregate tests include sieve analysis, specific gravity.

2. RAP materials

RAP binder and RAP aggregate were tested. Centrifuge Extractor and Rota Vapor were used to extract and recover RAP binders from different types of RAP mixes

RAP binder tests include penetration, viscosity, soft point, and Superpave performance tests (not including aging after RTFOT and PAV).

RAP aggregate tests include sieve analysis, specific gravity, etc.

Step 3 Foamability of bitumen

Different virgin bitumens were foamed to test their foaming properties by laboratory foam bitumen plant. Test conditions, including added water content, bitumen temperature, and air pressure, were considered in the foaming test in order to study effects of conditions on the bitumen foamability.

After being tested and analyzed, the most important foaming parameters, half-life and the maximum expansion ratio under the optimum foaming condition (OFC) should be determined for production of foamed asphalt mixes. This step will provide the foundation for mix design in the Step 4.

Step 4 Mix design

Initially, gradation of the foamed asphalt mix was determined through the literature review and investigation of Hong Kong’s practice. Then OMC and DBC were determined. The former was tested using the modified compaction method for Marshall specimens with reference to ASTM D 1557. Indirect tensile strength test (TST) was used to determine the DBC. DBC was the bitumen content at which soaked specimens could reach the maximum.

After obtaining OFC, DBC and OMC, mixture was produced. Specimens containing different RAP contents were prepared for the following performance study.

Step 5 Study of permanent deformation

In this step, dynamic creep test was carried out to investigate properties of permanent deformation. Based on the test, effects of bitumen grade, content of RAP materials and ageing of RAP on resistance of foamed asphalt mixes to permanent deformation were studied. Different content of RAP (e.g. 0%, 20%, 40%, 60%) was added to the mix.

Step 6 Study of Fatigue properties

The NU-10 manufactured by Cooper Research Technology Ltd. was adopted to perform fatigue test for foamed asphalt mixes. NU-10 is a versatile testing machine. It can test not only the strength property and creep/deformation properties, but also the fatigue property of mixture (Cooper 2003). Like study of permanent deformation, effects of bitumen grade, content of RAP materials and ageing of RAP on fatigue resistance of the foamed asphalt mixes were studied.

Step 7 Study of Moisture susceptibility

In this study, evaluation method for moisture susceptibility of foamed asphalt mixes was determined based on the literature review. Specimens of two conditions, namely dry and soaked conditions, were tested by indirect tensile method and dynamic creep test to investigate the susceptibility of moisture-conditioned specimens to indirect tensile strength and permanent deformation.

In order to investigate the moisture susceptibility on fatigue properties and effect of conditioning methods on the fatigue properties, specimens under two moisture conditions (the soaked and the freeze-thaw conditions) were also tested by the indirect tensile fatigue test.

Finally, a FA mix with the best overall properties was recommended to use in Hong Kong based on tests and analysis of the mix properties.

In addition, comparison were made to hot mixed asphalt concrete in order to investigate the property difference between FA mix and hot mixed asphalt concrete.

CHAPTER THREE FOAMABILITY OF BITUMENS

3.1 LITERATURE REVIEW AND TASKS

Studying the use of foamed bitumen for cold-in-place recycle includes two aspects: one is to study the foamability of bitumen, and the other is to study the property of the foamed asphalt mixture. This chapter focuses on the former study. The latter will be introduced in other chapters.

3.1.1 History Of The Foaming Systems

The art of manufacturing foamed bitumen has a long and colorful history.

- The first break through come in 1957, when Prof. Ladis Casanyi of Iowa State University, demonstrated the addition of foam bitumen to marginal quality aggregates. He used a system where steam under pressure was introduced into hot bitumen (Csanyi 1957 and Csanyi 1962). In practice it was found to be totally impractical due to the complexity of his equipment and the inability of control the amount of steam that was added. He patented the method for combining bitumen with an aggregate material.
- The second break through came in 1971 when Mobil Oil Corporation patented their foaming system in Australia. After years of research in the late sixties and the early seventies, the process was modified by Mobil, the steam was replaced with cold atomized water (between 1.5% and 2.5% of cold water). The water was injected under high pressure into the hot bitumen in a specially designed expansion chamber, the produced foamed bitumen was then sprayed through nozzles onto unheated, damp aggregate which were then mixed to produce stabilized material.
- After the Mobil's patent rights expired in 1991, some new foaming systems were developed. The Scandinavian system, Nesotec OY 1994, a new generation system, was developed by Nestor Salminen, this was followed by Savalco Sweden and other "Home Made" Systems (Van der Walt et al 1999).

Currently foamed bitumens are used in many countries (i.e. South Africa, Australia, Canada, Mexico, Europe, Middle East and the Scandinavian Countries). There are three basic foaming processes currently available in their development order (Van der Walt et al 1999):

- The Mobil System (1971), a steam addition system;
- The Scandinavian System (Nesotec OY, Finland, 1994), a water saturation process;
- The Savalco System (Sweden), a combination of Mobil and Ladis Csanyi,

The Wirtgen system used in this research, developed in the mid-1990's, is a variation of the Mobil process. It injects both cold water and air into the hot bitumen in an expansion chamber.

3.1.2 Physical And Chemical Aspects Of Bitumen Foam.

Bitumen foaming process is a method of reducing the viscosity of bitumen to allow a dramatic improvement in wetting at the bitumen-aggregate interface. This is achieved by substantially increasing the surface area of the bitumen/aggregate contact and lowering the interfacial tension.

When water is heated rapidly above its boiling point it becomes stream. The rapid nature of this expansion and the subsequent cooling as the foam leaves the nozzles, together with the rapid contact made with damp and cold aggregates creates a multi phase composite system.

Foamed bitumen in this case will consist of bitumen, air, water vapour and water. As the added water is less than 2% in most instances and steam loss does occur, the morphological structure of foam is mostly air and bitumen. However some water will incorporate from the contact of the hot foam with cold damp aggregates (at optimum moisture content (OMC)) (Maccarrone et al 1993)).

Coating occurs at the aggregate foam interface at the moment of contact. Residual mobility aids in compaction. The foam collapses as the foam structure is unstable.

The fundamentals of the foaming process require consideration before the factors influencing the characteristics of foamed bitumen can be analyzed. The laws governing the behaviour of the bitumen during foaming are primarily physical, although chemistry, e.g. foaming agents, does also play a role (Jenkins et al 1999).

The moment that a cold water droplet (at ambient temperature) makes contact with the bitumen at 170 ^{0}C to 180 ^{0}C, the following events occur (Wirtgen GmbH. 2001):

- The bitumen exchanges energy with the surface of the water droplet heating the droplet to a temperature of 100 ^{0}C and cooling the bitumen.
- The transferred energy of the bitumen exceeds the latent heat of steam resulting in explosive expansion and the generation of steam. Steam bubbles are forced into the continuous phase of bitumen under pressure, in the expansion chamber.
- With spraying from the nozzle the encapsulated steam expands until a thin film of slightly cooler bitumen holds the bubble intact through its surface tension.
- During expansion, the surface tension of the bitumen film counteracts the ever-diminishing steam pressure until a state of equilibrium is reached.
- Due to the low thermal conductivity of bitumen and water, the bubble can remain stable for a period of time, usually measurable in seconds.

3.1.3 Characteristics Of Foamed Bitumen.

The foamed bitumen is characterized in terms of maximum expansion ratio and half-life. The **maximum expansion ratio (ERMax)** of the foam is defined as the ratio between the maximum volume achieved in the foam state and the final volume of the binder once the foam has dissipated. The **half-life (HL)** is the time, in seconds, between the moment the foam achieves maximum volume and the time it dissipates to half of the maximum volume.

The expansion ratio is used in many references to express the maximum foaming property. This is not accurate. Because expansion ratio is a function of time, it varies at different time. However there is only one maximum value, this value is the maximum expansion ratio. On the other hand, the expansion ratio can be used to describe decay property of the foamed bitumen, detailed introduction is explained in section 3.4.

The foaming characteristics of a specific bitumen are influenced by numerous factors, the most important being the following (Brennen et al 1983, Castedo-Franco et al 1983, Wirtgen GmbH 2001):

- Temperature of the bitumen, the foaming characteristics of the most bitumens improve with higher temperatures;
- Amount of water added to the bitumen, generally, the maximum expansion ration increases with an increase in the amount of water added, whilst the half-life decreases;
- Pressure under which the bitumen is injected into the expansion chamber, low pressures negatively affect both the maximum expansion ratio and half-life; and
- The presence of anti-foaming agents; such as silicone compounds

Characteristics can be normally reflected graphically. Fig 3.1 is a typical foaming characteristic. The example in Fig. 3.1 shows that working with a temperature 170 ^{0}C and adding 2.5% water will produce a foamed bitumen with an expansion ratio of 11 and half-life of 9 seconds.

The "best" foam is generally considered to be the one which optimizes both expansion and half-life. However, when foaming characteristics are extremely poor (both the maximum ratio is less than 5 and the half-life is below 5 seconds), it is difficult to achieve an acceptable mix. Consideration should then be given to using bitumen from a different source, or adding a foaming agent.

In addition, excessive asphaltenes in the bitumen composition has a negative influence on the foaming characteristics. Generally the greater the proportion of asphaltenes, the poorer the foam (Wirtgen GmbH 2001).

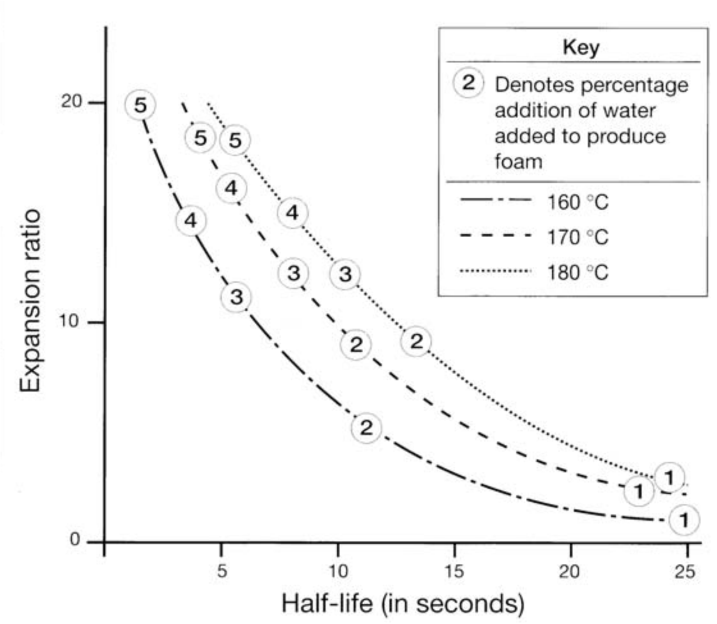


Fig.3. 1 Ideal foaming characteristics (Wirtgen GmbH 2001)

Unlike hot-mixed asphalt, material stabilized by foamed bitumen is not black. This is because the coarser particles of aggregate are not coated and are usually free of bitumen. When foamed bitumen comes into contact with aggregate, the bitumen bubbles burst into millions of tiny "spots" that seek out and adhere to the fine particles, specifically the minus 0.075 mm fraction. This results in a bitumen-bound filler that acts as a mortar between the coarse particles. There is therefore only a slight darkening in the colour of the material after treatment.

Abel (1978) found that bitumen which contained silicones could have reduced foaming abilities. By using certain surface-active additives it is possible to produce highly expanded and stable foamed bitumens with maximum expansion ratios greater than 15 and half-lives greater than 60 seconds (Maccarrone et al, 1995). Kendall et al (1999) also stated because the bitumen manufactured in Australia contained silicone which negatively affected foaming property of bitumen, 0.5% foaming agent was added to the bitumen in their trial project in order to improve the foaming characteristics. Holleran and Ky (1995) pointed out that special surface-active additives were used both in the bitumen phase and the water phase to produce a highly expanded and stable foam, typically this meant a volume expansion greater than 14 and a half-life greater than 60 seconds.

Lee (1981) also stated that there were no criteria as to what constituted a satisfactory foam. Ruckel et al (1983) recommend limits of 8-15 for the expansion ratio and at least 20 seconds for the half-life. Bowering and Martin (1976) showed that the cohesion and compressive strength of mixes were significantly greater when high expansion (15:1) foamed bitumen was used. For soil stabilization the recommended maximum expansion ratio was 8 to 15 and half-life was a minimum of 25 seconds (Bowering and Martin 1976, Bowering 1970). The cold recycling manual of Wirtgen Co. stated that

ERMax should be larger than 15 and half-life should be within 5 and 10 seconds for high-quality foamed bitumen (Wirtgen Gmbh. 2001).

Generally, only maximum expansion ratio and half-life are used to describe the foaming properties of the bitumen in most literature reference. The optimum foaming condition means that the maximum expansion ratio and half-life used to control the production of the foamed asphalt mix. Almost all of application adopts this method. However, characteristics of foamed bitumen should include decay property, which can be described by the decay curve. Herein the "foamability" is used to characterize all properties of the foamed bitumen, including the maximum expansion ratio, half-life and decay property.

Decay curve records the change of the expansion ratio with time. It bridges the maximum expansion ratio and half-life. If decay function of a foamed bitumen is obtained, its ERMax and half-life can be calculated from the decay function.

Decay property of bitumen can be clearly described by its decay function. Different bitumens have different decay properties, some decay fast in the foaming process, whilst the others decay slowly. The processing conditions affect decay property of the foamed bitumen. However, study on decay curve of foamed bitumen is seldom found in the literatures. Jenkins et al (2000) carried out foaming test of different bitumens, and then used the Foam Index to describe their foamability, the bitumen was selected based on the value of the Foam Index.

So far, few study answers how the viscosity affects bitumen's foamability and its decay property. Viscosity has a great impact on bitumen's property, hence it is required to be tested for mix design and construction in many countries. Therefore it is necessary to study the effect of viscosity on the decay property of the bitumen.

3.1.4 Tasks

Bitumens will show different foamability. It is clear that there are no absolute limits governing these characteristics when using foamed bitumen as a stabilizing agent. Various criteria occur in the literature references. This is due to many factors influencing bitumen foamability, including bitumen type, bitumen temperature, added water content, air pressure, and composition of bitumen. Therefore, it is necessary to study the foamability of bitumen used in this recycling research.

There are two tasks in the foamability study:

- Task 1: Study of foaming property
- Task 2 Study of decay property

In Task 1, the maximum expansion ratio and half-life of bitumens being used in Hong Kong will be tested. After analysis, the optimum foaming condition of bitumen will be determined to control the production of the foamed bitumen.

In Task2, the decay property of each bitumen will be studied based on the foaming tests. The Aim of this study is to obtain the decay functions for bitumens. Because foaming test is onerous, the obtained functions can be used to guide application of the same bitumens used in the future.

The next section will introduce experiment design and foaming test. Section 3.3 and 3.4 will describe foaming property study and decay property study. Section 3.5 will summarize results obtained in the foamability study.

3.2 EXPERIMENT DESIGN AND FOAMING TEST

3.2.1 Introduction To Test Equipment

Foamed bitumen is produced by injecting a small amount of atomized water into hot bitumen in a specially designed expansion chamber. Fig. 3.2 illustrates the process of the foamed bitumen in the expansion chamber. The Wirtgen WLB 10 plant, as shown in Fig. 3.3(a), was designed to produce foamed bitumen under laboratory condition. The foamed bitumen produced by this unit is similar to that produced by the foamed bitumen systems mounted on large recycling machines. In this study, bitumen will be foamed by Wirtgen WLB 10.

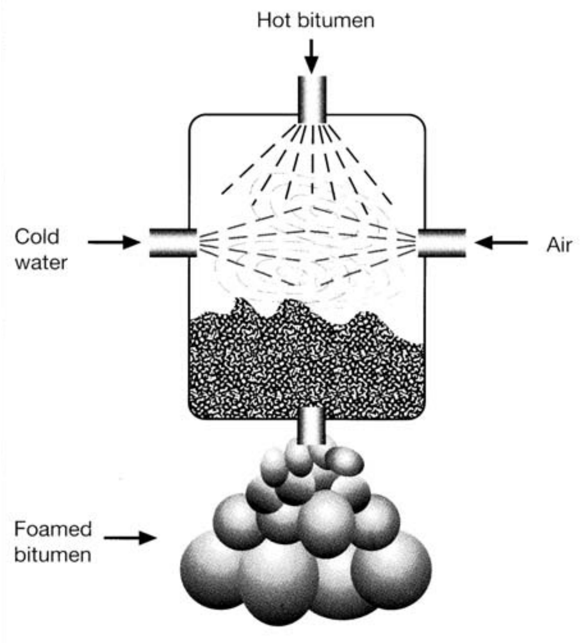
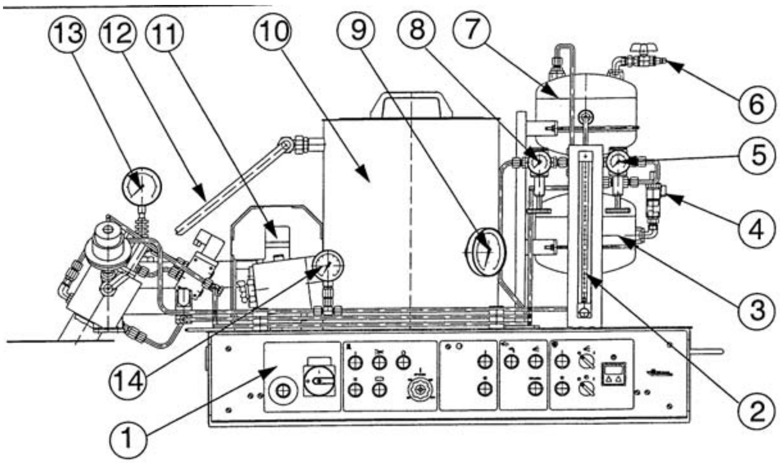


Fig.3. 2 Expansion chamber of foamed bitumen plant

(a) Photo of the machine



(b) Configuration (Wirtgen GmbH 2000)

Fig.3. 3 Wirtgen WLB 10 laboratory foamed bitumen unit

Note: In Fig 3.3 (b), 1=control unit, 2=flow meter/adjuster (l/h), 3=water tank, 4=connection port (external water supply), 5=manometer/adjuster (water tank pressure), 6=connection port (external compressed air), 7=compressed air tank, 8=manometer/adjuster (compressed air for foam production), 9=thermometer (bitumen tank temperature), 10=bitumen tank, 11=bitumen bump, 12=bitumen tank return line, 13=manometer (hot bitumen pressure during foam production), 14=manometer (water pressure during foam production).

The Wirtgen WLB 10 plant is used to determine the foaming properties of a bitumen sample and to produce the foamed bitumen required for preparing treated samples in order to determine the optimal application rate for stabilization (such as in the mix design procedure and construction control). The foamed bitumen produced by the Wirtgen WLB 10 is mixed with a material sample in a Hobart mixer for mix design purposes.

The Wirtgen WLB 10 plant mainly consists of air supply sub-system, water supply sub-system, bitumen tank and heating sub-system, bitumen circulation sub-system, expansion chamber, control unit and circuit sub-system, etc. Fig 3.3(b) illustrates the configuration of this machine.

Before testing, the Wirtgen WLB 10 should be prepared for use according the following procedures (Wirtgen GmbH 2000):

1. Filling the water tank
The water tank can only be filled when there is no air pressure in the system. A level indicator shows the water level in the water tank. After filling, the water tank and its relevant valves must be closed.

2. External connections
Connect the plant to a pressurized air supply (minimum 8 bar).
Connect the plant to a power supply.
Turn on the main circuit breaker.

3. Heating the bitumen to the required temperature
Switch on the heating circuits.
Add bitumen to the kettle (minimum working volume of bitumen is 5 litres).
Ideally, the bitumen sample should be placed in a forced-draft oven overnight at 100 ^{0}C. The following morning, increase the temperature to 180 ^{0}C for approximately 2 hours before pouring the bitumen into the kettle.

4. Circulation of the bitumen in the system
The bitumen pump should only be started once all bitumen is fluid; the pump has reached its operating temperature and air pressure is present in the system. The pump operating temperature is indicated when the “pump heating” signal light turns off.

If the bitumen pressure gauge registers pressure during circulation, then there is a blockage in the system and the pump must be switched off immediately and the blockage must be removed.

5. Setting the timer
The discharge timer regulates the opening time for the bitumen valve at the entrance to the expansion chamber, thereby allowing for a metered amount of bitumen to be discharged into the expansion chamber.

6. Checking the bitumen discharge flow rate

The bitumen discharge flow rate should be regularly checked, especially when changing bitumen grades. This is achieved by the following procedure:

(1) Close off the water flow to the expansion chamber at the shut of valve;
(2) Switch off the airflow at the control panel;
(3) Set the discharge time at the timer for a period of 5 seconds;
(4) Press the automatic discharge button at the timer;
(5) Collect the bitumen discharged through the expansion chamber in a steel container;
(6) Weigh the discharged bitumen;
(7) Calculate the bitumen flow rate by dividing the discharged bitumen by the discharge time. This procedure is repeated with discharge times of 2 and 4 seconds to achieve and average (in grams per second).

7 Set the foaming water flow

Place a container under the nozzle to collect the foaming water. Manually operate the water and air switches on the control panel to start the water flow. The flow rate can be adjusted by turning the knob at the bottom of the flow meter. The flow rate is read at the bottom edge of the washer fixed to the top of the measuring cone in litres per hour.

The water flow rate is calculated according to Eq. 3.1 (Wirtgen GmbH. 2001)

$$Q_{Water} = \frac{Q_{Bit} * W_{add} * 3.6}{100} \tag{3.1}$$

where, Q_{Bit} =bitumen flow rate (g/s)

Q_{Water} = water flow rate (l/h)

W_{add} =foam water content (%)

3.2.2 Materials

Bitumen grade and rheology affect its foamability. Bitumen selection is largely influenced by ambient temperatures. Hard bitumens (Penetration grade <100) are usually preferred in hot climate. Hong Kong is located in the tropical region, therefore the bitumen with penetration grade less than 100 can be considered in this study.

Finally Shell Penetration-grade 60 and Shell Penetration-grade100 are selected in the foamability investigation. They were provided by Shell Hong Kong Ltd. Shell 60 bitumen is suitable to be used in the tropical climate in southern China, it is the only one used in the paving industry in Hong Kong. Shell 100 bitumen is suitable to be used in northern China, and is selected to compare with shell 60.

In this study, the basic properties of penetration, softening point and high temperature viscosity of the two bitumens were tested in accordance with corresponding specifications respectively (ASTM D5 1997, ASTM D36 1995 and ASTM D4402 2002). Due to foaming temperature being in the range of 160^{0}C to 200^{0}C, bitumen viscosity is tested with Brookfield rotary viscometer, a kind of high temperature viscometer recommended by SHRP. Test results are shown in Table 3.1. It can be found that viscosity of Shell 60 is larger than that of Shell 100. Viscosity difference between the two bitumens increases with an increase of temperature, the value of Shell 60 is 1.264 times and 3.9 times of that of Shell 100 at 135 ^{0}C and 180 ^{0}C respectively. Fig. 3.4 demonstrates the result of temperature versus viscosity. It is obvious that at the same temperature, viscosity of Shell 60 is higher than that of Shell 100. The values of the former at 160, 170 and 180 ^{0}C are shown to be 1.42, 1.69 and 3.9 times those of the latter respectively. The difference between two bitumens' viscosity increases with increase of temperature.

Table 3. 1 Basic properties of Shell 60 and Shell 100

Properties	Bitumen	
	Shell 60	**Shell 100**
Penetration (25 ^{0}C, 100g, 5s) (0.01mm)	66.6	91.2
Softening Point (Ring and Ball) (^{0}C)	49.5	46.3
Viscosity at 135 ^{0}C (mPa.s)	468.0	370.2
Viscosity at 160 ^{0}C (mPa.s)	96.5	68.0
Viscosity at 170 ^{0}C (mPa.s)	50.0	29.5
Viscosity at 180 ^{0}C (mPa.s)	19.5	5.0

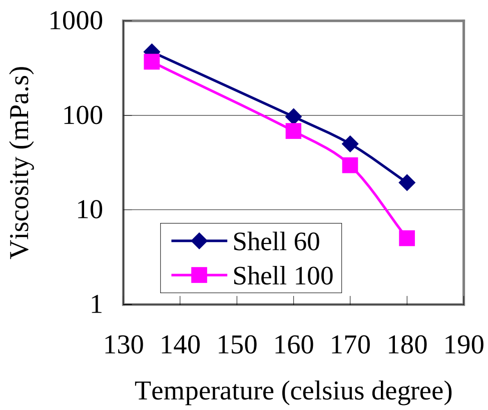


Fig.3. 4 Temperature versus viscosity of two bitumens

3.2.3 Testing Methodology

In this study, bitumen temperature (Temp), water content (WC) and air pressure (AP) were selected as factors to investigate their effects on the foamability by varying values of these factors. Four levels of temperature were set in test: viz. 160°C, 170°C, 180°C and 190°C. Five levels of water content were set: viz. 1%, 2%, 3%, 4% and 5%, by mass of bitumen. Four levels of air pressure were applied in the test: namely 1bar, 3bar, 5bar and 7bar (1 bar is equivalent to 100 kPa).

After analyzing results of the Shell 60, it is found that 1 bar pressure adversely affects both the ERMax and half-life. This finding is consistent with that of Wirtgen Cold Recycling Manual (Wirtgen GmbH 2001). Hence 1 bar level was canceled in the foaming test of the Shell 100, only three levels of air pressures (3bar, 5bar and 7bar) were applied in the Shell 100's test. There were totally 80 conditions (4 Temps X 5 WCs X 4 APs) for Shell 60 and 60 conditions for Shell 100.

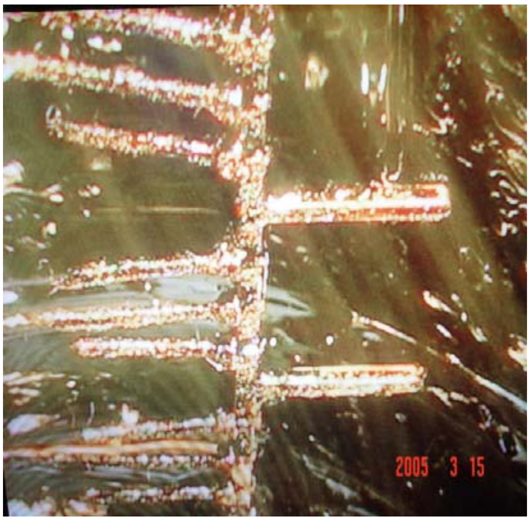
The two bitumens were subjected to foaming test under each condition using WLB10 laboratory foam bitumen plant. For each condition, 3 duplicate tests were conducted.

Generally ERMax and half-life are recorded by visual observation in the foaming test. However, accuracy of the half-life is usually affected by operator's skill and experience. To minimize the human error on recording of the half-life results, digital video recorder was used to record the whole foaming process.

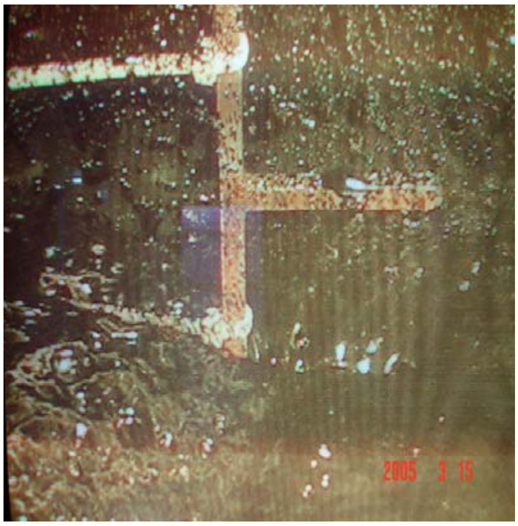
The recording was performed from the point when bitumen began to spray to the point when the bitumen foam almost completely dissipated. Since it took a long time for the bitumen foam to completely dissipate, it was assumed that the end of record time was 80 seconds after spraying, at which time expansion ratio of the foamed bitumen almost remained stable.

WLB 10 laboratory plant sprayed 500 grams of bitumen each time. It spent about 5 seconds in terms of flow rate of bitumen. It was assumed that time was zero when bitumen spray was finished. Therefore recording time was in the range of minus 5 to 80 seconds.

During the whole testing, the foaming process was recorded by video recorder. The expansion ratios of each foaming test were written down through watching videotape at each second from the spraying time to 15th second, and 20th second, 30th second, 60 second and 80th second. Decay curve, a process curve of bitumen foaming with time, was plotted based on the recorded expansion ratios. In the mean time, two parameters of foaming properties, the maximum expansion ratio and half-life, were also written down by watching videotape and recorded. Fig 3.5 shows processing of data recording by watch videotape. Fig3.5 (a) is the picture taken at the beginning of the spraying, and Fig 3.5 (b) is another picture taken when the expansion ratio reached the ERMax.



(a) at the beginning of spraying



(b) at ERMax

Fig.3. 5 Recording of data by watching videotape

3.3 FOAMING PROPERTIES

3.3.1 Results

After recording, ERMaxs and half-lives of the two bitumens are obtained and listed as shown in Table 3.2 and Table 3.3. Each value in these two tables is the averages of results of triple tests under different conditions.

3.3.2 Effect Of Factors On Foaming Properties

ERMaxs and half-lives were analyzed with multiple analysis of variance (MANVOA) in order to examine the effect of factors and their interaction on the test results. MANVOA was carried out using Minitab software (Minitab 2000). In addition to the influencing factors themselves, the interaction of factors will also affect the testing results. A α-level of 0.05 was selected. The response variables were ERMax and half-life. Factors were temperature, air pressure and water content.

The most important statistic in the ANOVA table is the P-value, which is the probability computed in the F-test to reject the null hypothesis when it is true (Montgomery1997). There is a P-value for each term in the ANOVA table except for the Error and Total. The P-value of a term can determine whether the effect of this term on the testing result is significant. In brief, if the P-value is less than or equal to a α-level selected, the effect on this term is significant. If the P-value is larger than a α-level selected, the effect is not significant.

As for Shell 60 bitumen, the ANOVA of half-life is shown in the Table 3.4. This table indicates that there is no significant evidence for the effect of temperature on the half-life, because the P-value of this term is 0.110 and larger than α-level. Except temperature, there are significant evidences for the effects of water content, air pressure and their interaction, including interactions of temperature and air pressure, temperature and water content, on the half-life. Based on the ANOVA of ERMax of Pen 60 bitumen, it can be concluded that there are significant evidences for effects of all factors and their interaction on the ERMax.

Table 3. 2 ERMaxs and Half-lives of Shell 60 Bitumen

Temp (˚C)	AP (Bar)	WC (%)	ERMax	HL(s)	Temp(˚C)	AP(Bar)	WC(%)	ERMax	HL(s)
160	1	1	10.2	11.9	180	1	1	7.5	14.0
		2	15.0	6.6			2	14.2	5.3
		3	20.3	5.3			3	18.3	3.9
		4	23.8	6.7			4	20.7	2.9
		5	25.0	6.4			5	21.8	2.7
	3	1	8.5	14.8		3	1	6.5	25.8
		2	16.2	4.2			2	14.2	5.1
		3	19.7	4.0			3	21.0	3.4
		4	23.0	5.3			4	23.5	2.6
		5	26.0	7.5			5	26.3	3.0
	5	1	8.4	14.1		5	1	7.0	30.2
		2	15.7	5.3			2	13.7	7.2
		3	20.5	4.7			3	20.7	4.2
		4	21.7	5.3			4	26.3	3.6
		5	24.5	4.9			5	30.3	4.0
	7	1	6.7	21.8		7	1	5.7	33.9
		2	14.2	4.9			2	13.2	7.8
		3	20.0	4.1			3	20.8	3.5
		4	22.7	4.5			4	25.3	3.5
		5	26.3	5.5			5	30.2	4.2
170	1	1	9.3	13.1	190	1	1	8.7	7.8
		2	15.8	6.0			2	11.5	3.2
		3	20.8	4.0			3	11.8	4.5
		4	24.0	2.6			4	11.7	4.4
		5	24.2	3.2			5	11.9	4.6
	3	1	8.5	15.4		3	1	7.0	22.8
		2	15.5	4.0			2	13.0	6.2
		3	21.5	3.3			3	18.0	3.0
		4	22.8	3.0			4	23.0	2.8
		5	23.3	3.8			5	25.3	3.3
	5	1	6.8	32.0		5	1	6.0	44.0
		2	15.3	5.1			2	13.2	6.9
		3	22.3	3.8			3	19.0	2.7
		4	27.5	4.1			4	25.2	2.8
		5	28.2	4.6			5	28.5	3.3
	7	1	6.6	27.2		7	1	6.7	33.4
		2	15.0	5.4			2	11.3	8.8
		3	21.0	4.7			3	20.2	3.4
		4	26.5	6.3			4	23.7	2.3
		5	30.5	7.1			5	28.8	3.0

Table 3. 3 ERMaxs and Half-lives of Shell 100 Bitumen

Temp (℃)	AP (Bar)	WC (%)	ERMax	HL(s)	Temp(℃)	AP(Bar)	WC(%)	ERMax	HL(s)
160	3	1	8.0	11.1	180	3	1	6.6	11.4
		2	12.0	7.9			2	10.5	6.0
		3	13.8	8.9			3	12.5	5.7
		4	15.2	10.2			4	13.8	5.8
		5	17.0	9.8			5	14.3	5.7
	5	1	7.1	10.5		5	1	6.9	10.7
		2	11.8	8.7			2	10.5	6.8
		3	14.0	8.8			3	12.0	6.6
		4	16.0	10.1			4	14.0	5.6
		5	18.0	10.8			5	15.5	6.3
	7	1	7.5	5.3		7	1	6.4	7.1
		2	12.2	9.2			2	8.8	6.2
		3	15.3	7.8			3	12.0	5.6
		4	17.3	7.7			4	14.5	6.1
		5	18.3	11.6			5	22.2	6.4
170	3	1	7.5	16.3	190	3	1	7.2	10.3
		2	12.0	5.6			2	8.8	5.3
		3	14.3	6.8			3	10.5	5.8
		4	16.0	8.4			4	13.3	7.7
		5	16.5	7.2			5	13.8	6.8
	5	1	6.7	8.8		5	1	7.3	14.1
		2	11.3	8.0			2	10.2	6.9
		3	13.3	7.4			3	11.8	5.2
		4	15.7	8.4			4	12.7	4.7
		5	16.5	8.0			5	15.5	6.3
	7	1	9.2	13.9		7	1	8.5	9.6
		2	11.6	3.4			2	9.2	12.7
		3	13.5	7.0			3	10.4	7.3
		4	15.8	7.2			4	14.5	4.4
		5	17.7	8.3			5	16.2	7.9

Table 3. 4 ANOVA Result of Half-life of Shell 60

Variance Source	Degrees of Freedom	Sum of Squares	Mean Squares	F-value	P-value
Temp	3	62.20	20.73	2.04	0.110
AP	3	637.35	212.45	20.92	0.000
WC	4	12736.2	3184.04	313.59	0.000
Temp*AP	9	274.18	30.46	3.00	0.002
Temp*WC	12	1012.53	84.38	8.31	0.000
AP*WC	12	2107.72	175.64	17.30	0.000
Temp*AP*WC	36	910.30	25.29	2.49	0.000
Error	160	1624.55	10.15		
Total	239	19364.99			

Note: *means interaction, WC*AP means the interaction of WC and AP.

As for Shell Pen 100 bitumen, except the interaction of temperature and air pressure, there are significant evidences for the factors on the ERMax. Except air pressure, the interaction of air pressure and temperature, there are significant evidences for factors on the half-life. The ANOVA of ERMax of Pen 100 is listed in the Table 3.5.

Table 3. 5 ANOVA Result of ERMax of Shell 100

Variance Source	Degrees of Freedom	Sum of Squares	Mean Squares	F-value	P-value
Temp	3	142.383	47.461	69.07	0.000
AP	2	26.139	13.07	19.02	0.000
WC	4	1913.792	478.448	696.25	0.000
Temp*AP	6	8.948	1.491	2.17	0.050
Temp*WC	12	53.805	4.484	6.52	0.000
AP*WC	8	51.81	6.476	9.42	0.000
Temp*AP*WC	24	83.439	3.477	5.06	0.000
Error	120	82.462	0.687		
Total	179	2362.777			

All MANOVA results are summarized in Table 3.6, From this table it is found that there are significant evidences for effects of all factors and their interaction on ERMax of Shell 60. Except temperature, effects of other factors and all interactions on half-life of Shell 60 are significant. MANOVA results of Shell 100 differ from those of Shell 60. Apart from the interaction of temperature and air pressure, all other factors and their interaction significantly affect ERMax of Shell 100. Except air pressure, interaction of temperature and air pressure, other factors significantly affect half-life of Shell 100.

Table 3. 6 MANOVA results of Shell 60 and Shell 100

Bitumen	Response	Term						
		Temp	AP	WC	Temp*AP	Temp*WC	AP*WC	Temp*AP*WC
Shell 60	ERMax	V	V	V	V	V	V	V
	HL	**X**	V	V	V	V	V	V
Shell 100	ERMax	V	V	V	**X**	V	V	V
	HL	V	**X**	V	**X**	V	V	V

Note: V denotes significant evidence, X denotes no significant evidence. * denotes the interaction. $\alpha=0.05$.

It is noted that temperature insignificantly affect the half-life of Shell 60 which has higher viscosity, on the contrary, it significantly affects the half-life of Shell 100 which has lower viscosity. Air pressure significantly affects the half-life of Shell 60, but it has no significant effect on that of Shell 100.

3.3.3 Optimum Foaming Condition (OFC)

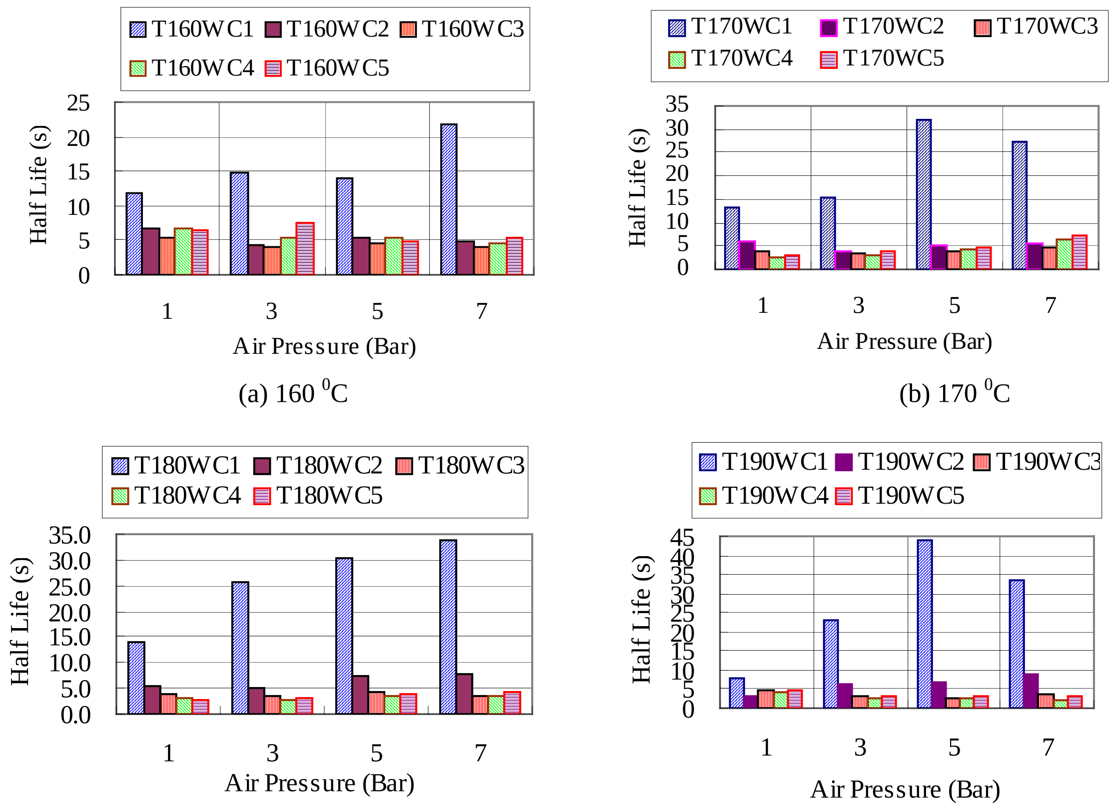
3.3.3.1 Determination of OFC

In general, the better the foaming characteristics, the better will be the quality of the resulting mix. There are no absolute limits governing these characteristics when using foamed bitumen as a stabilizing agent. Large expansion ratios obtained by sacrificing half-life, or vice versa, generally produce a poorer quality of mix than when both characteristics are optimal (see Fig 3.1) (Wirgten GmbH 2001).

However, when foaming characteristics are extremely poor (both ERMax is less than 5 and half-life is below 5 seconds) it is difficult to achieve an acceptable mixture. Therefore the “best” bitumen foam is generally considered to be the one which optimizes both ERMax and half-life (Wirtgen Gmbh. 2001 and Nataatmadja 2001).

In order to guide production of foamed asphalt mix in the next performance study, OFCs of the two bitumens should be determined based on the test results.

Half-lives of Shell 60 at different conditions are shown in Fig.3.6. From this figure, it can be found that most of the half-lives are less than 5 seconds when water content is larger than 2%. This implies that half-life is very poor when water content exceeds 2%. Therefore water content should not exceed 2% in order to obtain a good half-life.



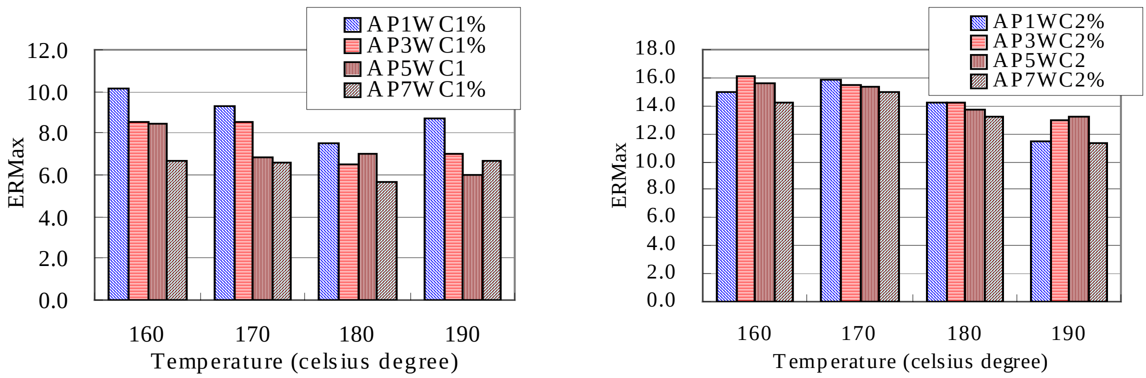
(a) 160 ^{0}C (b) 170 ^{0}C

(c) 180 ^{0}C (d) 190 ^{0}C

Fig.3. 6 Half-lives of Shell 60 at different conditions

Fig.3.7 demonstrates temperature effect on ERMax of Shell 60 at the condition of 1% and 2% water content. This figure indicates that when water content is less than or equal to 2%, ERMaxs at 160 and 170 ^{0}C are larger than those at 180 and 190 ^{0}C. This result is obviously different from the findings of Wirtgen Co., the foaming characteristics improve with high temperature (Wirtgen Gmbh. 2001). Therefore 180 and 190 ^{0}C are not considered in the determination of OFC.

When water content is less than 2%, half-lives at 1 bar and 3 bar are usually less than those at 5 bar and 7 bar in Fig.3.6. Hence 5 bar and 7 bar are preferred rather than 1 bar and 3 bar. In half of the cases (170 and 190 ^{0}C), half-lives at 5 bar are larger than those at 7 bar. Because 7 bar is too high and may cause "blast" of bitumen bubble, 5 bar is finally selected as the optimum air pressure.



(a) 1% water content (b) 2% water content

Fig.3. 7 Temperature effect on ERMax of Shell 60 at 1% and 2% water content

In summary, the optimum foaming condition can be found from the combination of 5 bar pressure, 160 and 170 ^{0}C temperature, 1% and 2% water content. Generally, the balance-point method is usually used to determine the optimum foaming condition. Balance-point is an intersection of ERMax curve and half-life curve. It optimizes both ERMax and half-life by maximizing the expansion ratio and half-life of the foamed bitumen.

Fig 3.8 demonstrates this optimization method. When water content is approximate 1.8%, the maximum expansion ratio and half-life obtain the "best" value. The ERMax and the half-life are approximately 8.5 and 14 seconds respectively.

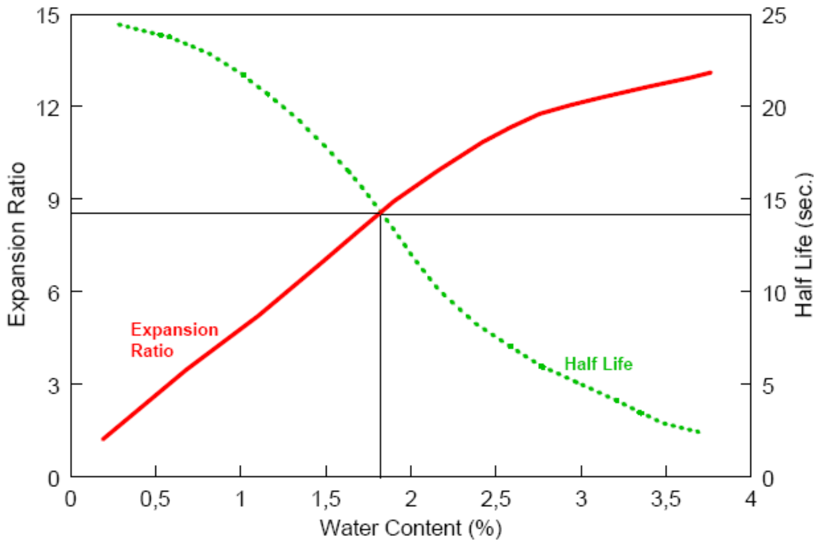
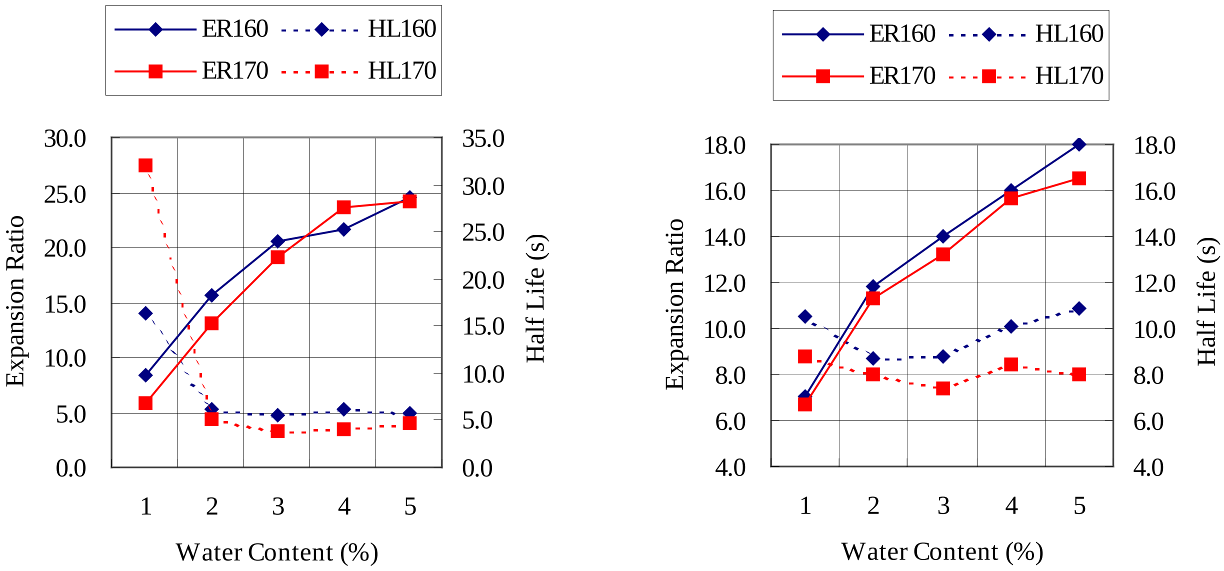


Fig.3. 8 Optimizing foam properties (Muthen 1998)

Fig.3.9 (a) demonstrates effect of water content on ERMax and half-life of Shell 60. This figure exhibits that at the condition of 170 ^{0}C and 1.7% water content, ERMax of 13 and half-life of 18 seconds can be achieved at the balance-point. Another balance-point occurs at the condition of 160 ^{0}C and 1.35% water content at this condition ERMax is 11 and half-life is 15.5 seconds. Comparing two balance-points, the condition of 5 bar pressure, 170 ^{0}C and 1.7% water content is selected as the optimum foaming condition for Shell 60 bitumen due to larger ERMax and half-life obtained.



(a) Shell 60 (b) Shell 100

Fig.3. 9 Effect of water content on ERMax and Half-life at condition of 160 ^{0}C, 170 ^{0}C and 5bar

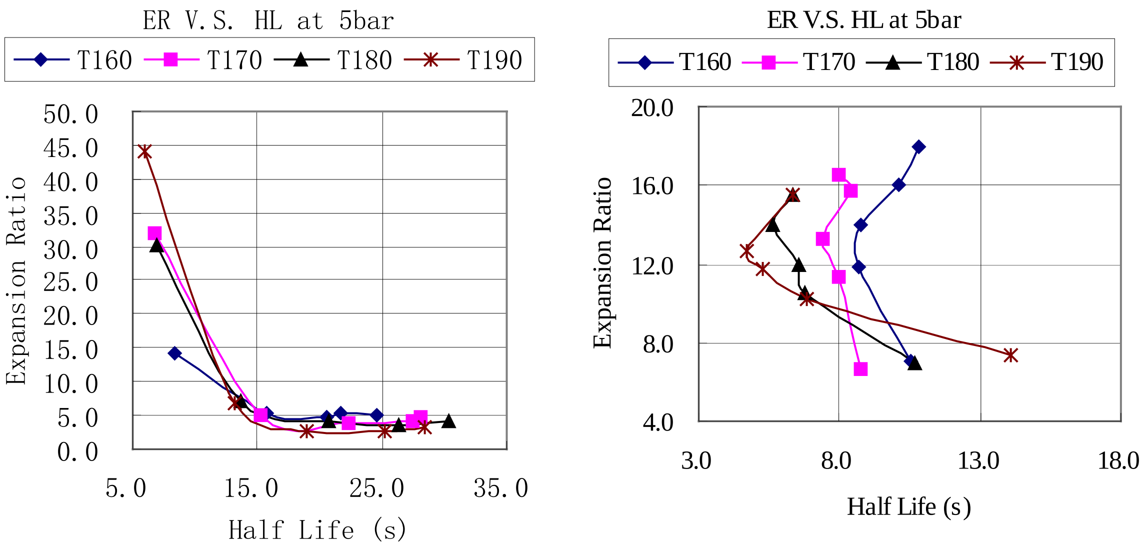
For Shell 100, effect of air pressure on foamability is similar to that of Shell 60. 5 bar is also selected as the optimum air pressure. It is also found that ERMax decreases with an increase of temperature. This implies that 160 ^{0}C can be considered as the optimum temperature.

Effect of water content on ERMax and half-life of this bitumen is shown as in Fig.3.9 (b). If the optimum foaming condition is determined using the balance-point method, it can be found that ERMax is very poor, no ERMax is larger than 10. Therefore this method is unsuitable for Shell 100.

When determining water content, it is noted that the selection with only high water content value which may cause lower half-life should be avoided. After comparison, 3% of water content can be determined eclectically. Hence the optimum foaming condition for Shell 100 is 3% water content, 5 bar pressure and 160 ^{0}C, ERMax is 14 and half-life is 8.8 at this condition. This result is almost the same as values recommended by Wirtgen manual (ERMax>15 and half-life=5~10 seconds).

3.3.3.2 ERMax versus Half-life

Based on test results, plots of ERMax versus half-life of the two bitumens were obtained and analyzed. Fig.3.10 contains the two bitumens' plots at 5 bar condition. For each a curve in these two figures, a point represents a water content.



(a) Shell 60 (b) Shell 100

Fig.3. 10 ERMax versus Half-life of the two bitumens at 5bar

Fig.3.10(a) exhibits that Shell 60 bitumen has a clear relationship between ERMax and half-life, ERMax decreases with an increase of half-life. This is due to the increase of ERMax and the decrease of half life with an increase in the amount of water added. It indicates that it is impossible to obtain the maximum value of both ERMax and half-life at the same time.

Fig.3.10(b), however, shows that there exists an inverse relationship between ERMax and half-life of Shell 100, the cause of which is not clear. At other air pressure conditions, the same conclusion of the two bitumens can be also drawn.

3.3.4 Difference Of Foaming Properties Between The Two Bitumens

3.3.4.1 Mechanism analysis of difference of foaming properties

Comparison of ERMax and half-life of the two bitumens exhibits that almost all ERMaxs of Shell 100 are smaller than those of Shell 60. The difference of ERMax between the two bitumens is very significant (Table 3.2 and Table 3.3). On the other hand, most of the half-lives of the Shell 100 are larger than those of Shell 60 (Table 3.2 and Table 3.3). Fig.3.11 illustrates the substantial ERMax difference between the two bitumens at 180 ^{0}C, 5bar and 3% water content.

Fig.3.4 exhibits temperature effect on viscosity of the two bitumens.

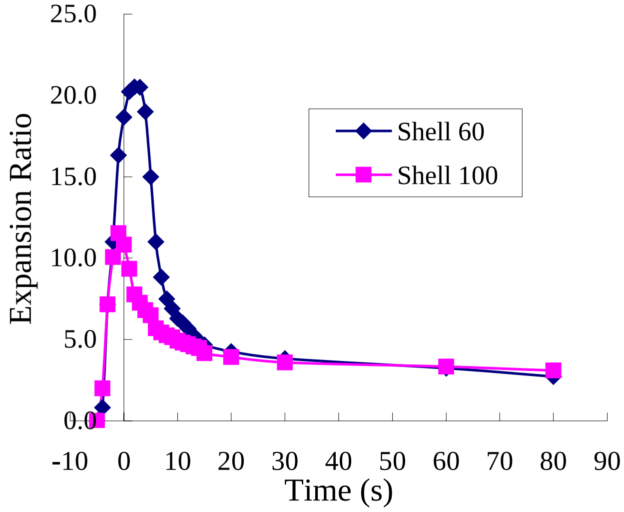


Fig.3. 11 Decay curves of the two bitumens at 180 ^{0}C, 5bar and 3% water content

There are two important causes of the collapse of foamed bitumen bubbles with time. One is the temperature reduction of the steam in bubbles due to their contact with ambient air at lower temperature. If the rate of temperature and pressure reduction within a bubble exceeds its recovery rate, the bubble will collapse. The other is the surpassing the elongation limit of bubble. If the steam pressure inside the bubble extends the bubble film beyond its ductile limit, the bubble will fail. Large bubbles will fail first allowing steam to escape. If a small water droplet is still present in the bubble, this low-energy bubble will collapse but will take a longer time (Jenkins et al 1999).

Bubbles of Shell 60 have larger surface tension due to their higher viscosity. They are more likely to collapse in the first mode than in the second mode. Numerous unbroken bubbles accumulate continually, resulting in a larger ERMax. It takes a longer time for expansion ratio to reach the maximum.

On the contrary, bubbles of Shell 100 have smaller surface tension; they are more likely to collapse in the second mode than in the first mode. Theoretically, before expansion ratio reaches ERMax, the number of unbroken bubbles of Shell 100 is smaller than that of Shell 60, and the height of these accumulated bubbles is lower. Therefore ERMax of Shell 100 is smaller than that of Shell 60, and the occurring time of Shell 100' ERMax comes earlier than that of Shell 60. This is the reason why ERMax occurring time of Shell 60 is at about 0 to 5 seconds, whilst that of Shell 100 is at about zero. In the test of Shell 100, "explosion" (simultaneously collapse) appeared in many conditions before expansion ratio reached the peak. This is due to simultaneous collapse of numerous bubbles. This fact confirms the above analysis.

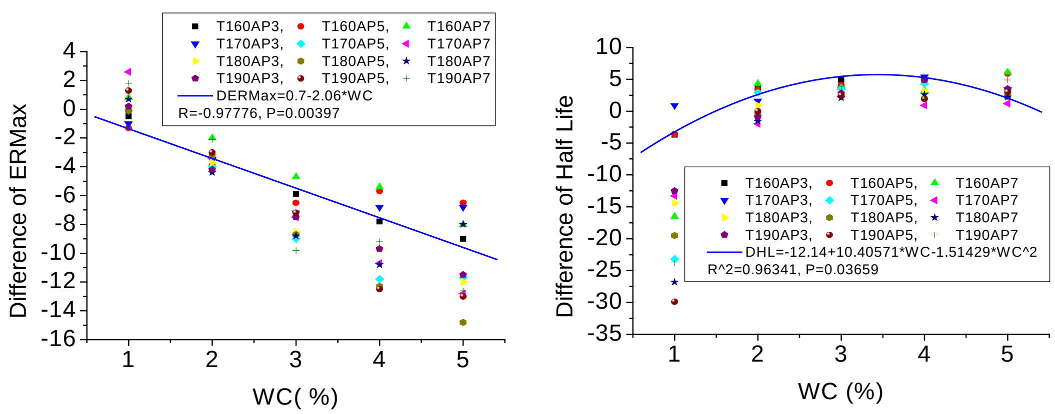
Temperature and air pressure in the bubble reduce with time in the course of test due to its contact with ambient air. After Shell 60 bitumen reaches its ERMax, many bubbles collapse in the first mode, and expansion ratio decreases swiftly. Therefore half-life of the Shell 60 is relatively short.

However, bubbles of Shell 100 are apt to fail in the second mode. Before expansion ratio reach the peak, many bubbles collapse, and new bubbles with less energy are then generated. It will take more time to collapse again for these low-energy bubbles. Hence half-life of Shell 100 is longer than that of Shell 60.

However, comparison of the foaming behavior of the two bitumens was not conducted at the different temperature selected such that the viscosity is the same.

3.3.4.2 Effect Of Water Content On Foamability Difference

With an increase of water content, between Shell 60 and Shell 100, there is a linear trend for the ERMax difference shown in Eq. 3.2, and a second order polynomial trend for the half-life difference shown in Eq. 3.3. Fig.3.12 demonstrates ERMax and half-life differences versus water content of the two bitumens. Fitting of these two bitumens shown in Fig.3.12 have the best good-of-fitness. Difference of ERMax/half-life is defined as the result of ERMax/half-life of Shell 100 minus result of Shell 60.



(a) ERMax (b) Half-life

Fig.3. 12 ERMax and Half-life differences versus water content

For the linear fitting, coefficient of relationship (R) can be used to represent the good-of-fitness, for the polynomial fitting, the coefficient of determination (R-square) can be used to represent the good-of-fitness. R and R-square of Eq.3.2 and Eq.3.3 are close to 1. Probabilities of test are smaller than the α-level (0.05) selected.

$$DERMax = 0.7 - 2.06 \times WC \tag{3.2}$$

R=-0.97776, P=0.00397

$$DHL = -12.14 + 10.4057 \times WC - 1.5143 \times WC^2 \tag{3.3}$$

R^2=0.96341, P=0.03659

Where: $DERMax=ERMax_{100}-ERMax_{60}$, $DHL=HL_{100}-HL_{60}$.

3.4 DECAY PROPERTY

Decay properties of the two bitumens and effects of viscosity on their foamability are presented in this section. In order to establish decay functions to describe the decay curves, non-linear fitting and best subset regression methods are adopted. Effect of viscosity on decay property is also discussed.

3.4.1 Results And Analysis

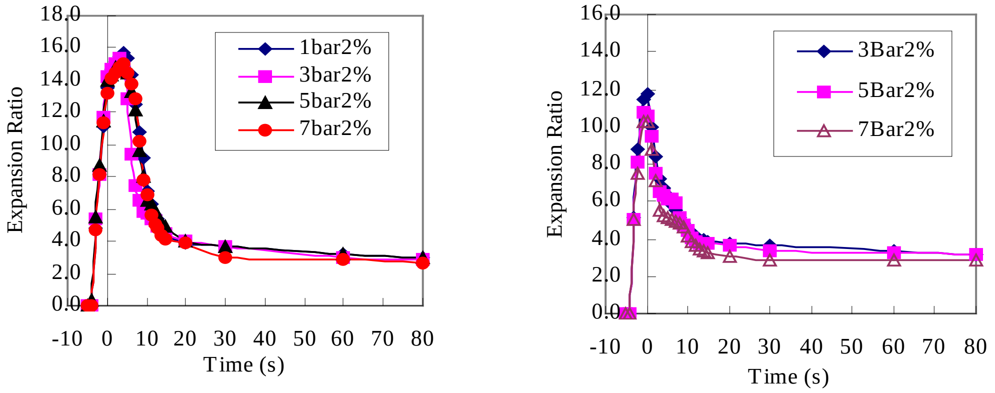
Based on the video results, the expansion ratios of each foaming test were written down through watching videotape at each second from the spraying time to 15th second, and 20th second, 30th second, 60th second and 80th second. Decay curve was plotted based on the recorded expansion ratios.

Decay curves under all conditions were plotted. They were analyzed and conclusions are drawn as follows:

(1) Air pressure has little effect on decay curves; curves at various air pressures are very close to each other. Fig. 3.13 illustrates a case of the decay curves of the two bitumens at the condition of 170^{0}C and 2% water content.

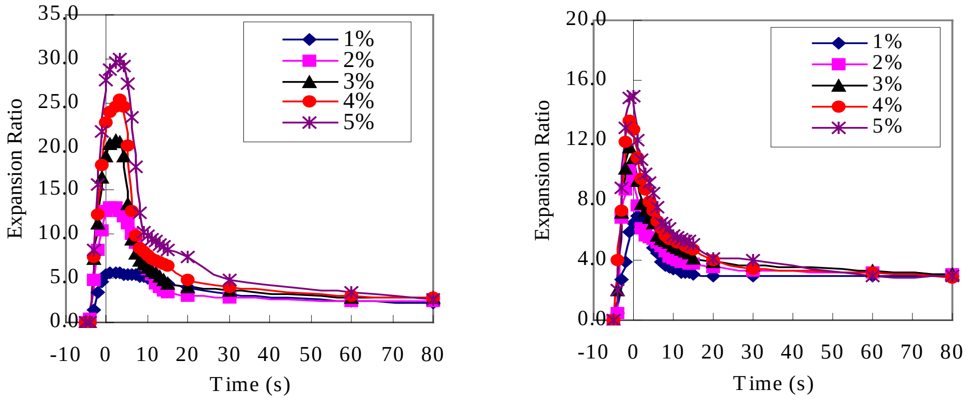
(2) Like decay curves of the two bitumens at the condition of 180^{0}C and 5 bar pressure illustrated in Fig. 3.14, water content significantly affects decay curves. ERMax of the two bitumens increases with an increase of water content.

(3) The occurring time of ERMax ranges from 0 to 5 seconds for Shell 60, and is at about 0 for Shell 100. For the two bitumens, the occurring time of ERMax advances with an increase of temperature. Fig. 3.15 demonstrates decay curves of the two bitumens at the condition of 3% water content and 5 bar.



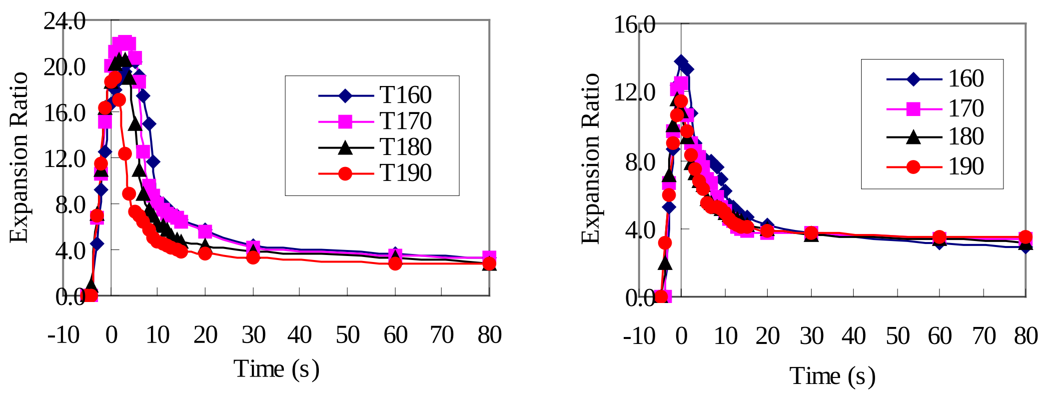
(a) Shell 60 (b) Shell 100

Fig.3. 13 Effect of Air Pressure on Decay Curve at 170^{0}C and 2% Water Content



(a) Shell 60 (b) Shell 100

Fig.3. 14 Effect of Water Content on Decay curve at 180^{0}C and 5 bar Pressure



(a) Shell 60 (b) Shell 100

Fig.3. 15 Effect of Temperature on Decay Curve at 3 % Water Content and 5 bar Pressure

(4) Decay curves show that expansion ratio of Shell 100 decreases abruptly when it reaches the peak of curve. Expansion ratio of Shell 60, however, drops slowly when it moves over the peak, and then drops sharply again after a period of time. This may be due to the viscosity difference between the two bitumens.

3.4.2 Establishment Of Decay Functions

3.4.2.1 Selection of Functions

In order to establish decay functions of the two bitumens, suitable equations must be found to fit the decay curves. Decay curves of the two bitumens under various conditions are non-linear and they have their own characteristics. Therefore two bitumens' decay curves can be characterized by two kinds of function.

From decay curves of the two bitumens, it can be found that the expansion ratio is zero when the bitumen begins to be sprayed, it increases with time and peaks the maximum at about 0-5 seconds. Expansion ratio drops when it moves over the peak. In the end, , expansion ratio keeps stable after a long period of slow dropping.

For Penetration-grade 60 bitumen, each decay curve has a maximum expansion ratio, asymptote, and an inflection point at which the expansion ratio changes its dropping from fast speed to slow speed. Thus the equation used to describe decay curves has to meet the above requirements. For Penetration-grade 100 bitumen, each decay curve has a maximum expansion ratio, an asymptote, and a decay rate. The equation used to fit decay curves has to meet these requirements.

A four-parameter non-linear function shown in Eq. 3.4 meets the fitting requirements of Shell 60 bitumen, and is therefore selected to fit the decay curves of this bitumen (Ratkowsky 1990).

$$ER(t) = (\alpha_1 + \beta_1 t)\gamma_1^{\,t} + \delta_1 \qquad (t \geq 0) \tag{3.4}$$

where *ER(t)* is the expansion ratio at the time *t*, α_1, β_1, γ_1, δ_1 are coefficients, and $-1< \gamma_1<1$, *t* is time.

For Shell 100 bitumen, a three-parameter non-linear function shown in Eq. 3.5 meets the fitting requirements of Shell 100 bitumen, and is therefore selected to fit its decay curves (Ratkowsky 1990).

$$ER(t) = ER_0 + \alpha_2 \exp(-t/\beta_2) \qquad (t \geq 0) \tag{3.5}$$

where *ER(t)* is the expansion ratio at the time *t*, ER_0 is value of the asymptote, *t* is time, α_2 and β_2 are coefficients, and $\beta_1>1$.

The explanation of parameters of above two equations are introduced in detailed in Section3.4.3.2.

3.4.2.2 Fitting Method and Initial Values of Parameters

Eq. 3.4 has four parameters and a process variable *t*, i.e. $\mathbf{ER}=\mathbf{ER_1}(t, \boldsymbol{\theta}_1)$, and $\boldsymbol{\theta}_1=(\alpha_1, \beta_1, \gamma_1, \delta_1)^T$. Eq. 3.5 has three parameters and a process variable *t*, i.e. $\mathbf{ER}=\mathbf{ER_2}(t, \boldsymbol{\theta}_2)$, and $\boldsymbol{\theta}_2=(EP_0, \alpha_2, \beta_2)^T$. $\boldsymbol{\theta}_1$ and $\boldsymbol{\theta}_2$ are vectors.

Parameters of these two equations are different at various testing conditions, hence they are functions of various testing conditions, i.e. $\boldsymbol{\theta}_1=\boldsymbol{\theta}_1$ (Temp, WC, AP), and $\boldsymbol{\theta}_2=\boldsymbol{\theta}_2$ (Temp, WC, AP). Therefore fitting of decay curves of the two bitumens can be translated into another form: two non-linear fittings of the parameters which are functions of testing conditions. For the purpose of fitting the decay curves, the following two-stage method is adopted:

Firstly, parameter $\boldsymbol{\theta}_1$ and $\boldsymbol{\theta}_2$ will be fitted. Then $\boldsymbol{\theta}_1$and $\boldsymbol{\theta}_2$ are considered as responses, and regression will be carried out to fit them with testing conditions.

In order to assure the success of non-linear fitting, the best way is to try to find suitable initial values of parameters. This can make the convergence faster (Bate and Watts 1988). Parameters' initial values can be obtained by boundary conditions of two equations.

For Shell 60 bitumen, two boundary conditions of Eq. 3.4 are as follows:
when t=0, ER=$\alpha_1+\beta_1$=0;
when $t \to \infty$, ER$\to \delta_1$.

Base on observation in the test, it is found that ER keeps stably about 1.5 after 9~10 minutes. Hence δ_1 is 1.5.

On the other hand, another two boundary conditions can be expressed using the occurring time of ERMax and inflection point. The ERMax and inflection point occur at $t_0 = -1/\log\gamma_1 - \alpha_1/\beta_1$ and $t = -2/\log\gamma_1 - \alpha_1/\beta_1$ respectively. Therefore the initial values of parameter α_1, β_1, γ_1, δ_1 of Eq. 3.4 can be obtained by solving four boundary equations. For example, initial values of parameters of Eq. 3.4 under the condition of 160 ^{0}C and 5 bar are $\boldsymbol{\theta}_1$=（5, 2.3, 0.8, 1.5）^{T}.

For Shell 100 bitumen, two boundary conditions of Eq. 3.5 are as follows:
when t=0, ER=$ER_0+\alpha_2$=0;
when $t \rightarrow \infty$, ER$\rightarrow$ ER_0.

When t=0, $\beta_2 = -\frac{\alpha_2}{ER^{(1)}(0)}$, Here $ER^{(1)}(0)$ is the first order derivative of Eq. 3.5 at t = 0. Through analysis of decay curves of Shell 100 bitumen, it can be found that the expansion ratio drops very fast after the expansion ratio moves over the peak, and the curve exhibits as a line in the small range of t=0 to t=1. Therefore $ER^{(1)}(0)$ can be approximately expressed by the decay curve in the small range from t=0 to t=1.

Using the above three boundary condition equations, the initial values of parameters of Eq. 3.5 can be solved. For example, parameters' initial values of this equation under the condition of 160 ^{0}C and 5 bar are θ_2=（1.8, 8.1, 5.5）^{T}.

3.4.2.3 Fitting Results

Non-linear fitting had been carried out for decay curves of Shell 60 and Shell 100 bitumen under all conditions using Eq. 3.4 and Eq. 3.5 respectively. Parameters of these two equations at various conditions were obtained.

Fitting result of Eq. 3.4 exhibits that average coefficient of determination of fittings is 96.74%. This result indicates that non-linear fitting of Eq. 3.4 is very good.

For Eq. 3.5, the average coefficient of determination of fittings is 96.39%. It also shows that non-linear fitting of Eq. 3.5 is very good.

3.4.2.4 Best Subset Regression Analysis of the Parameters and Residuals Analysis

Best subset analysis identifies the best fitting regression models that can be constructed with the specified predictor variables. Determining variable for regression model using this method should consider two statistic items: one is the value of Cp, another is coefficient of determination (R^2).

Typically the subset should provide the largest R^2 when comparing models of the same size. Small Cp is being searched in the best subset regression analysis. Since a small Cp value indicates that the model is relatively precise in estimating the true regression coefficients and predicting the future responses Minitab (2000).

In this analysis, α_1、β_1、γ_1、δ_1, of Eq. 3.4 are considered as responses, and Temp, WC and AP are variables. Table 3.7 shows the best subsets of parameter α_1. From this table, it can be found that among the three variables WC strongly affects the parameter α_1. Cp is 9.4 and R^2 is 77.4 when there is only WC in the subset. Cp rapidly decreases from 9.4 to 4.6 if AP is added into the subset; at the same time R^2 increases from 77.4 to 79.3. It indicates that variable AP also significantly affects the parameter α_1. When variable Temp is added into the subset, Cp decreases slightly from 4.6 by 0.6, and R^2 only increases from 79.3 by 0.7. Hence variables which significantly affect parameter α_1 are AP and WC.

Table 3. 7 Best Subset Analysis of Parameter α_1

Variables	R^2	C-p	Temp	AP	WC
1	77.4	9.4			X
1	1.8	289.2		X	
1	0.8	292.7	X		
2	79.3	4.6		**X**	**X**
2	78.2	8.6	X		X
2	2.5	288.4	X	X	
3	80.0	4.0	X	X	X

Note: X denotes variable included in the analysis.

The best fitting model of parameter α_1 with variable AP and WC can be established using regression method. The result is shown as Eq. 3.6. The estimated standard deviation (S) of this regression is 2.531.

$$\alpha_1 = 0.391AP + 3.52WC \tag{3.6}$$

(S=2.531)

Using the same method, best subsets of β_1, γ_1, δ_1 have been analyzed. The results are listed in Table 3.8. It is found that significant variable of the response β_1 is WC. Temp and WC strongly affect variable γ_1 and δ_1. Regression equations of β_1, γ_1, δ_1 are shown as Eq. 3.7 to Eq. 3.9, estimated standard deviations of these fittings are 2.819, 0.064 and 0.617 respectively.

Table 3. 8 Summary of Best Subset Analysis

Response	Temp	AP	WC
α_1		√	√
β_1			√
γ_1	√		√
δ_1	√		√
ER_0	√		√
α_2	√	√	√
β_2	√		√

Note：√ denotes factor significantly affect the response.

$$\beta_1 = 2.71WC \tag{3.7}$$

(S=2.819)

$$\gamma_1 = 1.57 - 0.00413Temp - 0.0384WC \qquad (3.8)$$

(S=0.064, R^2=56.4%)

$$\delta_1 = 0.0115Temp + 0.528WC \qquad (3.9)$$

(S=0.617)

For Eq. 3.5, best subsets of ER_0, α_2, β_2 has been analyzed too. Their results are also listed in Table 3.8. It is found that significant variables of the response α_2 are Temp, AP and WC. Temp and WC strongly affect variable ER_0 and β_2. Regression equations of ER_0, α_2, β_2 are shown as Eq. 3.10 to Eq. 3.12. Estimated standard deviation of ER_0, α_2, β_2 are 0.402, 1.531and 1.790 respectively.

$$ER_0 = 0.0171Temp + 0.130WC \qquad (3.10)$$

(S=0.402)

$$\alpha_2 = 0.00648Temp + 0.238AP + 1.97WC \qquad (3.11)$$

(S=1.531)

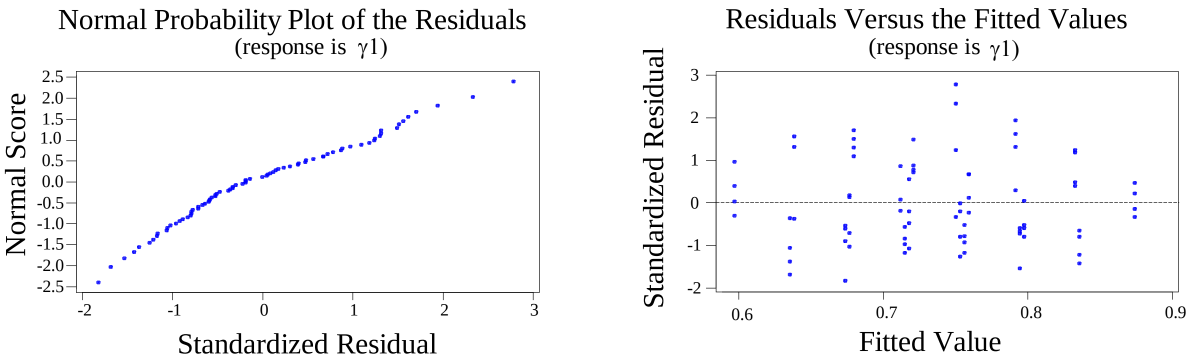
$$\beta_2 = 0.0201Temp - 0.820WC \qquad (3.12)$$

(S=1.790)

A good regression fitting has to meet two requirements. Firstly, residuals of the predicted values versus test values are randomly distributed. Secondly, probability distribution of the residuals is normal (Bate and Watts 1988).

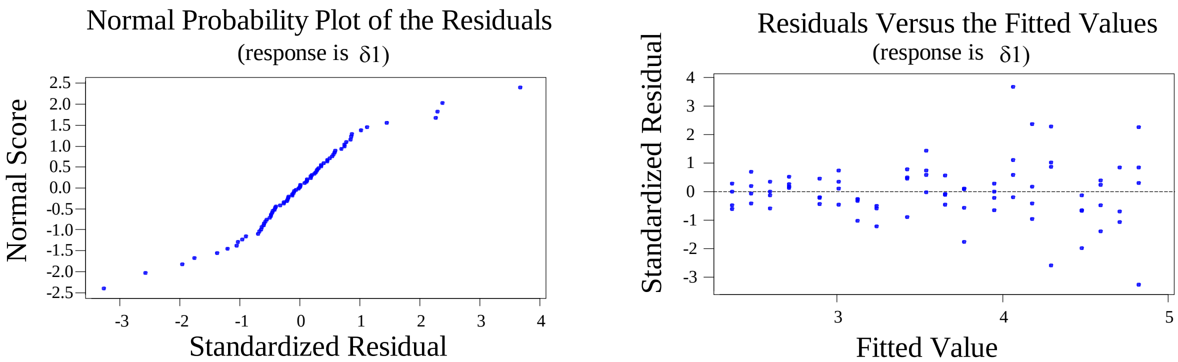
For Eq. 3.4, residual analysis had been carried out to check the good fitness of parameter α_1, β_1, γ_1, δ_1. It is found that residual distributions of parameter α_1 and γ_1 are normal, while those of parameter β_1 and δ_1 are normal in most instances (in the range of standardized residual from -2 to 2).

The residuals versus fitted value of parameter α_1 and γ_1 are randomly distributed. Random distributions of parameter β_1 and δ_1 are not very good for very small or very large fitted values, but their distributions are good for other fitted values. Fig. 3.16 and Fig. 3.17 illustrate the results of residual analysis of parameter γ_1 and δ_1. Residual analysis plots of α_1 and β_1, which are better than those of γ_1 and δ_1, are omitted here. The above residual analysis indicates that the fittings of parameters of Eq. 3.4 are good on the whole.



(a) (b)

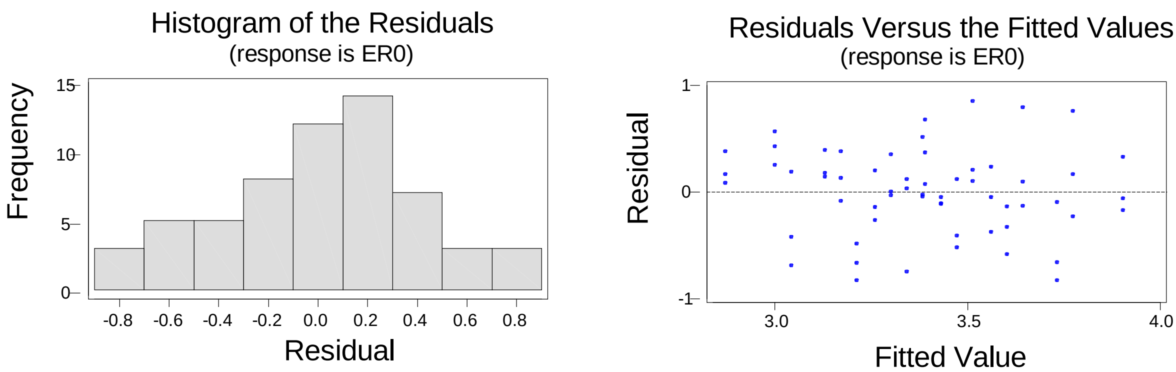
Fig.3. 16 Residual analysis of parameter γ_1



(a) (b)

Fig.3. 17 Residual analysis of parameter δ_1

For Eq. 3.15, residual analysis for parameter ER_0, α_2, β_2 have been also performed. It is found that residual distributions of these three parameters are normal, and the residuals versus fitted value of them are randomly distributed. Residual analysis indicates that the fittings of parameters of Eq. 3.5 are good. Fig. 3.18 illustrates the results of residual analysis of parameter ER_0.



(a) (b)

Fig.3. 18 Residual analysis of parameter ER_0

3.4.3 Application And Significance Of The Decay Functions

3.4.3.1 Application

Decay functions links the maximum expansion ratios and half-lives of the two bitumens well. Moreover, decay functions can also be used to evaluate the foamability differences of Shell 60 and Shell 100 bitumen under various conditions instead of onerous foaming test of the two bitumens, because parameters of decay functions embody effects of foaming conditions. For other bitumens, evaluation work can be done only based on their decay functions established.

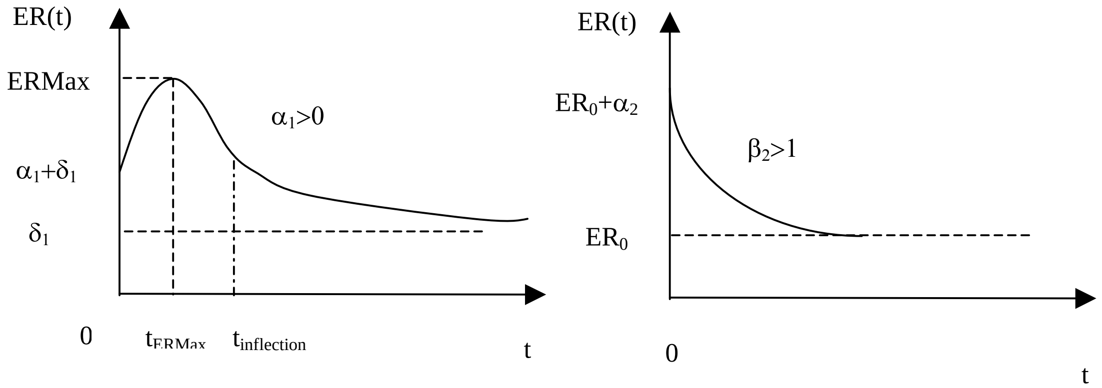
The ERMax and half-life of Shell 60 bitumen under various conditions can be calculated by Eq. 3.4, Eq.3.6, Eq. 3.7, Eq.3.8 and Eq. 3.9. Therefore foamability characteristics of this bitumen can be evaluated.

At first, the expansion ratio peaks its maximum at $t_0 = -1/\log\gamma_1 - \alpha_1/\beta_1$, then the time $t_{1/2}$ when the expansion ratio decreases from the ERMax by 50% is obtained using Eq. 3.4. The difference between t_0 and $t_{1/2}$ is the half-life.

The ERMax and the half-life of Shell 100 bitumen under various conditions can be calculated by Eq. 3.5, Eq. 3.10, Eq.3.11 and Eq.3.12. Firstly, the expansion ratio peaks its maximum at t=0. Secondly, the time $t_{1/2}$ when the expansion ratio decreases from the ERMax by 50% is obtained using Eq. 3.5. Therefore the half-life is $t_{1/2}$.

3.4.3.2 Meanings of parameters

Parameters in Eq. 3.4 and Eq. 3.5 have their specific implications. For Eq. 3.4 (Fig. 3.19(a)), meanings of parameters are as follows:



(a) Eq. 3.4 (b) Eq. 3.5

Fig.3. 19 Parameters' meanings of Eq. 3.4 and Eq. 3.5

(1) δ_1 controls the position of the asymptote: the asymptote moves up with an increase of δ_1, and moves down with a decrease of δ_1. δ_1 is the asymptote parameter.

(2) γ_1 controls the position of the inflection point: the inflection point moves right when γ_1 increases, and moves left when c decreases. γ_1 is inflection-point parameter.

(3) α_1 and β_1 control ERMax and its occurring time. ERMax increases with an increase of α_1 and β_1, and decreases when α_1 and β_1 decreases. If α_1 increases and β_1 decreases, ERMax point moves left; and if α_1 decreases and β_1 increases, ERMax moves right. Hence α_1 and β_1 are parameters which reflect the ERMax and its occurring time.

For Eq. 3.5, ER_0 denotes value of the asymptote. α_2 is the amplitude of expansion ratio, which means the variation range of expansion ratio when $t>0$. β_2 is a decay constant, which reflects the decay rate when expansion ratio drops from the maximum value ($\alpha_2 + ER_0$) to the final value. The large β_2 is, the fast expansion ratio decays. Fig. 3.19(b) illustrates the meanings of these three parameters.

3.4.3.3 Significance of decay function

Eq. 3.6 and Eq. 3.7 reflect that parameters α_1 and β_1 of the decay function of Shell 60 bitumen increase with increase of added water content; It is found that ERMax increases with an increase of α_1 and β_1. Hence, ERMax also increases with increase of the added water content. This theoretical analysis reveals the reason why ERMax increases with an increase of the water content shown in Fig. 3.14.

Many references only described the phenomenon of Fig 3.14 (Wirtgen GmbH 2001), however, they did not explain it. Decay function of foamed bitumen gives the answer.

Because in the ERMax stage, the number of bubble which have the maximum volume but not collapse is the maximum, the low-viscosity bitumen bubble can easy to coat the mix, once the bubble collapse during the mixing, it change into little bubble or “black dot”, and easily disperse among the mix. Therefore more large bitumen bubbles can mix with aggregates in the ERMax stage. Increasing ERMax will provide better mixing of aggregate and foamed bitumen, and improve the performance of the foamed asphalt mix.

It can be found from Eq. 3.9 that parameter δ_1 increases with increases of temperature, and/or water content. This implies that asymptote line rises in this situation, and it will take a longer time for all bitumen foams to completely collapse. When δ_1 increases, the ERMax also increases. This result will also help aggregate and foamed bitumen mix better.

If air pressure and water content don't change, parameters α_1 and β_1 will not change. Parameter γ_1 will decrease in the condition of a increasing temperature (see Eq. 3.6 to Eq. 3.8). This will cause increasing of ERMax (i.e. $t_0 = -1/\log\gamma_1 - \alpha_1/\beta_1$). Hence it confirms that ERMax improves with higher temperatures. Decay function can also be used to analyze the effect of temperature on the foaming properties.

For Shell 100 bitumen, it is clear from Eq. 3.10 that ERMax, i.e. ER_0, will improve with increase of temperature or water content.

Eq. 3.11 indicates that temperature, air pressure and water content positively affect the parameter α_2. That means that increase of any of these two conditions will raise the asymptote.

Eq. 3.12 reflects that decay rate increases when temperature increases. This may be due to lower viscosity of the bitumen at higher temperatures. Lower viscosity will cause faster collapse-speed of the bitumen foam.

Based on the above analysis for parameters of the two decay functions, it is clear that temperature (in the range of 160 to 200 ^{0}C) and water content have direct influence on the foaming properties of foamed bitumen. They positively enhance the ERMax, which may help improve the performance of the foamed asphalt mix. Hence study of decay properties of foamed bitumen provides a guide for foaming production.

However, significant difference exists among bitumens from different suppliers. The foamability study in this research is only for Shell bitumen. Conclusion drawn from above analysis may not be applicable to other bitumens.

If the water content exceeds an appropriate range, on the other hand, it maybe a disadvantageous to the half-life. This can be confirmed from analysis of the foaming properties in section 3.3. Generally, the way to determine the water content is to use the balance-point method.

3.5 CONCLUSION

In this chapter, foaming tests under various conditions were carried out to investigate the foamability of two bitumens: Shell 60 and Shell 100.

Testing data on ERMaxs and the half-lives of the two bitumens under each condition were obtained. MANOVA was conducted to examine the effects of factors on ERMax and half-life. The optimum foaming conditions of the two bitumens were determined based on effect analysis of ERMax and half-life. From the view of viscosity, mechanism analysis of foamability difference between two bitumens was performed, and effect of water content on this difference was discussed.

Decay curves of the two bitumens under each condition were obtained and analyzed. A four-parameter power function and a three-parameter exponential function were established to fit decay curves of Shell 60 and 100 bitumens respectively based on non-linear fitting method. Decay functions' parameters were obtained by the best subset regression method. Meanings of these parameters and application of the decay functions were discussed.

Conclusion can be drawn as follows:

(1) Factors

- MANOVA indicates that there is no significant evidence for temperature effect on half-life of Shell 60, for air pressure effect on half-life of Shell 100. The interaction of temperature and air pressure has no significant effect on half-life of the two bitumens.

- Air pressure has little effect on the decay curve, whilst water content significantly affects the latter. ERMax of the two bitumens increases with an increase of water content.

(2) Viscosity of bitumen

- Viscosity makes an important impact on foaming properties of the two bitumens. Almost all testing results of ERMaxs of Shell 100 with lower viscosity are smaller than those of Shell 60 with higher viscosity. On the other hand, most testing results of half-lives of the Shell 100 are larger than those of the Shell 60.

- Analysis shows that bubble of Shell 60 is most likely to collapse in the first mode, whilst bubble of Shell 100 is in the second mode. Test results confirm that mechanism analysis is rational.

- Due to viscosity difference of the two bitumens, Shell 60 achieves the maximum expansion ratio at about 0 to 5 seconds, and Shell 100 does that at about 0 second.

• Decay curves exhibit that expansion ratio of Shell 100 decreases abruptly when it climbs to the peak whilst expansion ratio of Shell 60 drops at relatively slower speed when it moves over the peak, and then drops sharply again after a period of time.

(3) ERMax vs. half-life

• Shell 60 exhibits a clear inverse relationship between ERMax and half-life. However Shell 100 do not show this clear relationship. There is a significant difference in foamability between the two bitumens.

(4) Decay functions

• It is found that non-linear fittings of decay curves of two bitumens are good. Residual analysis shows that the regressions of parameters of two decay functions are good. Decay functions' parameters have their own specific implications.

• Two decay functions can be used to calculate the maximum expansion ratios and half-lives of Shell 60 and 100 bitumens under various conditions. This method can also be used to evaluate the foamability difference of two bitumens instead of onerous foaming tests.

• Decay property study indicates that temperature (in the range of 160 to 200 ^{0}C) and water content have direct influence on the foaming properties of the foamed bitumen. Results of decay property study also provide a guideline for production of foamed bitumen.

(5) Optimum foaming condition

• Test result indicates that water content with more than 2% adversely affects half-life. Majority of half-life at this condition cannot exceed 5 seconds. The condition of 5 bar pressure, 170 ^{0}C and 1.7% water content is selected as the optimum foaming condition for Shell 60 bitumen. The optimum foaming condition for Shell 100 is the combination of 3% water content, 5 bar pressure and 160 ^{0}C

CHAPTER FOUR MIX DESIGN

4.1 CONSIDERATION AND METHOD

Several parameters need to be considered in mix design to ensure that the final mix has optimized properties and that rate of cure and mechanical properties are suitable for the intended application.

In this chapter, the key design parameters that need to be considered in a mix design for foamed asphalt materials are discussed. The objective of a foamed asphalt mix design is to select the mix properties, one of which is bitumen content, in order to achieve:

- optimum values for laboratory-measured properties;
- structural and functional requirements of the in-service mix, and,
- retentions of the relevant engineering properties at in-service conditions of temperature, moisture and loading conditions.

As introduced in Chapter 1, property tests of the foamed asphalt mix are limited to in the laboratory. The engineering properties at in-service conditions are not within this research scope. Hence the design bitumen content will only be determined by laboratory test. However, engineering properties will also be studied in laboratory by simulating the field conditions. These studies will be introduced in the Chapter 5, 6 and 7.

Bitumen content, and other aspects, such as the moisture content, gradation of the asphalt mix will be discussed in this chapter.

4.1.1 Consideration

1. Bitumen characterization and preparation

The foaming characteristics of a particular bitumen type need to be optimized for producing foamed asphalt mixes. This is achieved by measuring the half-life and the maximum expansion ratio of foamed bitumen produced under various conditions. Usually five tests are conducted with the foaming water content varying from 1 percent to 5 percent at 1 percent increments. Temperature of the bitumen before foamed should be in the range 160° C - 200° C. Additives may be used to catalyze the foaming. However, conducting full test with these parameters has a significant cost implication.

The optimum foaming condition has been determined in Chapter 3. The foamed bitumen for mix preparation will be prepared under the determined OFC.

2. Aggregate characterization and preparation

The grading and the PI of the aggregates should be determined for the mix design. The grading is adjusted, if required, by adding fines or coarse material so that the final grading conforms to the grading envelope. Materials which have a PI greater than 10 and lower than 16 should be treated with 1 percent lime to reduce the PI. If PI is greater than 16, 2% lime should be added into the aggregate. 1% cement can be used into the aggregate when PI is lower than 10 (Wirgten GmbH 2001). In addition, it is a common practice to add 1 - 2 percent cement to the mix to aid in bitumen adhesion (Lewis, 1998).

The aggregates should be oven dried to a constant mass. The dried aggregates are riffled into 5 batches of 10 kg each. These will be used to produce 5 batches of foamed asphalt samples at various bitumen contents. For each batch, 5~8 specimens are made depending on the requirement of testing. When determining the OMC by the soaked ITST, 3 soaked specimens were required. However, 8 specimens were required in this research in order to investigate the moisture susceptibility of FA mix to ITS and volumetric property of FA mix, 3 for soaked ITS, 3 for dry ITS, and 2 for volumetric property.

3. Bitumen content (BC) for trial mixes

An appropriate range of foamed bitumen contents (see Tables 2.6 and 2.7) is selected for the trial mixes. Five batches of trial mixes are normally prepared at bitumen contents differing by one percent.

4. Moisture content

The moisture content for mixing and compaction is a crucial mix design parameter, which has generally been selected as the ‘fluff’ point of the aggregates (70 - 80 percent modified AASHTO OMC). However, it is recommended by Muthen that the moisture content for mixing and compaction be selected at OMC minus the BC for the Marshall method (Muthen 1998).

In this research, there was no soil in the mix. Moisture added is only to help mixing and compaction, absorption is unnecessary to be considered.

5. Mixing and compaction

Each 10 kg sample of aggregate and the required mass of foamed bitumen (and lime or cement if required) are mixed in a mechanical mixer at the moisture content described as above. The foamed asphalt mixes are stored in sealed containers to prevent moisture loss, until all five batches have been mixed. Duplicate samples are removed from each foamed asphalt mix batch for the determination of moisture content and bitumen content. The remaining foamed asphalt will be used to prepare compacted foamed asphalt specimens for further testing. Eight samples were prepared from each batch in this research. Six samples for indirect tensile tests and two samples for volumetric evaluation. The two samples for volumetric evaluation were not needed to compact according the requirement of the maximum theoretical specific gravity. The content of each batch were determined by the 3 dry ITS specimens.

Specimens are compacted, using the Marshall hammer, to a 100 mm nominal diameter and 65 mm nominal height. Specimens of this size normally require about 1,15 kg of material. A compactive effort of 75 blows on each face is recommended.

6. Curing, testing and design bitumen content determination

The foamed asphalt specimens should be subjected to an accelerated curing procedure before undergoing any tests, for example 3 days' oven curing at 60° C. In this study, this method was used to cure specimens. Indirect tensile strength testing is conducted to determine the ultimate strengths of both dry and soaked samples. The dry and soaked indirect tensile strength should meet the requirement respectively recommended by Maccarone (1995).

The **design bitumen content** should be selected as the bitumen content at which the soaked indirect tensile strength is at a maximum. Resilient modulus testing at the design bitumen content is recommended for structural design by some project. Dynamic Creep testing at the design bitumen content is suggested in order to evaluate the permanent deformation characteristics of the foamed asphalt mixes.

4.1.2 Design Methods

The most commonly used mix design method for foamed bitumen has been that based on Marshall stabilities and densities.(Acott and Myburgh 1983, Tia and Wood 1983, Sakr and Manke 1984, Bissada 1987, Muthen 1998, Wirgten GmbH 2001) Generally, it has been observed that the Marshall stabilities of foamed asphalt mixes tended to increase to a maximum as the bitumen contents increased.

The Marshall design criterion used to determine the design bitumen content is when the ratio between the wet and dry stabilities is at a maximum, i.e. the bitumen content at which the mix retains most of its strength when soaked. However, in recent years asphalt design methods, including foamed bitumen mix design, have shown a shift away from the Marshall methods, the emphasis now being placed on dynamic test procedures such as the dynamic creep test and the indirect tensile test.

Based on experience in Australia, Lancaster et al (1994) recommended that the bitumen content selected for foamed asphalt mixes be based on the highest resilient modulus value. However, following recommendations by Lewis (1998), it is proposed that the indirect tensile strength test be used to select a design bitumen content.

As accepted by many highway agencies, the performance-related mix design method of Superpave was also applied in mix design of the cold-recycled mixtures for some trial roads (Lee et al 2003, Mallick et al 2001, Thomas and Kadrmas 2003). However, application of Superpave technology in cold-recycled mixture design is limited in laboratory and in trial. In order to solve some problems with performance reliability, specifically the lack of a uniform, defined design procedure, and problems with raveling, thermal cracking, compaction problems, low early strength and extended curing time, performance-related tests and specification for cold in-place recycling is worthy to be further studied.

Based on the review of previous published investigations (Lewis 1998, Muthen 1998 and Wirtgen GmbH 2001), the procedure of Marshall mix design for the foamed asphalt mix is proposed. Procedures recommended in this mix design still need to be validated by laboratory investigation and are subject to change; this work is not in this research scope. The detailed laboratory procedures are listed in the Appendix of this chapter.

4.2 GRADATION AND MATERIALS

4.2.1 Gradation

Table 2.7 shows that a wide range of aggregates may be used with foamed-bitumen, ranging from crushed stone to silty sands. In order to assure satisfactory performance of foamed asphalt mix, the aggregate grading should meet some requirements. The grading and its requirement of the foamed asphalt mix have been reported extensively in the literature references. RAP materials may be used for grading aggregates. In most cases, grading adjustment should be conducted.

Fig.2.6 illustrates the foam stabilizing grading chart recommended by the Mobil (Akeroyd and Hicks 1988). There are three zones suitable for different trafficked roads in this chart. Vorobieff and Preston (2004) explained the preferred gradation for foamed asphalt mixes (see Fig. 2.7). The design guide of UK for cold in-place recycling also gives the requirement of the particle size distribution (see Table 2.3) (Milton and Earland 1999).

A basic principle of the foamed asphalt mix is that the ratio of fines (minus 0.075 mm particles) content to bitumen content is critical because excess bitumen in the mortar will result in a loss of strength and stability. Generally, the fine content of the aggregate should preferably be above 5 percent and not more than 20 percent.

Based on the literature review, the grading specifications of the foamed asphalt mixes recommended by Mobil Australia, UK and SABITA Ltd & CSIR Transportek in South Africa are summarized in Table 4.1 and illustrated in Fig. 4.1.

Table 4. 1 Gradings of the foamed asphalt mixes recommended by CSIR, Mobil and UK

Nominal max size (mm)	Cumulative % Passing							
	CSIR		Mobil		UK (ideal grading)		UK (acceptable grading)	
	Min	Max	Min	Max	Min	Max	Min	Max
50	—	—	100	100	100	100	—	—
37.5	—	—	89	100	94	100	—	—
28(26.5)	73	100	78	100	—	—	—	—
20(19)	63	100	66	100	66	100	100.0	100.0
14(13.2)	52	88.5	58	87	—	—	—	—
10(9.5)	44	75	48.5	74	48	75	75.0	100.0
(6.7)	36	65	—	—	—	—	—	—
5(4.75)	29	55	35	57	35	57	57.0	95.0
2.36	23	43.5	25	42	25	42	42.0	77.0
1.18	18	38	18	33	—	—	—	—
0.6	14	31	14	28	13	28	28.0	52.0
0.3	10	27	10	23	10	24	24.0	45.0
0.15	7	22	7	22	—	—	—	—
0.075	5	20	5	20	5	20	20.0	35.0

Note: Numbers of sieve size in bracket are used by CSIR.

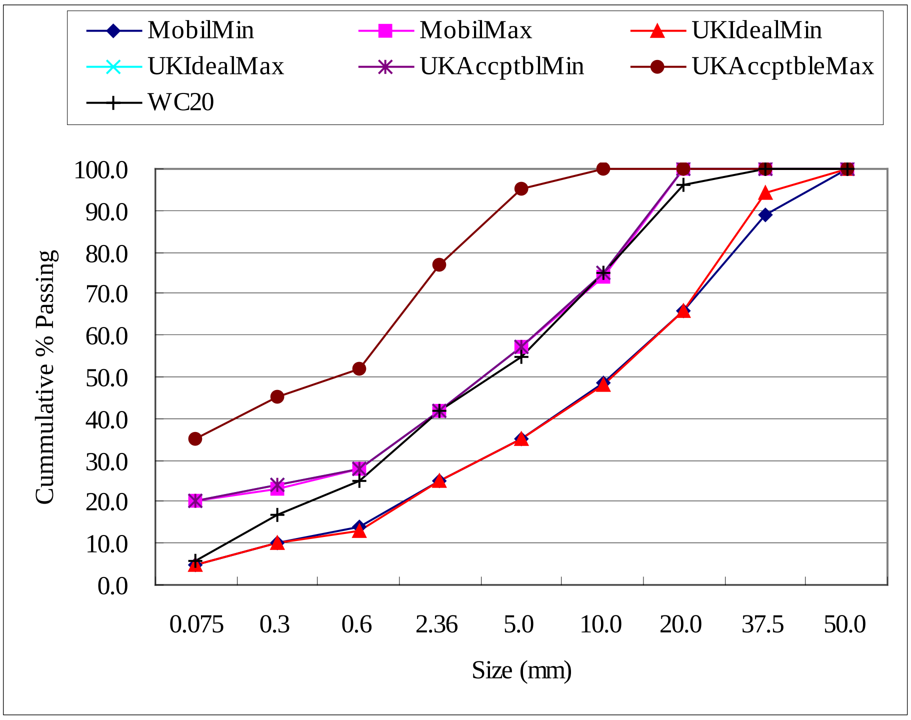


Fig.4. 1 Foamed asphalt mix gradings of CSIR, Mobil, UK and WC20

Note: WC20 in introduced below in detailed.

The bituminous road in Hong Kong is typically composed of full-depth asphalt pavement, including three layers, i.e. wearing course, base course and road base. Fig. 4.2 illustrates this pavement structure. In freeway and trunk road, a friction course, namely the porous asphalt layer, is generally paved on the top to improve traffic safety and lower the traffic noise. Table 4.2 lists gradings and corresponding bitumen contents recommended (HKSAR 2002).

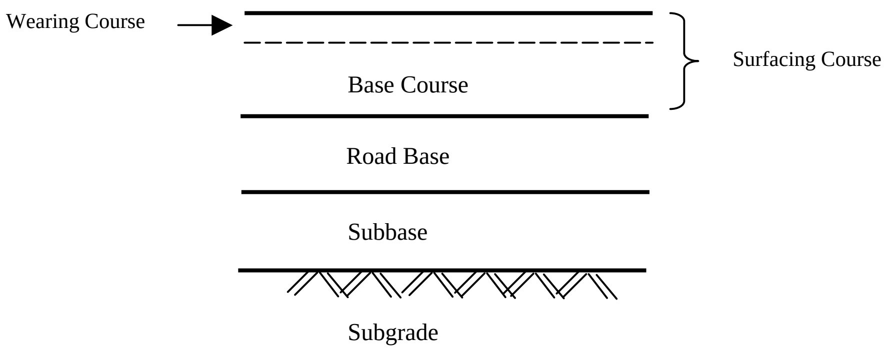


Fig.4. 2 Hong Kong's typical flexible pavement

Table 4. 2 FA Design grading and bituminous asphalt mix grading specifications of Hong Kong

Nominal max size (mm)	Cumulative % Passing					
	Grading specification of Hong Kong					FA mix grading
	Wearing Course		Base Course		Roadbase (Recipe mix)	WC 20
	10	20	28	37.5	37.5	20
50	-	-	-	100	100	**-**
37.5	-	-	100	91-100	90-100	**-**
28	-	100	91-100	70-94	70-94	**100**
20	-	91-100	85-95	62-84	62-84	**96**
14	100	78-90	72-87	55-76	-	**88**
10	87-100	68-84	55-75	49-67	49-67	**75**
5	62-80	54-72	35-53	37-55	37-55	**55**
2.36	42-58	42-58	25-40	27-43	27-43	**42**
1.18	34-48	34-48	15-30	20-35	-	**35**
0.6	24-38	24-38	12-24	13-28	13-28	**25**
0.3	16-28	16-28	8-18	7-21	7-21	**17**
0.15	8-18	8-18	5-12	4-14	-	**11**
0.075	4-8	4-8	3-6	2-8	2-8	**6**
BC(%)	6.0-7.0	5.0-5.5	4.5-5.0	4.0-4.5	3.0-4.0	**-**

In order to improve the mix design, Monismith et al (1985) provided the recommendation as following to select mix grading for three kinds of pavement:

- Dense-grade mix composed of aggregates with rich surface texture is proposed to improve pavement's rutting-resistance. This mix is easy to be compacted. If the air void is less than 3%, the susceptibility to rutting will decrease quickly.

- For the purpose of keeping mix's durability, it should be noted that air void should be as low as possible in the mix design so as to reduce the possibility of bitumen ageing, freezing and thawing and ingress of moisture.
- It is preferable to select the dense-grade mix to avoid moisture-induced damage.

Hong Kong's climate is sub-tropical. May to August months are hot and humid with frequent showers and occasional thunderstorms. In this period, temperatures often exceed 31 C in the afternoons whereas at night, generally remain around 26 C with high humidity.

Hence asphalt pavement in Hong Kong ordinarily endures the environmental effect of high-temperature and rich moisture. Permanent deformation and moisture-induced damage are typical failures in this region. Hong Kong's weather condition and their effects on mixture performance should be considered in the mix design.

Foamed asphalt mix is generally paved as road base or base course covered a hot-mixed asphalt layer due to its poor abrasion and large air void. With respect to weather condition in Hong Kong, the dense-grade type is an appropriate selection for FA mixes, when they are paved as road base, on the base of the following consideration:

- In order to improve permanent deformation of mix, the appropriate content of the course aggregate should be retained;
- In order to mitigate the moist-induced damage and improve the moisture susceptibility of mix, it should be noted that air void in the mix should be lowered. The content of fine aggregate should be appropriate, low content of fine aggregate will cause larger air void.
- The dense-grade mix is adopted as much as possible in order to avoid mix segregation and poor workability in the construction process. This is the reason why the continuous dense-grade mixture is recommended in the mix design in many countries (Sha 1998).
- As a road base, the FA mix should have good fatigue-resistance. The fatigue-resistance of the dense-grade mix is better than that of open-grade mix.
- The content of filler is in the range of 5% to 20% so as to assure good properties of the foamed bitumen mixes.

The design FA grading was determined on three considerations: the first is the maximum size of the RAP, the second is to meet the requirement of preferred grading recommended by Mobil or CSIR, the third is that the grading can be compatible with the grading of the existing base course used in Hong Kong.

Generally, RAP material comes from the wearing course and friction course in Hong Kong. Size of the milled material is less than 20 mm. Due to the size of RAP material and consideration of construction workability, the wearing course with 20mm of the nominal max size used in Hong Kong is preliminarily selected as the grading of FA mix. In comparison of this grading with those recommended by CSIR, UK and Mobil, it can be found that the preliminary grading roughly falls in the envelope of the preferable grading. In fact this grading is basically compatible with the base course.

Finally, the design grading of FA mix is determined based on adjustment of the Hong Kong's wearing course with 20mm nominal max size. The name of this grading is determined as WC 20. Table 4.2 list the detailed grading, and its maximum size is less than 28 mm. Fig. 4.1 illustrates this WC 20 too.

To investigate effect of RAP content on performance of the FA mix, four contents of each RAP material, i.e. 0%, 20%, 40% and 60% will be added into the FA mix; the whole percentage of the mix is 100%. 0% and 20% represent low RAP contents, 40% and 60% represent high RAP contents. 0% of RAP is the foamed-treated mix without RAP material.

Two RAP materials with different ageing and two types of bitumen are considered so as to investigate the effects of RAP materials and the bitumen grade on performance of the foamed asphalt mixes in this research.

In this research, the combined mix had no soil; mix's PI didn't exceed 10. Hence, only 1% cement was used in the mix. The aggregates were determined in the mix design with reference to ASTM D 1557.

4.2.2 Properties Of The Materials

Materials include two parts: i.e. RAP materials and virgin materials. There are two types of RAP materials: RAP#1 and RAP#2. They were provided by local contractor and milled from two freeway sections in Hong Kong. Before being milled, RAP#1 has been used for 4 years and RAP#2 has been used for 6 years. Virgin aggregate is a kind of granite rock typically used in road paving industry in Hong Kong, also provided by local contractor.

As introduced in Chapter 2, two types of Shell bitumen, i.e. Shell penetration-grade 60 and Shell penetration-grade 100, are selected as bitumen. In addition, cement is also added into the FA mix in order to improve strength of the mixture. Tests of all materials and their basic properties are introduced as follows.

1. Virgin materials

(1) Virgin aggregates

Virgin aggregates were sieved into various groups with different sizes for mix design. Their apparent specific gravities (ASG) were tested in accordance with ASTM C127 (2001) and ASTM C128 (2001) and results are list in Table 4.3. Virgin aggregates are different form of soil; their PIs are very low.

Hence it is unnecessary to test them.

Table 4. 3 Apparent specific gravities of virgin aggregates and RAP materials

Size (mm)	Virgin Aggregate	RAP#1	RAP#2
28	2.581	-	2.547
20	2.583	2.472	2.506
14	2.585	2.453	2.421
10	2.583	2.429	2.341
5	2.591	2.369	2.342
2.36	2.596	2.394	2.333
1.18	2.612	2.264	2.279
0.6	2.611	2.242	2.285
0.3	2.613	2.302	2.317
0.15	2.653	2.310	2.401
0.075	2.622	-	2.276
<0.075	2.626	-	2.177

(2) Bitumen

Basic properties of Shell 60 and Shell 100, including penetration, softening point and high temperature viscosity, were tested in accordance with ASTM D5 (1997), ASTM D36 (1995) and ASTM D4402 (2002). In addition, two SHRP performance-grade tests, high-temperature and intermediate-temperature dynamic shear rheology (DSR) tests were carried out for virgin bitumens and their critical temperatures were obtained. Critical temperature is defined as the temperature at which the bitumen's G*/sin δ reaches the critical criteria (critical criteria can be referred SHTP-A-379 1994). Results of all these testes are listed in Table 4.4 and 4.5.

Table 4. 4 Basic properties of virgin and recovered bitumens

Properties	Virgin Bitumen		Recovered RAP Bitumen	
	Shell 60	Shell 100	RAP#1	RAP#2
Penetration (25 ^{0}C, 100g, 5s) (0.01mm)	66.6	91.2	17.8	7.0
Softening Point (Ring and Ball) (^{0}C)	49.5	46.3	71.7	113.5
Viscosity at 135 ^{0}C (mPa.s)	468.0	370.2	1449	572000

Table 4. 5 Critical Temperatures of virgin and recovered RAP bitumens

Ageing	Property	Virgin Bitumen		Recovered RAP Bitumen (aged)	
		Shell 60	Shell 100	RAP1	RAP2
Original	DSR G*/sin δ	76.9	69.4	102.3	129.2
RTFO*	DSR G*/sin δ	70.7	70.4	108.2	156.4
PAV*	DSR G*sin δ	22.9	25.7	32.8	46
	PG	PG 70-*	PG 70-*	PG 88-*	PG 88-*

Note: Due to the low-temperature, DSR test is not conducted, the low-temperature performance grades of bitumens cannot be determined.

The purpose of DSR test is to investigate the rutting resistance, fatigue property and critical temperature of the bitumens, and to compare the properties of the two bitumens. It is found that Shell 60 has better fatigue property than Shell 100 due to lower PAV critical temperature (22.9 ^{0}C) of the former than that (25.7 ^{0}C) of the latter. For the two bitumen, critical temperatures of RTFO samples reflect that these two virgin bitumens have same rutting-resistance.

(3) Cement

Due to PI of RAP and virgin aggregates being less than 10, 1 percent of cement will be added into the foamed asphalt mix to enhance their adhesion. The cement is made in Guangzhou, China. Its ASG is 3.0 g/cm^3.

2. RAP materials

Before the mix design, grading of RAP materials can be obtained by sieve test. The extraction and recovery test of RAP materials must also be performed to investigate the properties of these aged bitumens.

After RAP materials were sieved, gradings of the two kinds of RAP materials were obtained and are listed in Table 4.6. It can be found from this table that the content of fine RAP aggregates is very low. Portions passing 0.3 mm of the two RAP materials are 0.9% and 0.6% respectively. This result reflects that RAP#1 is smaller than RAP#2 overall.

Table 4. 6 Sieve analysis of RAP materials

Sieve Size (mm)	Cumulative Passing (%)			
	Raw		Recovered	
	RAP#1	RAP#2	RAP#1	RAP#2
28	100.0	97.3	100.0	100.0
20	93.7	85.2	99.2	94.6
14	76.8	56.7	94.8	87.8
10	58.3	39.7	88.5	78.9
5	29.3	18.7	70.6	64.0
2.36	13.4	8.0	52.2	49.3
1.18	5.7	3.2	38.5	37.3
0.6	2.3	1.3	28.9	28.0
0.3	0.9	0.6	20.5	19.5
0.15	0.3	0.3	13.4	12.1
0.075	0.1	0.1	7.2	6.7
<0.075	0.0	0.0	0.0	0.0
Bitumen content (%)	4.6	4.4	-	-

The ASGs of each size of RAP aggregates were tested and results are listed in Table 4.3.

There are several methods to extract bitumen from the recovered asphalt. ASTM D2172 introduces 5 methods, including Centrifuge method, Reflux method and etc. Typical ways to recover the bitumen include Abson method and Rotavapor method. Abson method has been the standard recovery method used for many years. The Rotavapor method has recently gained in popularity. These two recovery methods with three solvents, i.e. trichloroethylene, toluene and N-Propyl bromid (NPB) in the NCHRP 9-12 research project were evaluated and it was found that the Rotavapor recovery treatments indicated similar precision with coefficients of variation much lower than that of the Abson recovery (5-20%

compared to 38%-69%) (McDaniel et al 2000).

In Hong Kong, trichloroethylene has been prohibited due to its damage to atmosphere. Hence dichloromethane, which is recommended by Hong Kong road paving industry, replaced trichloroethylene as the solvent in the extraction asphalt mixture in this research.

RAP materials were extracted by centrifuge method using dichloromethane. RAP bitumens were recovered by Rotavapor method. The extraction and recovery were all performed in the fume hood.

After extraction and recovery, the recovered bitumens and aggregates were obtained. Sieve test was carried out for recovered aggregates and results are summarized in Table 4.6. It is clear that there is insignificant difference between two extracted aggregates. Compared with result of two raw RAP materials, the different between them may be due to the fact that RAP#2 was aged more than RAP#1. The contents of 20mm and 14 mm aggregates of the two RAP materials are not in the range of the design grading.

The RAP bitumen contents of the two RAP materials were tested by ashing method (ASTM D 2172). Bitumen contents are 4.6% and 4.4% for RAP#1 and RAP#2 respectively. They are close to each other, but lower than the design bitumen content (5~5.5%).

The penetration, softening point and viscosity of the recovered RAP bitumen were tested by the same methods for virgin bitumens, shown in Table 4.4. It can be concluded that the recovered bitumen of RAP#2 is more aged than that of RAP#1, softening point and viscosity at 135 ^{0}C of the former are 1.58 times and 394.8 times those of the latter. DSR test was further performed for the two recovered RAP bitumens. Critical temperature under original, RTFO and PAV are also listed in Table 4.5.

It can be found from Table 4.5 that recovered bitumen of RAP#2 is more aged than that of RAP#1, because RAP#2 bitumen has higher DSR results in three ageing conditions (original, RTFO and PAV) than RAP#1. This result implies that the former has better rutting-resistance but poorer fatigue property than the latter, although they are classified in the same high-temperature grade (88 ^{0}C). This result confirms with what was obtained from the basic property tests.

4.3 OPTIMUM MOISTURE CONTENT (OMC)

4.3.1 Introduction

Moisture is required for several reasons: (a) to soften and breakdown agglomerations in the aggregates, (b) to aid in bitumen dispersion during mixing and for field compaction; (c) to function as lubricant within the aggregate matrix. The percentage of water adhered by the aggregate therefore plays a major role in the adhesion and the coating (or “mastic”) that is achieved on the larger particles (Van de Walt et al 1999).

Various proposals on OMC to ensure the best mixing and compaction results have been documented. Ruckel et al (1983) recommend that the moisture-density relationship be considered in the formulation of trial mixes.

Investigations by Mobil Oil suggest that the optimum moisture content for mixing lies at the “fluff point” of the aggregate, i.e. the moisture content at which the aggregates have a maximum loose bulk volume (70 % - 80 % mod AASHTO OMC). However, the fluff point may be too low to ensure adequate mixing (foam dispersion) and compaction, especially for finer materials. Lee (1981) found that the optimum mixing moisture content occurred in the range of 65 to 85 percent of the modified AASHTO OMC for the aggregates. Bissada (1987) confirmed the optimum mixing moisture content for mixing of the aggregates.

Sakr and Manke (1985) developed a relationship (Eq. 4.1) to calculate the moisture content for maximum density of foamed asphalt mixes, which considers the modified AASHTO OMC, and percentage of fines (PF) of the aggregate and the bitumen content (BC). As suggested by the equation, the higher the bitumen content, the lower the compaction moisture content.

$$MMC = 8.92 + 1.48(OMC) + 0.40(PF) - 0.39(BC) \tag{4.1}$$

where *MMC*=moisture content of mixing and compaction (%),

OMC= the modified AASHTO OMC (%),

PF= percentage of fine (%),

BC=bitumen content (%).

The OMC for mixing is approximately 10 to 20 percent higher than the compaction moisture (MMC), as predicted by Eq. 4.1. In order to prevent the time-consuming task of drying the mix after mixing (to achieve the MMC), Sakr and Manke suggested that the MMC be used for both mixing and compaction, as no significant differences in mix properties were observed when this procedure was used.

The amount of water added to mix was approximately 70 percent of the OMC recommended by Wirtgen Co. This method was adopted to determine the added water in this research. The added water content could be calculated by the Eq.4.2 (Wirtgen GmbH 2001):

$$W_{add} = 0.7 \times W_{OMC} - W_{moist} + 0.6 \tag{4.2}$$

where W_{add} =water content to be added to sample (% by mass),

W_{OMC} =optimum moisture content (% by mass),

W_{moist} =moisture content in sample (% by mass).

Castedo and Wood (1983), Muthen (1998) recommended that the OMC of the mix occurred when the total fluid content (water + bitumen) is approximately equal to the OMC of the aggregate for modified impact compaction.

Because the OMC obtained by the gyratory compaction method is lower than that by Modified AASHTO method, Maccarrone et al (1995) advocated OMC for mixing and compaction when gyratory compaction is used for the preparation of foamed asphalt.

4.3.2 OMC Test

There are two methods for determination of the OMC according to AASHTO specification: ASTM D 698 and ASTM D 1557 (corresponding to AASHTO T90 and AASHTO T180). The difference between these two methods is due to the compaction effort.

For ASTM D 698, the sample is compacted in a 4 or 6 in (101.6 or 152.4 mm) diameter mold with a 5.5-lbf (24.4-N) rammer dropped form a height of 12 in. (305 mm) producing a compaction effort of 12400 ft-lbf/ft^3 (600 kN-m/m^3). This method is suitable to determine the relationship between water content and dry unit weight of soils (compaction curve) for the pavement material of the lightly trafficked road. However, it is insufficient for determine the compaction characteristics of pavement material of the heavily trafficked road. In order to achieve this purpose, another testing method, i.e. ASTM D1557, is provided.

For ASTM D 1557, the sample is compacted in a 4 or 6 in (101.6 or 152.4 mm) diameter mold with a 10-lbf (44.5-N) rammer dropped form a height of 18 in. (457 mm) producing a compaction effort of 56000 ft-lbf/ft^3 (2700 kN-m/m^3).

In Hong Kong, traffic on the freeway and trunk road is very heavy. Hence the FA mix used as the road base of these roads must be evaluated by ASTM D1557 method, not ASTM D698 method. In this research, the OMCs of all FA mixes will be tested in accordance with ASTM D1557 (2000).

Since the maximum particle size of the aggregate is not greater than 20mm, more than 20% by mass of the material is retained on the 4.75-mm sieve and 13% (<20%) by mass of the material is retained on the 10-mm sieve, the OMC of aggregates can be tested by the Method B of ASTM D1557. Specimen can be compacted in a 100-mm diameter mold.

In the test, the oversize fraction, the portion of total sample retained on the 19-mm sieve, is not used in performing the compaction test, correction must be made to the unit weight and water content of the test specimen (or the appropriate field in place density test specimen) using practice ASTM D4718 (2001). The test procedures and its correction are listed in the APPENDIX B of this chapter.

4.3.3 Result And Analysis

Aggregates containing different percentage of RAP contents (0%, 20%, 40% and 60%) are tested in accordance with above procedures. There are totally 7 groups of aggregates (the OMC of 0%RAP#1 is the same as that of 0%RAP#2). Specific gravities of aggregates, including finer and oversize, are calculated using the basic property data obtained in Section 4.2. Testing results of OMC are summarized in Table 4.7 and 4.8.

In the compaction test, the maximum dry unit weight is defined as the maximum value of the compaction curve. The optimum moisture content is the water content at which the aggregate can be compacted to the maximum dry unit weight.

The dry unit weight and water content of each group are plotted in Fig. 4.3 and Fig. 4.4. From these two figures, the OMC of each group can be determined. OMC results of all groups are listed in Table 4.9.

Table 4. 7 OMC results of aggregates containing RAP#1

Specimen	W_F (%) Water content of the fine	V (m^3) Volume of the mold	M_t (kg) Mass of the mold	M_{md} (kg) Mass of the moist specimen and mold	ρ_m (Mg/m^3) Moist density	ρ_d (Mg/m^3) Dry density	γ_d (kN/m^3) Dry unit weight	Gs Specific gravity of soils	W_{sat} (%) Water content for complete saturation	W_c Water content of the oversize	C_w Corrected water content	G_m Bulk specific density of the total specimen	$C\delta_d$ Corrected unit weight of the total specimen
RAP#1%0													
RAP#1%0(3%)	4.04	0.0006183	2.1873	3.5154	2.15	2.06	20.25	2.58	9.52	0.005809	3.18	2.53	21.23
RAP#1%0(4%)	5.40	0.0006198	2.1931	3.5542	2.20	2.08	20.43	2.58	9.09	0.010256	4.31	2.53	21.38
RAP#1%0(5%)	6.47	0.0006199	2.1877	3.5805	2.25	2.11	20.69	2.58	8.48	0.014183	5.21	2.53	21.59
RAP#1%0(6%)	7.64	0.0006191	2.1908	3.6057	2.29	2.12	20.82	2.58	8.19	0.022915	6.30	2.53	21.70
RAP#1%0(7%)	8.20	0.0006192	2.1941	3.5952	2.26	2.09	20.51	2.58	8.91	0.035552	7.04	2.53	21.44
RAP#1%20													
RAP#1%20(3%)	4.12	0.0006183	2.1874	3.4898	2.11	2.02	19.84	2.51	9.52	0.010033	3.34	2.50	20.83
RAP#1%20(4%)	5.35	0.0006198	2.1932	3.5334	2.16	2.05	20.13	2.51	8.81	0.013029	4.34	2.50	21.07
RAP#1%20(5%)	6.53	0.0006199	2.1877	3.5558	2.21	2.07	20.32	2.51	8.36	0.016241	5.30	2.50	21.23
RAP#1%20(6%)	7.51	0.0006191	2.1910	3.5933	2.27	2.11	20.66	2.51	7.55	0.024946	6.25	2.50	21.51
RAP#1%20(7%)	8.04	0.0006192	2.1942	3.5901	2.25	2.09	20.46	2.51	8.02	0.030848	6.80	2.50	21.35
RAP#1%40													
RAP#1%40(3%)	4.25	0.0006183	2.1873	3.4653	2.07	1.98	19.44	2.45	9.53	0.010003	3.44	2.47	20.45
RAP#1%40(4%)	5.40	0.0006198	2.1931	3.4903	2.09	1.99	19.47	2.45	9.45	0.016667	4.47	2.47	20.48
RAP#1%40(5%)	6.43	0.0006199	2.1877	3.5190	2.15	2.02	19.79	2.45	8.65	0.019382	5.31	2.47	20.74
RAP#1%40(6%)	7.42	0.0006191	2.1909	3.5491	2.19	2.04	20.03	2.45	8.06	0.021762	6.11	2.47	20.93
RAP#1%40(7%)	7.70	0.0006192	2.1942	3.5504	2.19	2.03	19.94	2.45	8.26	0.032536	6.59	2.47	20.87
RAP#1%60													
RAP#1%60(3%)	4.20	0.0006183	2.1874	3.4453	2.03	1.95	19.15	2.39	9.31	0.008556	3.37	2.44	20.16
RAP#1%60(4%)	5.26	0.0006198	2.1931	3.4699	2.06	1.96	19.19	2.39	9.19	0.012235	4.25	2.44	20.19
RAP#1%60(5%)	6.26	0.0006199	2.1877	3.4886	2.10	1.97	19.37	2.39	8.72	0.015461	5.08	2.44	20.34
RAP#1%60(6%)	7.22	0.0006191	2.1908	3.5099	2.13	1.99	19.49	2.39	8.41	0.026417	6.07	2.44	20.44
RAP#1%60(7%)	7.96	0.0006192	2.1942	3.5134	2.13	1.97	19.35	2.39	8.76	0.035511	6.85	2.44	20.33

Note: Finer : oversize= 75: 25; value in brackets in the first column is the moisture content.

Table 4. 8 OMC results of aggregates containing RAP#2

Specimen	W_F (%) Water content of the fine	V (m^3) Volume of the mold	M_t (kg) Mass of the mold	M_{md} (kg) Mass of the moist specimen and mold	ρ_m (Mg/m^3) Moist density	ρ_d (Mg/m^3) Dry density	γ_d (kN/m^3) Dry unit weight	Gs Specific gravity of soils	W_{sat} (%) Water content for complete saturation	W_c Water content of the oversize	C_w Corrected water content	G_m Bulk specific density of the total specimen	$C\delta_d$ Corrected unit weight of the total specimen
RAP#2%20													
RAP#2%20(3.5%)	4.09	0.0006183	2.1892	3.4961	2.11	2.03	19.92	2.49	9.05	0.0080	3.26	2.48	20.86
RAP#2%20(4.5%)	6.13	0.0006198	2.1938	3.5425	2.18	2.05	20.11	2.49	8.58	0.0115	4.88	2.48	21.02
RAP#2%20(5.5%)	7.17	0.0006199	2.1923	3.5808	2.24	2.09	20.50	2.49	7.66	0.0231	5.95	2.48	21.33
RAP#2%20(6.5%)	7.58	0.0006192	2.1895	3.5919	2.26	2.11	20.65	2.49	7.31	0.0251	6.31	2.48	21.45
RAP#2%20(7.5%)	8.22	0.0006191	2.1890	3.5584	2.21	2.04	20.04	2.49	8.74	0.0389	7.14	2.48	20.96
RAP#2%40													
RAP#2%40(3.5%)	4.74	0.0006183	2.1891	3.4694	2.07	1.98	19.39	2.42	9.11	0.0087	3.77	2.42	20.32
RAP#2%40(4.5%)	6.11	0.0006198	2.1937	3.4997	2.11	1.99	19.47	2.42	8.89	0.0133	4.92	2.42	20.39
RAP#2%40(5.5%)	7.22	0.0006199	2.1923	3.5351	2.17	2.02	19.81	2.42	8.03	0.0163	5.82	2.42	20.66
RAP#2%40(6.5%)	7.51	0.0006191	2.1890	3.5444	2.19	2.04	19.97	2.42	7.64	0.0209	6.15	2.42	20.79
RAP#2%40(7.5%)	8.13	0.0006192	2.1893	3.5405	2.18	2.02	19.79	2.42	8.09	0.0344	6.96	2.42	20.65
RAP#2%60													
RAP#2%60(3.5%)	5.18	0.0006183	2.1891	3.4622	2.06	1.96	19.20	2.34	8.33	0.0109	4.16	2.37	20.07
RAP#2%60(4.5%)	6.36	0.0006198	2.1937	3.4909	2.09	1.97	19.30	2.34	8.07	0.0128	5.09	2.37	20.15
RAP#2%60(5.5%)	7.11	0.0006199	2.1923	3.5100	2.13	1.98	19.46	2.34	7.64	0.0170	5.76	2.37	20.29
RAP#2%60(6.5%)	7.49	0.0006191	2.1890	3.5209	2.15	2.00	19.63	2.34	7.22	0.0189	6.09	2.37	20.42
RAP#2%60(7.5%)	8.09	0.0006192	2.1891	3.5160	2.14	1.98	19.44	2.34	7.69	0.0311	6.84	2.37	20.27

Note: Finer : oversize= 75: 25; value in brackets in the first column is the moisture content.

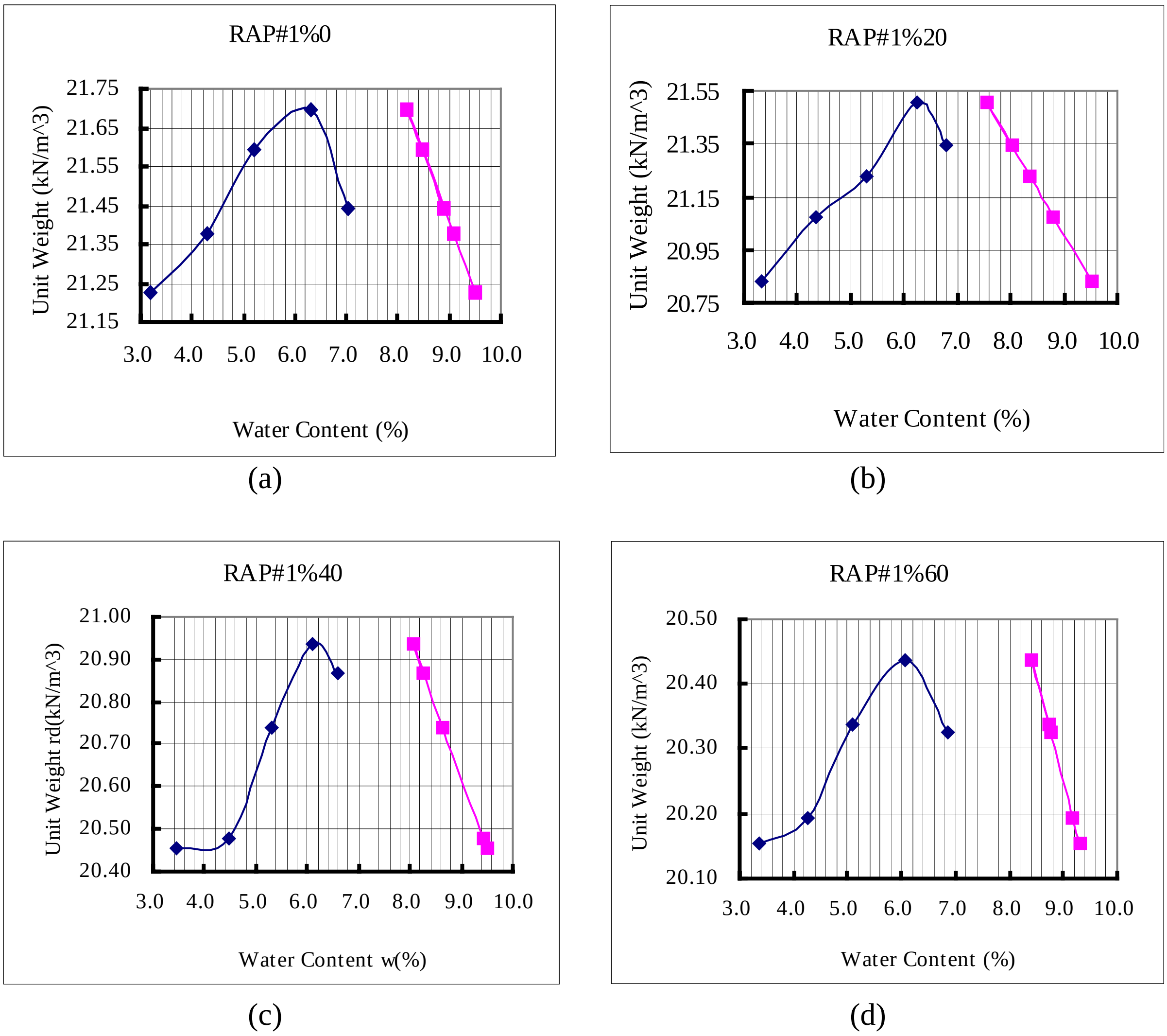
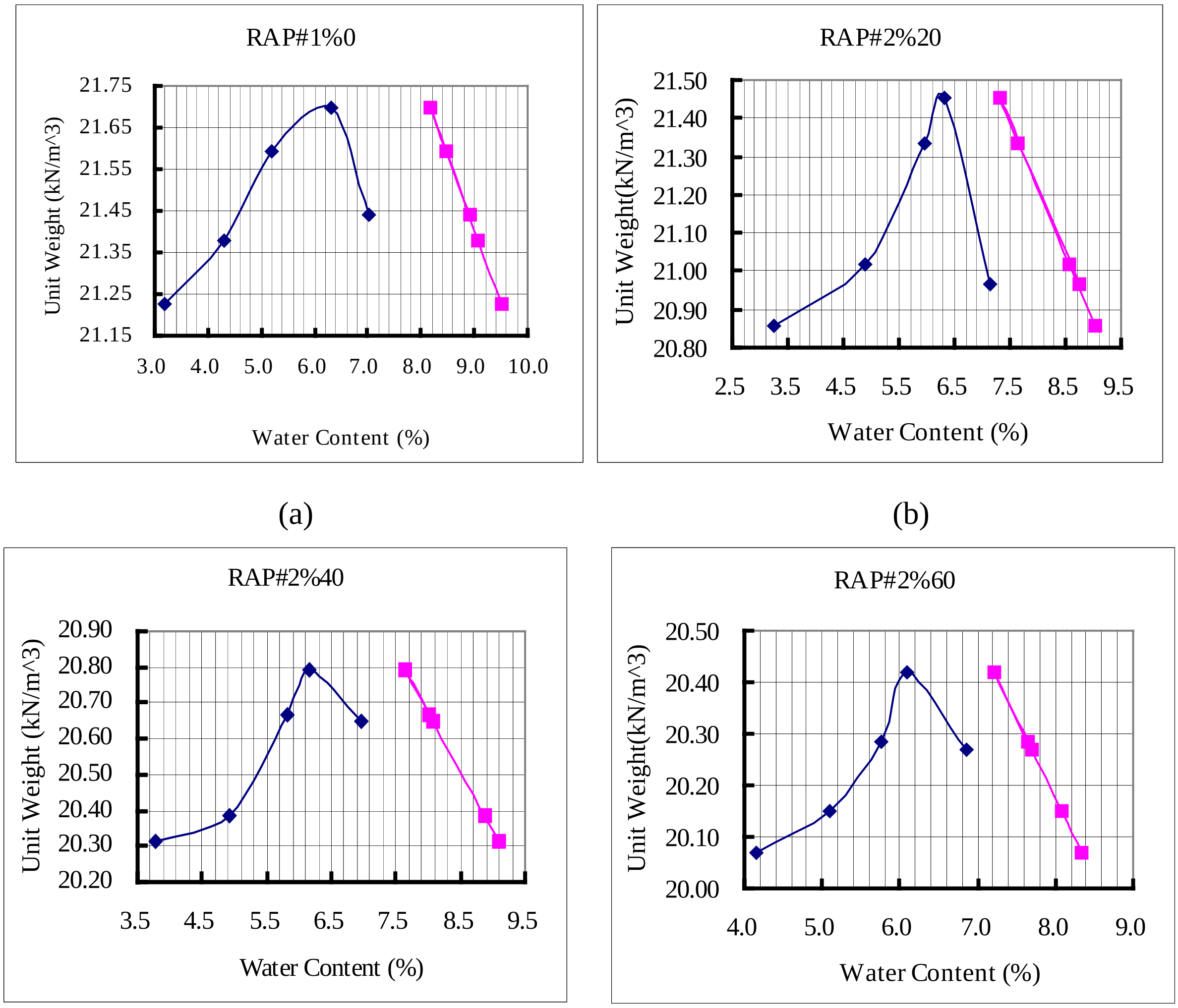


Fig.4. 3 Plots of the dry unit weight vs. water content of RAP#1 materials



(c) (d)

Fig.4. 4 Plots of the dry unit weight vs. water content of RAP#2 materials

Table 4. 9 OMC results of each group aggregate

Group	OMC (%)	γ_d (KN/m^3)	ρ_d(Mg/m^3)
RAP#1%0	6.30	21.70	2.21
RAP#1%20	6.25	21.51	2.19
RAP#1%40	6.11	20.93	2.14
RAP#1%60	6.07	20.44	2.09
RAP#2%0	6.27	21.70	2.21
RAP#2%20	6.31	21.45	2.19
RAP#2%40	6.15	20.79	2.12
RAP#2%60	6.09	20.42	2.08

4.4 DETERMINATION OF DESIGN BITUMEN CONTENT

4.4.1 Consideration And Method

1. Difference between FA mix and hot-mixed asphalt mix

There are two major differences between cold and hot bitumen mixes. First, a curing period is required for cold mixes both in the laboratory before testing, and in the field before being subjected to traffic. Second, significant strength losses for cold mix occur when the materials get wet.

The foaming process leads to a surface expansion of the bitumen and results, at the same time, in a reduction of its viscosity. A distinct difference between foamed mixes, and asphalt and emulsion stabilized mixes is the way in which the bitumen is dispersed through the aggregate. In the later case the bitumen tends to coat all particles whilst in the foamed mixes the larger particles are not fully coated, this means that the coarser particles have thinner films or even have no films.

Aggregate coating is an important consideration for cold mixes. Aggregate coating is essential to ensure moisture resistance. The most critical aggregate fraction to be coated is the fine portion as this is most affected by moisture.

The foamed bitumen disperses itself among the finer particles forming a mortar which binds the mix together. This partial coating gives the appearance of only a slightly dark colour to the mix compared to a much darker colour that will result if the same material were to be prepared to produce asphalt. This colour difference can be shown in Fig. 4.5, in which the left is FA mix specimen and the right is hot-mixed asphalt specimen. Fig. 4.6 illustrates the foam-coating of aggregates in the foamed asphalt mix, it shows the reason of the color difference between two kinds of mixes.

Fig.4. 5 Comparison of FA mix specimen and hot-mixes asphalt specimen

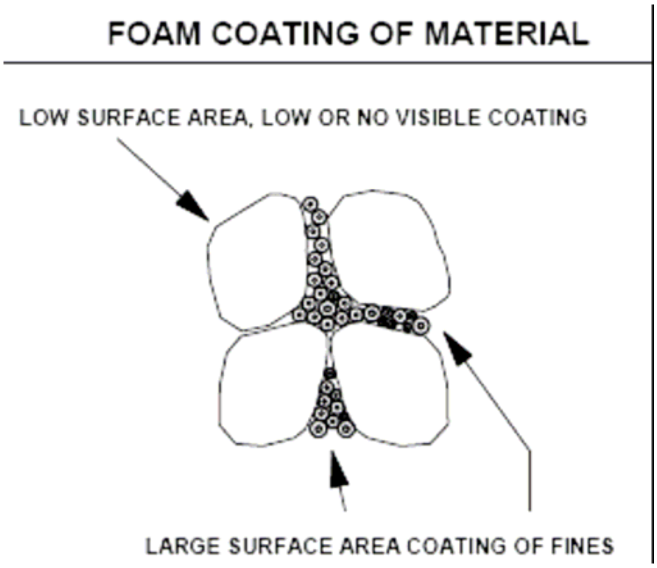


Fig.4. 6 Foam-coating of aggregates

2. Consideration

The bitumen content included in a foamed asphalt mix is a function of the aggregate type and moisture content, and the desired mix properties. Generally, foamed asphalt mix can be prepared at lower bitumen contents and has about the same properties as conventional cold mixture at higher bitumen contents. Typical bitumen contents for foamed asphalt mix range from 3 to 6 percent.

(1) Thickness of bitumen film

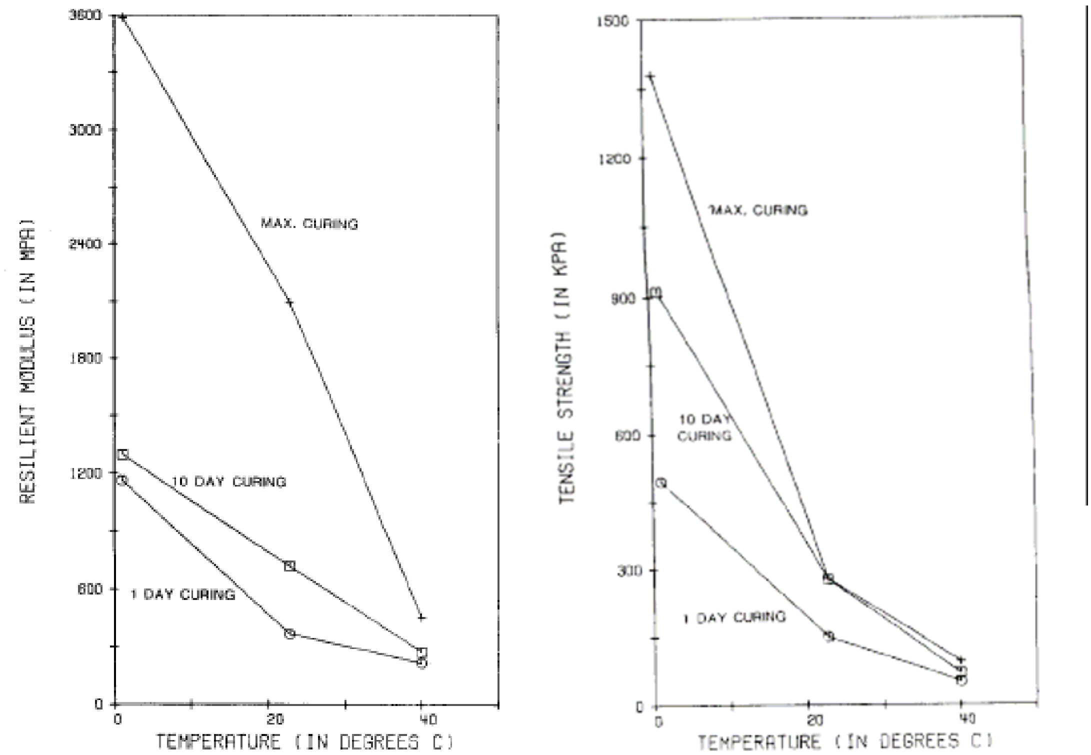
The most significant factors affecting the film thickness of the bitumen are adhesion between the aggregate, the bitumen temperature, viscosity, cohesion, and the surface tension as well as quantity of the bitumen at mixing temperature. When the films are too thick, the bitumen simply lubricates the aggregate particles. When the films are too thin there may not be enough bitumen for coating with a resulting decrease in mixture stability and high strength loss when tested wet.

Brennan (1983) reported the effect of water on stability decreased with increasing bitumen content as did the percent adsorbed water. Abel (1978) reported that the lower viscosity bitumen foamed better than higher viscosity bitumen but the higher viscosity bitumen produced better aggregate coating.

(2) Curing

Foamed asphalt mixes have a longer curing time than the conventional, generally 1-2 days, whilst their curing time is shorter than cement or emulsion bound pavements thus allowing early trafficking. Abel (1978) emphasized that the curing of FA mixes should be done after mixing when the water in the mix evaporated.

Lab test needs to simulate both short term and long term curing in the field. Accelerated curing in the oven must be calibrated with field experience and with curing tests at ambient temperature. Van Wijk and Wood (1982) investigated effects of the curing time and temperature on resilient modulus and tensile strength. Fig 4.7 shows curing time and temperature have significant influence on properties of the FA mix. Resilient modulus and tensile strength of specimen after 10-day curing at 20 ^{0}C are approximately two times these after 1-day curing at the same temperature. At 40 ^{0}C, there is no significant difference among 1-day curing, 10 day curing and maximum curing.



(a) Resilient modulus (b) Tensile strength

Fig.4. 7 Effect of curing time on resilient modulus and tensile strength

Holleran and Ky (1995) pointed out that oven cured samples gave similar modulus results as field cores taken 12 months after construction, hence 3-day curing at 60 ^{0}C appeared relevant for 12 months field curing for foamed bitumen systems.

Engelbrecht et al (1985) also reported that curing temperature, length of curing and moisture conditions dramatically affected the strength of foamed asphalt mixes prepared using either the sand and recycled RAP materials.

In the mix design procedure of the foamed asphalt mixes, Muthen (1998) proposed the 3-day oven curing at 60 ^{0}C in the specimen preparation. However Wirtgen Co. recommends using the 3-day oven curing at 40 ^{0}C (Wirtgen GmbH 2001).

Nataatmadja (2001) investigated effects of the curing methods on properties of the foamed asphalt mixes. Three curing methods, i.e. 28 days of air curing at ambient temperature (25 ^{0}C), 3 days of curing at 40 ^{0}C, and 3 days at 60 ^{0}C were compared. It was found that the 60 ^{0}C curing produced the highest modulus, which was probably associated with a change in bitumen characteristics within the specimen due to bitumen ageing. Curing at 60 ^{0}C would result in an overestimation of the resilient modulus compared with specimens cured at an ambient temperature. Whereas the 60 ^{0}C curing temperature could reflect the temperature regime and the long-term moisture content in some situations, the 3-day curing at 40 ^{0}C would probable be more suitable in this particular case. Nataatmadja found that a specimen cured for 3 days at 40 ^{0}C produced modulus values similar to that of 7 days of air cuing.

Based on above literature review, the 3 days oven drying at 40 ^{0}C is selected as the accelerated curing method in this research.

(3) Compaction method

Nataatamadja (2001) reported that Marshall compaction method produced higher resilient modulus values compared with the gyratory compaction method. Moreover, Marshall compaction showed an optimum bitumen content associated with the maximum density. The gyratory compactor produced specimens that were less dependent on bitumen content but had higher density values.

Gyratory compaction method has been applied extensively in the hot-mixed asphalt design. However, applying this compaction method in mix design of the cold-recycled materials is still in research. In this research, FA mix specimens are prepared by Marshall compaction method with 75 blows for each of two faces.

3. Method

The foamed asphalt mix is generally used as a road base or base course covered a hot-mixed asphalt mix as surfacing layer. Hence this kind of material is required to have better fatigue resistance to repeated loading.

Due to the larger aggregates partially coated by bitumen, the strength of FA mix is mainly contributed by adhesion of the mortar between aggregates. Moisture-resistance of the FA mix is very different from that of hot-mixed asphalt; effect of moisture on this mix is significant. Therefore moisture of FA mix must be considered in the determination of bitumen content as well as indirect tensile properties.

Resistance to indirect tensile can be characterized by the indirect tensile strength test (ITST). The indirect tensile testing is often used to investigate the crack-resistance at low temperature and fatigue-resistance of asphalt mixes (Christensen 1998). It is also adopted to select the design bitumen content in the FA mix design.

There are several typical methods, such as Lottman Method, AASHTO T283 method, for evaluating the moisture effect on the asphalt mix. In Chapter 7, the effect of the moisture on FA mixes will be evaluated by laboratory testing, including moisture effect on the permanent deformation, strength and fatigue performance. In the mix design, soaking after vacuum condition is often used to simulate the worst condition of FA mix in the service period, and the soaked specimen is tested by the indirect tensile test in order to determine the design bitumen content. The soaking treatment can be referred to Appendix A.

Wirtgen GmbH (2001) and Muthen (1998) also recommended that the bitumen at which the soaked ITS is maximum is taken as the design bitumen content for the foamed bitumen treated mix. This recommendation emphasizes the moisture effect in the worst condition. Lancaster et al (1994) recommended that the bitumen content selected for foamed asphalt mixes be based on the highest resilient modulus value. Kendall et al (1999) reported that the bitumen content was designed to meet a minimum soaked resilient modulus of 1500 MPa.

In this research, the indirect tensile strength test is used to select the design bitumen content. The optimum bitumen content is considered as the bitumen at which the soaked indirect tensile strength is maximum. This method simultaneously considers the moisture effect on the FA mixture and resistance of this mixture to indirect tensile. It is commonly accepted and used in the FA mix design.

4.4.2 Testing, Results And Analysis

1. Testing

For the convenience of analyzing and depicting the testing results, each combination/group with different RAP content, different RAP materials and bitumen type is coded with letter. There are totally 14 groups (see Table 4.10). Because there was no detailed information to guide the mix design of FA, the bitumen content range was large. It was assumed that bitumen content decreased with an increase of the RAP content. This assumption should be further confirmed.

Table 4. 10 Coding and bitumen content range of each group

Bitumen	Shell 60							Shell 100						
RAP		RAP #1			RAP #2				RAP #1			RAP #2		
%RAP	0%	20%	40%	60%	20%	40%	60%	0%	20%	40%	60%	20%	40%	60%
Group	A	B	C	D	F	G	H	J	K	L	M	O	P	Q
Bitumen content range (%)	2-6	1.5-5.5	1-5	0.5-4.5	1.5-5.5	1-5	0.5-4.5	2-6	1.5-5.5	1-5	0.5-4.5	1.5-5.5	1-5	0.5-4.5

Based on Tables 2.6 and 2.7, an appropriate range of foamed bitumen contents is selected for the trial mixes (see Table 4.10). Herein it is assumed that the bitumen content of each group decreases as RAP content increase. Five batches of each group are normally prepared at bitumen contents differing by one percent.

Each 10 kg sample of aggregate and the required mass of foamed bitumen (and 1% cement) are prepared and mixed in a mechanical mixer at the moisture content which was calculated by Eq. 4.2. The foamed asphalt mixes are stored in sealed containers to prevent moisture loss, until all five batches have been mixed.

Duplicate samples are removed from each foamed asphalt batch for the determination of moisture content and bitumen content. The remaining foamed asphalt will be used to prepare compacted foamed asphalt specimens for further testing: six samples for indirect tensile tests (3 for dry ITS and 3 for soaked ITS) and 2 samples for volumetric evaluation.

The maximum theoretical specific gravity (MTSG) is tested in accordance with ASTM D2041 (2000). The dry gravity is calculated according to Eq.(A.6) in the APPENDIX A. Specimens are compacted, using the Marshall hammer, to a 100 mm nominal diameter and 65 mm nominal height. Specimens of this size normally require about 1,15 kg of material. A compaction effort of 75 blows on each face is recommended.

All foamed asphalt specimens must be subjected to 3 days oven drying at 40 ^{0}C before undergoing any tests. As well as dry and soaked ITS tests, resilient modulus test at the design bitumen content will be carried out. Dry density and air void of FA mixes are also to be tested. Detailed procedures of all tests can be referred to A.3 to A.11 in the APPENDIX A.

2. Results and analysis

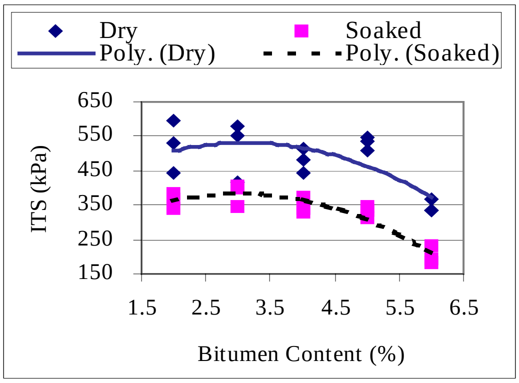
After testing for all groups, results of dry and soaked ITS, dry density, MTSG and air void are obtained and listed in Table 4.11and 4.12. Plots of bitumen content vs. dry and soaked ITS of each group are illustrated in Fig 4.8 and Fig.4.9. Three ITSs of each trial mix are all shown in the two figures.

Table 4. 11 ITSs and volumetric properties of FA mixes treated by Shell 60

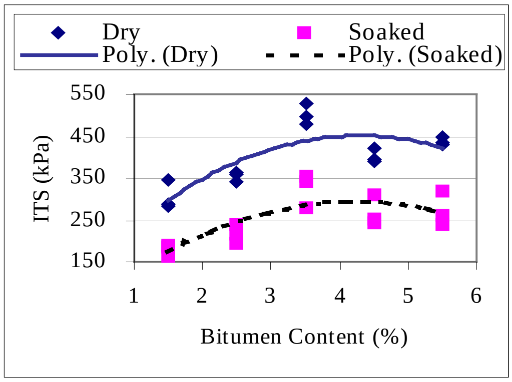
Group and Trial mix	BC	Air Void (%)	Dry Density (Kg/m^3)	MTSG	Dry ITS (kPa)	Soaked ITS (kPa)	ITS Ratio
A1	2	12.84	2.104	2.414	521.97	363.72	0.697
A2	3	11.61	2.078	2.351	516.53	384.82	0.745
A3	4	12.58	2.033	2.325	478.35	346.56	0.724
A4	5	12.51	2.011	2.299	483.47	331.15	0.685
A5	6	14.92	1.951	2.293	346.04	204.16	0.590
B1	1.5	12.35	2.079	2.372	305.36	179.55	0.588
B2	2.5	12.07	2.052	2.333	354.73	215.94	0.609
B3	3.5	11.83	2.030	2.303	501.33	323.71	0.646
B4	4.5	13.05	1.996	2.295	401.11	268.87	0.670
B5	5.5	12.30	1.982	2.260	437.03	273.22	0.625
C1	1	11.26	2.086	2.350	355.85	229.70	0.645
C2	2	11.45	2.047	2.312	393.26	283.90	0.722
C3	3	11.89	2.027	2.300	427.58	303.94	0.711
C4	4	12.50	1.998	2.283	485.20	324.10	0.668
C5	5	11.80	1.977	2.241	502.30	264.16	0.526
D1	0.5	12.93	2.081	2.390	381.05	310.27	0.814
D2	1.5	11.94	2.034	2.310	394.78	311.04	0.788
D3	2.5	11.07	2.005	2.254	367.88	306.09	0.832
D4	3.5	12.40	1.972	2.251	376.83	319.99	0.849
D5	4.5	11.42	1.964	2.217	377.31	284.07	0.753
F1	1.5	10.84	2.101	2.356	421.76	303.70	0.720
F2	2.5	10.24	2.085	2.323	440.75	374.04	0.849
F3	3.5	11.04	2.058	2.313	425.41	355.50	0.836
F4	4.5	11.45	2.018	2.279	462.77	370.11	0.800
F5	5.5	12.16	1.982	2.257	380.48	332.56	0.874
G1	1	11.37	2.097	2.366	408.37	313.57	0.768
G2	2	10.66	2.069	2.316	401.27	325.26	0.811
G3	3	8.48	2.049	2.238	466.82	344.85	0.739
G4	4	9.50	2.002	2.212	455.27	326.03	0.716
G5	5	9.68	1.968	2.179	371.96	285.45	0.767
H1	0.5	13.12	2.096	2.412	373.04	307.74	0.825
H2	1.5	9.70	2.059	2.280	319.85	316.14	0.988
H3	2.5	9.21	2.027	2.233	400.71	349.11	0.871
H4	3.5	9.77	2.001	2.217	379.80	359.80	0.947
H5	4.5	10.38	1.968	2.196	332.19	309.16	0.931
Average							0.752

Table 4. 12 ITSs and volumetric properties of FA mixes treated by Shell 100

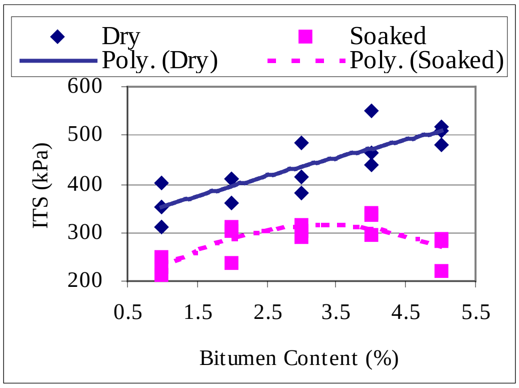
Group and Trial mix	BC	Air Void (%)	Dry Density (Kg/m^3)	MTSG	Dry ITS (kPa)	Soaked ITS (kPa)	ITS Ratio
J1	2	10.93	2.147	2.410	450.46	311.75	0.692
J2	3	10.73	2.109	2.362	402.90	335.11	0.832
J3	4	12.58	2.055	2.351	359.02	360.11	1.003
J4	5	12.69	2.039	2.336	322.44	308.70	0.957
J5	6	13.07	2.012	2.315	302.43	329.76	1.090
K1	1.5	11.47	2.116	2.390	348.46	316.17	0.907
K2	2.5	11.40	2.089	2.358	397.87	348.47	0.876
K3	3.5	12.37	2.051	2.341	384.55	339.38	0.883
K4	4.5	12.48	2.008	2.294	338.87	272.45	0.804
K5	5.5	11.24	1.985	2.236	275.45	234.49	0.851
L1	1	12.55	2.081	2.380	307.19	250.66	0.816
L2	2	12.34	2.042	2.330	328.34	275.33	0.839
L3	3	12.88	2.008	2.305	293.25	266.94	0.910
L4	4	12.99	1.976	2.271	282.21	253.11	0.897
L5	5	12.43	1.959	2.237	245.43	249.46	1.016
M1	0.5	11.98	2.046	2.325	269.19	208.43	0.774
M2	1.5	11.64	2.036	2.304	333.01	258.93	0.778
M3	2.5	11.48	2.027	2.290	346.68	293.86	0.848
M4	3.5	10.64	2.002	2.240	297.60	267.80	0.900
M5	4.5	11.47	1.951	2.204	234.54	240.02	1.023
O1	1.5	13.92	2.093	2.431	357.23	304.70	0.853
O2	2.5	13.33	2.074	2.393	387.32	330.73	0.854
O3	3.5	12.88	2.044	2.346	327.48	345.18	1.054
O4	4.5	13.08	2.018	2.322	340.64	355.15	1.043
O5	5.5	14.34	1.965	2.294	244.87	267.31	1.092
P1	1	11.89	2.087	2.368	320.72	236.08	0.736
P2	2	11.78	2.040	2.312	406.52	274.00	0.674
P3	3	11.71	2.017	2.285	338.64	296.20	0.875
P4	4	10.46	2.000	2.234	333.35	279.19	0.838
P5	5	11.15	1.978	2.226	305.42	210.93	0.691
Q1	0.5	13.61	2.032	2.352	261.50	182.38	0.697
Q2	1.5	13.40	2.014	2.326	319.14	231.13	0.724
Q3	2.5	12.08	1.994	2.268	261.30	230.99	0.884
Q4	3.5	11.98	1.970	2.238	315.19	223.45	0.709
Q5	4.5	12.04	1.950	2.217	265.43	217.93	0.821
Average							0.864



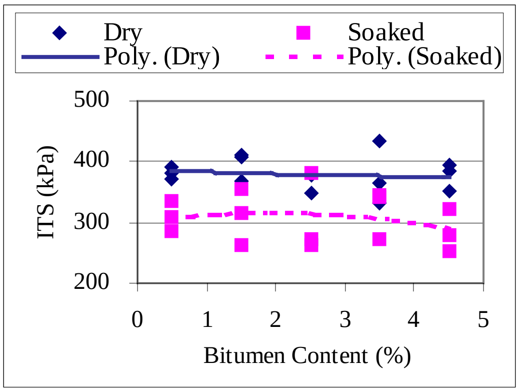
Group A



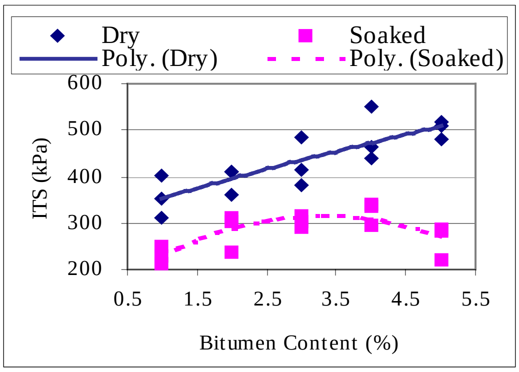
Group B



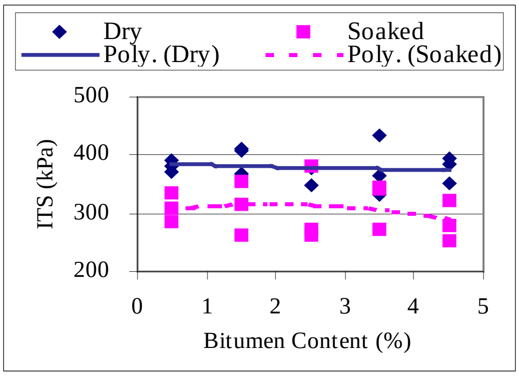
Group C



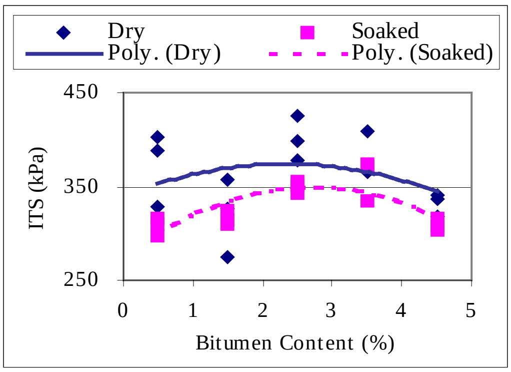
Group D



Group F

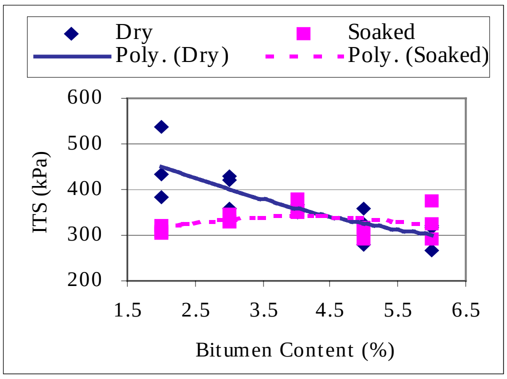


Group G

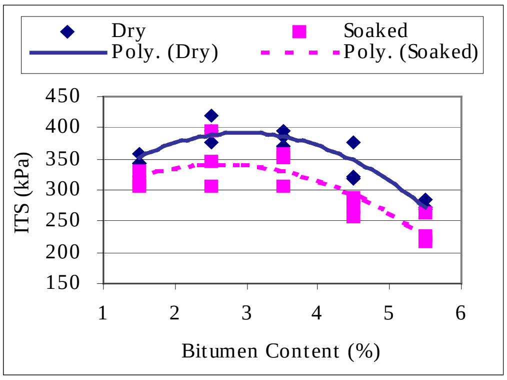


Group H

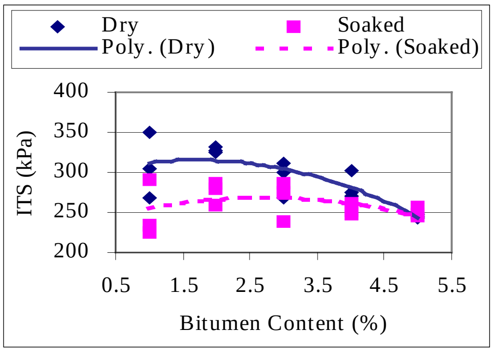
Fig.4. 8 Plots of bitumen content vs. dry and soaked ITS (Group A~H)



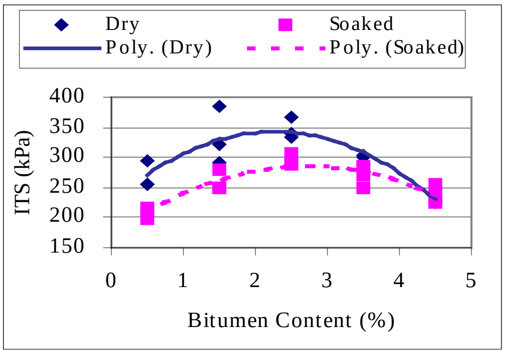
Group J



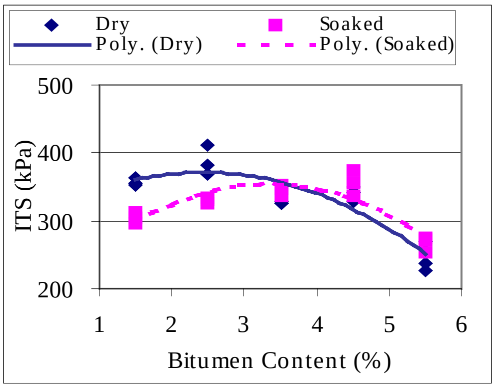
Group K



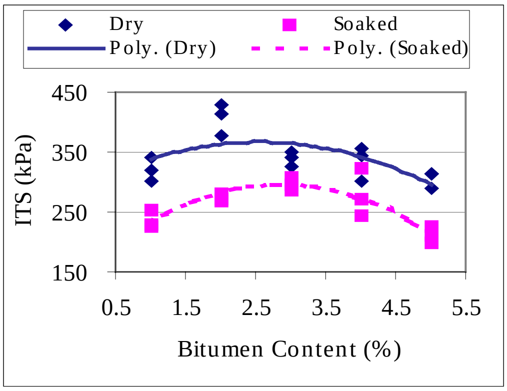
Group L



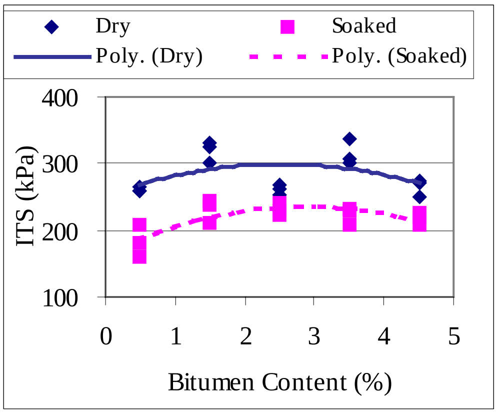
Group M



Group O



Group P



Group Q

Fig.4. 9 Plots of bitumen content vs. dry and soaked ITS (Group J~Q)

It can be found from Fig. 4.8 and Fig. 4.9 that the design bitumen contents of all FA mix groups are within 2.5~3.5%. Deviations between the design bitumen contents of various FA mixes are within1%. Therefore the early assumption that bitumen content decreased with an increase of the RAP content is not appropriate.

No matter which RAP material (RAP#1 or RAP#2) is utilized, Fig.4.8 illustrates that design bitumen content of the FA mix treated by Shell 60 is about 3.5% when the added RAP content is 0% or 20% (Group A, B, F), is about 3.0% when the added RAP content is 40% or 60% (Group C, D, G, H). Hence 3.5% is recommended for FA mixes containing 0% and 20% RAP materials, and 3.0% for FA mixes containing 40% and 60% RAP materials.

For Shell 100 bitumen, the same conclusion can also be drawn. The design bitumen contents of the FA mix treated by Shell 100 are selected as 3.5% and 3.0% for the low-added RAP content (0% and 20%, corresponding to Group J, K, O) and the high-added RAP content (40% and 60%, corresponding to Group L, M, P, Q).

Ruckel et al (1983) recommended that the bitumen content was 4% when percentage of aggregate passing 4.75mm was larger than 50% and percentage of filler passing 0.075mm was within 5~7.5%. In this study, the percentage of aggregate passing 4.75mm is 55%, and filler content is 6%. The above testing results are close to the bitumen content recommended by Ruckel et al.

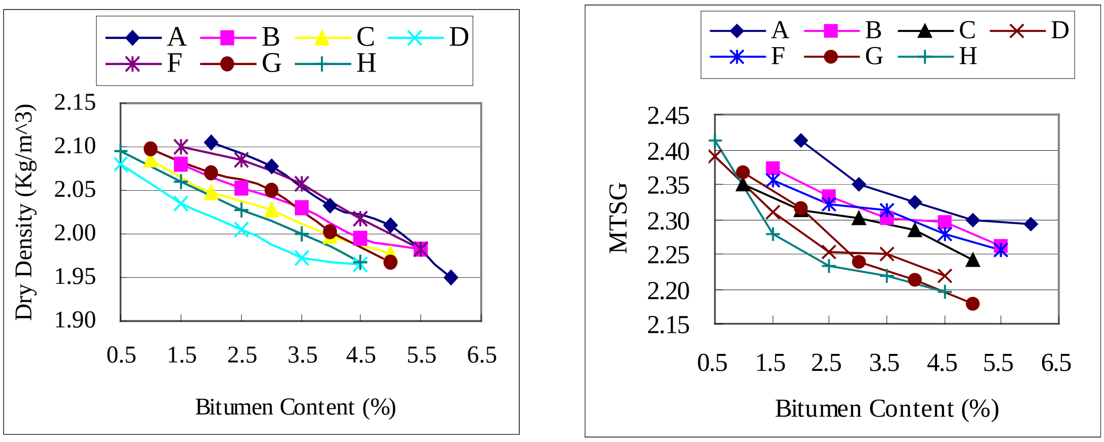
When the added RAP content is lower (0% and 20%), the aged RAP material does not significantly impact on the FA mix, they may be regarded as the virgin aggregate. Hence design bitumen content is higher (3.5%). On the contrary, RAP material will impact on the FA mix at high-added content, the design bitumen is lower (3.0%).

Fig.4.8 and Fig.4.9 indicate that there is no design bitumen content in some cases if it is determined as the content at which dry ITS is at a maximum. Therefore the method used to determine the design bitumen content in this research is rational and practical.

4.5 VOLUMETRIC AND MECHANICS PROPERTIES

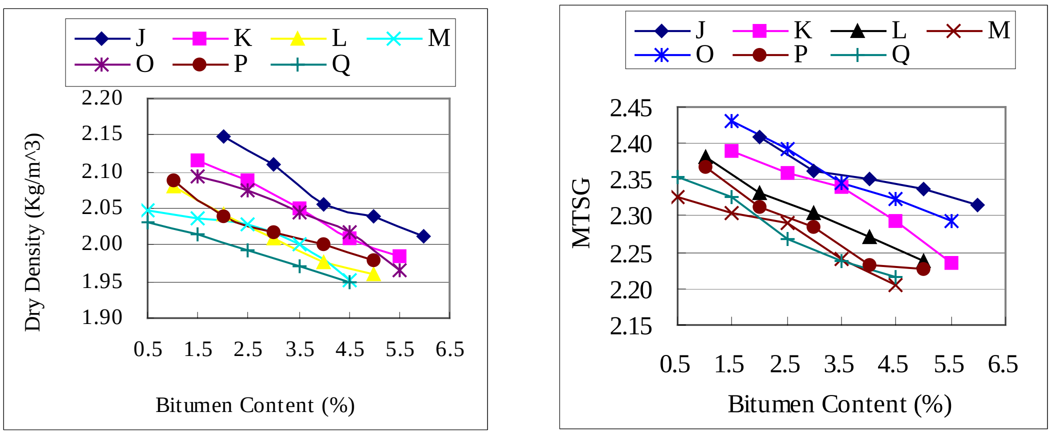
1. Volumetric properties

The volumetric properties of all groups of FA mixes are evaluated in this section. Results of dry density, MTSG and air void are listed in Tables 4.11 and 4.12. Fig.4.10 and Fig. 4.11 illustrate the dry densities and MTSGs of FA mixes treated by Shell 60 and Shell 100 respectively.



(a) Dry Density (b) MTSG

Fig.4. 10 Dry densities and MTSGs of FA mixes treated by Shell 60



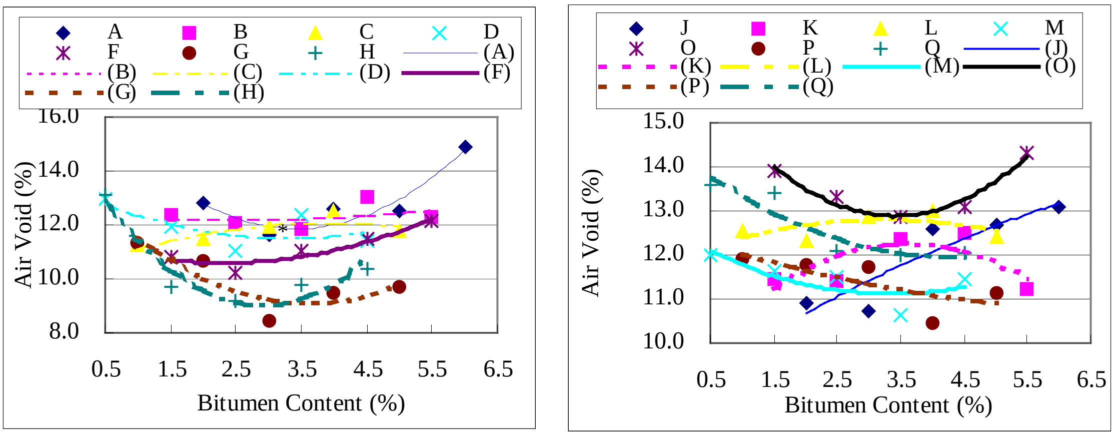
(a) Dry Density (b) MTSG

Fig.4. 11 Dry densities and MTSGs of FA mixes treated by Shell 100

It is clear from Fig.4.10 and Fig.4.11 that dry density and MTSG decrease with an increase of bitumen content no matter which RAP material is added or which bitumen is used to treat.

For Shell 60 bitumen, dry densities of FA mixes containing RAP#2 material are larger than those of FA mixes containing RAP#1 material at the same RAP content (F>B, G>C and H>D). For Shell 100 bitumen, except for Group L, dry densities of mixes containing RAP#1 material are larger than those of mixes containing RAP#2 material at the same RAP content (i.e. K>O, M>Q).

Air voids of all groups are plotted in Fig.4.12. Fig 4.12 (a) demonstrates a second order polynomial relationship for the FA mixes treated by Shell 60 (Group A~H) between air void and bitumen content. When bitumen content is smaller, air void is higher. The air void decreases as bitumen content increases, and reaches the bottom at 2.5% to 3.5% bitumen content. Then air void increases with an increase of bitumen content. This result is different from hot-mixed asphalt concrete. Because more bitumen may cause conglomeration of the mortar in the mix, the number of the small size particle decrease. Therefore the air void of FA mix decreases when the bitumen content exceeds the DBC



(a) Shell 60 (b) Shell 100

Fig.4. 12 Air voids of FA mixes treated by Shell 60 and Shell 100

Except for Group J, K, and L, Fig.4.12 (b) demonstrates a similar relationship between air void and bitumen content for those mixes treated by Shell 100 bitumen as that of mixes treated by Shell 60. The lowest air void approximately locates at 2.5% to 3.5% BC, which corresponds to the design bitumen content.

The air void of FA mix is smaller at lower bitumen content. As bitumen increases in the mix, more mortars made by filler and bitumen fill the void in the mix, which results in a decrease of air void. However, if bitumen content is higher than the design bitumen content, many smaller mortars gather into larger ones, and these larger mortars fill into the void. Because smaller mortars become less, the air void becomes higher.

Findings of volumetric properties in this study indicate that there is a BC (approximate 2.5~3.5%) which can produce the lowest air void for FA mixes. This BC corresponds to the design bitumen content at which FA mix has the lowest air void and highest soaked ITS. Hence lower air void helps to improve moisture susceptibility of FA mix, although the dry ITS of this mix is not the highest. On the other hand, the design bitumen content can also be selected as the content at which the air void is the lowest. This is consistent with that design bitumen content can be obtained at which the soaked indirect tensile strength is at a maximum.

2. Mechanics properties

(1) Indirect tensile strength

As a structural layer of the pavement, the FA mix must meet requirement of strength. Experts in ARRB (Maccarrone,1995) recommended that the dry and soaked indirect tensile strength should be at least 200 kPa and 100 kPa respectively. From Table 4.11 and 4.12, it is clear that all of dry and soaked indirect tensile strength meet the requirement recommended by Maccarrone.

For mixes treated by Shell 60 bitumen, all of soaked ITSs are larger than 170 KPa, majority of the soaked ITSs are within 250~400 KPa. The average ratio of soaked ITS to dry ITS is 0.752. Generally, the soaked ITSs of mixes incorporating RAP#1 material (Group B, C, D) are smaller than those of mixes incorporating RAP#2 material (Group F, G, H).

All dry ITSs of mixes treated by Shell 60 bitumen are in the range of 350 to 550 KPa. No matter which RAP material is added into the FA mix, the ITS of mix incorporating 60% of RAP materials is the lowest.

For mixes treated by Shell 100 bitumen, all of soaked ITSs are larger than 180 KPa, and the majority of the soaked ITSs are within 220~360 KPa. The average ratio of soaked ITS to dry ITS is 0.864. Compared with mixes treated by Shell 60 bitumen, mixes treated by Shell 100 have lower ITSs. Similar to finding for mixes treated by Shell 60, mixes containing RAP#1 material (Group K, L, M) have lower soaked ITS than mixes containing RAP#2 material (Group O, P, Q). This finding reflects that moisture susceptibility of FA mix containing RAP#1 material is better than that of mix containing RAP#2 material.

All dry ITSs of mixes treated by Shell 100 bitumen are in the range of 230 to 450 KPa. No matter which RAP material is added into the FA mix, the ITS of mix incorporating 60% of RAP materials is the lowest. This implies that 60% of RAP content is disadvantage to ITS of FA mix no matter which bitumen is used to treat FA mixes.

It is clear from Fig.4.8 and Fig.4.9 that there is a second order polynomial relationship between soaked ITS and bitumen content for all groups. The same finding can be obtained for dry ITSs of all groups except for Group C, D, F, G and J.

This second order polynomial relationship indicates that ITS of FA mix is smaller when the bitumen content is lower. ITS increases with an increase of the bitumen content. It obtains the maximum at the design bitumen content. Further increase of the bitumen content will cause a decrease of ITS. These convex-style second polynomial curves of ITS correspond to those concave-style second polynomial curves of air void.

(2) Indirect tensile stiffness modulus (ITSM)

ITSM is an important dynamic parameter used in the bituminous pavement design. For example, it is a parameter of the pavement structure design in AASHTO Pavement Design Guide. It has been reported in CSIR Transportek's projects that the loading time of 100 ms for the standard indirect tensile resilient modulus test may be too harsh for foamed asphalt samples. Lancaster et al (1994) proposed a loading time of 50 ms (at 25° C) for foamed asphalt resilient modulus test. He also recommended that the acceptance criteria for foamed asphalt mixes, with respect to resilient modulus, be at least 1500 MPa and 6000 MPa for soaked and dry specimens respectively. However, the resilient modulus requirements depend on the structural requirements of the pavements.

Holleran and Ky (1995) found that results on oven cured samples (3 days at 60 ^{0}C) indicated that the 2% bitumen was optimum for the foamed asphalt mix. Dry modulus of the foamed asphalt mix (3100 MPa) was much higher than for the two emulsions (1500 MPa).

ITSM test was carried out by the Nottingham Asphalt Tester (NU-10) for FA mix at the design bitumen content in accordance with BSI DD 213:1993 (2000). Differing from the loading time recommended by Lancaster, the loading rise-time of NU-10 Tester for testing the ITSM of FA mixes is 124 milliseconds. Hence it cannot compare the results of FA mix with Lancaster's criteria.

ITSM results of all groups are listed in Table 4.13 and plotted in Fig 4.13. Herein it is assumed that for each bitumen content, mixes containing no RAP materials have same ITSM value.

Table 4. 13 ITSM results of all FA mixes

%age of RAP	Shell 60		Shell 100	
	RAP#1	RAP#2	RAP#1	RAP#2
0	1878.0	1878.0	2299.0	2299.0
20	1591.0	1467.8	1697.5	1355.7
40	1577.8	1621.0	1572.0	1639.2
60	1243.3	1733.0	1524.7	1558.5

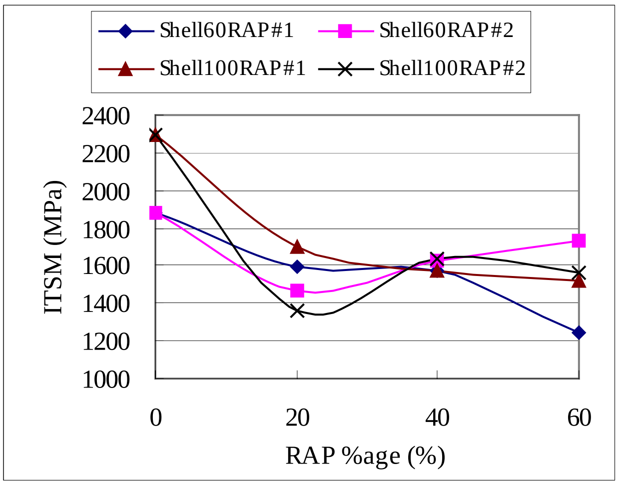


Fig.4. 13 Results of ITSM vs. RAP content of all groups

ITSMs of mixes treated by Shell 60 are in the range of 1240 to 1880 MPa. Values of mixes treated by Shell 100 are within 1350 to 2300 MPa. Hence the latter mixes have slightly higher ITSM than the former mixes.

No matter which bitumen is used to treat, it can be found from Fig 4.13 that ITSM decreases with an increase of RAP content for those mixes incorporating RAP#1 material. The maximum ITSM is obtained at 0% RAP content, not at minimum air void.

For mixes incorporating RAP#2 material, the minimum ITSM appears at 20% RAP content for two bitumen treatments. It has been found that minimum air void appears at 40% RAP content. However the maximum ITSM doesn't occur at 40% RAP content. Hence there is no direct correlation between the minimum air void and the maximum ITSM.

4.6 CONCLUSION

In this chapter, design method of the foamed asphalt mix is presented.

1. Grading

Based on the consideration of Hong Kong's weather conditions, size of the RAP materials, grading requirement of the pavement used in Hong Kong, a WC20 grading was proposed for the road base layer incorporating RAP materials. It meets the requirement of the preferred grading of the foamed asphalt mix recommended by CSIR, UK and Mobil Australia.

2. Materials

Two types of bitumen (Shell 60 and Shell 100), two RAP materials (RAP#1 and RAP#2) and four added RAP contents (0%, 20%, 40% and 60%) were considered in the mix design in order to investigate effects of the bitumen grades, RAP type and RAP content on the foamed asphalt mixes in further property tests of these mixes. There are totally 14 groups/combinations of mixes.

Apparent specific gravity test and sieve analysis were carried out for virgin and RAP aggregates. Basic properties (including penetration, softening point, high temperature viscosity) and DSR tests at three ageing conditions (original, RTFO and PAV) of virgin and recovered RAP bitumen were tested. All bitumen property tests indicate that RAP#2 is more aged than RAP#1.

3. OMC

The optimum moisture condition is a critical parameter of the construction. OMC tests were performed in accordance with the modified AASHTO OMC method for eight groups of FA mixes (4 RAP contents X 2 RAP types). OMC results show that optimum moisture content and dry unit weight decrease as RAP content increases for two RAP materials

4. Design bitumen contents

FA mix significantly differs from hot-mixed asphalt material in two aspects: curing period and coating of the aggregates. In FA mix, the foamed bitumen disperses itself among the finer particles forming a mortar which binds the mix together; larger aggregates are partially coated by bitumen. Hence this mix is largely affected by moisture, and strength losses will occur when it get wet. On the other hand, FA mix specimen need to undergo an accelerated oven curing before being tested due to water added into the mix.

In this research, the indirect tensile strength test is used to select the design bitumen content. The optimum bitumen content is considered as the bitumen at which the soaked indirect tensile strength is maximum. This method simultaneously considers the moisture effect (the worse condition) on the FA mixture and resistance of this mixture to indirect tensile. All specimens compacted by Marshall compaction method are subjected to three-day curing in oven at 40 ^{0}C before any test is conducted.

The design bitumen contents of all FA mix groups are in the range of 2.5 to 3.5%. It is found that optimum bitumen content is close to 3.5% when RAP content is 0% and 20%, and close to 3.0% when RAP content is 40% and 60%. Therefore, no matter which RAP material (RAP#1 or RAP#2) is added, design bitumen content of the FA mix treated by the two bitumens is determined as 3.5% for the lower RAP content (0% and 20%), as 3.0% for the higher RAP content (40% and 60%).

5. Volumetric properties

Volumetric properties of all groups of FA mixes were evaluated in this section. It is clear that dry density and MTSG decrease with an increase of bitumen content no matter which RAP material is added or which bitumen is used to treat.

A second order polynomial relationship (concave curve) between air void and bitumen content is shown for FA mixes. When bitumen content is smaller, air void is higher. The air void decreases as bitumen content increases, and reaches the bottom at 2.5% to 3.5% bitumen content. Then air void increases with an increase of bitumen content.

Relationship between air void and bitumen reflects that there is a BC (approximate 2.5~3.5%) which can produce the lowest air void for FA mixes. This BC corresponds to the design bitumen content at which FA mix has the lowest air void and highest soaked ITS. Hence the design bitumen content can also be selected as the content at which the air void is the lowest.

6. Mechanics properties

Indirect tensile strength test indicates that the dry and soaked indirect tensile strengths of mixes are higher than 200 kPa and 100 kPa respectively. ITS test also reflects that FA mixes containing RAP#1 material have higher soaked ITS results than those mixes containing RAP#2 material. It is found that 60% of RAP content is disadvantage to ITS result no matter which bitumen is used to treat FA mixes.

There is a second order polynomial relationship (convex curve) between soaked ITS and bitumen content for all groups. These convex second polynomial curves of soaked ITS correspond to those concave second polynomial curves of air void, at 3.0% to 3.5% BC the soaked ITS reaches the maximum and air void reaches the minimum.

As well as indirect tensile strength, ITSMs of FA mixes were also tested. No matter which bitumen is used to treat, it can be found that ITSM decreases with an increase of RAP content for those mixes incorporating RAP#1 material. However, for mixes incorporating RAP#2 material, the minimum ITSM appears at 20% RAP content for treatment of the two bitumens.

CHATPER FIVE PROPERTIES OF PERMANENT DEFORMATION

5.1 INTRODUCTION

Flexible pavement distresses are numerous and varied. Of these distresses, permanent deformation, the accumulation of small deformations of pavement materials, is a type of densification and /or repeated shear deformations under applied wheel loads. This type of deformation is caused by consolidation, lateral movement, or both, of the pavement layer under traffic (Cooley et al 2000). In either case, permanent deformation appears as longitudinal depressions in the wheel paths of the roadway. This failure mode results in a loss of serviceability of the bituminous pavement, and can pose certain safety risks as well. This deformation is also commonly referred to as rutting.

In 1970's, investigation by AASHTO indicated that distress caused by permanent deformation in the all kinds of pavement distresses proportioned up to 30% (Majidzadeh et al 1979). In Japan, 80% of maintenance cost of main asphalt roads was used to repair, overlay or repave the pavement due to distress of the existing pavement surfacing caused by permanent deformation (Nievelt 1988, Dawley et al 1990).

With increases in axle loads, load repetitions, tire pressures, and asphalt concrete thickness, permanent deformation of the modern road is becoming a major problem for pavement management and design. A large amount of efforts are focused on the research in order to mitigate this type of bituminous pavement distress.

Rutting is usually used to describe the permanent deformation of the asphalt mixture. Asphalt Institute (1996) stated that a properly designed HMA mixture possessing a strong interlocking aggregate structure combined with a binder of adequate stiffness can significantly reduce a pavement's susceptibility of this type of deformation failure.

In general, permanent deformation of pavement can be mainly classified into two types: structural rutting and instability rutting. In the early study, structural rutting was regarded as the uppermost type of permanent deformation. Based on this idea, a number of studies included criteria for limiting values of subgrade strain to levels that precluded rutting at the pavement surface. Examples include the Shell model; the Asphalt Institute model; and the State of Kentucky methodology (Sousa and Weissman 1994). Some have recommended limitations on vertical subgrade stress (rather than strain), others have utilized statistically-based rut depth prediction equations. In the Asphalt Institute's rutting model, for example, rutting of pavement is mainly contributed by permanent deformation of the subgrade (FHWA-RD-98-012 1998). The failure criterion of permanent deformation is defined as pavement condition with a total rutting depth of 13 mm to 19 mm. However, there is a significant deviation between rutting model's prediction and measured value in field due to the assumption of main contribution of subgrade to permanent deformation of pavement structure.

Because of rutting model's limitations mentioned above, many researchers carried out further studies on the instability-rutting properties of asphalt mix layers. Results of AASHTO testing road indicated that the maximum rutting depth would occur at 10 inches under the pavement surface (Sousa et al 1991). This implies permanent deformation mainly occurs in the asphalt mix layer. The National Center for Asphalt Technology (NCAT) also developed an empirical rutting model (Brown and Cross 1992). This model shows that there is a good correlation between rutting model's prediction and measured value, it also indicates that rutting mainly occurs at 3~4 inches under the pavement surface. The air void of asphalt mixture is required to be close to 3% and not larger than 7% in order to mitigate early rutting depth of the pavement, or the pavement is prone to have early rutting.

So far researchers already agree that permanent deformation of pavement is contributed by densification and repeated shear deformation of asphalt mixture, and the latter is regarded as the main factor which usually causes the pavement rutting problem in the operation period (SHRP-A-415 1994).

Study on permanent deformation of bituminous pavement includes two aspects: rutting model, susceptibility of pavement material to permanent deformation and its factors. The rutting model can be established based on permanent deformation properties of each bituminous layer by using the layer-strain method or viscoelastic method (Sousa et al 1991). The former uses either linear or nonlinear elastic theory to analyze the pavement structure in predicting the rut depth. In viscoelastic closed form methodologies, moving wheel loads can be considered in conjunction with time-dependent material properties to define the state of stress and strain at particular points in the pavement structure.

Many factors can affect the shear resistance of a mix, especially the asphalt binder. Binder film thickness should be adequate for coating the aggregate and providing cohesion, but too much binder can actually have a lubricating effect, reducing the effectiveness of the aggregate skeleton and creating an unstable mix that is prone to premature rutting (Brosseaud et al 1993). Binder grade can also affect rutting performance. In general, the higher the binder grade, the stiffer the binder, and the greater the rutting resistance (Stuart and Izzo 1995, Netemeyer 1998).

Aggregate properties are important as well. In fact, it has been shown that the rutting resistance of a mix may be enhanced more by strengthening the aggregate gradation than by changing the grade or amount of binder.

Other volumetric properties of the asphalt mixture may also affect rutting, such as voids in the mineral aggregate (VMA) and voids filled with asphalt (VFA). By providing appropriate VMA, it is believed that rutting may be minimized, and mixture durability will be enhanced. But it is important to remember that these values are functions of other, more fundamental, properties of the asphalt mixture.

Compaction characteristics, characterized by the rate of densification during laboratory compaction, can also provide information relative to a pavement's resistance to permanent deformation. In theory, the greater the compaction effort required to compact a sample, the greater its shear resistance.

Permanent deformation of bituminous materials is a complex phenomenon that depends on the properties of the bitumen and of the aggregate structure. It is recognized that asphalt mixes have a rate and temperature dependent behavior. Hence temperature affects pavement rutting, especially in summer day. The effect of the moisture on rutting of asphalt mixture has caused researcher's attention recently. Many studies pointed out that when moisture went into the mixture the binder would be stripped from the aggregate under repeated traffic loading, serious rutting would occur under high temperature and high moisture conditions. Sousa (1994) and Bennett (1994) stated that moisture damage in some mixes might play an important role in development of permanent deformations and recommended that moisture should be considered as an important factor in establishing rutting model of asphalt mixture.

Though many factors are known to contribute to rutting, the precise relationships between aggregate/binder interactions and rutting characteristics are still unknown.

In this research, foamed asphalt mix incorporating with RAP is used as base course or road base material. Because rutting of pavement is a overall result, each bituminous mixture layer can contribute to permanent deformation. However, foamed asphalt mix is quite different from HMA in bitumen-aggregate cohesion. It is necessary to study the permanent deformation property of this kind of material. The objective aims at analyzing susceptibility of FA mixes to permanent deformation and effects of bitumen grade, RAP content and ageing of RAP on the susceptibility.

Research scope will be limited to laboratory tests. The permanent deformation of FA mixes and its effects will be presented in this chapter. Effect of moisture on permanent deformation will be described in Chapter 7. Rutting model of FA mix and permanent deformation of other bituminous material are not involved in this research.

Results presented in this chapter are meaningful to the application of RAP recycling. It can provide the foundation for establishing rutting model of pavement containing RAP materials.

5.2 EXPERIMENTAL AND EVALUATION METHODS

5.2.1 Experimental Methods

The above section is only a general introduction on the permanent deformation test and evaluation. In this section, the experimental and evaluation method of permanent deformation are extensively discussed.

The resistance of bituminous mixtures to permanent deformation is evaluated in research laboratories using a number of different types of equipment. This can be accomplished by two different tests: empirical and fundamental.

The characteristics of empirical test is to simulate the traffic loading on pavement by tracking the loaded wheel to and fro on the specimen whose condition is similar to pavement condition in order to obtain the rut depth under a specified load cycle.

Typical empirical testing equipments include the French Pavement Rutting Tester (PRT), the Hamburg Wheel-Tracking Device (WTD), Georgia Loaded Wheel Tester (Georgia LWT), the Asphalt Pavement Analyzer (APA), Nottingham’s Wheel Tracker (Stuart et al. 1999, Brown and Bell 1979, Kandhal. and Mallick 1999). Accelerated loading facility (ALF) is another kind of empirical equipment used in situ (Stuart et al. 1999). Shell’s Laboratory Test Track (LTT), different from equipments used in laboratory and in situ, is a type of accelerated pavement testing equipment (Lijzenga 1999).

Purpose of empirical tests by above equipments is to obtain rut depth to evaluate permanent deformation of pavement material in the laboratory or in field condition. The unique advantages of rut depth tested by empirical equipments are that the stress state applied to the sample is somewhat similar to that which occurs in the field, and it is relatively inexpensive and easy to operate. Its disadvantages are that the data cannot be used in mechanistic pavement analysis and cannot be used to determine the modulus of the mixture used by thickness design procedures.

These empirical testing methods are briefly introduced as follows:

The French PRT tests a slab for permanent deformation using a smooth, rubber tire inflated to 600 ± 30 KPa. Each slab has a length of 500 mm, a width of 180 mm, and a thickness of 50 or 100 mm. The applied load is 5000 ± 50 N and the test is performed at the temperature of interest. The maximum number of cycles is 30,000 (60,000 wheel passes).

The Hamburg WTD tests a slab of hot-mix asphalt submerged in hot water by rolling a steel wheel across its surface. The device simultaneously tests two slabs using two reciprocating solid steel wheels, each having a width of 47 mm. The applied load is 685 N and the average speed is 1.1 km/h. The rut depth in each slab is measured continuously by a linear variable differential transformer (LVDT). After each user-specified increment of wheel passes is applied, the device stores the maximum rut depth along the 200-mm wheel path relative to a rut depth of zero for the first wheel pass. It does not calculate an average rut depth. There are various criteria for adjudging pass/fail rut depth, and one of which is 10 mm at 20000 passes. The creep slope from this device, which is the number of wheel passes needed to create a 1-mm rut depth, is used to evaluate the mixtures for their resistance to rutting. The creep slope is a measure of rutting resistance before moisture starts to significantly damage the specimen. Even so, some moisture damage may be included in the creep slope. Higher creep slopes indicate more resistance to rutting. The customary test temperature for the Hamburg WTD is 50ºC

The Georgia LWT measures rutting susceptibility for three replicate beams by rolling a steel wheel across a pressurized hose positioned on top of each asphalt concrete beam at 40 ^{0}C. If the average rut depth for three replicate beams exceeds 7.6 mm, the mixture is considered by Georgia Department of Transportation to be susceptible to rutting.

The Asphalt Pavement Analyzer (APA) is a multifunctional loaded wheel tester used for evaluating permanent deformation, fatigue cracking and moisture susceptibility of both hot and cold asphalt mixes. A repeated load is applied to the test specimens in a dry or wet condition. A typical test runs 8,000 cycles and takes 2 hours and 15 minutes to finish. In this period of time the APA can predict how an asphalt mix will perform during its lifespan. The APA features an automated data acquisition system that obtains all rutting measurements and plots them in a graphical and numeric format.

The Nottingham Wheel tracker is capable of carrying out tests in accordance with British Standard BS598-Part110:1998 and the relevant proposed CEN standard. The distance of travel is 230 mm, rate of tracking is 42 passes/minute. The test is usually carried out at a temperature of either 45 ^{0}C or 60 ^{0}C. Specimen’s wheel-tracking rate and rut depth are measured by applying a load to the single rubber-wheel under standard test conditions, of 520 N. Test will stop until the wheel-tracking deformation reaches a depth of 15 mm or for 45 minutes.

The LTT is a piece of equipment at the Shell Research and Technology Center in Amsterdam. The circular track has an outer diameter of 3.25 m and a width 0.7 m. Pavement sections can be tested on permanent deformation with wheel loadings of 20 kN and a wheel velocity of 16 km/h. the track is temperature controlled to 60 ^{0}C.

The ALF is a full-scale pavement tester that applies 20 years of traffic loadings in less than 6 months. It applies a load, ranging from 43 to 100 kN, through a wheel assembly that models one-half of single truck axle. A dual tire or wide base (super single) tire can be used. It travels at a maximum speed of 18.5 km/h over a 10-m test section. Approximately 8600 wheel passes can be applied per day. To simulate highway traffic, the ALF loads the pavement in one direction, the load can be laterally distributed to simulate traffic wander. The FHWA has evaluated other laboratory tests used to predict the rutting and fatigue cracking susceptibilities of asphalt mixtures. The resistance to rutting is being predicted using wheel-tracking devices, a creep test and a repeated load test. The results from these tests will also be compared to ALF pavement performance.

The fundamental tests evaluate permanent deformation of bituminous materials by analyzing stress or strain of the specimen under a specified testing condition, such as loading rate, temperature, moisture, loading force, etc. The Superpave Shear Tester (SST), triaxial test, uniaxial test, including static creep test, dynamic creep test (repeated loading test (RLA)) are important fundamental tests (SHRP-A-379 1994, Carpenter and Vavrik 2001, BSI 1996, Kim ea al 1997). Unlike empirical tests, result of fundamental tests can be used in the mechanism analysis of pavement structure. These tests are briefly described as following:

The SST is a servo-hydraulic machine that can apply both axial and shear loads at constant temperatures to a cylindrical (150x150 mm) specimen using closed-loop control. The AASHTO TP7-94 that contains a detailed description of the SST test in the different modes of operation. The information obtained from the SST is used to compare data of proposed mixture with another mixture with known performance under the same conditions at identical temperatures. However, this equipment is very expensive.

In the RLA test, the specimen is subjected to repeated applications of axial stress. This is considered to be more simulative of traffic loading than static creep. There are two kinds of test methods: Australian test method and British method. The Australian method applies a square load pulse of 1 second duration at 450 KPa, a 1.5 second rest pulse follows. The test is run at 50 ^{0}C, and is unconfined. The accumulated strain is recorded after each load cycle. The output data used in this analysis are the A and B coefficients of the best fit line on the log plot of accumulated strain and load repetitions. The British method also applies the square load pulse of 1 second duration, however followed by a 1 second rest period. In the United Kingdom the standard test consists of 1800 load cycles with a maximum axial stress of 100 KPa at a temperature of 30 ^{0}C, the axial strain at the end of the test is taken as a measure of resistance to permanent deformation.

In the triaxial test, by appropriate control of the confining pressure and the axial load, virtually any combination of the hydrostatic (compressive) and deviatoric (axisymmetric) stresses can be induced in the specimen. Because of more expensive equipment, and more complicated testing technology, fewer triaxial than uniaxial compression tests on asphalt concrete have been reported in the literature. There are two kinds of triaxial tests: static and dynamic.

5.2.2 Evaluation Method

All tests of permanent deformation in laboratory and in field evaluate either rut depth by tracking simulation or accumulated axial strain by repeated axial load.

1. Wheel tracking method

There is an important concept, creep slope. In the following description, this concept will be used for strain, not for rut depth. Therefore the wheel tracking method and the creep slope of rutting are introduced firstly.

Rut depth is the most significant parameter for pavement performance (Brown and Gibb 1996). Characterization of performance in wheel tracking tests may be better represented by the mean deformation rate. This is certainly helpful when dealing with repeated load axial when a specimen goes to failure and no representative ultimate strain can be recorded.

Williams (2003) explored the relationships of HMA mixture properties to rutting susceptibility as measure by the utilization of Wheel Tracker. Two analyses involving regression procedures were performed in an attempt to develop mathematical relationships between laboratory measurements of rutting susceptibility and HMA mixture characteristics. The overall conclusion of this analysis was that while many factors play a role in the rutting characteristics of HMA samples, regression procedures were unable to determine valid mathematical relationships.

In the French PRT test, slope taken from log rut depth vs. log cycle plots can be compared. Rut-susceptible mixtures generally have higher slopes, but there is no French specification on the slope. It should be noted that the slopes from the French PRT might be based on cycles and not on passes (Stuart et al 1999).

For Hamburg WTD test, the number of wheel passes being used in the United States is 20000. The allowable rut depth is recommended as 4 mm for 10000 wheel passes, and 10 mm for 200000 wheel passes. However, the City of Hamburg uses a maximum allowable rut depth of 4 mm at 19200 wheel passes.

The post compaction consolidation, creep slope, stripping inflection point, and stripping slope, shown in Fig.5.1, can also be analyzed. The post-compaction consolidation is the deformation (mm) at 1000 wheel passes. It is called post-compaction consolidation because it is assumed that the wheel densifies the mixture within the first 1000 wheel passes.

The creep slope is used to measure rutting susceptibility. It is the inverse of the rate of deformation (wheel passes per 1-mm rut depth) in the linear region of the plot between the post-compaction consolidation and the stripping inflection point. Creep slopes have been used to evaluate rutting susceptibility instead of rut depths because the number of wheel passes at which moisture damage starts to affect performance varies widely from mixture to mixture. Furthermore, the rut depths often exceed the maximum measurable rut depth of 25 to 30 mm, even if there is no moisture damage.

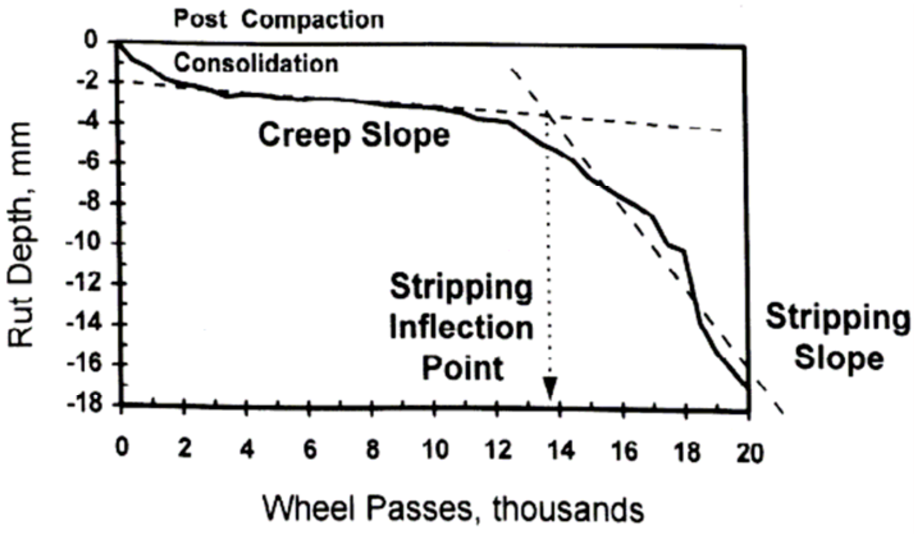


Fig.5. 1 Rut depth vs. number of wheel passes

The stripping inflection point and the stripping slope are used to measure moisture damage. The stripping inflection point is the number of wheel passes at the intersection of the creep slope and the stripping slope. This is the number of wheel passes at which moisture damage starts to dominate performance.

Overall, the French PRT at 60 ^{0}C, Georgia LWT at 40 ^{0}C, Hamburg WTD at 50 ^{0}C provided similar conclusions in the validation of asphalt binder and mixture tests that measure rutting susceptibility. Therefore, Stuart et al (1999) drew a conclusion that any one of these tests can be used to estimate rutting potential at high temperatures.

2. Fundamental experiment method

In the laboratory, SST, tracking equipments and repeated load axial test apparatus are often used to evaluate permanent deformation.

Sosnovske et al (1994) conducted a study using SST and French PRT. They drew a conclusion that rutting was directly proportional to void content, and mixes containing the gravel were more prone to rutting. It was found from the SST test that the dense-graded mixes outperformed the open graded mix at 40 ^{0}C in the rutting potential. There is a linear relationship between SST result and French PRT result.

In order to compare coarse-graded Superpave mixes with fine-graded Superpave mixes in terms of resistance to rutting, Kandhal and Cooley Jr. (2002) tested the susceptibilities of 14 mixes using APA, SST and repeated load confined creep test.

In order to evaluate the performance of recycled pavements in comparison to virgin (control) asphalt pavements, Kandhal et al (1995) tested creep modulus of the recycled mixtures by dynamic creep testing and found that recycled mixes had higher creep modulus than the control mixes. This implies that recycled mixes have higher resistance to permanent deformation.

The dynamic creep test is a repeated axial load test, however, the standard test condition vary between different countries, which makes direct comparison of results difficult. The Transport Research Laboratory (UK) conducted a comparative study between the repeated load axial test, indentation test, repeated load axial test, vacuum repeated load axial test and a wheel tracking test. The results showed that the vacuum repeated and the wheel tracking tests were highly correlated, with correlation coefficients of 0.99 (Stephenson and Bullen 2002)

Carpenter and Vavrik (2001) established a permanent deformation model using repeated load triaxial test. The relationship for the permanent strain, in micro-strains, accumulated at the onset of tertiary failure (in this stage the mixture is stripped and fail) is as following:

$$Strain = 78740.9 - 9474767 \times US + 17628564 \times CS - 390.78 \times LI - 174.944 \times UI + 539.156 \times CI - 307.44 \times C\text{mod} + 232.04 \times E\text{mod}$$

R^2=0.998 Std. Error=187.3 (5.1)

Where *Strain* is the accumulated permanent strain in micro-strains

US-Slope of the upper line,

CS-Slope of crossover point,

LI-Intercept of the lower line,

UI-Intercept of upper slope line,

CI-Intercept of crossover point,

Cmod-Compressive modulus,

Emod-Extension modulus.

The representation of stable behavior of permanent deformation is the log-log plot of permanent stain against load repetitions with a linear relation. The relationship developed for the slope is as following:

$$B = -1.945 - 49.34 \times LS - 0.01254 \times LI + 0.00339 \times UI - 0.0162 \times CI + 0.005723 \times C\text{mod} - 0.00234 \times E\text{mod}$$

R^2=0.888 (5.2)

Where *B*-slope of permanent strain,

LS-Slope of lower line,

Other parameters-see Eq. (5.1)

Kim et al (1997) studied the rate sensitivity of asphalt concrete in triaxial compression. The creep results show that small deviator-induced dilation that is nearly fully recoverable during the test period, and which, in some applications, can be neglected or approximated by an elastic response. The deviatoric response, on the other hand, is not recoverable, and can be modeled as a sum of elastic, plastic, viscoelastic, and viscoplastic strains, all being linear function of the deviator stress.

Many studies on moisture condition, curing condition, temperature susceptibility of foamed asphalt mix have been conducted. However, few studies on permanent deformation had been found in the literature. Bissada (1987) used static creep test to evaluate the creep deformation of foamed-asphalt-sand mixture, he found that this mixture was less susceptible than the corresponding hot-asphalt mixtures to permanent deformation.

3. Relationship between bitumen and mix's permanent deformation

Despite the extensive research on permanent deformation carried out within SHRP, the basic mechanisms which offer resistance to this mode of failure within a mixture are still not clearly understood. Hence many researchers carried out studies to find the relationship between rutting of asphalt mix and bitumen in recent years.

When mixtures are subjected to deformation upon application of a stress, the aggregates act as load-bearing entities while the bitumens deform in response to the applied stress. When mixtures with same aggregate type and gradations are used, it is therefore quite natural to expect a good relationship between the properties of the mixtures and those of the bitumens for their rutting behavior though most often, a good correlation is not observed based on literature search.

In a project of evaluating the properties of polymer modified bitumens and relationship to mix and pavement performance, Maccarrone et al (1995) found that polymer modified bitumens offer improved performance in respect to both rutting and fatigue cracking compared to conventional bitumen. This is to say that former bitumens are more rut resistant than the latter. However the ranking of the bitumen based on measurements of Superpave rutting-evaluation parameter is not entirely in agreement with results of wheel tracking tests, nor with results of dynamic creep tests.

Shenoy et al (2003) found that average rut depth after 6000 wheel passes of the French PRT had poor correlation with the $G^*/\sin\delta$ values at the frequency of 0.9 radians/s for the mastics (R^2=0.45) and was a little better for the bitumens (R^2=0.64). The creep slope from the Humburg WTD had no correlation with the $G^*/\sin\delta$ values at the frequency of 0.125 radians/s for the mastics (R^2=0.05) and poor correlation for the bitumens (R^2=0.42).

Christensen (1998) pointed out that hot-mix asphalt concrete was a composite, composed of crushed stone, sand, bitumen and perhaps other addictives, its mechanical behavior was much more complicated than that of bitumen alone.

Oliver and Tredrea (1998) also stated that the SHRP parameter for rutting resistance ($G^*/\sin\delta$) did not give an acceptable correlation for a data set which included both unmodified and polymer modified bitumens but did correlate well for unmodified bitumens alone.

4. Validation experiments for permanent deformation

White et al (2000) conducted a study of the actual Accelerated Pavement Tester (APT, a fixed apparatus similar to ALF's function) data to calibrate the creep rate model using a finite element method analysis. An additional analysis was made using the creep rate model calibrated for Purdue University's Wheel Tracker. It is found that at 5000 passes, APT rut depths for both high and low densities were 30% higher than those of the Purdue Wheel Tracker. However, APT rut depths at 5000 passes are about equal to Purdue Wheel Tracker rut depths at 20000 passes. This suggests there is a 3:1 scale shift between APT test results and Purdue Wheel Tracker results.

In the Shell Pavement Design Method (SPDM), the effect of the bituminous binder on permanent deformation is modeled via a relation between the stiffness of the asphalt mix and its bituminous binder under long loading time conditions. Lijzenga (1999) used Shell's laboratory test track to correlate stiffness relationship obtained in laboratory and in field. He pointed out that stiffness relationships derived from the SST and the unconfined dynamic creep test were found to be in line with the relations obtained from the accelerated pavement deformation tests in the laboratory test track, and with those obtained from the repeated load axial test. However, this result is limited to one type of dense-grade asphalt mix, and the approach needs to be confirmed for other type of mixes.

However, Brown and Gibb (1996) stated that various laboratory tests on small specimens of material (known as element tests), whether in compression, shear or under wheel tracking, had not been extensively validated against realistic rut development under full-scale or pilot-scale loaded wheels.

5.3 TESTING METHODOLOGY

5.3.1 Selection Of Test

From above literature review, it can be found that bituminous mixture is a time, temperature, and stress dependent material. This material is subject to repeated loading, and exhibits elastic, plastic, viscoelastic, and viscoplastic responses. The elastic properties do not contributed to permanent deformation and are therefore modeled by modulus of elasticity and Poisson's ratio. Plastic properties contribute to permanent deformation, which is cumulative under repeated loading.

On the theoretical side, the work of Hills relating to the development of permanent deformation under static creep loading provided a useful insight into the respective roles of bitumen and aggregate. However, in applying the static creep test to practice, Van der Loo had to introduce an empirical factor to allow for the real effects of repeated loading (Brown and Gibb 1996). Monismith and Tayabali (1988) clearly demonstrated the significant differences in results between compression tests carried out under static and under repeated load conditions. They also demonstrated the importance of good mix grading in providing long term deformation resistance

Under static loading there is an immediate strain but no further accumulation with time for the elastic materials. Repeated loading, however, do cause permanent strains to build up. It is for this reason that repeated loading is considered essential when measuring permanent deformation resistance of bituminous mixtures in order to properly asses the response of the aggregate structure (Brown and Gibb 1996).

All tests of permanent deformation in laboratory evaluate either rut depth by tracking simulation or accumulated axial strain by repeated axial load. However creep strain can be used in mechanistic analysis. Since the ultimate strain contains initial strain caused by densification, which is affected by different compaction, therefore it is imperfect to use only the ultimate axial strain to evaluate the permanent deformation for laboratory specimens. Susceptibility of asphalt mixture to permanent deformation can be evaluated using the information obtained from the linear portion of the dynamic creep curve in the log-log scale plot. Hence repeated load axial creep test is selected to evaluate the susceptibility of FA mix to permanent deformation in this research. The specimen preparation by using Marshall method for repeated load axial creep test is simple and convenient.

In this research, dynamic creep test under square shape load pulse was conducted for FA mixes in accordance with British Standard DD 226 (BSI 1996). A load cycle consists of a stress application of 1 second duration followed by a 1 second rest period. According to BS DD 226, a standard test consists of 1,800 load cycles with a maximum axial stress of 100 KPa at a test temperature of 30 ^{0}C.

5.3.2 Determination Of Testing Temperature

Generally, maximum temperature is usually considered to be the worst condition and is adopted for the dynamic creep test. In Hong Kong, the summer average maximum temperature is 30.7 ^{0}C, and the average maximum temperature of continual seven-day may reach 35 ^{0}C (HKSRA 2003). Calculated by SHRP's temperature formula (Eq. (5.3)) (SHRP-A-648. 1994), the maximum temperature of pavement surface is 61.44 ^{0}C。

$$T_{surf} = T_{air} - 0.00618Lat^2 + 0.2289Lat + 24.4 \tag{5.3}$$

where T_{surf} is the temperature of pavement surface (^{0}C), T_{air} is the air temperature (^{0}C), and *Lat* is the latitude (degree), Hong Kong's latitude is 22 degree.

FA mix is usually used as a base course covered with a hot surfacing asphalt mix. In view of the typical pavement structure of a freeway in Hong Kong, road base is located at about 30 cm (11.8 inches) under the surface. The temperature of this road base can be calculated by FHWA's equation (Eq. (5.4)) (SHRP-A-648. 1994):

$$T_d = T_{surf}(1 - 0.063d + 0.007d^2 - 0.004d^3) \tag{5.4}$$

where T_d is the temperature of layer located at d inches under pavement surface.

When the average maximum air temperature of continual seven-day in a year is 35 ^{0}C, the temperature of layer located at 11.8 inches under pavement surface is calculated as 34.41 ^{0}C.

Hence 35 ^{0}C is used as the temperature of dynamic creep test. The maximum axial stress is 100 KPa. The dynamic creep test consists of 1,800 load cycles, one cycle contains 1 second load duration and 1 second rest. Testing parameters of NAT 10 for dynamic creep test are not adjusted and in accordance with BSI DD 216. Test results are the axial strains under 1,800 load cycles which constitute the dynamic creep curve.

5.3.3 Experiment Consideration And Apparatus

In the evaluation of permanent deformation of FA mix, RAP content, RAP type and bitumen grade are considered as factors. Two RAP materials, RAP#1 and RAP#2, will be added in the FA mixes. As mentioned in Chapter 4, RAP#2 is more aged than RAP#1. Shell penetration-grade 60 and Shell penetration-grade 100 will be used as binders. In the testing, 4 RAP dosages, i.e. 0%, 20%, 40% and 60%, will be added in the mixes. Moisture effect on permanent deformation of FA mix will be studied; however, this study will be introduced in Chapter 7.

In this research, only one grading, WC 20, is considered. It is a modified gradation of the Hong Kong specification of bituminous mixture. Specimen preparation was introduced in last chapter. However the dimension stipulated by BSI DD 216 of the specimen for dynamic creep test cannot eliminate the end effect of the specimen.

As mentioned above, volumetric properties have significant effect on the permanent deformation of HMA mixes. It is necessary to investigate effect of air voids on permanent deformation of FA mixes.

Since air void of cold-recycled FA mix is usually larger than 10%, as introduced in literatures, and percentage of water absorbed is more than 2%, the saturated surface-dry method (ASTM D 2726) and paraffin-coated method (ASTM D 1188) are not suitable for determining the air void of FA mix.

In the tests, the moist bulk density of FA mix and the moisture content of specimen during compaction will be tested in accordance with ASTM D 3203 (1994) and ASTM D1557 (2000) respectively. Hence the dry bulk density of mix can be determined using these results. The maximum theoretical relative density (MTRD) of FA mix will be tested in accordance with ASTM D2401 (1994). All materials should be dried before MTRD is tested because FA mixture contained moisture after compacted. Finally air void can be calculated using dry bulk density and MTRD.

Two faces of each specimen will be coated with a thin layer of silicone grease coating in turn with graphite flakes or powder in order to eliminate the influence of unevenness of specimen face on the test result. Before being tested by applying 100 KPa axial stress, the specimen should be pre-loaded with a conditioning load equivalent to a stress of 10 KPa for 600 seconds

In order to compare deformation properties of FA mix with those of hot mix, two hot mixes (AC-20 and PA-10) were selected, dynamic creep test was also carried out for these two hot mixes.

5.3.4 Experiment Apparatus

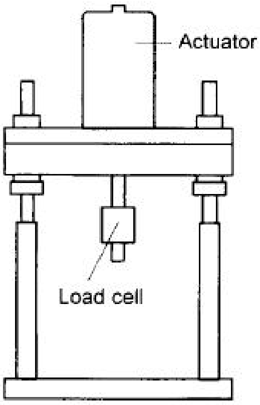
The repeated load axial test is conducted by the Nottingham Asphalt Tester (NAT-10), as shown in Fig. 5.2, in this research. The NAT-10 is a pneumatic loading device developed at the University of Nottingham, U.K., and manufactured by Cooper Research Technology Ltd. It is capable of carrying the ITSM test, four-point-bending-beam fatigue test, indirect tensile fatigue test, indentation test, static creep test, as well as repeated load axial test. The repeated load axial test is performed using the NAT-10 according to BS DD 226 (BSI 1996).

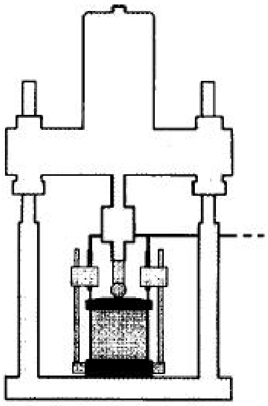
The NAT-10 is composed of temperature control cabinet, testing frame, loading control sub-system, data collection sub-system, computer and interface sub-system. Fig.5.2 (a) illustrates this testing facility. Fig.5.2 (b) illustrates testing frame and repeated load axial test sub-system.

Different from the Australian method which applies a square load pulse of 1 second duration at 450 kPa followed a 1.5 second rest period, the NAT applies the square load pulse of 1 second duration at 100 kPa followed by a 1 second rest period. All specimens are subjected to 1800 load cycles at 35 ^{0}C, and are unconfined. The accumulated strain is recorded after each load cycle.

Fig.5.3 illustrates the pre-coated sample and testing process.

(a) The Nottingham Asphalt Tester





(b) Test frame (c) Repeated load axial testing sub-system

Fig.5. 2 The Nottingham Asphalt Tester and its sub-system

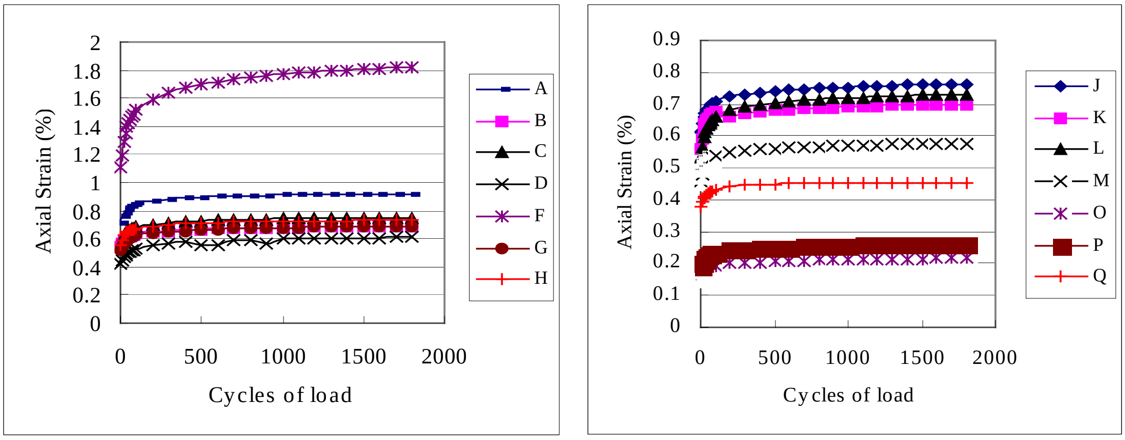
(a) Pre-coated sample (b) Testing process

Fig.5. 3 Sample and testing process

5.4 RESULTS AND ANALYSIS

5.4.1 Dynamic Creep Curve And Ultimate Axial Strain

After the tests, dynamic creep curves of all groups were obtained and depicted in Fig. 5.4(a) and Fig. 5.4(b). It can be found that there are significant differences among these curves. Each dynamic creep curve consists of two parts, one is the curve segment reflecting the densification of mixture, and the other is the line segment exhibiting the stable-developing axial strain. Due to a relative small applied axial stress and a short loading period of 1,800 cycles, failure of specimen did not occur in the test.



(a) Shell 60 (b) Shell 100

Fig.5. 4 Results of dynamic creep curves

The ultimate strains of all groups after 1,800 load cycles are listed in Table 5.1. Fig. 5.5 demonstrates mean columns and error bars (I-shapes in the plot, half I-shape represents the standard deviation) of these strains. It is apparent that variance of each group's ultimate strains is not large. The results exhibit that except the ultimate strains of Group O, P and Q, all other ultimate strains roughly decrease with an increase of RAP content. This implies that the increase of RAP content may improve the susceptibility of permanent deformation of FA mixes.

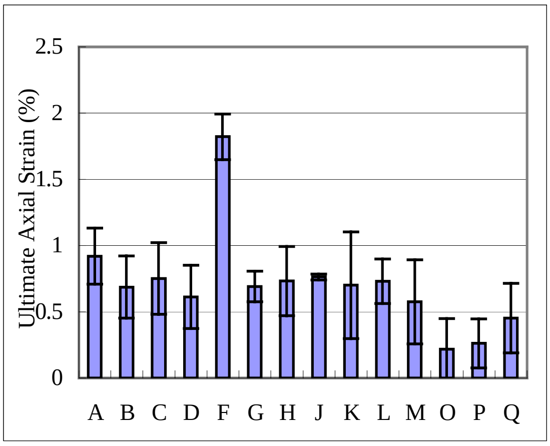


Fig.5. 5 Ultimate strains of all groups

Table 5. 1 Summary of test results of all groups and hot asphalt mixes

Mix		Ultimate Strain (%)	CSS	Intercept (Microstrain)	SCSM (MPa)	Air Void (%)
FA Groups	A	0.920	0.162	8917.5	604.3	12.8
	B	0.687	0.147	6623.5	627.8	11.7
	C	0.752	0.170	7231.3	486.1	11.9
	D	0.611	0.259	5520.2	262.0	11.7
	F	1.822	0.130	13088.3	126.1	12.3
	G	0.690	0.189	6575.1	431.4	10.8
	H	0.732	0.138	7088.8	628.1	11.1
	J	0.763	0.149	7375.8	558.6	12.9
	K	0.700	0.132	6681.7	666.1	12.2
	L	0.730	0.161	7023.8	551.0	12.9
	M	0.576	0.094	5594.2	1136.6	11.6
	O	0.217	0.068	2052.2	1335.8	12.5
	P	0.261	0.065	2499.8	1052.3	12.9
	Q	0.452	0.080	4527.1	912.1	12.7
Hot Asphalt Mixes	AC-20	0.336	0.297	2870.3	302.0	7.0
	PA-10	0.920	0.654	8089.4	128.5	17.5

5.4.2 Creep Strain Slope (CSS) And Intercept

If axial strains (in microstrain) under 1,800 load cycles are depicted in the plot of log strain versus log load cycles, all strains still exhibit a curve relationship with load cycles. Although this curve can be linearly fitted, the relationship coefficient is small (see Fig. 5.6 and Eq. 5.5).

After 500 cycles, a linear relationship for all groups between axial strains and load cycles can be found. Furthermore, the strains caused by the applied load after 600cycles versus load cycles demonstrates a linear relationship with a high relationship coefficient (see Eq. 5.6). Hence the last two thirds of dynamic creep curve in the log-log plot can be used to evaluate the development of permanent deformation of FA mixes.

$$y^{3/3}=0.2439x+2068.1 \quad (R^2=0.6155) \qquad (5.5)$$

$$y^{2/3}=0.069x+2287.5 \quad (R^2=0.9357) \qquad (5.6)$$

Where $y^{3/3}$ denotes strain under the load cycles ranging from 0 to 1800, $y^{2/3}$ denotes strain under the load cycles ranging from 600 to 1800 (log scale), x denotes load cycles (log scale).

Fig. 5.6 demonstrates results of the two fittings. The slope of the fitted linear equation of the two thirds of dynamic creep curve in the log-log plot, namely creep strain slope (CSS), reflects the trend of axial strain. Lager CSS indicates less resistance of FA mixes to permanent deformation. CSS can be calculated by the following equation:

$$CSS = \frac{\log \varepsilon_{1800} - \log \varepsilon_{600}}{\log 1800 - \log 600} \quad (5.7)$$

where ε_{1800} denotes the strain at 1800th cycle, and ε_{600} denotes the strain at 600th cycle.

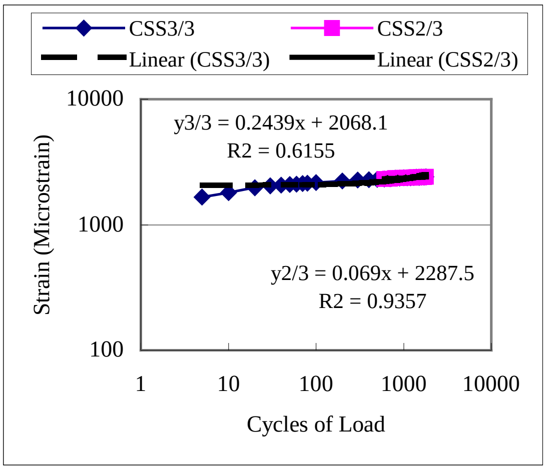


Fig.5. 6 CSS and Intercept of two fittings

The Intercept of the fitted linear equation denotes roughly the initial axial strain of FA mixes, which reflects the permanent deformation in the densification stage. The more Intercept is, the larger the initial permanent deformation is.

The initial permanent deformation of specimen compacted in laboratory, which is not caused by load cycles, is often affected by compaction method. However CSS excludes the initial permanent deformation. Therefore CSS can be used to characterize permanent deformation susceptibility of FA mixes under load repetition.

CSSs of all groups are summarized in Table 5.1. It assumes that for each bitumen, the results of mixes with 0 percent of RAP#1 and 0 percent of RAP#2 are identical. Fig. 5.7 demonstrates the column chart with error bar of CSSs versus RAP content. Error bars of all groups indicate that there is high deviation between replicate specimens of each group.

The figure also shows that all CSSs of groups stabilized by Shell 100 are smaller than those of groups stabilized by Shell 60. This means that softer bitumen helps the FA mixes to lower their susceptibility to permanent deformation. This result is different from the hot-mixed asphalt concrete. CSSs of FA mixes containing Shell 100 and RAP#2 (Group O, P and Q) are smallest among all the groups. It implies that ageing affects the CSS of FA mix stabilized by Shell 100. More aged RAP material added would result in less susceptibility of FA mix to permanent deformation.

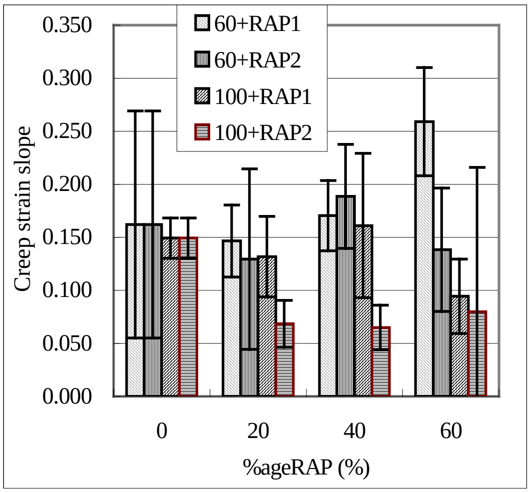


Fig.5. 7 CSSs of all groups vs. RAP content

One way analysis of variance (ANOVA) was conducted to analyze the effect of RAP content on CSS. In the ANOVA, mean comparisons of CSSs for each pair of RAP contents were tested using Tukey's method (Minitab Inc 2000); the individual error was set as 5 percent.

The results are summarized in Table 5.2. For each pair of FA groups, if the confidence interval of their mean difference excludes zero, means of these two FA groups are significantly different, otherwise means of FA groups are insignificantly different. For example, the first pair of numbers in Table 5.2, (-0.152, 0.183), gives 95% confidence interval for the mean of CSS of the mix (Group A) with 0 percent of RAP#1 stabilized by Shell 60 minus the mean of CSS of the mix (Group B) with 20 percent of RAP#1 stabilized by the same bitumen. This confidence interval includes zero, therefore CSSs of two groups are insignificantly different.

From Table 5.2, it can be found that all 95% confidence intervals of CSS differences include zero. This result indicates that there is insignificant difference in each pair of CSSs, and it means that RAP content does not significantly affect susceptibility of FA mixes to permanent deformation. This may be due to a large variance of CSS.

Table 5. 2 Mean comparison of CSSs for all groups at each pair of RAP contents

	60_RAP#1			60_RAP#2			100_RAP#1			100_RAP#2		
	0	20	40	0	20	40	0	20	40	0	20	40
20	-0.152 0.183			-1.256 0.569			-0.097 0.132			-0.103 0.264		
40	-0.176 0.159	-0.191 0.144		-0.939 0.886	-0.595 1.229		-0.126 0.102	-0.144 0.085		-0.100 0.268	-0.180 0.187	
60	-0.264 0.070	-0.280 0.055	-0.256 0.079	-0.888 0.936	-0.545 1.279	-0.862 0.962	-0.059 0.169	-0.077 0.152	-0.048 0.181	-0.036 0.331	-0.117 0.251	-0.120 0.247

Multiple analysis of variance (MANOVA) was conducted to test the effects of all factors, including RAP content, bitumen grade, ageing of RAP, and their interaction, on the CSSs with confidence probability of 95%, and α =0.05.

MANOVA output is listed in Table 5.3. The result indicates that only bitumen has significant effect on CSSs, because probability of F-test of bitumen grade, P value, is 0.038 (less than α =0.05). This is consistent with the above finding (see Fig. 5.7 and its explanation). RAP content and ageing of RAP dose not significantly affect CSSs.

Table 5. 3 MANOVA of CSSs

Source	DF	SeqSS	Adj SS	Adj Ms	F	P
%RAP	3	0.05238	0.05238	0.01746	0.53	0.667
RAP	1	0.0000	0.0000	0.0000	0.0000	0.993
Bitumen	1	0.15597	0.15597	0.15597	4.71	0.038
%RAP*RAP	3	0.10437	0.10437	0.03479	1.05	0.384
%RAP*Bitumen	3	0.07908	0.07908	0.02636	0.8	0.506
RAP*Bitumen	1	0.04848	0.04848	0.04848	1.46	0.235
%RAP*RAP*Bitumen	3	0.0956	0.0956	0.03187	0.96	0.423
Error	32	1.06063	1.06063	0.03314		
Total	47	1.59651				

Note: * denotes interaction.

Intercepts of the fitted linear equations of all groups are listed in Table 5.1. Fig. 5.8 is the scatter chart of all Intercepts. It demonstrates a poor correlation between Intercept and RAP content. Although the relationship coefficient is small, this relationship exhibits that Intercept decreases with an increase of RAP content. It can be inferred that the initial axial strain decreases when RAP content increases, i.e. increasing of RAP content helps to reduce permanent deformation.

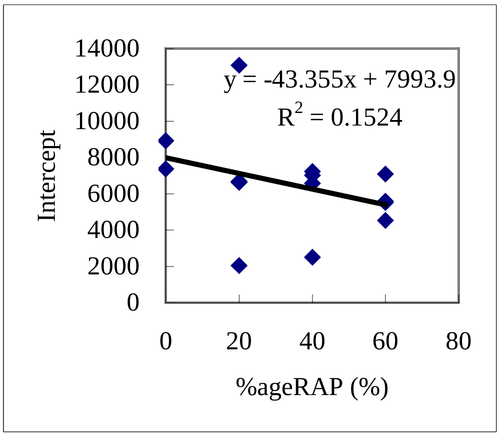


Fig.5. 8 Intercept versus RAP content

5.4.3. Secant Creep Stiffness Modulus (SCSM)

Dynamic creep stiffness modulus is the ratio of the applied stress to the total axial strain caused to the specimen after specific load cycles at testing temperature. Disadvantage of this modulus is that it is a transient modulus, and in its calculation the total axial strain contains the initial strain caused by densification. Hence dynamic creep stiffness modulus cannot reflect the susceptibility of the permanent deformation.

For the purpose of characterizing the permanent deformation in stable developing stage, the information in the last two thirds of dynamic creep curve is used to calculate the secant creep stiffness modulus. SCSM can be calculated by Eq. (5.8):

$$S_{\sec(35^0C)} = \frac{\sigma_0}{\varepsilon_{(35^0C,1800)} - \varepsilon_{(35^0C,600)}} \qquad (5.8)$$

where $\varepsilon_{(35^0\mathrm{C},600)}$, $\varepsilon_{(35^0\mathrm{C},1800)}$ are axial strains at 35 ^{0}C caused to the specimen after 600 load cycles and 1,800 load cycles respectively, $S_{\sec(35^0C)}$ is SCSM at 35 ^{0}C, σ_0 is the applied stress, in kPa. In this research, σ_0 =100 kPa.

SCSM is obtained from the stable developing stage of axial strain; it does not contain the information of initial axial strain caused by densification. Hence it significantly reflects the susceptibility of FA mixes to permanent deformation.

SCSMs of all groups are also listed in Table 5.1. These SCSMs range from 126 to 1,335 MPa. Results of SCSMs and CSSs are depicted in Fig. 5.9. There is a good correlation between these two parameters. The relationship coefficient of the fitted curve, R^2, is 0.70. It is apparent that CSS deceases with an increase of SCSM. So if the secant creep stiffness of FA mix is being enhanced, susceptibility of FA mixes to permanent deformation will drop as the consequence. Thus CSS and SCSM are considered to be useful parameters for evaluating susceptibility of FA mixes to permanent deformation.

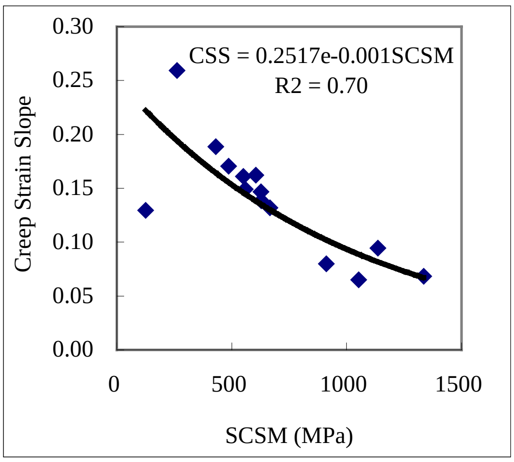


Fig.5. 9 Secant Creep Stiffness Modulus vs. CSS

5.4.4. Air Voids

Air voids of FA mixes are listed in Table 5.1. They are in the range of 10.4 to 13.6% with the mean of 12.1%, and the standard deviation of 0.8%. Although FA mix is designed as dense gradation, the air voids of all groups are larger than those of conventional hot asphalt mix.

Being different from the variance of CSS, air-void's variance of FA specimens compacted by Marshall Method in laboratory is small. Fig. 5.10 demonstrates the results of CSSs versus air voids, and it shows that there is no correlation between CSS and air void. This is to say air void does not significantly affect susceptibility of FA mixes to permanent deformation. This result is quite different from that of hot asphalt mix.

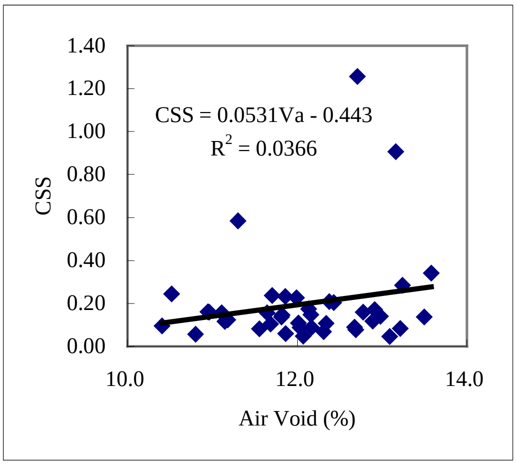


Fig.5. 10 CSS vs. Air Void

5.4.5 Comparison Of FA Mixes And Hot Asphalt Mixes

In this research, comparing the difference of permanent deformation between FA mixes and hot asphalt mixes was conducted. Two types of hot asphalt mixes were selected. One is AC-20, which is a conventional hot asphalt mix having the same normal maximum size as FA mixes. The design air void of AC-20 is 7%. In China, AC-20 has two types, one is a dense grading with the air void of 4~6%, the other is open grading with air void of 6~10%. The AC 20 selected here belong the latter type. The other hot mix is PA-10, which is a porous wearing course modified by SBS. The design air void of PA-10 is 17.5%.

CSSs, SCSMs, Intercepts, Ultimate Strains, air voids of FA mixes and the two hot asphalt mixes are also listed in Table 5.1. Fig. 5.11 demonstrates the mean columns and error bars of all FA groups and hot asphalt mixes. It exhibits that variance of SCSMs of all FA groups are significant and larger than those of hot asphalt mixes. Except SCSM of Group F is less than those of AC-20 and PA-10, and SCSM of Group D is slightly less than that of AC-20, all other SCSMs of FA mixes are larger than those of AC-20 and PA-10. These results reflect that FA mixes have smaller trend of accumulated axial strain than the two hot asphalt mixes, and almost all of creep moduli of FA mixes are higher than those of AC-20 and PA-10. It can be seen that all CSSs of FA mixes are smaller than those of AC-20 and PA-10.

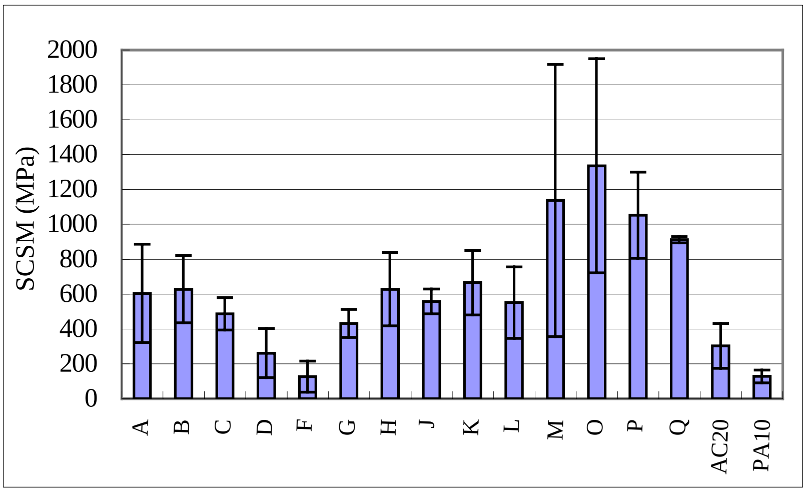


Fig.5. 11 Comparison of SCSM between FA mixes and hot asphalt mixes

For initial axial strain, almost all of Intercepts of FA mixes are larger than that of AC-20; however, almost all Intercepts of FA mixes are smaller than Intercept of PA-10. This result may be duo to air void of FA mixes lies at between air void values of AC-20 and PA-10.

From the above analysis, it can be drawn that susceptibility-resistance of FA mixes is better than that of AC-20 and PA-10 even though FA mixes has a larger air void than AC-20. This may due to the difference of bitumen-aggregate adhesion between cold mix and hot mix. Stiffness of FA mix is mainly formed by interlocking force, while strength of hot mix is formed both by cohesion and interlocking force.

5.5 CONCLUSTION

In this chapter, literature review of experimental methods for evaluating permanent deformation was conducted. Dynamic creep test was selected to evaluate characteristics of permanent deformation for foamed asphalt mixes.

In order to analyze the effects of bitumen grade, RAP content and ageing of RAP, two bitumen types and two RAP materials with different ageing were selected for preparing specimens subjected to the dynamic creep test. All tests were conducted using the Nottingham Asphalt Tester. Testing results were analyzed through creep strain slope, Intercept and secant creep stiffness modulus. Mean comparison and MANOVA were performed to analyze factors' effects on the CSSs. Comparison of permanent deformation between FA mixes and hot asphalt mixes was carried out. Conclusion has been drawn as follows:

(1) No failure of FA mixes occurs in the dynamic creep test after 1,800 load cycles with an axial stress of 100 KPa at 35 ^{0}C. All dynamic creep curves contain two segments: densification segment and stable developing segment.

(2) Three parameters, CSS, Intercept and SCSM are used to evaluate susceptibility of FA mixes to permanent deformation. CSS embodies the trend of permanent deformation in the stable developing stage, i.e. CSS reflects the resistance to permanent deformation. SCSM represents stiffness of FA mixes in the stable developing stage. Intercept reflects the initial strain. These three parameters can effectively characterize permanent deformation of FA mixes.

(3) Mean comparison using Tukey's method reveals that RAP content has no significant effect on CSS. The result of MANOVA exhibits that there is a significant effect of bitumen grade on CSS; CSSs of FA mixes stabilized by Shell 100 are smaller than those of mixes stabilized by Shell 60. Therefore, large-penetration bitumen will help FA mixes to reduce their susceptibility to permanent deformation. MANOVA also confirms that ageing of RAP and RAP content insignificantly affect susceptibility of FA mixes to permanent deformation.

(4) There is a good correlation between CSS and SCSM. CSS decreases when SCSM increases, i.e. FA mix with high creep strength will have low susceptibility to permanent deformation.

(5) CSSs, Intercepts, SCSMs of FA mixes exhibit high variance except for air voids and ultimate axial strains. It is difficult to establish correlation between CSSs and air voids.

(6) Comparison of test results between FA mixes and hot asphalt mixes confirms that susceptibility and creep strength of FA mixes are better than those of AC-20 and PA-10. Initial axial strains of FA mixes are between those of AC-20 and PA-10.

CHAPTER SIX FATIGUE PROPERTIES

6.1 INTRODUCTION

Failures in bituminous asphalt pavements can usually be classified as (a) rutting, a stability problem; (b) progressive cracking, a fatigue problem; or (c) fracture, a strength problem. Pavement distress resulting from repeated bending or fatigue of asphalt pavements has been a well-recognized problem since 1948 (Matthews et al 1993). Since 1960's, extensive efforts have been made to study the fatigue performance of the hot-mixed bituminous mixtures (Pell 1962, Adedimila. and Kennedy1976).

Fatigue is a damage generated by repetition of a big number of loading with small amplitude. Fatigue life is the number of cycles respecting a given fatigue criterion. Each vehicle on a road causes micro damage leading to a loss of rigidity. For the hot-mixes asphalt mixture, with few hundred of cycles and small strains, behavior can be regarded as linear viscoelastic. But with several tens of thousands of cycles and small strains, damage phenomena appear and the material shows "**FATIGUE**".

It is possible to establish the fatigue equation of small asphalt mixture specimens by using any of the many test methods available. However, fatigue is probably the most difficult to deal with from a design viewpoint. Fatigue life of asphalt pavement can be estimated by fatigue line which is obtained from the laboratory test. But fatigue line must be given suitable field calibration to account for pulse duration differences, rest periods between loads, thermal effects, and structural interactions that cannot be duplicated efficiently in the laboratory. The shift factor of calibration has varied from 6 to 70 and even higher and reflects differences between field and laboratory test conditions (Carpenter et al 2003). Fatigue data of this nature produce the coefficients in many of the fatigue models used in thickness design.

FA mix is often used as the roadbase layer covered with a hot-mixed asphalt material. Some engineers think that bitumen as a stabilizer can reduce the stiffness and increase the flexibility of the bound layer. Theyse (1999) pointed out that the foaming-treated material does not crack due to shrinkage as does cement treated material.

Some preliminary fatigue beam testing has already been carried out on foamed bitumen stabilized material. Ramanujam and Jones (2000) found that the fatigue potential of foamed bitumen appeared somewhat greater than hot-mixes asphalt, the variability, however, of the test results was very high. Chiu et al (2000) pointed out that the FA mixes exhibited much higher stiffness than emulsion treated material in the ambient temperature; the strains of the former were 2 to 3 times higher than those of the latter before failure. Little et al (1983) found that fatigue potential of the FA mix was well below that of the hot-mixed asphalt or the good-quality emulsion-stabilized bases; FA mix was also substantially below the line of low fatigue potential.

However, the fatigue properties of the FA mix are not extensively studied and cannot be accurately determined. This may be due to the poor adhesion of the FA mix. In order to characterize fatigue in mixture and pavement design procedures, it is necessary to describe the behavior of FA mix under repeated stressing of the tyre encountered in situ.

This chapter presents results on the fatigue study of FA mixes, with the WC 20 gradation as described in Chapter 4. The main objectives of this study are as follows:

- Establish the fatigue equation and discuss fatigue laws,
- Compare difference of the fatigue lives between FA mixes and the hot-rolled asphalt mixes,
- Analyze deformation and fatigue failure characteristics,
- Determine effects of bitumen grade, RAP type and RAP content on the fatigue properties,
- Analyze the nonlinear characteristics of strain and stress relationship.

The study on the fatigue properties of FA mixes is limited to the efforts in the laboratory. In this chapter, the research approach is briefly reviewed followed by fatigue tests, results, and analysis.

6.2 RESEARCH METHODOLOGIES

6.2.1 Factors Affecting Fatigue Response

Included herein is a brief summary of available information on factors affecting the fatigue response of asphalt mixes that are comprised of bitumen and aggregates.

1. Specimen fabrication

The primary objective of specimen fabrication or compaction is to produce "realistic" test specimens that reasonably duplicate the corresponding in-situ mixture paving in all major respects including composition, density, and engineering properties. For HMA, the specific ranking of compaction devices in terms of their abilities to consistently simulate the engineering properties of field cores is as follows (Von Quintus et al 1991):

i. Gyratory-shear compactor
ii. California kneading compactor or Mobile steel wheel simulator
iii. Arizona vibratory/kneading compactor
iv. Marshall hammer

Michaut and Bilal (2002) established a relationship between the fatigue deformation value and mix compaction, bitumen content. Hartman et al (2001) investigated the effect of different laboratory compaction procedures (namely, roller, Marshall, vibrating hammer, and gyratory techniques) on the indirect stiffness and fatigue properties of the two standard Irish bituminous mixtures (namely, hot-rolled asphalt (HRA) and dense base-coarse macadam (DBM)). The roller compaction method produced specimens of lower stiffness, similar to site compacted samples. The influence of the compaction method on the fatigue strength of asphalt mixes would appear to be mixture dependent; mixes with grading profiles that are designed for aggregate interlock were found to have higher fatigue strength.

2. Mode of loading

In laboratory tests, fatigue response has been shown to be a function of mode of loading. In the loading conditions of the controlled-stress mode, the load or stress amplitude remains constant during testing; of the controlled-strain mode, the deformation or strain amplitude is maintained constant. A brief summary of the two loading modes is presented in Table 6.1 (SHRP-A-404).

Table 6. 1 Comparative evaluation of controlled-stress and controlled-strain loading

Variables	Controlled-stress (Load)	Controlled-Strain (Deflection)
Thickness of asphalt mixture layer	Comparatively thick asphalt bound layers	Thin asphalt-bound layer, < 3 inches
Definition of failure; cycles	Well-defined since specimen fractures	Arbitrary in the sense that the test is discontinued when the load level has been reduced to some proportion of its initial value; for example to 50 percent of the initial level
Scatter in fatigue test data	Less scatter	More scatter
Required number of specimens	Smaller	Larger
Simulation of long-term influences	Long-term influences such as aging lead to increased stiffness and presumably increased fatigue life	Long-term influences leading to stiffness increase will lead to reduced fatigue life
Magnitude of fatigue life, N	Generally shorter life	Generally longer life
Effect of mixture variables	More sensitive	Less sensitive
Rate of energy dissipation	Faster	Slower
Rate of crack propagation	Faster than occurs in situ	More representative of in-situ condition
Beneficial effects of rest periods	Greater beneficial effect	Lesser beneficial effect

The fatigue modulus is particularly important for thin asphalt pavements over unbound base material. In this situation, the relatively high modulus of the asphalt may have very little impact on the overall pavement stiffness, or on the level tensile strain in the asphalt. The asphalt layer functions essentially in strain control mode, rather than stress control mode, and may be subject to a high strain level. In Australia, the controlled strain method is generally used since the majority of asphalt surfacing layers are relatively thin (Stephenson and Bullen 2002).

Depending on temperature, the results of fatigue tests may be quite different (see Fig. 6.1). Test results may also lead to different mixture designs: accordingly, attempts have been made to determine what mode of loading best simulates actual pavement conditions (Rao Tangella et al 1990). A frequent question in mix evaluation is whether the mode-of-loading selected for the laboratory testing will influence results of the mix evaluation process. Tayebali et al (1994) found that stiffer mix generally had greater fatigue resistance at a given stress level in controlled-stress testing, had lesser fatigue resistance at a given strain level in controlled-strain testing. In general, fatigue lives under controlled-strain loading were approximately 2.4 times greater than those under controlled-stress loading. When test results were interpreted in terms of the performance expected of the pavements in which they were placed, it appeared that controlled-stress and controlled-strain testing might yield similar mix rankings provided they yielded comparable estimates of flexural stiffness.

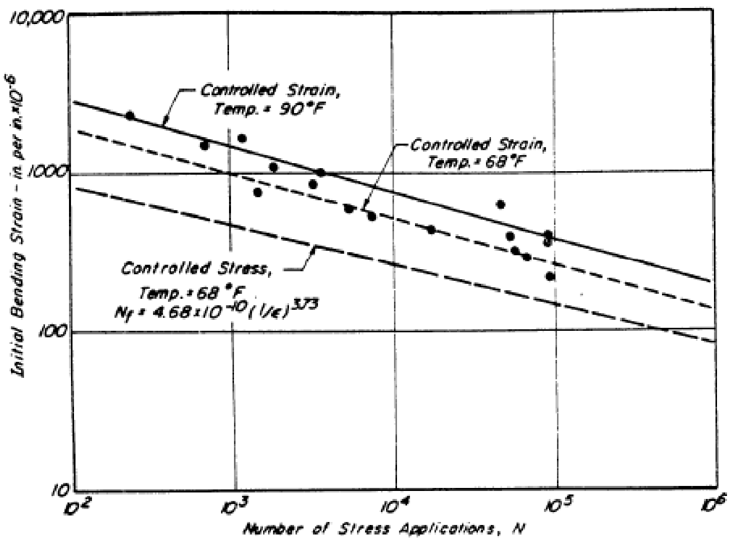


Fig.6. 1 Comparison of laboratory controlled-strain and controlled-stress fatigue data (Monismith et al 1977)

3. Mixture variables

In general, for continuously graded mixes, the two primary factors affecting fatigue response are bitumen content and air void content. Aggregate type seems to have less influence. Quantitatively, the effect of bitumen content and void content on the fatigue life of asphalt mixtures can be ascertained by a correction factor proportional to (Pell and Cooper 1975):

$$\frac{V_B}{(V_B + V_V)} \qquad (6.1)$$

where V_V is the air void volume (percent) and V_B is the bitumen volume (percent) and

$$V_B = \frac{[P_{asp} \cdot G_{agg} \cdot (1 - V_V)]}{[100 \cdot G_{asp} + P_{asp} \cdot G_{agg}]} \qquad (6.2)$$

where P_{asp} is the percent by weight of asphalt mix(aggregate basis), G_{asp} is the specific gravity of asphalt mix, and G_{agg} is the specific gravity of the aggregate.

Boussad et al (1996) found that mix viscoelastic behavior in the linear domain was governed by the bitumen, nevertheless, aggregates gave a fingerprint of prime important through slope and intercept of the log-log fatigue line. For heavy-duty pavements (with thick bitumen-bound layers), a mix of high stiffness should be utilized by incorporating a stiff bitumen--it may be necessary to temper this requirement where thermal stresses can lead to cracking. For light-duty pavements (with thin bitumen-bound layers), the mixture should be made as flexible as possible, with lower stiffness bitumen and more open gradations. Alternatively, mixes containing a gap grading appear to produce better fatigue response than the continuously graded mixes normally used in the United States (Freeme and Marais 1973).

4. Loading and environmental variable

Loading and environmental variables have both direct and indirect implications. Direct implications include the shape and duration of the load pulse used in the laboratory and the test temperature. Fig. 6.2 shows loading patterns generally used in the laboratory. Raithby and Ramshaw (1972) found that fatigue lives under cyclic square wave and cyclic triangle wave were 0.42 and 1.45 times that under sinusoidal wave (see Table 6.2).

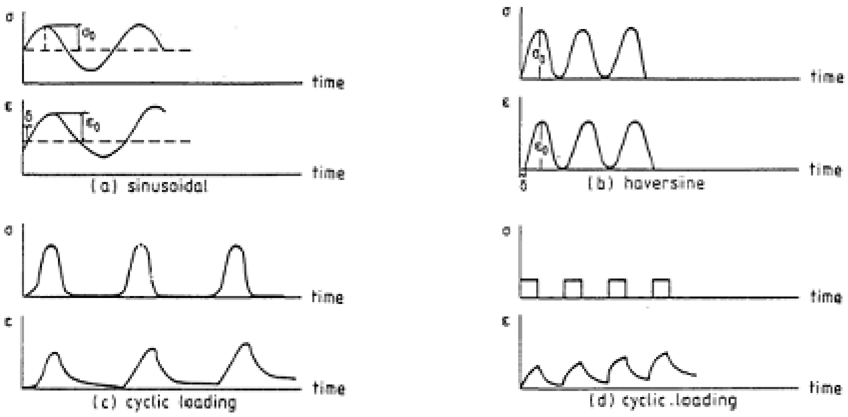
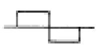
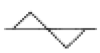


Fig.6. 2 Types of loading patterns (SHRP 1994)

Table 6. 2 Effect of shape of waveform on fatigue life (Raithby and Ramshaw 1972)

Waveform	Temp, ^{0}C	Stress Amp MN/m^2	Initial Strain Amp*	Geometric Mean Fatigue Life, Cycles	Relative Lives
	25		1.7×10^{-4}	24690	0.42
	25	$\pm$ 0.33 (48 psi)	1.2×10^{-4}	58950	1.0
	25		0.67×10^{-4}	85570	1.45

*These represent values after approximately 200 cycles.

In order to investigate the effect of test periods on the fatigue response of asphalt concrete mix, Hsu and Tseng (1996) theoretically analyzed a series of third-point bending beam tests and found that the number of repetitions to failure increased with the rest period and was more significantly exhibited at higher temperature.

6.2.2 Laboratory Fatigue Tests

A variety of methods have been developed for the fatigue testing of bituminous mixtures. A SHRP program has been conducted in order to evaluate various laboratory fatigue tests with the objective of recommending a relatively simple test (or tests) which can best simulate field conditions. Procedures of the fatigue tests which involve a variety of test techniques, equipment types, specimen configurations, types and modes of loading, test conditions (for example, frequency of loading, temperature, etc.), and analysis procedures were assessed (SHRP 1994, Matthews et al 1993). Table 6.3 lists the advantages, disadvantages, and limitations of each method. Necessary data on field correlations were not available; hence, it was not possible to take this factor into consideration in the evaluation. These tests are simply introduced as follows.

1. Simple flexure

A number of' different types of flexural equipment have been developed to study the fatigue characteristics of asphalt-concrete mixes including (but not limited to): flexure tests in which the loads are applied repeatedly or sinusoidally under center-point or third-point loading (Franchen and Verstraeten 1974), rotating cantilever beams subjected to sinusoidal loads, and trapezoidal cantilever beams subjected to sinusoidal loads or deformations (Pell 1962).

Maupin (1977) pointed out that testing in the constant stress mode in the flexural fatigue test showed the stiffer mixes exhibiting longer fatigue lives than the mixes with low stiffness. Carpenter and Vavrik (2001) carried out the flexural fatigue test in constant strain at 10 Hz in a temperature cabinet at 20 ^{0}C in accordance with AASHTO TP-94. The goodness of fit for the K_2 coefficient (slope of the fitted fatigue-stress equation in the log-log scale) in the regressed model indicated that there might be a stronger influence of material composition.

Stephenson and Bullen (2002) tested the beams at 20 ^{0}C using a four point bending method. The controlled strain mode of loading with a haversine wave form was selected. Initial stiffness was taken as the stiffness after 50 loading cycles and fatigue defined as the point where the stiffness reduced to 50% of the initial value. They found the lime filler produced the stiffest mix with the shortest fatigue life whereas the flyash filler produced the least stiff mix but with the longest fatigue life.

2. Supported flexure

This test can provide with a direct relationship between fatigue life and stress/strain developed by loading beams or slabs that are supported in various ways to directly simulate in-situ modes of loading and sometimes to simulate a more representative stress state.

3. Direct axial

This test can provide with a direct relationship between fatigue life and stress/strain developed by applying pulsating or sinusoidal loads, uniaxially, with or without stress reversal.

Table 6. 3 Comparison of fatigue test methods

Method	Application of Test Results	Advantages	Disadvantages and Limitations	Simulation of Field Conditions	Simplicity	Overall Ranking
Repeated flexure test	Yes σ_b or ε_b, S_{mix}	1. Well known, widespread 2. Basic technique can be used for different concepts 3. Results can be used directly in design 4. Options of controlled stress or strain	Costly, time consuming, specialized equipment needed.	4	4	I
Direct tension test	Yes (through correction) σ_b or ε_b, S_{mix}	1.Need for conducting fatigue tests is eliminated. 2. Correlations exist with fatigue test results.	In the LCPC methodology: 1. The correlations based on one million repetitions 2. Temperature only at 10^0C. 3. Use of EQI (thickness of bituminous layer) for one million repetitions only.	9	1	I
Diametral repeated load test	Yes $4\sigma_b$ and S_{mix}	1. Simple in nature 2. Same equipment can be used for other tests 3. Tool to predict cracking.	1.Biaxial stress state 2. Underestimates fatigue life	6	2	II
Dissipated energy method	ϕ, ψ, S_{mix} and σ_b or ε_b,	1. Based on a physical phenomenon 2. Unique relation between dissipated energy and N.	1.Accurate prediction requires extensive fatigue test data. 2. Simplified procedures provides only a general indication of the magnitude of the fatigue life.	5	5	III
Fracture mechanics tests	Yes K_I, S_{mix} curve (a/h-N) calibration function (also K_{II})	1. Strong theory for low temperature 2. In principle the need for conduction fatigue tests eliminated.	1. At high temperature, K_I is not a material constant. 2. Large amount of experimental data needed. 3. K_{II} (shear mode) data needed. Link between K_I and K_{II} to predict fatigue life to be established. 4. Only stable crack propagation state is accounted for.	7	8	IV
Repeated tension or tension and compression	Yes σ_b or ε_b, S_{mix}	1. Need for flexural fatigue tests eliminated.	1. Compared to direct tension test, this is time consuming, costly and special equipment required.	8	3	
Triaxial repeated tension and compression test	Yes σ_d or σ_c, S_{mix}	1. Relatively better simulation of field conditions	1. Costly, time consuming, and special equipment needed. 2. Imposition of shear strains required.	2	6	
Repeated flexure test on elastic foundation	Yes σ_b or ε_b, S_{mix}	1. Relatively better simulation of field conditions, 2. Tests can be conducted at higher temperatures since specimens are fully supported.	1. Costly, time consuming, and special equipment required.	3	7	
Wheel track test (Laboratory)	Yes σ_b or ε_b	1. Good simulation of field conditions.	1. For low S_{mix} fatigue is affected by rutting due to lack of lateral wandering effects. 2. Special equipment required.	1	9	
Wheel Track (field)	Yes σ_b or ε_b	1. Direct determination of fatigue response under actual wheel loads.	1.Expensive, time consuming. 2. Relatively few materials can be evaluated at one time. 3. Special equipment required.	1	10	

4. Diametral Or Indirect Tensile Fatigue Test (ITFT)

The diametral fatigue test is an indirect tensile test conducted by repetitively loading on a cylindrical specimen with a compressive load which acts parallel to and along the vertical diametral plane. This loading configuration develops a reasonably uniform tensile stress in the specimen perpendicular to the direction of the applied load and along the vertical diametral plane.

The loading configuration is illustrated in Fig. 6.3. Loads can be applied with various devices including electro-hydraulic and pneumatic systems. Usually a haversine load pulse is employed. Kennedy and Anagnos (1983) used a loading time of 0.4 second and a rest interval of 0.6 second (60 repetitions per minute). Khosla and Omer (1984) used a loading time of 0.05 second and a frequency of 20 repetitions per minute.

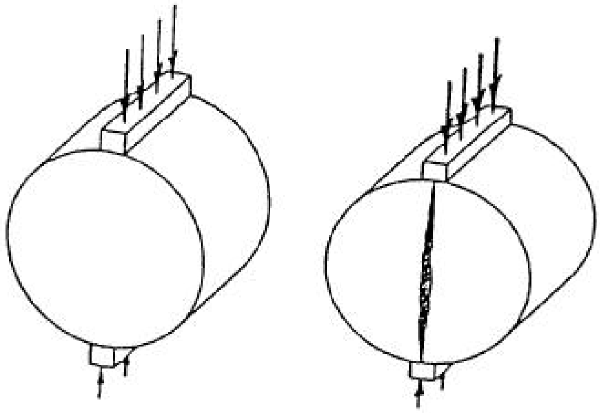


Fig.6. 3 Loading configuration and failure in diametral test

Test specimens are usually 4 inches in diameter and 2.5 inches high. Load is transmitted to the sides of the right circular cylinder through a 0.5 in. wide loading strip. Under a line load of sufficient magnitude, the diametral specimen would fail near the load line due to compression. The compressive stresses are greatly reduced by distributing the load through a loading strip, however, and a sufficiently large load will actually induce a tensile failure along the vertical diameter (Read and Collop 1997).

Stresses at the center of the specimen under a strip load (see Fig. 6.4) are as follows:

$$\sigma_x = [\frac{(2 \cdot P)}{\pi \cdot a \cdot h}][\sin 2\alpha - \frac{\alpha}{2R}] \qquad (6.3)$$

$$\sigma_y = [\frac{-6 \cdot P}{\pi \cdot a \cdot h}][\sin 2\alpha - \frac{\alpha}{2R}] \qquad (6.4)$$

where *P* is the applied load, *a* is the width of loading strip, *h* is the height of specimen, *R* is the radius of specimen, 2α , is the angle at the origin subtended by the width of loading strip, σ_x is the indirect tensile stress (horizontal) at the center of the specimen, and σ_y is the indirect compressive stress (vertical) at the center of the specimen. At the center of the specimen, the vertical compressive stress is three times the horizontal tensile stress.

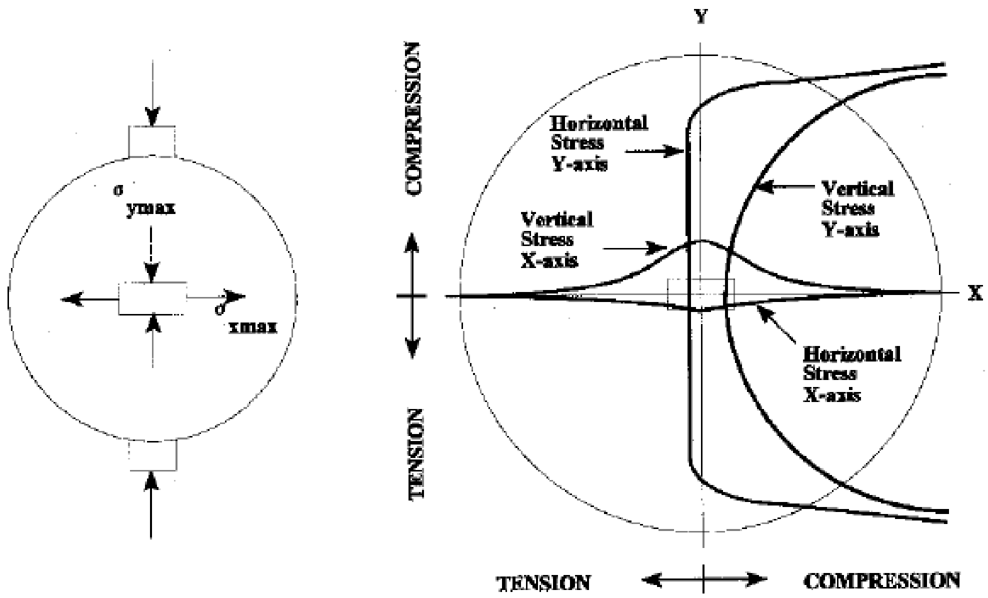


Fig.6. 4 Stress distributions and element in the diametrical specimen (Read and Collop1997)

In addition to the biaxial state of stress, two more differences exist between the flexural beam and diametral tests. These are: (1) permanent deformation which is usually prohibited in flexural tests but permitted in diametral tests and (2) stress reversal which is impractical in diametral tests. However, the existing biaxial state of stress possibly better represent the field conditions.

Due to simplicity and effectiveness of this method for characterizing materials in terms of "fundamental" properties, many researchers used this method to study the fatigue property of asphalt mix. Kennedy (1977) found that the relations of the fatigue law (slopes and relative positions) were material or project dependant. The coefficient of variation for fatigue life, which was relatively large, ranged from 30 to 80 percent. Read and Collop (1997) pointed out a high degree of correlation was demonstrated between results from the ITFT and those from two more fundamental test methods, the trapezoidal cantilever and the uniaxial tension-compression test. The repeatability and the reproducibility of the ITFT were considered as good. Partl and Piber (2002) found that fatigue predictions were in better agreement with the data collected from the in-situ material for a period of 10 years. However, the fatigue models with more accuracy could not be established. Said et al (2002) also confirmed that higher stiffness modulus of roadbase layer caused its lower strains.

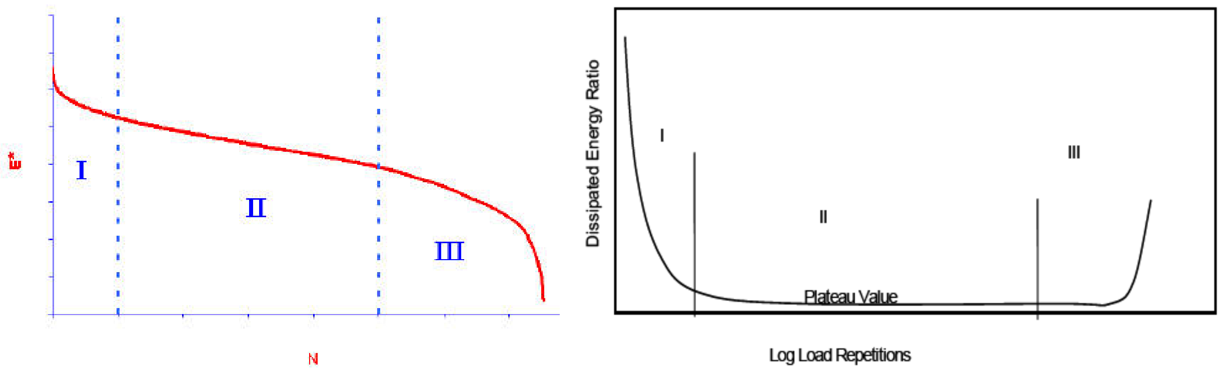
5. Triaxial

It can provide with a direct relationship between fatigue life and stress/strain developed by testing similar to direct axial testing but with confinement. This test better represents the state of stress in situ than most other laboratory tests. However, it is costly and time consuming, requires specialized equipment.

6. Fracture tests and energy theory

Another approach for characterizing the fatigue response of asphalt concrete makes use of the principles of fracture mechanics. In this method, fatigue is considered to develop in three phases: (1) crack initiation, (2) stable crack growth, and (3) unstable crack propagation (Philippe 2002). It is assumed that the second phase consumes most of the fatigue life and, consequently, it is for this phase that quantitative models based on fracture mechanics have been proposed. Simple-flexure method and tension test can be used to define response by this methodology.

The complex modulus of the asphalt mixes reduces during test due to the fatigue of material. This reduction can produce heating which is proportional to the dissipated energy. Hence fatigue properties of asphalt mixes can be studied by dissipation theory. Fig.6.5 (a) shows the variation of the typical complex modulus for bituminous mixes in the fatigue test, there are three phases during the fatigue test. In the Phase I, complex modulus reduces as repetition of loading increases until the micro cracking occurs. Phase II shows a linear part of the curve, the complex modulus steadily develops due to fatigue damage. In Phase III, macro cracking is appearing in the material and finally the specimen fails. Fig. 6.5 (b) demonstrates the dissipated energy ratio corresponding to three phase of the complex modulus.



(a) Typical complex modulus variation (Philippe 2002)

(b) Typical dissipated energy ratio (Carpenter et al 2003)

Fig.6. 5 Variation of complex modulus and dissipated energy ratio in the fatigue test for bituminous mixes.

Development of the reduction of complex modulus can also be studied by damage theory. Benedetto et al (1996) analyzed the fatigue properties of bituminous mixes using damage theory and found that heating had predominant effect in Phase I, non homogeneous large degradations occurred in Phase III. Lee et al (2002) established a fatigue model based on the energy theory and fracture mechanics. The two material parameters of classical fatigue equation can be represented and calculated by the viscoelastic material properties.

7. Wheel-tracking tests

In order to better simulate the effects of a rolling wheel on the pavement and to better understand the pattern of crack initiation and propagation, the wheel-tracking machine can be used to study fatigue characteristics of asphalt slabs in laboratory. Results can be expressed in terms of three fatigue stages associated with the development of crack initiation (N1), real crack (N2), and failure of the slab (N3). Van Dijk (1975) presented fatigue data using this method after measuring strains at the bottom of slabs and detecting the crack initiation and propagation. He suggested that controlled-strain data might be more appropriate to define pavement cracking than controlled-stress data since the former included the influence of crack propagation on the number of load repetitions associated with unserviceability. Laboratory controlled-stress tests appear to provide conservative results. He also noted that the difference between the development of crack initiation, N1, and the development of real cracks, N2, was related fairly well to the difference between fatigue results measured under controlled-stress and controlled-strain conditions.

In order to obtain full-scale field simulation, circular and longitudinal test tracks have been designed and constructed in a number of different countries. Well-known examples include the circular tracks located at Nantes, France, and at Pullman, near the Washington State University campus, and the Federal Highway Administration's ALF (Accelerated Loading Facility). The tracks are often divided into sections, each with a different pavement structure, and loads are applied by several sets of dual truck tires. However, this is an expensive method, and special equipment is required.

8. Recommendation of fatigue test

The literature review above has been conducted to identify the available methods for fatigue testing. It can be found that the indirect tensile fatigue test (ITFT) has second-best overall ranking (Table 6.3) as it presented a simple and inexpensive test method utilizing cylindrical specimens that can be cored from a pavement or prepared in the laboratory.

The biggest problem of ITFT test is probably that either compressive or shear failure in the specimen may occur under certain test conditions (high load and/or high temperature). However, it was found that, under the recommended conditions (loading time of 120 ms and test temperature of 20 ^{0}C), tensile failure occurred first in all cases for all hot-mix asphalt mixes (Read and Collop 1997).

Read and Collop found ITFT gave similar results compared to the other two test methods: the trapezoidal cantilever test and the uniaxial tension-compression test. The ITFT was capable of evaluating polymer modified mixes too. Due to the high reproducibility, the good repeatability, the simplicity and the speed of the test, the ITFT was considered to be suitable as a UK national Standard (BSI DD ABF 1995). Hence it was adopted as the fatigue test for FA mixes in accordance with BSI DD ABF (1995) in this study.

6.3 FATIGUE TESTS AND RESULTS

6.3.1 Materials And Specimen Preparation

As the same as aforementioned evaluation of the permanent deformation for the FA mixes described in the previous chapter, two RAP materials, i.e. RAP#1 and RAP#2, were added into the mixes; two bitumens, i.e. Shell penetration-grade 60 and Shell penetration-grade 100 were used as binders. To assess the impact of the added RAP content on the fatigue properties, 3 RAP dosages, i.e. 0%, 20% and 60%, were selected.

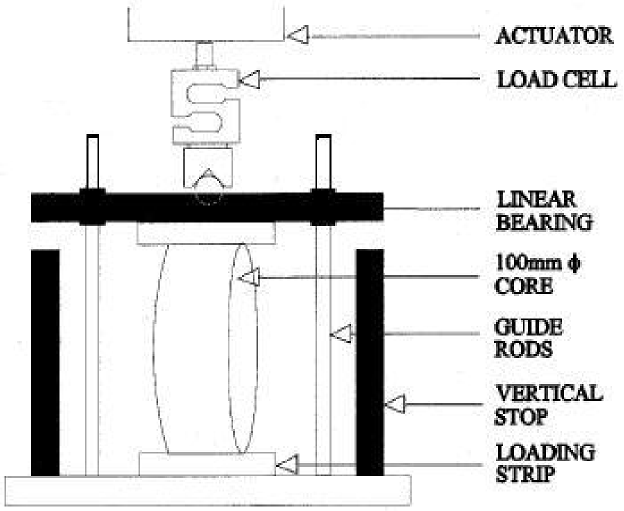
For the combination of three RAP contents, two RAP materials and two bitumens, there are 10 groups of FA mixes in total, i.e. A, B, D, F, H, J, K, M, O and Q (Table 4.10). Specimens of each FA group were prepared at the optimum foaming condition, the optimum moisture content and design bitumen content of this group. All cylindrical specimens with a diameter of 100 mm and the height of 40 mm were compacted by Marshall compactor and were cured at 40 ^{0}C for 3 days in the oven before being tested. Detailed description of the specimen preparation was introduced in Chapter 4.

6.3.2 Tests And Results

Because the FA mixes are paved as road base layer with more than 3 inches, generally, it is thicker than that of the hot-mix surfacing layer. For comparatively thick bitumen bound layer, the controlled-stress mode of loading is more suitable for fatigue testing. Hence all specimens of the FA mixes were tested in the controlled-stress mode.

In this study, the Nottingham Asphalt Tester 10 (NAT-10) was used to perform ITFT according to BSI DD ABF (BSI 1995). The NAT-10, consisting of test system, fatigue testing frame, top and bottom loading strips, a built-in LVDT and data logging equipment, is a pneumatic servo device which can adjust the loading force. It used a haversine loading waveform with a rise time of 124 ms at a pulse frequency of approximately 0.67 Hz. The cabinet of the NAT-10 constantly kept temperature at 20 ^{0}C (Copper Ltd. 2002).

Before testing, dimensions of the test specimens were measured and recorded. The specimens were then conditioned at the test temperature for 4 hours before being placed in the test frame, centered on and with the flat faces of the specimen perpendicular to the bottom loading strip. The top loading strip was then placed on the frame so that it is located centrally on the specimen. The loading was then applied via the test apparatus. Fig. 6.6 shows the schematic of the ITFT and testing process.



(a) Schematic of the ITFT

(b) Testing process

Fig.6. 6 Indirect tensile fatigue test in the NAT-10

Each specimen was tested at a different target tensile-fatigue stress (also called fatigue stress level). However, the stress level was known for FA's ITFT, in this research, the trial method was used to find the appropriate stress level (see the following description). The fatigue life is defined as the total number of loading application that causes a fracture of the specimen. During testing, the NAT-10 automatically recorded the applied load, loading repetitions and vertical deformation under each loading until the specimen was broken; the vertical deformation was collected by the built-in LVDT.

The target tensile-fatigue stress is the maximum horizontal tensile stress at the center of the specimen, $\sigma_{x,mas}$ (in MPa), and can be calculated by following formula:

$$\sigma_{x,\max} = \frac{2 \times P_L}{\pi \times d \times t} \times 1000 \qquad (6.5)$$

where P_L is the vertically applied line loading (kN);

d is the diameter of the test specimen (mm); and

t is the thickness of the test specimen (mm).

The maximum tensile horizontal strain, $\varepsilon_{x,\max}$ (in microstrain), at the centre of the specimen can be calculated from Poisson's ratio, the maximum tensile stress at the centre and the indirect tensile stiffness modulus at the maximum tensile stress level. The equation is as following:

$$\varepsilon_{x,\max} = \frac{\sigma_{x,\max} \times (1 + 3\nu)}{S_m} \times 10^9 \qquad (6.6)$$

where $\sigma_{x,mas}$ is the maximum tensile stress at the center of the specimen (KPa);

ν is Poisson's ratio (assumed to be 0.35 proposed by BSI DD ABF for bitumen bound materials); and

S_m is the indirect tensile stiffness modulus at $\sigma_{x,mas}$ (MPa).

The indirect tensile stiffness modulus (S_m) is a parameter related to the non-linear viscoelastic nature of bitumen and stress level. For each specimen, it was tested in accordance BSI DD 213 (2000) using a controlled stress version of the indirect tensile stiffness test (ITST) of the NAT-10.

According to BSI DD ABF (1995), no less than five specimens for mixes with a maximum nominal aggregate size of 20 mm and below are required in the ITFT in order to establish the fatigue relationship. For hot-mixed asphalt mixes, the target test stress level for the first specimen to be tested shall be 600 KPa unless this cannot reliably be obtained, when the first target stress level shall be 500 KPa. It should be noticed that the fatigue life of the maximum stress level is at least ten times greater the minimum fatigue life.

However, 500 KPa of the maximum tensile stress is very high for FA mixes, and the specimen cannot support this stress level. Hence, the trial method was used in the ITFT. The maximum target stress level was found for each group until the specimen couldn't support the loading. In order to give as a wide range of the fatigue lives as possible, specimens were tested with an increment/decrement of 20 to 50 kPa. Fatigue test for each group was stopped when the number of tested specimens was at least more than five, and the fatigue life ratio of the minimum stress level to the maximum stress level is at least ten times.

After ITFT and ITST, results for 10 FA groups are listed in Table 6.4, including maximum tensile stress and maximum tensile strain at the center of the specimen, vertical deformation, indirect tensile stiffness modulus, cycles to failure. The maximum tensile strains (ε_t) were calculated by Eq. 6.6. It is found that the maximum target stress level of each group ranges from 250 to 300 KPa.

Table 6. 4 Results of ITFT and ITST

Group	Spec. No.	Stress (kPa)	Vert. Deform. (mm)	Stiff. M.(MPa)	Cycles to failure	ε_t (microstrain)*
A	A7-130	130	0.8	3889.5	114824	68.52
	A3-150	150	1.1	2104.5	1157	146.12
	A9-170	170	1.1	2955	5872	117.94
	A10-190	190	1.2	1289	2107	302.17
	A11-210	210	1.3	1292.5	866	333.08
	A12-230	230	1.6	1641	1489	287.32
B	B11-150	150	1	2426	2108	126.75
	B13-190	190	1.2	1459	2052	266.96
	B14-210	210	1.6	1505.5	835	285.95
	B15-230	230	1.5	1267	321	372.14
	B16-250	250	1.7	1495.5	255	342.69
	B17-270	270	1.8	897	136	617.06
D	D2-100	100	0.8	2744.5	299770	74.69
	D11-150	150	1	2518.5	23332	122.10
	D12-170	170	1.2	3327.5	41996	104.73
	D13-190	190	1.2	3355	14152	116.10
	D14-210	210	1.5	2546	5066	169.09
	D15-230	230	1.5	2512.5	2180	187.66
	D16-250	250	1.7	2300	901	222.83
	D20-270	270	1.8	1941	647	285.16
	D18-290	290	2	1298	322	458.01
F	F11-150	150	1.1	2180.5	10722	141.02
	F12-170	170	1.1	1178.5	14421	295.71
	F13-190	190	1.2	1550	6606	251.29
	F14-210	210	1.4	2611	12149	164.88
	F17-250	250	1.8	1506	2261	340.31
	F18-270	270	1.9	618.5	132	894.91
H	H18-130	130	0.9	1482.5	9190	179.76
	H11-150	150	1.2	1662	7583	185.02
	H12-170	170	1.1	1708.5	6940	203.98
	H13-190	190	1.3	675	88	577.04
	H15-210	210	1.4	1318	809	326.63
	H17-230	230	1.5	674.5	341	699.04
J	J2-100	100	0.6	6871	85279	29.84
	J3-150	150	1	2964.5	29196	103.73
	J9-170	170	1	3212	27750	108.50
	J4-190	190	1.1	1698.5	2448	229.32
	J1-210	210	1.4	1581.5	1842	272.21
	J10-230	230	1.5	2261.5	58	208.49
K	K11-150	150	1	2370.5	9040	129.72
	K12-170	170	1.1	2873.5	15558	121.28
	K14-210	210	1.5	1731	1016	248.70
	K15-230	230	1.5	1206	361	390.96
	K16-250	250	1.7	1093	253	468.89
	K17-270	270	1.8	1212	345	456.68
	K18-290	290	1.9	953.5	84	623.49
M	M11-150	150	1	2541.5	12926	120.99
	M12-170	170	1.2	2148	1140	162.24
	M14-190	190	1.3	2217.5	3506	175.65
	M15-210	210	1.4	1096	518	392.79
	M16-230	230	1.6	1506	461	313.08
	M17-250	250	1.8	1201.5	398	426.55
O	O11-150	150	1.1	4464.5	141222	68.88
	O12-170	170	1.2	2221.5	23559	156.88
	O14-210	210	1.3	4435	37787	97.07
	O15-230	230	1.5	2456.5	3226	191.94
	O16-250	250	1.6	2839	935	180.52
	O17-270	270	1.7	2024	622	273.47
	O18-290	290	1.9	3115	721	190.85
	O19-310	310	2	2317.5	717	274.22
Q	Q21-130	130	0.9	2794.5	5013	95.37
	Q18-150	150	1.2	2043	3425	150.51
	Q12-170	170	1.2	2149.5	11363	162.13
	Q13-190	190	1.2	1830	3223	212.84
	Q14-210	210	1.4	1367.5	1081	314.81
	Q15-230	230	1.6	1209.5	539	389.83

Note: *Strain is calculated.

6.4 ANALYSIS

6.4.1 Fatigue Laws

Generally, the fatigue relationship between the initial strain and cycles to failure can be expressed as Eq. 6.7. It is a strain-fatigue equation (SHRP 1994).

$$N_f = k1(\varepsilon_t)^{k2} \qquad (6.7)$$

where ε_t is the initial strain (microstrain);

N_f is the cycles to failure;

k1 and *k2* are two material parameters.

The fatigue relationship can be also expressed by applied stress and cycles to failure as Eq. 6.8. It is a stress-fatigue equation.

$$N_f = k3(\sigma_t)^{k4} \qquad (6.8)$$

where σ_t are the magnitude of stress repeatedly applied at the centre of specimen (tensile stress level); *k3* and *k4* are material coefficients of Eq. 6.8.

Taking the logarithm (base 10) for both side of Eq. 6.7 gives:

$$\log N_f = \log k1 + k2 \log \varepsilon_t \qquad (6.9)$$

Hence *logk1* and *k2* denotes the intercept and the slope of the fitted fatigue line in the log-log plot.

In order to obtain the fatigue line of each group, ε_t against cycles to failure (N_f) was firstly depicted in the log-log plot for all testing data of this group; then linear regression analysis using the least square fitting method was carried out for data of $\log_{10}(\varepsilon_t)$ and $\log_{10}(N_f)$.

Regressed fatigue lines of 10 FA groups in the $\log N_f$ and $\log \varepsilon_t$ scales are plotted in Fig. 6.7. In this figure, the focal point as Ferreira et al stated (1987) couldn't be found, on which all fatigue lines converges to a point. On the contrary, many fatigue lines cross over each other. This phenomenon indicates that focus model is not suitable to describe the fatigue lines of FA mixes.

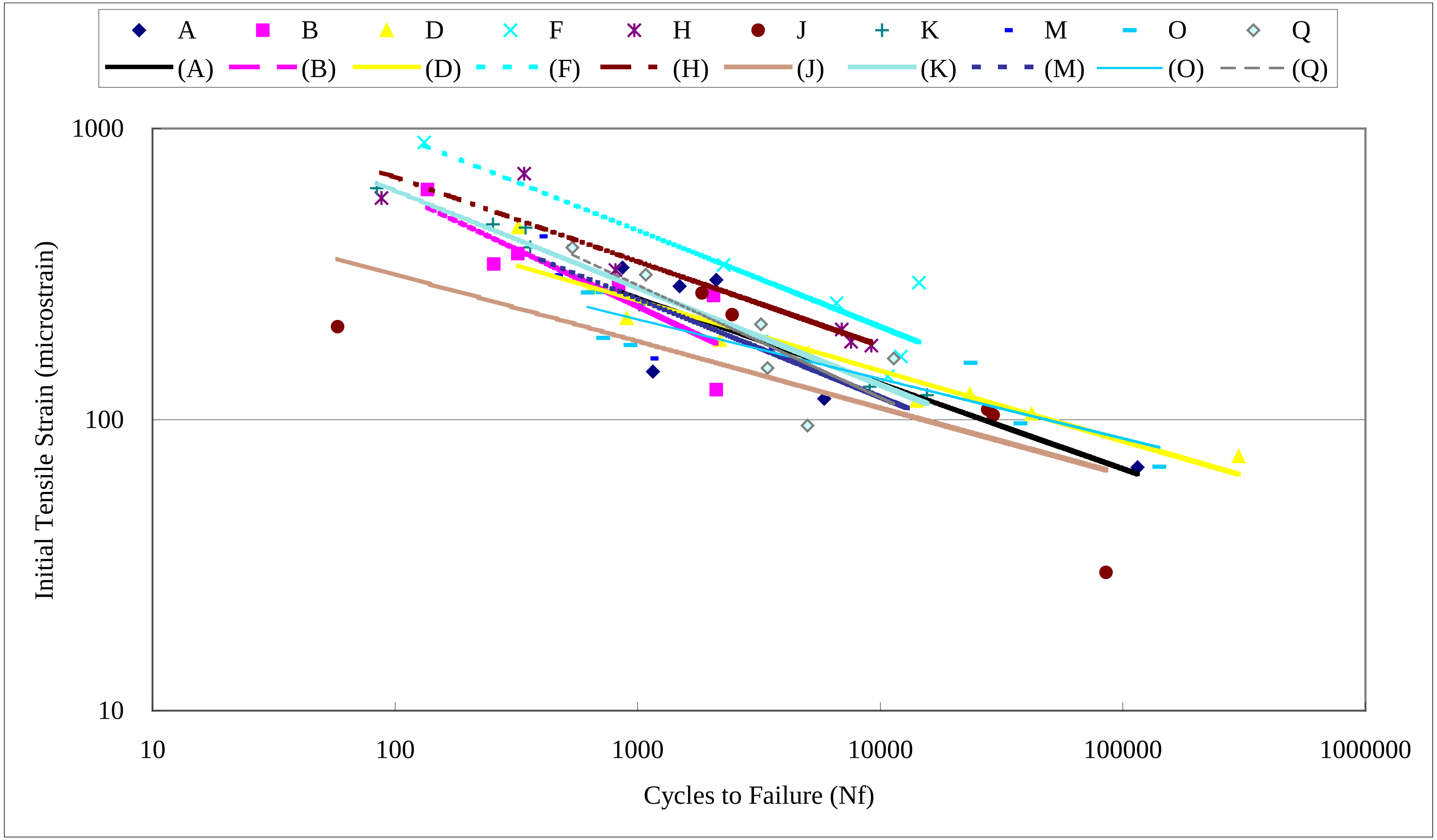


Fig.6. 7 ITFT measurements for 10 FA mixes

6.4.1.1 Material parameters

The material parameters of Eq. 6.7 and Eq. 6.8 of 10 FA groups were obtained after regression. Results, including k1, k2, k3, k4 and coefficients of determination (R-squires), are listed in Table 6.5 and Table 6.6 respectively. As well as each group, data of 10 FA groups were all fitted together; the results are listed in the row of Overall. Comparison for results in two tables indicates that in seven groups R-squares of strain-fatigue equations are higher than those of stress-fatigue equations. The overall R-square (0.7301) of the strain-fatigue equation is higher than that (0.5979) of stress-fatigue equation. Hence, the strain-fatigue equation (Eq. 6.7) is more suitable to describe the fatigue relationships of FA mixes; and it is used to characterize the fatigue of FA mixes in this study.

Table 6.5 also lists the material parameters of two hot-rolled mixes tested by the ITFT, which come from Read and Collop's work. These two materials are two standard Irish asphalt mixes. One is the SBS modified hot-rolled asphalt (HRA(SBS)) mix with 7.3% bitumen and 4% air void content, the other is the dense bitumen macadam (DBM50) with 4.2% bitumen content and 6% air void content (Read and Collop 1997). The DBM 50 was tested at 13.5 ^{0}C, and HRA(SBS) was tested at 18 ^{0}C.

Table 6. 5 Material parameters of strain-fatigue equation for FA mixes and hot-rolled mixes

Group	Temperature (^{0}C)	$N_f=K1(\varepsilon_t)^{K2}$ K1	K2	R^2	Cycles at 100 microstrains
A	20	1.71E+11	-3.404	0.7098	2.66E+04
B	20	1.23E+09	-2.548	0.7485	9.81E+03
D	20	9.82E+11	-3.720	0.9196	3.56E+04
F	20	1.00E+11	-3.018	0.8193	9.21E+04
H	20	6.10E+11	-3.454	0.8844	7.54E+04
J	20	7.97E+12	-4.363	0.558	1.50E+04
K	20	2.25E+10	-2.998	0.9838	2.28E+04
M	20	1.42E+10	-2.962	0.8025	1.70E+04
O	20	2.91E+14	-4.890	0.8392	4.84E+04
Q	20	2.34E+09	-2.587	0.6717	1.57E+04
Overall*	20	5.71E+11	-3.591	0.7301	3.76E+04
28mm DBM 50**	13.5	2.45E+13	-3.922	0.868	3.50E+05
30/14 HRA(SBS Modified)**	18	1.29E+15	-4.149	0.952	6.52E+06

Note * Overall denotes data of 10 FA groups were all regressed together.
** The last two groups are hot-rolled asphalt mixes. DBM denotes dense bitumen macadam with 4.2% 100-Pen bitumen and 6% air void, HRA denotes hot rolled asphalt with 7.3% 50-Pen bitumen and 4% air void (After Read and Collop (1997)).

Values of k2 range from –2.548 to –4.890 for all FA mixes, and vary from –3.922 to –4.149 for the two hot-rolled mixes. Hence FA mixes and the two hot-rolled mixes have the same order of magnitude of k2. However, except for Group O, values of k1 for FA mixes, which range from 1.23E+09 to 7.92E+12, are far smaller than those for the two hot-rolled mixes, which rang from 2.45E+13 to 1.29E+15.

Kennedy (1997) pointed out that a larger portion of the difference between repetition loading of ITFT fatigue and that of other methods involved a biaxial state of stress in ITFT specimen. It is suggested that stress be expressed in terms of a stress difference, i.e., the maximum principle stress minus the minimum principle stress, which is approximately $4\sigma_t$ in the failure zone. The relationship between the fatigue life and the stress difference is shown in Eq. 6.10.

$$N_f = k5(4\sigma_t)^{k6} \qquad (6.10)$$

In this study, relationship between fatigue life and stress difference were also analyzed for 10 FA groups. Results of k5 and k6 are listed in Table 6.6. It is clear that values of slope (k6) do not change, which are equal to k4, because this relation merely shifts along the $\log(N_f)$ axis. Values of k5 ranging from 6.04E+19 to 1.38E+37, however, are significantly larger than those of k3 for FA mixes, and are also significantly larger than those of k1 for the hot-rolled mixes. Values of k6 for FA mixes are 1.2 to 7.5 times those of k2 for the hot-rolled mixes. That means the fatigue lives of FA mixes drop quickly as stress increases.

Table 6. 6 Material parameters of stress-fatigue equation for FA mixes

Group	Temperature (^{0}C)	$N_f=k3(\sigma_t)^{k4}$			$N_f=k5(4\sigma_t)^{k6}$		
		k3	k4	R^2	k5	k6	R^2
A	20	9.11E+29	-11.751	0.5294	1.1E+37	-11.751	0.5294
B	20	1.93E+16	-5.807	0.8631	6E+19	-5.807	0.8631
D	20	3.49E+24	-9.268	0.7106	1.3E+30	-9.268	0.7106
F	20	3.68E+26	-9.960	0.64	3.7E+32	-9.960	0.64
H	20	9.18E+29	-11.919	0.5852	1.4E+37	-11.919	0.5852
J	20	1.64E+27	-10.549	0.7159	3.7E+33	-10.549	0.7159
K	20	3.47E+22	-8.375	0.9096	3.8E+27	-8.375	0.9096
M	20	2.31E+22	-8.389	0.7484	2.6E+27	-8.389	0.7484
O	20	7.73E+24	-9.009	0.8705	2.1E+30	-9.009	0.8705
Q	20	1.35E+19	-6.988	0.5385	2.2E+23	-6.988	0.5385
Overall	20	1.22E+26	-9.901	0.5979	1.1E+32	-9.901	0.5979

6.4.1.2 Logk1 vs. k2

The scatter plot of logk1 vs. k2 for 10 FA groups is depicted in Fig 6.8. There is a strong linear relationship shown as Eq. 6.11 between the intercepts and slopes of strain-fatigue equation.

$$k2 = -0.453\log k1 + 1.6952 \qquad (R^2=0.9656) \qquad (6.11)$$

Fig. 6.8 demonstrates that the slope (k2) of the fatigue line deceases with an increase of the intercept (logk1). The smaller the intercept (logk1) is, the larger the slope (k2) is; and the slower the fatigue decreases.

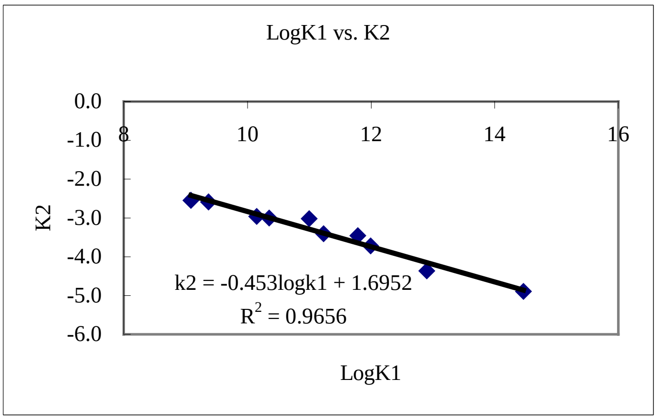


Fig.6. 8 logk1 vs. k2

6.4.1.3 Comparison of fatigue lives between FA mixes and the hot-rolled mixes

Generally, fatigue life at 100 microstrains (N_{f100}) is used as a performance indicator for road base materials (Read and Collop 1997). Table 6.5 lists the computed cycles to failure of FA mixes and the two hot-rolled mixes based on the regressed fatigue equations. Fig. 6.9 clearly demonstrates regressed fatigue lines of these two kinds of mixes. Comparing of cycles to failure at 100 microstrains of FA mixes and DBM (which is usually paved as road base layer), results of FA mixes are about 2.8% to 26.3% of that of the hot-rolled mixes. Among 10 FA mixes, only Group F and H have relatively higher cycles to failure at 100 microstrains. This comparison reflects that the fatigue lives of FA mixes are far smaller than that of DBM because k1 values of FA mixes are significantly smaller than that of DBM 50, although FA mixes and DBM have approximate order of magnitude of the slopes (k2).

Comparing FA mixes with the SBS-modified HRA, cycles to failure at 100 microstrains of FA mixes are only 0.2% to 1.4% of the latter's results. The regressed fatigue lines of the two hot-rolled mixes are also depicted in Fig.2. This figure clearly demonstrates the significant difference of fatigue lives between the FA mixes and the two hot-rolled mixes, although the testing temperature of the two hot-rolled mixes used by Read and Collop were lower than 20 ^{0}C.

Cycles to failure at 100 microstrains of FA mixes and two hot-rolled mixes were ranked as following:
HRA(SBS)>DBM(50)>F>H>O>D>A>K>M>Q>J>B

In the FA mixes, the adhesion between aggregates is provided by the mortar matrixes which consist of the foamed bitumen and fillers. Examination of the FA mixes revealed that some large aggregates were only partially coated with the foamed bitumen. Aggregates in the hot-rolled mixes, however, were all coated with the bitumen. Fig.6.10 shows specimens' central broken plates of FA mix and hot-rolled mix; it clearly demonstrates the difference between the FA mix and hot-rolled mix in the aggregate coated by bitumen (in the FA specimen, the black dots around the aggregates are bitumen mortar). Therefore adhesion between aggregates of the hot-rolled mixes provides a higher capacity to bear repeated fatigue loading than that of the FA mixes.

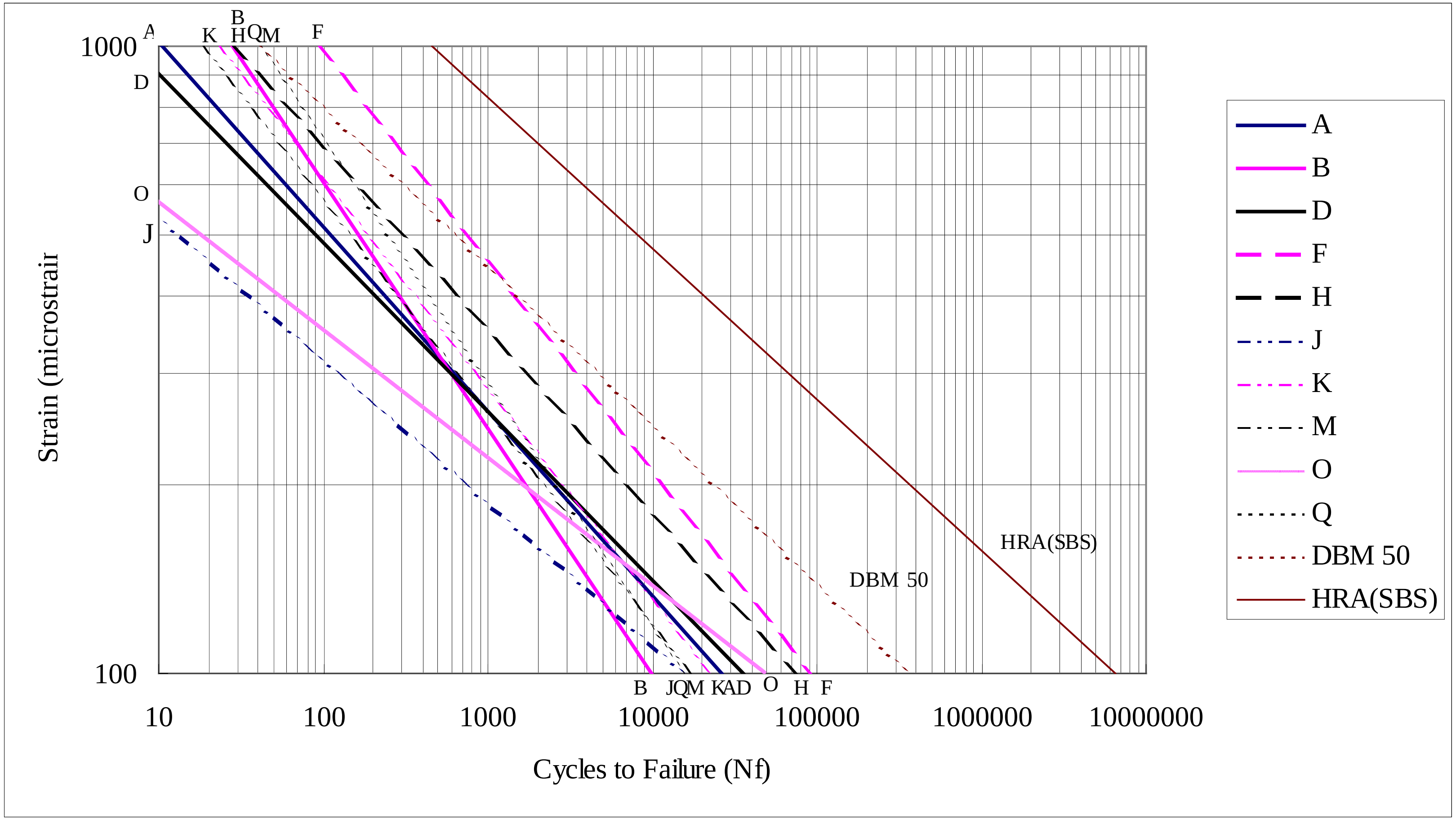
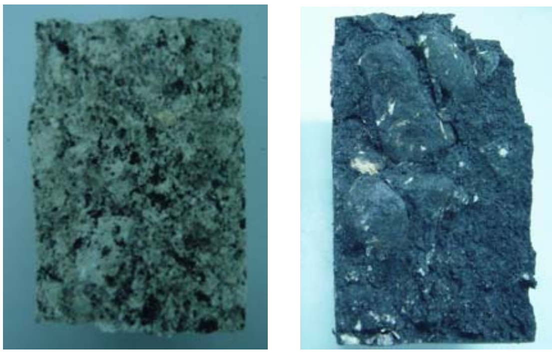


Fig.6. 9 Predicted cycles to failure at 100 microstrains for FA mixes and hot-rolled mixes



(a) FA specimen (b) Hot-rolled specimen

Fig.6. 10 Central broken plates of the FA specimen and hot-rolled specimen

Volumetric tests show that mean air voids of the 10 FA groups range from 8.5% to 15. % (see Table 4.11 and Table 4.12), significantly higher than those of the hot-rolled mixes. It is well known that air void of mix greatly affects fatigue properties. Hence high air voids and low adhesion in FA mixes result in their poor fatigue lives.

It should be noted that testing temperature of the two hot-rolled mixes was lower than 20 ^{0}C. If the hot-rolled mixes were tested at 20 ^{0}C, the difference of fatigue lives between FA mixes and the hot-rolled mixes would be more significant.

6.4.2 Deformation And Fatigue Failure Characteristics

6.4.2.1 Deformation

During the ITFT, the vertical deformation of the specimen under the repeated fatigue loading showed a steady increase trend. However, the increase rate is very small. Fig 6.11, a computer graph of the NAT-10, clearly demonstrates the deformation in the ITFT (blue curve) has an increase trend.

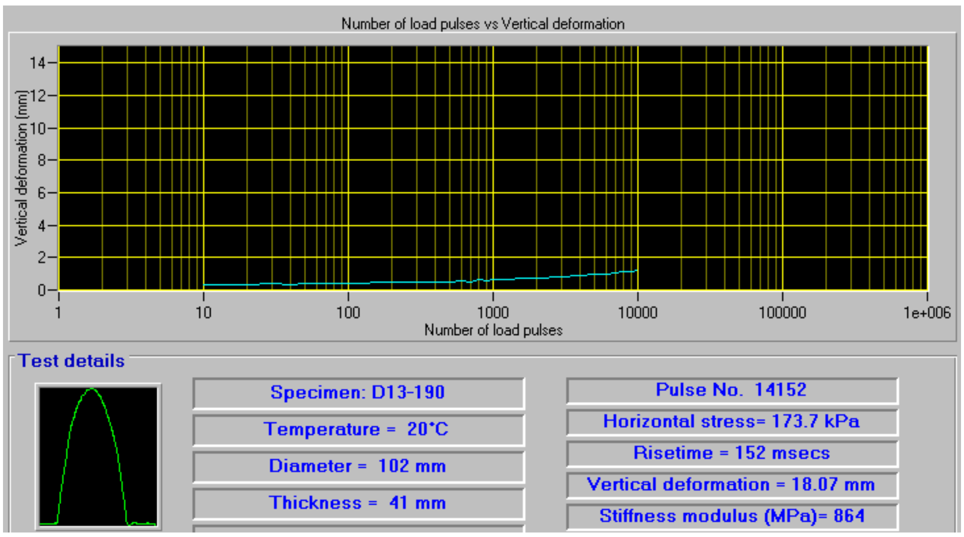
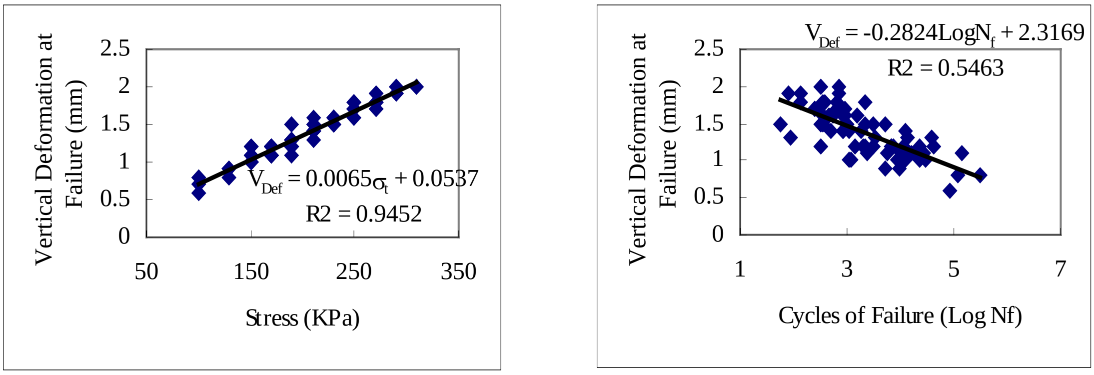


Fig.6. 11 Variation of the vertical deformation during the ITFT

Unlike the conventional hot-rolled mixes, the last stage of deformation (the unstable crack propagation) cannot be clearly found in the FA's ITFT. For all FA groups, stress applied on the specimens in the ITFT was below 310 kPa (in fact FA mixes cannot bear the stress larger than 310 kPa); the vertical deformation at failure ranged from 0.6 mm to 2.0 mm. Compared with the failure point of 9 mm for the vertical deformation of the hot-rolled mixes defined by Read and Collop (1997), the vertical deformation of FA mixes at failure is far lower than that of the hot-rolled mixes.

This result reflects that deformation of the FA mixes under the repeated loading substantially differs from that of hot-rolled mixes. In the ITFT condition, the hot-rolled mixes exhibit viscoelasticity under the fatigue repetition, as well as elasticity. However, FA mixes exhibit complete elasticity.

The relation between the vertical deformation at failure and stress level is shown in Fig. 6.12 (a), this relation can be expressed as Eq.6.12. R-square of this equation is very high. The linear relationship between the vertical deformation at failure and N_f can be described by Eq. 6.13 and is demonstrated in Fig. 6.12 (b).



(a) Vertical deformation vs. stress (b) Vertical deformation vs. N_f

Fig.6. 12 Relation between vertical deformation at failure and stress, N_f

$$V_{Def} = 0.0065\sigma_t + 0.0537 \qquad (R^2=0.9452) \qquad (6.12)$$

where V_{Def} is the vertical deformation at failure (mm),

σ_t is the tensile stress level (kPa).

$$V_{Def} = -0.2824\log N_f + 2.3169 \qquad (R^2=0.5463) \qquad (6.13)$$

However R-square (=0.5463) of Eq. 6.13 indicates that the variance between the vertical deformation and $\log N_f$ is somewhat large. This linear relationship is similar to that of the granular base material between the accumulated permanent strain and the fatigue life, found by Barksdale (1972).

6.4.2.2 Fatigue Failure Characteristics

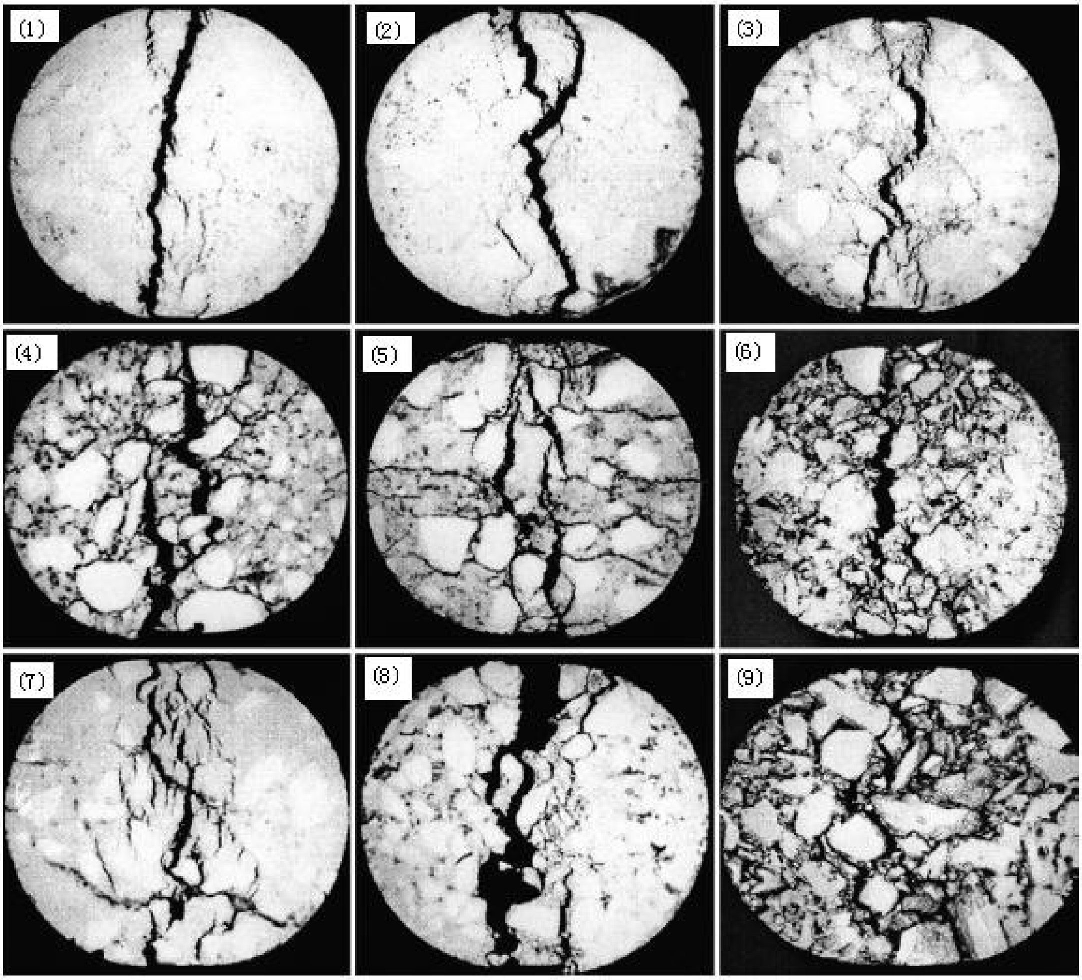
The ITFT shows that all specimens of FA mixes do not have apparent plastic deformation before failure; the failure of FA mixes is a kind of violent facture (small deformation before abrupt and violent split). When they failed at the end of the ITFT, it can be found that there were two kinds of failure patterns: ideal failure and single cleft as shown in Fig. 6.13. No specimen failed due to localized crushing or excessive deformation failure. Double split and combined failure were yet not found. For FA mixes, the fracture plane of many specimens showed uncoated aggregates; damage area of the failed specimens was narrow.

The hot-rolled mixes, however, have many failure patterns in the ITFT, including ideal failure, single cleft, double cleft, double split, multiple cleft, localized crushing failure, combined failure (see Fig. 6.13 (c)). Except for ideal failure and single cleft failure, other patterns have wide damage area or large vertical deformation. The hot-rolled mixes exhibit viscoelasticity under the load repetition; their vertical deformation is significantly larger than FA mixes'.

It is evident that the fatigue failure characteristics of the hot-rolled mixes are substantially different from those of the FA mixes due to the different bitumen coating between these two mixes. Since aggregates of the hot-rolled mixes are completely coated by bitumen, the hot-rolled mixes can support higher fatigue stress than the FA mixes. Plastic deformation can occur before specimens of the hot-rolled mixes fail. However, aggregates in FA mixes are partially coated by bitumen, FA mixes can only bear lower fatigue stress and deform very small. Hence this material is suitable to be paved as the lower layer (e.g. road base), unsuitable as the upper layer (e.g. surfacing or base course) except for the base course of the low traffic volume road.

(a) Ideal failure of FA mixes

(b) Single Cleft failure of FA mixes



(c) Failure patterns of the hot-rolled mixes (Hartman et al 2001)

Note: (1) Ideal failure (HRA); (2) Single cleft (HRA); (3) Double cleft (HRA); (4) Double split (DBM); (5) Multiple cleft (DBM); (6) Localized crushing failure (DBM); (7) Combined failure, Multiple cleft/crushing (HRA); (8) Combined failure, Multiple cleft/crushing (HRA); (9) Total deformation failure (DBM).

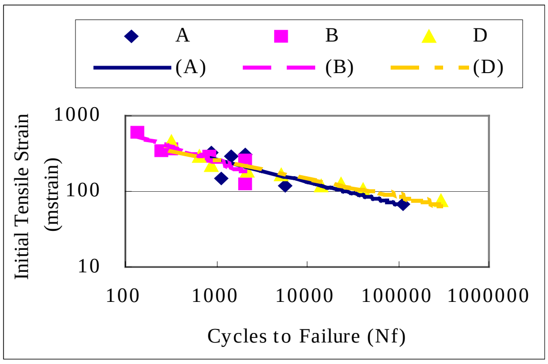
Fig.6. 13 Fatigue failure patterns on ITFT specimens for FA mixes and the hot-rolled mixes

6.4.3 Effects On Fatigue Properties

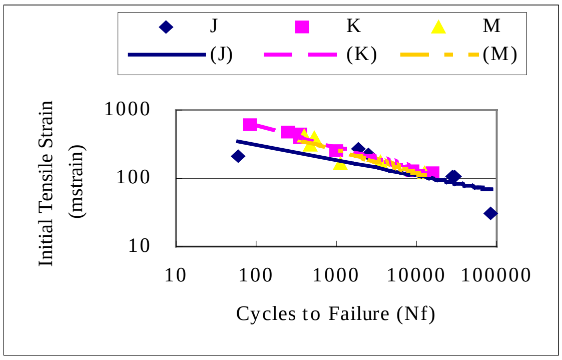
6.4.3.1 Discussion

Testing data are further presented graphically in Fig. 6.14 and Fig. 6.15 in terms of the influencing factors, including bitumen grade, RAP type and RAP content. Fig. 6.14 shows the effect of the RAP content on the fatigue lines. In this figure, only Fig 6.14 (b) clearly displays that predicted failure cycles of Group K and M at 100 microstrains are better than that of Group J. This means that when Shell 100 is used as stabilizer, mixes containing RAP materials may be superior to those without RAP materials. However, Fig. 6.14 (a), Fig. 6.14 (c) and Fig. 6.14 (d) all show that there is no clear relationship between RAP content and fatigue line.

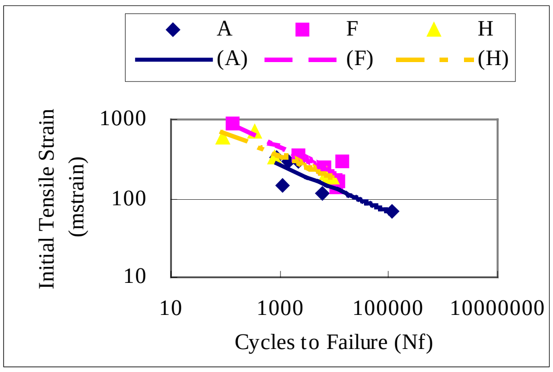
Fig.6.15 shows the relationships between the fatigue lines and bitumen types/RAP types. It is difficult to find a clear relationship between fatigue lines and RAP types as well. In order to investigate the influence of three factors on fatigue properties, the analysis of variance was carried out and is presented in the following section.



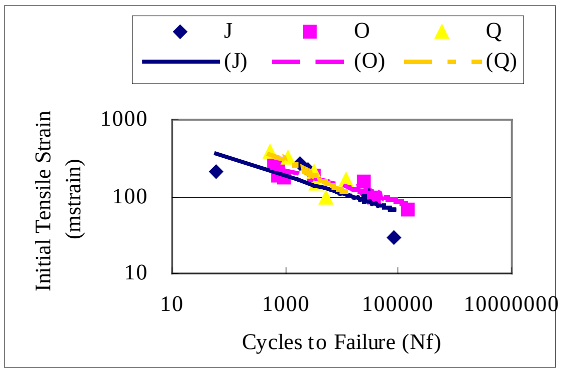
(a) Shell 60 RAP#1



(b) Shell 100 RAP#1

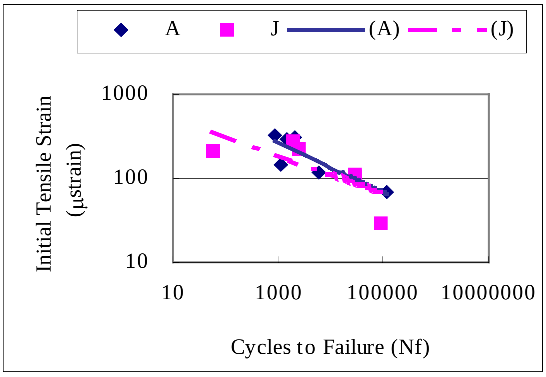


(c) Shell 60 RAP#2

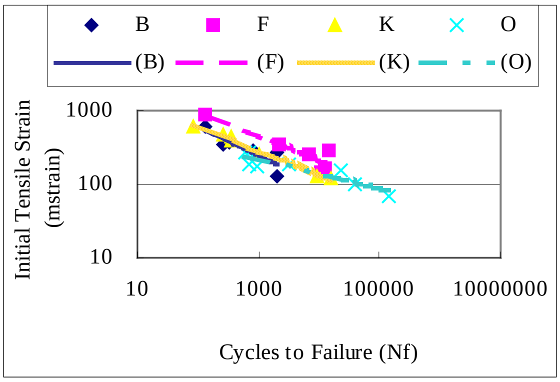


(d) Shell 100 RAP#2

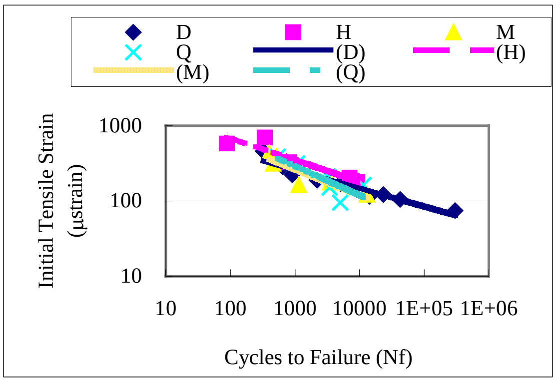
Fig.6. 14 Effects of RAP content on fatigue properties (Letter in parentheses denotes the fitted line)



(a) 0% RAP



(b) 20% RAP



(c) 60% RAP

Fig.6. 15 Effects of bitumen types and RAP types on fatigue properties (Letter in parentheses denotes the fitted line)

6.4.3.2 Analysis of variance (ANOVA)

It is difficult to conduct ITFT for a large amount of specimens due to the fact that this test is time-consuming in nature. Although the number of FA mixes' specimens are not very large, the analysis of variance can explore at least the probable relationship between fatigue properties of FA mixes and their influencing factors.

The RAP content included three levels: i.e., 0%, 20% and 60%. RAP type had two levels: RAP#1 and RAP#2. Bitumen type included two levels: Shell 60 and Shell 100. The responses were two material parameters (k1 and k2) and fatigue life at 100 microstrains (N_{f100}).

General linear model (GLM) was used to carry out ANOVA. The α level of 0.10 was selected. The most important statistic in ANOVA table is the P value, which is the probability computed in F-test to reject the null hypothesis when it is true (Montgomery 1997). Probabilities of F-test less or equal than 0.10 are considered to have significant effects.

k1 and k2 were first evaluated to see whether there were effects on them caused by variations from the RAP content, RAP type and bitumen grade. Table 6.7 shows the results of ANOVA. This table shows that all probabilities in k1 and k2 models are larger than the selected α. Hence it can be concluded that the three factors and their interactions have no significant effects on the two material parameters.

Table 6. 7 Analysis of variance for K1 and K2

Source	Degree of Freedom	Sum Squares	Mean Squares	F-value	Probability
General Linear Model: K1 versus Bitumen, %RAP, RAP					
Bitumen	1	7.77E+27	7.77E+27	1.1	0.404
%RAP	2	1.33E+28	6.67E+27	0.94	0.514
RAP	1	7.06E+27	7.06E+27	1	0.423
Bitumen*%RAP	2	1.35E+28	6.76E+27	0.96	0.511
Bitumen*RAP	1	7.09E+27	7.09E+27	1	0.422
%RAP*RAP	2	1.42E+28	7.09E+27	1	0.499
Error	2	1.41E+28	7.06E+27		
Total	11				
General Linear Model: K2 versus Bitumen, %RAP, RAP					
Bitumen	1	0.5693	0.5693	3.12	0.219
%RAP	2	1.0628	0.5314	2.91	0.256
RAP	1	0.2468	0.2468	1.35	0.365
Bitumen*%RAP	2	2.3583	1.1791	6.46	0.134
Bitumen*RAP	1	0.1437	0.1437	0.79	0.469
%RAP*RAP	2	1.2513	0.6256	3.43	0.226
Error	2	0.3651	0.1826		
Total	11				

Note: %RAP = percentage of RAP added. * = interaction of factors.

Table 6.8 lists results of ANOVA for N_{f100}. The probability of RAP is 0.10 (equal to the selected α). Hence the RAP type significantly affects the N_{f100} at the 90% confidence level. However, other factors and their interaction have no significant effect on the fatigue life at 100 microstrains.

Table 6. 8 Analysis of variance for N_{f100}

Source	Degree of Freedom	Sum Squares	Mean Squares	F-value	Probability
General Linear Model: N_{f100} versus Bitumen, %RAP, RAP					
Bitumen	1	1.46E+09	1.46E+09	6.79	0.121
%RAP	2	1.06E+09	5.25E+08	2.45	0.290
RAP	1	1.79E+09	1.79E+09	8.33	0.100
Bitumen*%RAP	2	7.97E+08	7.97E+08	3.72	0.194
Bitumen*RAP	1	1.50E+09	7.47E+08	3.49	0.223
%RAP*RAP	2	4.46E+08	2.23E+08	1.04	0.490
Error	2	4.29E+08	2.14E+08		
Total	11				

Note: %RAP = percentage of RAP added. * = interaction of factors.

6.4.3.3 Correlation between material parameters and basic property tests

In order to establish the relationship between two material parameters of the fatigue equation and responses of basic property tests of FA mixes, such as ITS, ITSM, air void and bitumen content (see Chapter 4), analysis for correlation between results (k1, k2 and logk1) of fatigue tests and responses of the basic property tests was conducted.

Results of correlation analysis are shown in Table 6.9. The correlation coefficient between k2 and ITS is minus 0.383, which is the biggest absolute value among all correlation coefficients. Unfortunately, there is no significant correlation between material parameters (k1, k2 and logk1) and responses of the basic property tests because all of absolute values of correlation coefficients are far below 1. Therefore it is impossible to establish a reliable relationship to predict the two parameters by the responses of the basic property tests. This conclusion confirms the results of study on the cold recycled mixes obtained by Ferreira et al (1987). For the hot-mixed asphalt, however, Maupin (1977) found that there was a good correlation between slope, intercept and ITSM, ITS. The difference between Maupin's finding and the result in this study reflects that certain properties of FA mixes is different from those of hot-mixed asphalt mixes.

Table 6. 9 Correlation between material parameters and basic property tests

Group	%RAP	$N_f=K1(\varepsilon_t)^{K2}$			ITS (MPa)	ITSM (MPa)	Air void (%)	Bitumen Content (%)
		K1	Log K1	K2				
A	0	1.71E+11	11.232	-3.404	461.64	1878.0	12.81	3.5
B	20	1.23E+09	9.088	-2.548	313.14	1591.0	11.73	3.5
D	60	9.82E+11	11.992	-3.720	290.34	1243.3	11.70	3
F	20	1.00E+11	11.001	-3.018	335.38	1467.8	12.30	3.5
H	60	6.10E+11	11.786	-3.454	335.40	1733.0	11.11	3
J	0	7.97E+12	12.902	-4.363	447.00	2299.0	12.86	3.5
K	20	2.25E+10	10.353	-2.998	389.66	1697.5	12.15	3.5
M	60	1.42E+10	10.154	-2.962	321.35	1524.7	11.59	3
O	20	2.91E+14	14.465	-4.890	358.46	1355.7	12.51	3.5
Q	60	2.34E+09	9.369	-2.587	310.54	1558.5	12.70	3
Correlation coefficients: K1					0.0267	-0.3112	0.2260	0.2786
Correlation coefficients: K2					-0.3831	-0.1440	-0.2776	-0.2433
Correlation coefficients: LogK1					0.3331	0.0689	0.2144	0.2148

6.4.3.4 Fatigue life at 100 microstrains (Nf100)

Results of ITFT, including fatigue stress level, cycles to failure, vertical deformation and air void, are summarized in Table 6.10. In this table, the mean value m_x of the multiple measurements is given as the best estimate of the void content or ITS for each FA group. The standard deviation s_x indicates the scatter of the measurements and therefore quantifies the void content or ITS variability. Alternatively, the standard deviation may be substituted by its ratio to the mean value, which is indicated as the coefficient of variation v_x. The air void variance of FA specimens is good except for Group F (v_x=0.084). Some of ITS variances of FA specimens are somewhat large (e.g. Groups A and H).

Table 6. 10 Summary of the ITFT testing data

Group	Fatigue Stress Level (kPa)	Cycles to Failure in ITFT	Vert. Def. at failure in ITFT (mm)	Dry ITS (kPa) $m_x(v_x)$	Stress Level/ITS (%)	Air void (%) $m_x(v_x)$
A	130,150,170,190,210,230	866-114824	0.8-1.6	461.64(0.12)	28.16~49.82	12.81(0.032)
B	150,190,210,230,250,270	136-2108	1.0-1.8	313.14(0.07)	47.9~86.22	11.73(0.043)
D	100,150,170,190,210,230,250,270,290	322-299770	0.8-2.0	290.34(0.06)	34.44~99.88	11.70(0.044)
F	150,170,190,210,250,270	132-10722	1.1-1.9	335.38(0.08)	44.73~80.51	12.30(0.084)
H	130,150,170,190,210,230	88-9190	0.9-1.5	335.40(0.13)	38.76~68.57	11.11(0.056)
J	100,150,170,190,210,230	58-85279	0.6-1.5	447.00(0.06)	22.37~51.45	12.86(0.046)
K	150,170,210,230,250,270	84-15558	1.0-1.9	389.66(0.09)	38.5~74.42	12.15(0.033)
M	150,170,190,210,230,250	398-12926	1.0-1.8	321.35(0.05)	46.68~77.80	11.59(0.066)
O	150,170,210,230,250,270,290,310	721-141222	1.1-2.0	358.46(0.01)	41.85~86.48	12.51(0.035)
Q	130,150,170,190,210,230	539-11363	0.9-1.6	310.54(0.10)	41.86~74.06	12.70(0.047)

Note: The maximum value in the stress level column for each group is the maximum applied stress that the specimen can bear. In the ITS and air void columns, numbers out of brackets are m_x, numbers in the brackets are v_x.

When there is no RAP material in the FA mix (Groups A and J), it is apparent that N_{f100} (=2.66E+04) of FA mix stabilized by Shell 60 bitumen is larger than that (1.50E+04) of FA mix stabilized by Shell 100 bitumen.

For 4 FA mixes containing RAP#2 material, three groups (Groups F,H and O) have the largest N_{f100} values (Table 6.5). Furthermore, FA mixes containing RAP#2 and stabilized by Shell 60 bitumen (Groups F and H) have the largest N_{f100}, much larger than other groups. Hence the combination of the higher viscosity bitumen (e.g. Shell 60) and more aged RAP materials (e.g. RAP#2) is advantage to N_{f100} of FA mix.

Except for Group B, most of FA mixes stabilized by Shell 60 have larger results of N_{f100} than those stabilized by Shell 100. For those groups stabilized by Shell 100, only Group O has larger value.

It can be concluded from above analysis that FA mixes containing RAP#2 have larger results of N_{f100} than those mixes containing RAP#1. The higher viscosity bitumen (e.g. Shell 60) is more advantageous to N_{f100} of FA mix with/without RAP material than the lower-viscosity bitumen (e.g. Shell 100).

6.4.3.5 Fatigue life at the maximum stress (N_{fms})

Comparison of fatigue lives at the maximum fatigue stress (the smallest fatigue lives in Table 6.10) indicates that N_{fms} of FA mix containing 60% RAP#1 is larger than that of mix containing 20% RAP#1. For RAP#2 material, however, N_{fms} of FA mix containing 20% RAP#2 is larger than that of mix containing 60% RAP#2.

On the whole, fatigue lives under the maximum fatigue stress for the combination of RAP#1 and Shell 60 bitumen are higher than those for the combination of RAP#1 and Shell 100 bitumen. For the RAP#2, the result is the opposite.

Hence the higher viscosity bitumen (e.g. Shell 60) is advantageous to N_{fms} of FA mixes containing less aged RAP material (e.g. RAP#1 which has been used 4 years). The lower viscosity bitumen (e.g. Shell 100) is advantageous to N_{fms} of FA mixes containing more aged RAP material (e.g. RAP#2 which has been used for 8 years).

For RAP#1, 60% of RAP content prolongs the fatigue lives of FA mixes by more than 20%. This may be because the less aged RAP binder around the aggregate contributes to the fatigue resistance. For RAP#2, however, more RAP content may reduce N_{fms}.

These findings suggest that if FA mixes are used as base course of the low traffic volume (in this case the stress that FA mixes bear may be larger), it is better to use the harder bitumen for less aged RAP material and to use the softer bitumen for more aged RAP material.

6.4.4 Nonlinear Characteristics of FA Mixes

To obtain the relationship between the tensile stress and the initial strain at the stress, data of the stress and the strain were depicted for each group in the ε_t - σ_t plot. ε_t (microstrain) and σ_t (kPa) denotes the horizontal and the vertical axis respectively. Four methods, i.e., linear, logarithmic, power and exponential, were used to fit the data of each group. R-squares of four fittings for 10 groups are shown in Table 6.11.

Table 6. 11 Fitting results for stress and strain

Group	R-squares of Fitting				σ_t=k7.ε_t^{k8}	
	Linear	Logarithmic	Power	Exponential	k7	K8
A	0.777	0.7913	0.8216	0.7921	34.461	0.3039
B	0.8256	0.9078	0.9169	0.7898	22.79	0.3912
D	0.7527	0.9145	0.8308	0.6293	12.167	0.5444
F	0.5759	0.6124	0.574	0.5226	46.617	0.2599
H	0.6568	0.7263	0.7085	0.6302	31.605	0.3
J	0.7952	0.8755	0.9214	0.7881	30.878	0.3506
K	0.9455	0.9438	0.9478	0.9189	27.814	0.3624
M	0.821	0.8633	0.872	0.8157	30.442	0.3413
O	0.684	0.6728	0.6843	0.6732	24.172	0.4407
Q	0.9427	0.9761	0.9696	0.9048	20.499	0.4072
Overall	0.4243	0.5505	0.5888	0.4177	37.679	0.3074

It is found that for most groups the logarithmic and power equations are more suitable to express the relationship between stress and strain than linear and exponential equations. Further comparison of R-squares between logarithmic and power indicates that six groups have higher R-squares of power regression (in the shaded cells) than those corresponding values of logarithmic regression. In other groups, R-squares of power regression are slightly lower than those of logarithmic regression. Therefore the good-of-fitness of power regression is the best among the four methods; the relationship between stress and strain can be described by the power equation as the Eq. 6.14.

$$\sigma_t = k7\varepsilon_t^{k8} \qquad (6.14)$$

where *k7* and *k8* are coefficients, their values for each group are listed in Table 6.11.

In Table 6.11, 70% of R-squares for power equation are larger than 0.80. R-square of Group F is the lowest (0.574). Values of k8 vary from 0.26 to 0.54. Values of k7 vary from 12.17 to 46.62. Group D and F have the smallest and biggest k7 respectively.

However the overall R-square of all groups is lower than that of each FA group except for Group F. The low R-square indicates that variance between groups is large, although most of groups have their own higher R-squares.

Fig. 6.16 shows scatter dots of all data and the regressed overall curve. The nonlinear characteristics between stress and strain are clearly shown in this fatigue illustration. When σ_t is lower than 200 kPa, the regressed curve exhibits an apparent nonlinearity

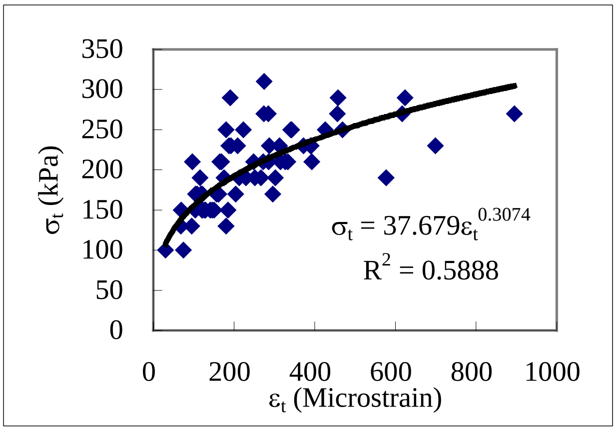


Fig.6. 16 Relationship between stress and initial strain for ITFT

Hence FA mixes have different stress-strain property from the hot-mixed asphalt, which is linear relationship. This nonlinearity may be caused by the adhesion among aggregates. In the FA mixes, the large aggregates are seldom and even not coated by the bitumen film (Fig. 6.10). The adhesion between aggregates is provided by mortars which scatter among the mixes. Therefore the FA mixes are more similar to the granular mixes, which are elastic but nonlinear.

Generally, nonlinear characteristics of the granular material is expressed by Eq. 6.15

$$E_r = k9\theta^{k10} \qquad (6.15)$$

where *k9* and *k10* are coefficient which describe the nonlinear characteristics of the granular material, θ is the principal stress. E_r is the elastic modulus. *k9* and *k10* are 2900 and 0.47 for sand and granular subbase material, 8300 and 0.71 for a high-quality, well-graded crushed stone base material (Chou 1975).

The relationship between the resilient modulus (M_r) and the average normal stress (p_m) for granular material is shown as Eq. 6.16 (Boyce 1976)).

$$M_r = 24000 p_m^{0.67} \qquad (6.16)$$

For FA mixes, the relationship between the indirect tensile stiffness modulus (ITSM) and the tensile stress was determined as Eq. 6.17

$$ITSM = k9'\sigma_t^{k10'} \qquad (R^2=0.2464) \qquad (6.17)$$

where *ITSM* is the indirect tensile stiffness modulus, σ_t is the tensile stress, $k9'$ and $k10'$ are two coefficients, $k9'$ =235380, and $k10'$ =-0.9151. $k10'$ is negative, this may be because under the larger stress level, the ITFT specimen is more apt to be split, the strain is larger. So ITSM is smaller.

Comparing Eq.6.17 with Eq. 6.15, it is found that $k9'$ is far higher than k9; $k10'$ is quite different from k10. This is because Eq.6.15 and Eq. 6.17 were established based on different testing method. Eq.6.15 is established by the triaxial test, deviate stress is applied to the specimen as well as the normal stress. While Eq. 6.17 is established by ITFT, only vertical loading applies on the specimen. Regardless of these differences, Eq. 6.17 still reflects the nonlinear characteristics of FA mixes; although the R-square (0.2464) is very low.

In order to investigate the effects of bitumen type, RAP type and RAP content on the non-linearity of the FA mixes, ANOVA was also performed, and result is shown in Table 6.12. The responses are $k9'$ and $k10'$. All probabilities of three factors in two responses' models are larger than 0.10 (α). Hence the three factors have no significant evidence of effects on $k9'$ and $k10'$, i.e., the three factors have no significant effects on the nonlinear characteristics of FA mixes. Usually, the material factors affect the fatigue property. However, in this research, the same finding cannot be obtained. This may be because the test method is not sensitive enough to detect the fatigue relationship although material factors may affect FA's fatigue behavior.

Table 6. 12 Analysis of variance for $k9'$ and $k10'$

Source	Degree of Freedom	Sum Squares	Mean Squares	F-value	Probability
General Linear Model: $k10'$ versus Bitumen, RAP Type, RAP Content					
Bitumen	1	0.001057	0.001057	0.12	0.746
RAP	1	0.006699	0.006699	0.74	0.428
%RAP	2	0.011496	0.005748	0.64	0.567
Error	5	0.045132	0.009026		
Total	9				
General Linear Model: $k9'$ versus Bitumen, RAP Type, RAP Content					
Bitumen	1	19.14	19.14	0.19	0.678
RAP	1	110.11	110.11	1.11	0.34
%RAP	2	205.95	102.97	1.04	0.419
Error	5	494.73	98.95		
Total	9				

Note: %RAP = percentage of RAP added.

Data of the indirect tensile stiffness modulus and initial tensile strain were fitted by power regression as Eq.6.18. The R-square is 0.8788, therefore there is a good power relationship between the ITSM and tensile strain. It actually embodies the nonlinearity of FA mixes

$$ITSM = 77242\varepsilon_t^{-0.6926} \qquad (R^2=0.8788) \qquad (6.18)$$

Fig. 6.17 shows the data curve of Eq. 6.18. Tensile strain is less than 200 microstrains when ITSM is larger than 2000 MPa; while tensile strain varies from 200 to 900 microstrains when ITSM is smaller than 2000 MPa.

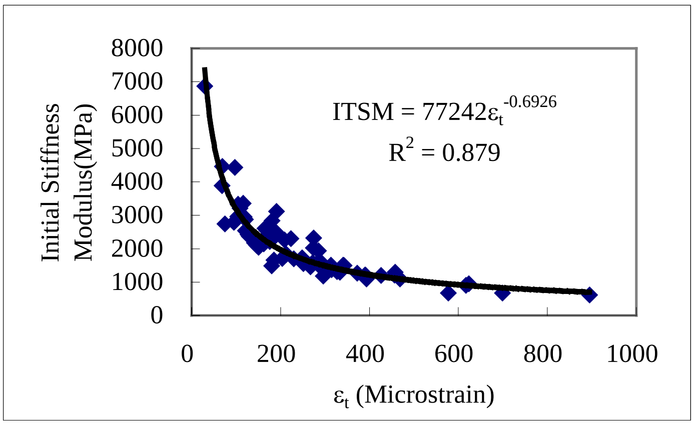


Fig.6. 17 Relationship between ITSM and initial tensile strain

Due to the nonlinear characteristics of FA mixes, strain-fatigue equation and stress-fatigue equation exhibit different good-of-fitness of regression. The fitting result of the former is better than that of the latter. The strain-fatigue equation is more suitable to characterize the fatigue of FA mixes, because the fatigue properties not only depend on the loading, but also on the basic properties, such as ITSM, of these materials.

6.5 CONCLUSION

Fatigue properties of FA mixes have not been extensively studied. In order to address fatigue characteristics in these mixes and pavement design procedures, it is necessary to know the behavior of these materials under repeated loading. This chapter presents a literature review on research methodologies for fatigue study. The ITFT was selected to evaluate the fatigue properties for FA mixes using the NAT-10 in accordance with BSI DD 213 and BSI DD ABF in laboratory.

After analysis of testing results, fatigue equations of FA mixes were established, comparison of fatigue lives between FA mixes and the hot-rolled mixes was carried out, characteristics of the fatigue failure and materials' effects on the fatigue properties were analyzed, nonlinear characteristics of FA mixes was discussed. Based on the results presented above, the following conclusion has been drawn:

(1) The developed strain-fatigue equation can satisfactorily characterize the fatigue data for FA mixes. There is a strong linear relationship between the intercepts (logk1) and slopes (k2) of the fatigue equation.

(2) Comparison of the fatigue lives at 100 microstrains between FA and hot-mixed asphalt indicates that the fatigue lives of FA mixes are far smaller than those of the hot-rolled mixes. This is due to the much higher air void and relatively lower adhesion of the FA mixes than those of the hot-rolled mixes.

(3) The vertical deformation at failure of FA mixes is far smaller than that of the hot-rolled mixes. For FA mixes, linear relationship between the failed vertical deformation and stress level is very strong; while linear relationship between the failed vertical deformation and cycles to failure (N_f) is poor.

(4) Unlike the hot-rolled mixes, in the ITFT, FA mix cannot bear fatigue loading higher than 310 kPa; vertical deformation at failure ranges 0.6 to 2.0 mm. FA mix exhibits an apparently violent fracture at failure in the ITFT; no plastic and viscoelastic deformation occur in the fatigue tests. Only ideal failure and single cleft failure were found when FA mixes failed. FA mix differs substantially from the hot-rolled mix in fatigue failure characteristics.

(5) RAP type significantly affects the fatigue life of FA mix at 100 microstrain at the 90% confidence level.

(6) It is difficult to accurately predict the fatigue lives for FA mixes using the responses of the basic

property tests because there is poor correlation between material parameters and responses of basic property tests.

(7) FA mix containing RAP#2 has larger N_{f100} than FA mix containing RAP#1. The higher viscosity bitumen is more advantageous to N_{f100} of FA mix with and without RAP material than the lower-viscosity bitumen.

(8) The higher viscosity bitumen is advantageous to the fatigue life at the maximum fatigue stress level of FA mix containing less aged RAP material. The lower viscosity bitumen is advantageous to fatigue life at the maximum fatigue stress level of FA mix containing more aged RAP material.

(9) FA mixes exhibit nonlinear characteristics between stress and strain, which can be expressed by the power equation. ANOVA indicates that bitumen grade, RAP type and RAP content do not significantly affect this nonlinear characteristics. The nonlinearity of FA mixes explains why the strain-fatigue equation is more suitable to describe the fatigue relationships than the stress-fatigue equation.

CHAPTER SEVEN MOISTURE SUSCEPTIBILITY

7.1. INTRODUCTION

Environmental factors, such as temperature, air, and water, have significantly effects on the durability of bituminous mixtures. Moisture is a key element in deterioration of the bituminous mixture. The terms *moisture* and *water* are often used interchangeably, but there appears to be a difference between the actions of moisture vapor and liquid water in distress mechanisms such as stripping.

There are three mechanisms by which moisture can degrade the integrity of bituminous mixture: (1) loss of cohesion (strength) and stiffness of the bitumen film; (2) failure of the adhesion (bond) between the aggregate the bitumen, and (3) degradation or facture of individual aggregate particles when subjected to freezing (Terrel and AI-Swailm,1994).

Premature failure of pavements due to moisture is a common problem faced by various highway agencies around the world. Moisture can accelerate the development of permanent deformation, fatigue cracking and raveling of the pavement, and can drastically reduce a pavement's expected design life.

Rainfalls are frequent in many warm-weather areas in southern China. Therefore the effects of moisture on properties of bituminous mixtures in these areas become an important research topic, especially for the FA mixes in which larger aggregates are only partially coated.

Extensive effort has been made to study the moisture-induced damage of the hot-mixed bituminous pavement. However, few efforts to study the moisture susceptibility of FA mixes were reported. This chapter focuses on investigating moisture susceptibility of FA mixes incorporating RAP materials. The objective aims at systematically analyzing effects of moisture on indirect tensile strength, permanent deformation and fatigue properties of FA mixes, and effect of moisture conditioning method on FA mixes' fatigue properties.

7.2. LITERATURE REVIEW

7.2.1. Factors Influencing Moisture Susceptibility Of Asphalt Mixture

The development of tests to determine the moisture susceptibility of bituminous mixtures began in the 1930s (Terrel and Shute 1989). Since then numerous tests have been developed in an attempt to identify susceptibility of bituminous mixtures to water damage. Current testing procedures have attempted to simulate the strength loss or other damages that can occur in the pavement so that asphalt mixtures that suffer premature distress from moisture can be identified before construction. However, no currently used test is able to predict performance. The major difficulty in developing a test procedure is to simulate the field conditions to which the bitumen-aggregate mixtures are exposed. Table 7.1 summarizes factors that should be considered in evaluating moisture susceptibility of the bituminous mixtures.

Table 7. 1 Factors influencing response of mixtures to moisture susceptibility (Terrel and Shute 1989)

Variable	Factor
Existing condition	Compaction method Voids Permeability Time Water content
Materials	Bitumen Aggregate Modifiers or additives
Conditioning	Curing Dry versus wet Vacuum saturation Freeze-thaw Repeated loading Drying
Other	Traffic Environmental history Age

The effect of water on bituminous mixtures has been difficult to access because of many variables involved. One variable that affects results of the current evaluation methods is the air void content of the mixture. The very existence of these voids, as well as their characteristics, can play a major role in performance.

A major effect of air voids is illustrated in Fig. 7.1. If the mixtures shown in this figure were designed for a range of voids by adjusting the aggregate size, gradation and the bitumen content, a range of permeability would result. Mixture with minimal voids that are not interconnected would be essentially impermeable. When air voids increased beyond some critical value, they would become larger and interconnected; thus water could flow freely through the mixture (e.g. stone-mastic mixtures and porous asphalt mixtures). Between these two extremes of impermeable and open gradation mixtures is where most bituminous pavements are constructed.

The term pessimum voids in Fig. 7.1 can actually represent a concept of quantity (amount of voids in the mixture) and quality (size, distribution, and interconnection) as they affect the behavior and performance of pavements.

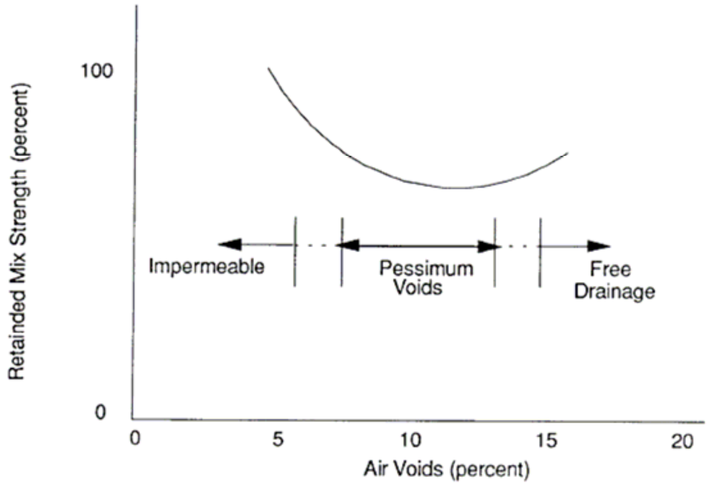


Fig.7 1 Relationship between strength of mixtures and air void content (After Terrel and Al-Swailmi 1994)

7.2.2. Conventional Methods

A moisture susceptibility (or sensitivity) test has a *conditioning* phase and an *evaluation* phase. The conditioning phase varies, but the attempt is to simulate the deterioration of the bituminous materials in the field.

The general methods of evaluating the conditioned specimens can be categorized into two types: (1) qualitative test, and (2) quantitative test. The qualitative tests include boiling water test (ASTM D 3625) and static-immersion test (AASHTO T182). The quantitative tests include Lottman test (NCHRP 246), Tunnicliff and Root conditioning (NCHRP 274), Modified Lottman test (AASHTO T 283) and immersion-compression test (AASHTO T 165).

The qualitative test may only be used for initial screening but not for detailed analysis. In the quantitative tests, the retained bitumen coating is examined following the conditioning process. Typically, the quantitative methods include strength, stiffness modulus, fatigue testing, in which a ratio is computed by dividing the result from the conditioned specimen by the result from an unconditioned specimen. If the ratio is less than a specified value, the mixture is determined to be moisture susceptible.

In Lottman test, specimens are divided into 3 groups of 3 specimens each. Group 1 is treated as control without any conditioning. Group 2 specimens are vacuum saturated (26 inches or 660 mm Hg) with water for 30 minutes. Group 3 specimens are vacuum saturated like Group 2 and then subjected to a freeze (-18 ^{0}C for 16 hours) and a thaw (60 ^{0}C for 24 hours) cycle (Lottman 1982). All 9 specimens are tested for resilient modulus (M_R) and/or indirect tensile strength (ITS) at 13 ^{0}C or 23 ^{0}C. Retained indirect tensile strength (ITSR) is calculated for Group 2 and Group 3 as follows:

$$ITSR=(ITS \text{ of Conditioned specimens})/(ITS \text{ of Control specimens}) \quad (7.1)$$

A minimum ITSR of 0.7 was recommended by Lottman who found that ITSR values of 0.7 to 0.75 differentiated distresses between stripping and non-stripping bituminous mixtures. It has been argued that Lottman procedure is too severe because the warm-water soaking of the vacuum saturated and frozen specimen can develop internal water pressure.

In NCHRP 274 test, six specimens are divided into two groups of three specimens each. Group 1 is treated as control without any conditioning. Group 2 is saturated by vacuum pump (20 inches or 508 mm Hg for about 5 minutes) with water to attain a saturation level of 55 to 80 percent, then soaked in water at 60 ^{0}C for 24 hours (Tunnicliff and Root 1984). ITS is conducted for all specimens at 25 ^{0}C.

AASHTO T 283 (also called Modified Lottman test) was proposed by Kandhal and was adopted by AASHTO in 1985. It combines the good features of Lottman test and the NCHRP 274 test. Six specimens are compacted to 6-8 percent air void content. Group 1 of three specimens is used as a control. Group 2 is saturated by vacuum pump (55 to 80 percent saturation) with water, and then subjected to one freeze and one thaw cycle as proposed by Lottman. ITS is conducted for all specimens at 25 ^{0}C. A minimum ITSR of 0.7 is usually specified (AASHTO T283 2003).

A large amount of studies on moisture susceptibility of hot-mixed bituminous mixtures can be found in the literature. Cheng et al. carried out the repeated load permanent deformation tests for asphalt mixture in dry and wet conditions respectively. They found that the wet condition accumulated more damage than the dry condition (Cheng et al 2003).

Schmidt and Graf (1972) conducted a research on the effect of water on the resilient modulus of bitumen-treated mixes. It was found that higher water content caused lower increase of the retained stiffness modulus with time. When water content decreased, the retained stiffness modulus would increase.

Cross and Voth (2001) studied the effects of conditioning on rut-depth of asphalt mixtures at 40 ^{0}C. Rut depths of samples after soaking were significantly greater than those of samples after saturation, freeze and thaw, and no conditioning. Although saturation and freeze-thaw were harsher than soaking, the former did not result in greater rut depth than the latter.

Wang et al (2004) also found that after soaking most of rut depths were greater than those after no conditioning. Saturation with freeze-thaw cycles, which was normally considered a harsher conditioning, did not result in larger rut depths of specimens. Epps et al. (2000) reported that the level of saturation did not significantly affect the magnitude of the moisture-conditioned tensile strength. More results from Epps can be referred to Epps et al 2000.

It was found that the addition of lime also significantly improved the moisture susceptibility of the bituminous mixtures subjected to multiple freeze-thaw cycles, while lime directly added into aggregate and bottom ash addition were insignificant (Zeng and Ksaibati, 2003).

McCann and Sebaaly (2003) carried out a research to evaluate the lime-treated HMA mixtures before and after multiple cycles of freeze-thaw using mechanical tests, including the resilient modulus, indirect tensile strength and simple shear test. Resilient modulus proved to be the best technique for measuring small reductions in strength. When the loss of strength due to moisture sensitivity exceeded 20%, the measurement of indirect tensile strength provided a better statistical correlation.

Moisture also significantly affects the fatigue properties of the bituminous mixtures, as well as strength and permanent deformation. Lottman et al (1988) and Shatnawi et al (1995) evaluated moisture sensitivity of asphalt mixtures respectively in terms of fatigue life and fatigue life ratio (FLR) through fatigue beam test. The fatigue life ratio was calculated by the following formula Shatnawi et al (1995):

$$FLR=FL_{wet}/FL_{dry} \tag{7.2}$$

Where FL_{wet} = fatigue life of conditioned specimens, and FL_{dry} =fatigue life of unconditioned specimens.

However, formula proposed by Lottman (Eq. 7.3) was very complicated. Fatigue life ratio was proportional to resistance to fatigue cracking. It was developed from the mechanics of materials relationship for the relative position of the wet and dry strain-fatigue line, the wet and dry pavement layer strains, and their intersections.

$$FLR=[(2\ dry\ ITS/dry\ Mr)]^{-wet\ k+dry\ k}\times (TSR^2/MrR^2)^{-wet\ k}\times (wet\ \varepsilon^{-wet\ k})/(dry\ \varepsilon^{-dry\ k}) \tag{7.3}$$

Where k=the inverse of the slope of log strain (ε) vs log repetitions of fatigue loading, and it was predicted by $k=-1.4\times10^{-3}Mr^{0.573}$; ε =the tensile bending strain due to wheel loads and was predicted for average condition by $\varepsilon=1.53\times10^{-3}Mr^{-0.187}$. *Mr* was the resilient modulus, *MrR* was the resilient modulus ratio. *ITS* was the indirect tensile strength, and *TSR* was the indirect tensile strength ratio.

However very few studies of the moisture susceptibility of FA mixes can be found in literature, especially the moisture susceptibility to fatigue and permanent deformation. Castedo-Franco et al (1984) reported that vacuum saturation weakened FA mixes, and durability was generally better with higher levels of bitumen content. Marquis et al. (2003) tested retained resilient modulus of the FA mixes by the submerging method, moisture susceptibility under cyclic loading in water showed that FA mixes were found to be not inferior to emulsion plus lime mixes. They also indicated that compaction effect (air void of FA mixes) was a key to achieving good performance of FA pavements.

7.2.3. Other Methods

The pore pressure generated under saturated undrained condition is significantly higher than that generated under drained condition. Mallick et al (2003) found that conditioning process of applying cyclic pressure of 35 to 210 kPa at 5 Hz and 60 ^{0}C was able to induce moisture damage within one hour. They also developed a new type of equipment with capacity of repetitive generation of the pore pressure and suction, causing hydraulic scouring of mixes, and causing progressive moisture damage with increasing number of cycles.

Knowledge of moisture storage capacity above the hygroscopic range is essential when calculating moisture, especially when different materials are to be connected. Janz (2001) investigated two different methods of saturating the HMA specimens before testing, namely capillary and vacuum saturation. He found that the capillary and vacuum saturated specimens gave different results. Absorption testes clearly showed that the capillary saturation was a vague concept. For this reason, vacuum saturation is generally to be preferred.

Ishai and Nesichi (1988) tested moisture susceptibility of asphalt mixtures under hot and humid climatic conditions by long-term hot immersion method. The specimens were immersed in the 60 ^{0}C hot water as long as 14 days. It was found that this method was superior to the traditional 1-day soaking test in detecting sensitive mixtures.

FWA and OH (1995) studied the effect of moisture content on asphalt mixes' properties by two conditioning methods: cyclic wetting-drying method they developed and 24-hour soaking method. Retained resilient modulus and retained indirect tensile strength were found to be affected by saturation levels and temperatures. No statistically significant difference existed between the freeze-thaw samples and the 24-hour soaked samples at the 95 percent level of confidence.

The bitumen-aggregate water interactions were studied using the environmental scanning electron microscope (ESEM) by Williams and Miknis (1998). The results of the ESEM experiments clearly demonstrated the separation of bitumen from aggregate surface during freeze-thaw cycling. Whether the separation of bitumen from aggregate surface could lead to the failure of the HMA mixtures was not investigated.

7.2.4. Limitation Of Evaluation Methods

Although many laboratory tests have been developed, implemented and modified by highway agencies to identify moisture susceptibility of bituminous mixtures, these test methods do not always yield accurate results either because they do not simulate field conditions or because they are subjective in nature (Tandon et al 1998, Terrel and Ai-Swailmi 1994).

So far, no single test has proven to be "superior" and no test has yet been established that is highly accurate in predicting moisture susceptibility of bituminous mixtures and estimating the life of the pavement.

Stuart (1986), Parker and Wilson (1986) found that a single pass/fail criterion could not be established that would enable the test results to correctly indicate whether bituminous mixtures were moisture sensitive. These are characteristics of all currently used test methods.

7.3. EXPERIMENTAL PROGRAM AND SAMPLE PREPARATION

7.3.1. Experimental Program

In this program, moisture susceptibility was only evaluated in laboratory; the effects of traffic and ageing of mixtures were not considered, effects of air voids and compaction methods on FA mixes were excluded in this program. Factors influencing FA mixes included bitumen type, RAP type and RAP content. After testing, effects of moisture on FA mixes' properties, including ITS, permanent deformation and fatigue, were to be analyzed.

No single test is proven to be better than the other. Hence moisture susceptibility of FA mixes can be investigated by one of the quantitative methods. Generally, conditioning processes of moisture susceptibility evaluation include: soaking, vacuum-saturating, freezing and thawing after vacuum-saturating. FA mixes, however, had high air voids and large water absorptions (>2%), the vacuum-saturating method was unsuitable for moisture evaluation for these mixtures. In this program, 24-hour soaking method was selected as the conditioning process for ITS and dynamic creep test. The procedures are as following:

(1) Soaked ITS

After partially vacuum-saturated with water (see Chapter 4), specimens of FA mixes were placed in 25 ^{0}C water bath and soaked for 24 hours. Then they were removed and their surfaces were dried. Finally, ITS test was conducted with a loading rate of 51 mm/minute at 25 ^{0}C for all specimens in accordance with ASTM D4867 (2002). ITS results under the dry and 24-hour soaked conditions were evaluated.

(2) Soaked dynamic creep

In view of the typical pavement structure of the freeway in Hong Kong, road base of FA mix is located at about 30 cm (11.8 inches) under the surface. The temperature of this road base was calculated by SHRP's and FHWA's equation (SHRP-A-648 1994) based on the meteorological information of Hong Kong (Hong Kong Observation 2004) (see Chapter 5). After calculation, 35 ^{0}C was specified as the temperature of dynamic creep test. Dynamic creep test was conducted in accordance with BSI DD 226 (1996). After testing, CSSs and SCSMs of FA mixes under the dry and soaked conditions were analyzed.

(3) Moisture condition ITFT

In order to investigate effects of moisture and conditioning methods on fatigue properties, two kinds of conditioning methods were selected in this program: freeze-thaw and 24-hour soaked conditions. Fatigue test were conducted in accordance with BSI DD ABF (1995) for these two conditioned specimens.

Due to the higher air voids and water absorptions, FA mixes were difficult to meet the requirement of saturation degree in AASHTO T283, e.g. the degree of saturation is between 55% and 80%. Hence only ITS was conducted for FA specimens after no-conditioning (dry condition), 24-hour-soaked conditioning (soaked condition) and the freeze-thaw conditioning (freeze-thaw condition) based on ITS procedures specified in the protocol of AASHTO T283. No requirement of saturation degree was specified for the soaked and freeze-thaw conditions.

In the freeze-thaw process, the compacted specimens were stored at room temperature for a period of 72 to 96 hours. All specimens were partially vacuum-saturated with water by applying a vacuum of 13-67 kPa absolute pressure (10-26 inches Hg. Partial pressure) for a short time (5-10 minutes). When the vacuum was removed, the specimens were submerged in water for a short time (5-10 minutes). After partially vacuum-saturation, specimens were put in a freezer at –18 ^{0}C for 16 hours, and then soaked in a 60 ^{0}C water bath for 24 hours in order to thaw the specimens. Then specimens were cooled in a 25 ^{0}C water bath for 2 hours.

For the freeze-thaw and the soaked conditions, specimens were wrapped with parafilm after cooled in a 25 ^{0}C water bath for 2 hours in order to maintain specimens' moisture during the whole fatigue testing. Then, the ITFT was performed for these specimens. Data of dry specimens came from Chapter 6.

7.3.2. Specimen Preparation

As the same as aforementioned evaluation of the permanent deformation and fatigue test described in the above chapters, two RAP materials, i.e. RAP#1 and RAP#2, were added into the mixes; two bitumens, i.e. Shell penetration-grade 60 and Shell penetration-grade 100 were used as binders. For ITS and dynamic creep testing, four RAP dosages were considered, namely: 0%, 20%, 40% and 60%. For the ITFT, three RAP dosages, i.e. 0%, 20% and 60%, were selected.

Hence, there are totally 14 FA groups in ITS and dynamic creep test (see Table 4.10); there are 8 FA groups in the ITFT, i.e. B, D, F, H, K, M, O and Q (see Table 4.10). Specimens of each FA group were prepared by the same way as in the test under the dry condition. Detailed description of the specimen preparation was introduced in Chapters 4, 5 and 6.

7.4 PROPERTIES ANALYZED

Currently, ITS and resilient modulus are used as the basic mechanical properties to determine moisture susceptibility of the bituminous mixtures from ITS ratio and resilient modulus ratio. Physical property ratios, e.g., fatigue life ratio and CSS ratio, also characterize bituminous materials' moisture resistance to a specific field distress such as fatigue cracking or wheel-path rutting.

The properties analyzed in this study are classified into two catalogues: mechanical properties and physical properties, including ITS, retained ITS (ITSR), CSS, CSS ratio (CSSR), SCSM, retained SCSM (SCSMR), US, US ratio (USR), fatigue life (FL), retained fatigue life (FLR). CSS, CSSR, US, USR, FL, and FLR belong to physical properties. ITS, ITSR, SCSM and SCSMR belong to mechanical properties.

1. ITS and ITSR

The ITS is defined as the maximum stress from a diametric vertical force that a specimen can withstand. It was computed using Eq. A.7.

The ITSR is used as a parameter to identify moisture-susceptive mixtures. The ITSR is defined as the ratio of the strength of conditioned (soaked) specimen to the strength of unconditioned (dry) specimen and is expressed mathematically as Eq. A.8.

2. CSS and CSSR

The CSS is defined as the rate of creep strain, which reflects the trend of axial strain (see Chapter 5). The CSSR is used as a parameter to identify moisture susceptive mixtures in permanent deformation. It is defined as the ratio of the CSS of conditioned (soaked) specimen to the CSS of unconditioned (dry) specimen and is expressed as Eq. 7.4

$$CSSR=CSS_{soaked}/CSS_{dry} \qquad (7.4)$$

where *CSSR* denotes creep strain slope ratio; CSS_{soaked} denotes CSS of conditioned specimen, and CSS_{dry} denotes CSS of unconditioned specimen.

3. SCSM and SCSMR

The SCSM is a function of the applied stress, and the axial strains after 600 cycles and 1800 cycles. It can be computed using Eq.5.5.

The SCSMR, which is used to identify moisture susceptive mixtures in permanent deformation, is defined as the ratio of the SCSM of conditioned (soaked) specimen to the SCSM of unconditioned (dry) specimen and is expressed mathematically as Eq. 7.5:

$$SCSMR=SCSM_{soaked}/SCSM_{dry} \quad (7.5)$$

where *SCSMR* denotes secant creep stiffness modulus ratio, $SCSM_{soaked}$ denotes the SCSM of conditioned specimen, and $SCSM_{dry}$ denotes the SCSM of unconditioned specimen.

4. US and USR

The ultimate strain is the creep strain of specimen after 1800 cycles in the dynamic creep test. The ultimate strain ratio is another parameter used to identify the moisture susceptive mixtures in permanent deformation and can be computed using the following formula:

$$USR=US_{soaked}/US_{dry} \quad (7.6)$$

where *USR* denotes the ultimate strain ratio, US_{soaked} denotes the US of conditioned (soaked) specimen, and US_{dry} denotes the US of unconditioned (dry) specimen.

5. FL and FLR

FL is the fatigue life under repetitions of a specific fatigue loading. FLR is the ratio of FL of conditioned (soaked or freeze-thaw condition) specimens and unconditioned (dry) specimen, and is calculated using following equation:

$$FLR_s=FL_{soaked}/FL_{dry} \quad (7.7)$$

$$FLR_f=FL_f/FL_{dry} \quad (7.8)$$

where FLR_s and FLR_f denote the FL ratio of soaked specimen and FL ratio of freeze-thaw specimen respectively, FL_{soaked} denotes the FL of soaked specimen, FL_f denotes the FL of freeze-thaw specimen, and FL_{dry} is the FL of dry specimen.

7.5 ANALYSIS AND EVALUATION

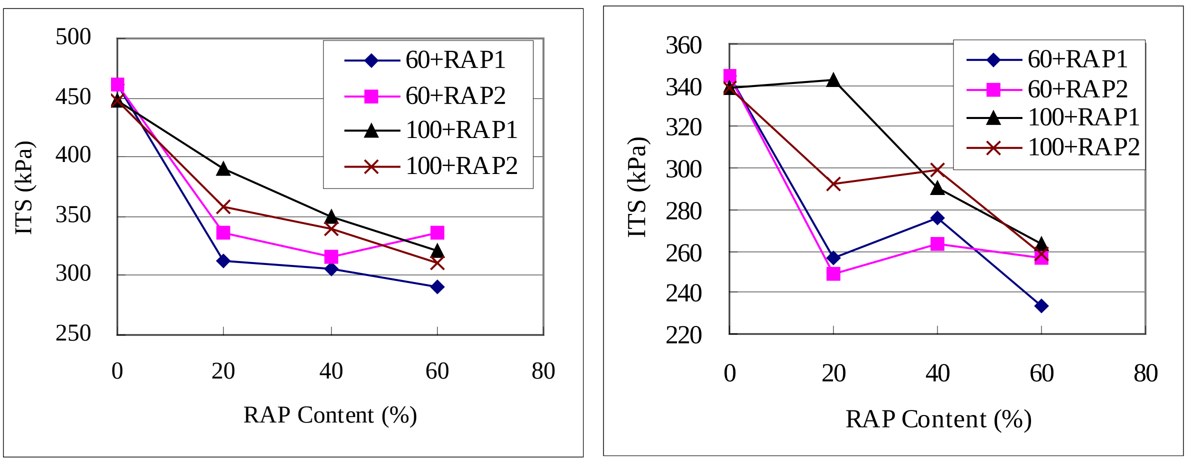
7.5.1 Indirect Tensile Strength

ITS results of each group under the dry and soaked conditions were averaged based on results of three specimens. All results of ITS and its mean value, coefficient of variation are listed in Table 7.2. The mean value m_x of the multiple measurements is given as the best estimate of the ITS for each FA group. The standard deviation s_x indicates the scatter of the measurements and therefore quantifies the ITS variability. Alternatively, the standard deviation may be substituted by its ratio to the mean value, which is indicated as the coefficient of variation v_x. ITS results are also depicted in Fig. 7.2.

Table 7. 2 Air voids and ITS Results of FA mixes and HMA mixes (at 25 ^{0}C)

Group	ITS_Dry (kPa) $m_x(v_x)$	ITS_Soaked (kPa) $m_x(v_x)$	Retained ITS	Air Void (%) $m_x(v_x)$
A	461.64 (0.12)	344.21 (0.06)	0.75	12.81 (0.032)
B	313.14 (0.07)	256.51 (0.11)	0.82	11.73 (0.043)
C	305.19 (0.10)	276.29 (0.05)	0.91	11.94 (0.042)
D	290.34 (0.06)	233.34 (0.08)	0.80	11.70 (0.044)
F	335.38 (0.18)	249.03 (0.02)	0.74	12.30 (0.084)
G	316.52 (0.06)	263.23 (0.03)	0.83	10.79 (0.033)
H	335.40 (0.13)	256.35 (0.05)	0.76	11.11 (0.056)
J	447.00 (0.06)	339.04 (0.04)	0.76	12.86 (0.046)
K	389.66 (0.09)	343.02 (0.04)	0.88	12.15 (0.033)
L	349.32 (0.07)	290.19 (0.07)	0.83	11.95 (0.048)
M	321.35 (0.05)	263.23 (0.11)	0.82	11.59 (0.066)
O	358.46 (0.05)	292.82 (0.05)	0.82	12.51 (0.035)
P	339.56 (0.10)	298.96 (0.10)	0.88	11.99 (0.046)
Q	310.54 (0.05)	258.94 (0.05)	0.83	12.70 (0.047)
PA10	500.02 (0.07)	370.34 (0.09)	0.74	17.5 (0.069)
AC 20	802.20 (0.11)	943.80 (0.08)	1.18	7.0 (0.054)

Note: 1. PA10=conventional porous asphalt mix used in Hong Kong, AC20=conventional dense asphalt concrete with 20 mm normal maximum size.
2. m_x denotes the mean value; v_x denotes coefficient of variation.



(a) Dry (b) Soaked

Fig.7 2 ITS results of all FA groups at 25 ^{0}C under two conditions

In this study, two types of HMA mixes, PA10 and AC20, are used to compare ITS results of FA mixes with those of HMA mixes. PA10 is a conventional porous asphalt mix used in Hong Kong, AC20 is a dense asphalt concrete mix with 20 mm nominal maximum size. Bitumen used in these two HMA mixes was Shell 60. ITS results of HMA mixes came from another project of Hong Kong Road Research Laboratory (HKRRL 2005), they are listed in Table 7.2

In dry condition, variations of Groups A, F, H and AC20 are larger than 10%; In soaked condition, variations of Groups B, M are larger than 10%. Table 7.2 also gives coefficients of variation of air void, which are smaller than 8.4%.

It can be found from Fig. 7.2 that ITS roughly decreases under both dry and soaked conditions when RAP content in FA mixes increases. In both dry and soaked conditions, FA mixes that were stabilized by Shell 100 have higher ITS results than those stabilized by Shell 60.

The commonly accepted evaluation method of moisture susceptibility for the HMA mixes has been proposed for a long time. Lottman et al made substantial efforts in the laboratory tests and field investigation of the moisture sensitivity for HMA mixes, and proposed a method for predicting and evaluating the moisture damage in asphalt concrete (Lottman 1978, Lottman 1982, Lottman 1986 and Lottman et al 1988). The requirement of ITSR$\geq$0.7 was one of Lottman's contributions.

So far, there is no standard method for evaluating the moisture susceptibility of the FA mixes. Under this situation, one approach to solving this problem is to adopt the HMA's criterion to evaluate the moisture susceptibility of the FA mixes. If ITSR results of FA mixes are larger than 0.7, they meet at least the same requirement of moisture susceptibility as HMA mixes.

Fig. 7.3 depicts ITSR results of all FA groups and two HMA mixes. It is found from this figure that RAP content does not significantly affect retained ITS. In fact all retained ITS results of FA mixes are higher than 0.7. Hence, all FA mixes meet the requirement of moisture susceptibility of AASHTO T283 in terms of ITSR. ITSRs of FA mixes with 0%RAP (Group A and Group J) are almost the smallest (0.75 and 0.76); it implies that RAP material helps FA mixes to improve their moisture susceptibility.

Table 7.2 shows that all ITSs of FA groups are smaller than those of PA10 and AC20. In average, ITSs of FA mixes are 43% of the dry AC 20 and 30% of the soaked AC20 respectively. Compared with PA10, ITSs of FA mixes are averagely 70% of the dry PA10 and 76% of the soaked PA10. However, Fig. 7.3 illustrates that 14 FA groups have larger ITSR than PA10 except that Group F has the same result as PA10, and 14 FA groups have smaller ITSRs than AC20. That is it to say moisture susceptibility of FA mixes is superior to PA10, but inferior to AC20.

ANOVA was performed in order to investigate effects of bitumen types, RAP types and RAP content on results of dry ITS, soaked ITS and ITS ratio. The significant evidence level (α) was set as 0.05. Probabilities less than 0.05 are considered to have significant effects (Montgomery 1997). Results of ANOVA are listed in Table 7.3.

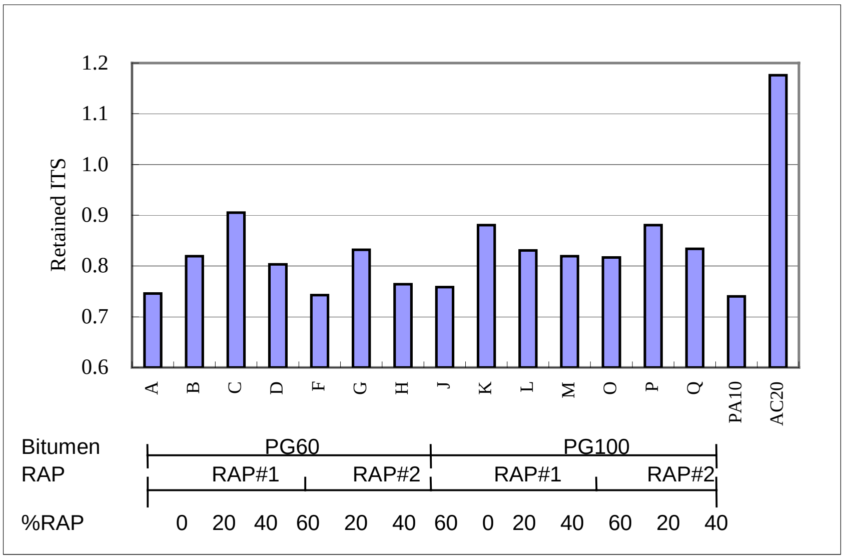


Fig.7 3 Retained ITSs of 14 FA groups and HMA mixes (25 ^{0}C)

Table 7. 3 ANOVA of ITS tests under dry and soaked conditions

Source	Degree of Freedom	Sum Squares	Mean Squares	F-value	Probability
General Linear Model: Dry ITS versus Bitumen, RAP, %RAP					
Bitumen	1	5351	5351	4.67	0.037
RAP	1	184	184	0.16	0.691
%RAP	3	78994	26331	22.99	0.000
Error	36	41230	1145		
Total	41				
General Linear Model: Soaked ITS versus Bitumen, RAP, %RAP					
Bitumen	1	4865.9	4865.9	11.5	0.002
RAP	1	467.5	467.5	1.1	0.301
%RAP	3	26382.8	8794.3	20.8	0.000
Bitumen*%RAP	3	6749.6	2249.9	5.3	0.004
Bitumen*RAP	1	580.7	580.7	1.4	0.250
Error	32	13559.5	423.7		
Total	41				
General Linear Model: Retained ITS versus Bitumen, RAP, %RAP					
Bitumen	1	0.005929	0.0	0.76	0.390
RAP	1	0.00801	0.0	1.02	0.318
%RAP	3	0.056575	0.0	2.41	0.083
Error	36	0.281448	0.0		
Total	41				

It is found from Table 7.3 that bitumen type and RAP content significantly affect the dry ITS; bitumen type, RAP content and the interaction of these two factors have significant evidence of effects on the soaked ITS. RAP type insignificantly affects the dry and the soaked ITS. Hence the ANOVA results explain the differences of FA mixes under the two conditions caused by bitumen and RAP content (see Fig.7.2). Bitumen type, RAP type and RAP content do not significantly affect the ITS ratio.

7.5.2 Permanent Deformation

7.5.2.1. Analysis of variance (ANOVA)

After dynamic creep testing, results of the dry and the soaked specimens were analyzed. CSS, CSS ratio (CSSR), SCSM, retained SCSM (SCSMR), US and US ratio (USR) were calculated and listed in Table 7.4. CSS, SCSM and US of each group shown in this table were averages of the three specimens.

Table 7. 4 Results of CSS, SCSM and US (35 ^{0}C)

Group	CSS_dry	CSS_Soaked	CSS ratio (%)	SCSM_ dry (MPa)	SCSM_ Soaked (MPa)	Retained SCSM (%)	US_dry (%)	US_Soaked (%)	US ratio (%)
A	0.1622	0.1825	1.126	604.33	448.52	0.743	0.92	1.07	1.16
B	0.1466	0.2398	1.636	627.82	343.66	0.547	0.69	0.93	1.36
C	0.1704	0.1936	1.136	486.12	239.91	0.494	0.75	0.63	0.83
D	0.2226	0.2083	0.936	261.99	440.50	1.681	0.59	0.72	1.22
F	0.1295	0.2918	2.254	126.14	282.84	2.242	1.39	1.02	0.74
G	0.1885	0.3268	1.733	431.41	169.88	0.394	0.69	1.01	1.46
H	0.1383	0.2974	2.151	628.09	270.00	0.430	0.73	0.80	1.09
J	0.1492	0.2042	1.368	558.62	398.01	0.712	0.76	0.81	1.06
K	0.1318	0.1891	1.434	666.09	413.01	0.620	0.70	0.70	0.99
L	0.1611	0.2337	1.450	551.03	352.25	0.639	0.73	0.78	1.06
M	0.0945	0.4465	4.726	1136.65	209.93	0.185	0.58	0.80	1.38
O	0.0684	0.2305	3.368	1335.80	350.20	0.262	0.22	1.02	4.69
P	0.0651	0.2103	3.232	1052.27	381.26	0.362	0.26	0.85	3.26
Q	0.0799	0.3866	4.839	912.11	215.31	0.236	0.45	1.19	2.64
PA10	0.654	N/A	N/A	302.00	N/A	N/A	N/A	N/A	N/A
AC20	0.297	N/A	N/A	128.50	N/A	N/A	N/A	N/A	N/A

ANOVA was performed in order to investigate effects of bitumen type, RAP type and RAP content on results of CSSs, SCSMs and USs under two conditions. The results of ANOVA are shown in Table 7.5.

It is found that bitumen type significantly affects dry CSS, dry SCSM and dry US; that is to say in dry condition bitumen affects permanent deformation. RAP content has significant effect on soaked CSS. The interaction of bitumen type and RAP type (or RAP content) also affects dry US. In dry condition, SCSM is affected by three factors and their interactions.

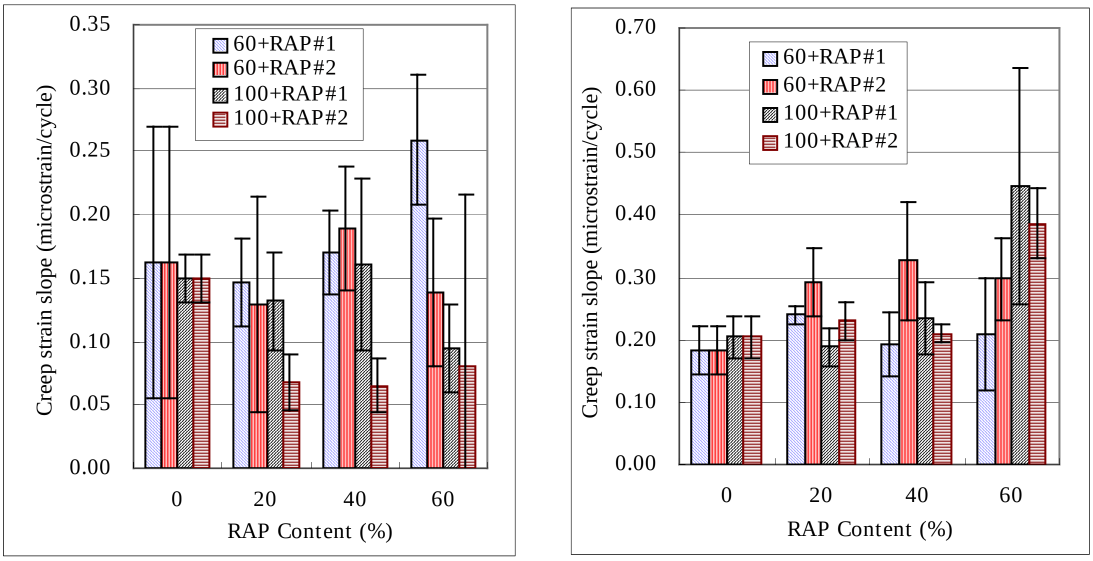
Table 7. 5 ANOVA of dynamic creep testing

Source	Degree of Freedom	Sum Squares	Mean Squares	F-value	Probability
(a) General Linear Model: Dry CSS versus Bitumen, RAP, %RAP					
%RAP	3	0.05238	0.01746	0.53	0.667
RAP	1	0.0000	0.0000	0.0000	0.993
Bitumen	1	0.15597	0.15597	4.71	0.038
%RAP*Bitumen	3	0.07908	0.02636	0.8	0.506
RAP*Bitumen	1	0.04848	0.04848	1.46	0.235
%RAP*RAP*Bitumen	3	0.0956	0.03187	0.96	0.423
Error	32	1.06063	0.03314		
Total	44				
(b) General Linear Model: Soaked CSS versus Bitumen, RAP, %RAP					
%RAP	3	0.19421	0.06474	3.12	0.040
RAP	1	0.05176	0.05176	2.5	0.124
Bitumen	1	0.04232	0.04232	2.04	0.163
%RAP*RAP	3	0.07703	0.02568	1.24	0.312
%RAP*Bitumen	3	0.08402	0.02801	1.35	0.276
%RAP*RAP*Bitumen	3	0.0987	0.0329	1.59	0.212
Error	32	0.66359	0.02074		
Total	46				
(c) General Linear Model: Dry SCSM versus Bitumen, RAP, %RAP					
Bitumen	1	792.08	792.08	28.71	0.000
RAP	1	186.19	186.19	6.75	0.014
%RAP	3	440.67	146.89	5.32	0.004
Bitumen*RAP	1	565.63	565.63	20.5	0.000
Bitumen*%RAP	3	268.39	89.46	3.24	0.035
RAP*%RAP	3	323.86	107.95	3.91	0.017
Bitumen*RAP*%RAP	3	331.65	110.55	4.01	0.016
Error	32	882.91	27.59		
Total	47				
(d) General Linear Model: Soaked SCSM versus Bitumen, RAP, %RAP					
Bitumen	1	16.07	16.07	0.24	0.625
RAP	1	205.73	205.73	3.11	0.087
%RAP	3	209.21	69.74	1.06	0.382
Error	32	2114.82	66.09		
Total	37				
(e) General Linear Model: Dry US versus %RAP, RAP, Bitumen					
%RAP	3	0.28978	0.09659	2.12	0.117
RAP	1	0.01438	0.01438	0.32	0.578
Bitumen	1	1.07141	1.07141	23.55	0.000
RAP*Bitumen	1	0.80432	0.80432	17.68	0.000
%RAP*Bitumen	3	0.37043	0.12348	2.71	0.061
Error	32	1.45562	0.04549		
Total	41				
(f) General Linear Model: Soaked US versus %RAP, RAP, Bitumen					
%RAP	3	0.05198	0.01733	0.21	0.891
RAP	1	0.21458	0.21458	2.56	0.118
Bitumen	1	0.00108	0.00108	0.01	0.910
Error	36	3.01525	0.08376		
Total	41				

7.5.2.2. Creep strain slope

Fig. 7.4 illustrates CSS mean columns with their I-shape error-bars of 14 FA mixes. It is apparent that the dry specimens have higher deviation than the soaked specimens.

In the dry condition, CSSs of mixes stabilized by Shell 100 are smaller than those of mixes stabilized by Shell 60 except that result of Group K (Shell 100 plus 20%RAP#1) is slightly larger than that of Group F (Shell 60 plus 20% RAP#2). This result means that softer bitumen helps FA mixes to lower their susceptibility to permanent deformation in dry condition. CSSs of mixes containing RAP#2 material and stabilized by Shell 100 (Group O, P and Q) are smallest among all FA groups. It implies that ageing of RAP material affects the CSS of mix stabilized by Shell 100. More aged RAP material added in mixes would result in less susceptibility to permanent deformation in dry condition. However Fig. 7.4(a) shows that RAP content does not affect the CSS significantly.



(a) Dry (b) Soaked

Fig.7 4 CSS results of 14 FA groups at 35 ^{0}C under two conditions

In the soaked condition (see Fig. 7.4(b)), five eighths of CSS results (0% RAP#1, 60% RAP#1, 0% RAP#2, 60% RAP#2 and 40% RAP#1) show that FA mixes stabilized by Shell 100 have higher CSSs than those stabilized by Shell 60. This conclusion is different from that in dry condition.

In summary, FA mixes stabilized by Shell 100 in dry condition have smaller CSSs, whilst more than half of FA mixes stabilized by Shell 60 in soaked condition have smaller CSSs. This result implies that Shell 60 may be superior to Shell 100 in view of susceptibility to permanent deformation in soaked condition.

Fig. 7.5 illustrates results of CSSR of all FA groups. All CSSR results are more than 1 except for that of Group D. Hence permanent deformation rate of FA mixes increase after soaked condition. It is noted that CSSRs of Group M, O, P and Q are larger than 3, especially for that of Group Q whose CSSR is 3.8. Hence moisture is found to severely affect permanent deformation susceptibility of these groups, whose moisture susceptibilities are very poor, although these groups have strong resistance to permanent deformation in dry condition.

Comparison of CSS between FA mixes and HMA mixes was not conducted because the latter had no data.

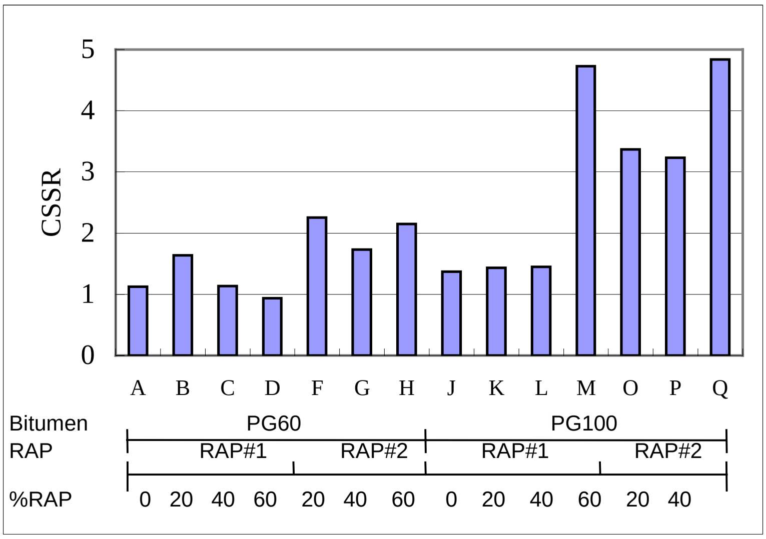
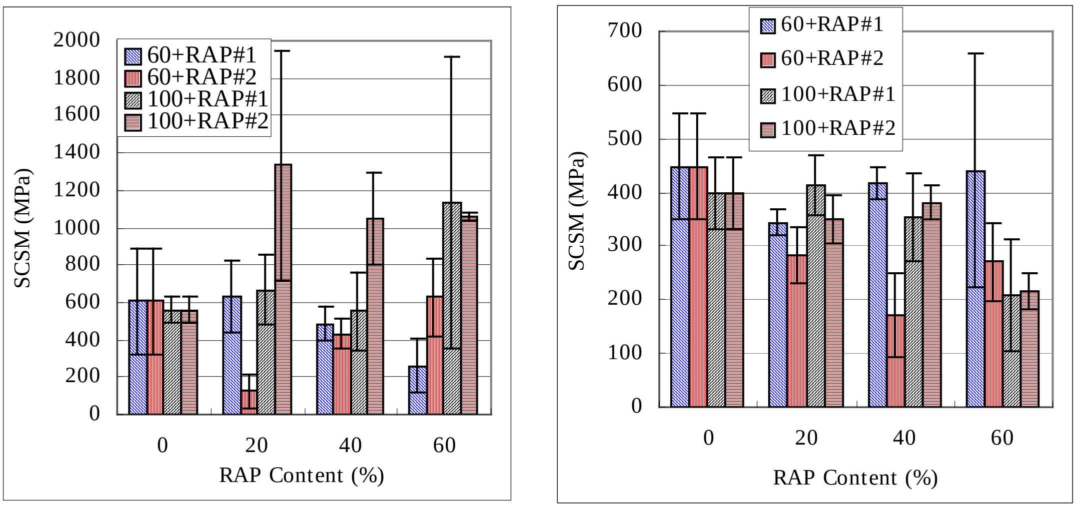


Fig.7 5 Retained CSSs of 14 FA groups (at 35 ^{0}C)

7.5.2.3 Secant creep stiffness modulus (SCSM)

After calculation, SCSMs in both dry and soaked conditions and retained SCSMs of 14 FA groups are summarized in Table 7.4. Fig. 7.6 depicts SCSMs' mean columns and error bars. It is found variations of groups under both dry and soaked conditions are significantly different, and some groups have very high values.



(a) Dry (b) Soaked

Fig.7 6 SCSM results of 14 FA groups at 35 ^{0}C under two conditions

Fig. 7.6(a) reflects that in dry condition FA mixes stabilized by Shell 100 have larger SCSM than those stabilized by Shell 60 except for groups containing zero percent of RAP material. Hence Shell 100 is superior to Shell 60 in improving the deformation resistance of FA mixes in dry condition. Table 7.5 statistically explains that three factors and their interactions affect significantly the dry SCSMs.

In soaked condition (see Fig. 7.6(b)), five eighths of SCSM results (0% RAP#1, 60% RAP#1, 0% RAP#2, 60% RAP#2 and 40% RAP#1) exhibit that FA mixes stabilized by Shell 100 have smaller SCSMs than those mixes stabilized by Shell 60. Therefore Shell 60 is better than Shell 100 to improve mixes' susceptibility to permanent deformation in soaked condition. This conclusion is the same as that in CSS analysis.

Results of SCSMR are demonstrated in Fig. 7.7. It is also found from this figure that Group M, O, P and Q have lowest retained SCSM. Hence these groups have poor resistance to moisture susceptibility. This conclusion is also the same as that in the CSS analysis. Therefore SCSM method and CSS method in analyzing the permanent deformation of FA mixes are consistent; SCSM indicates that the strength resists permanent deformation and CSS embodies the developing rate of deformation. Fig. 7.8 indicates that SCSMR decreases with an increase of CSSR.

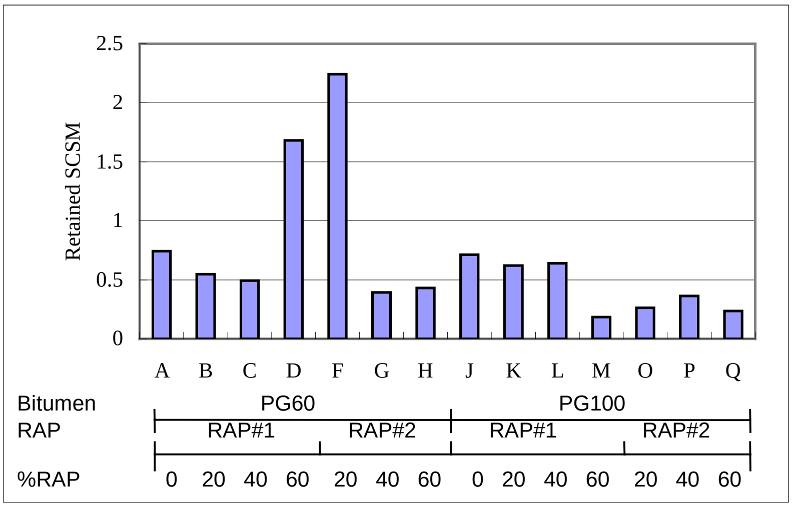


Fig.7 7 Retained SCSMs of 14 FA groups (at 35 ^{0}C)

Fig. 7.8 illustrates the relationship between SCSMR and CSSR as shown in Eq. (7.9). Their relationship can be expressed by a power function and the relationship coefficient is not large due to the abnormal datum of Group F.

$$SCSMr = 0.9415 \times CSSr^{-0.8619} \qquad \text{(R2=0.4071)} \qquad (7.9)$$

where $SCSM_r$ denotes retained SCSM, CSS_r denotes CSS ratio, relationship coefficient is 0.4071.

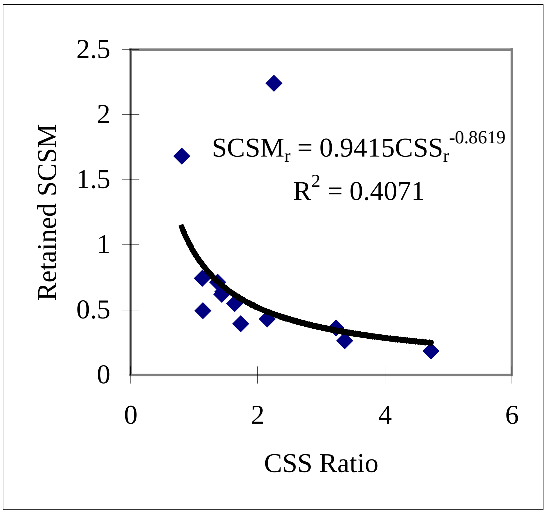
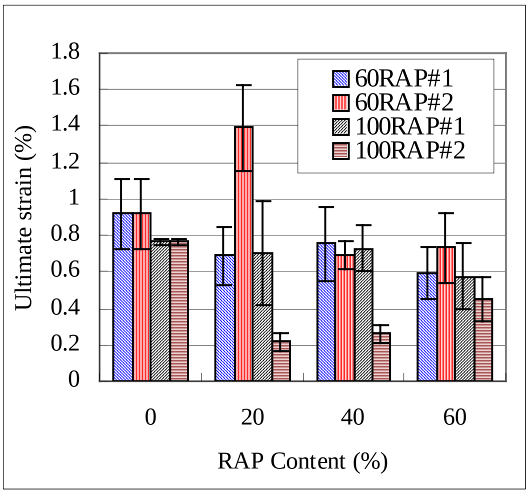


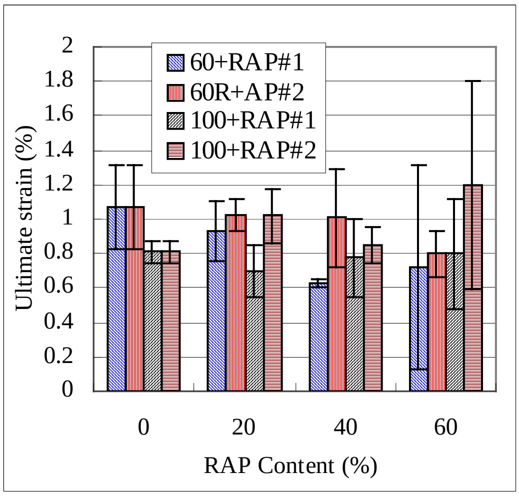
Fig.7 8 Retained SCSM vs. CSS ratio (at 35 ^{0}C)

7.5.2.4 Ultimate strain (US)

Results of ultimate strain, including mean columns and error bars, of 14 FA groups under dry and soaked conditions after 1,800 load cycles are depicted in Fig. 7.9. Under dry condition, the ultimate strains of Groups O, P, and Q are the smallest. These groups are incorporated with 20%, 40%, and 60% RAP#2 material and stabilized by Shell 100. They have best resistance to permanent deformation in dry condition.



(a) Dry



(b) Soaked

Fig.7 9 Ultimate strains of 14 FA groups at 35 ^{0}C under two conditions

This conclusion is consistent with that in CSS analysis. Six eighths of ultimate strains show that FA mixes containing RAP#2 material have lower strains than those mixes containing RAP#1 material after

1,800 load cycles in dry condition. It implies that ageing of RAP material provides significant impact on susceptibility of FA mixes to permanent deformation. Thus FA mix containing more aged RAP material exhibits less permanent deformation. Seven eighths of strain results (except those of Groups B and K) show that FA mixes stabilized by Shell 100 have less strains than those stabilized by Shell 60. Therefore it can be inferred that high-grade bitumen helps to reduce susceptibility of FA mixes to permanent deformation in dry condition.

However the same conclusion as that in dry condition cannot be drawn in soaked condition.

Ultimate strain ratios of all FA groups are depicted in Fig. 7.10. Results of Groups O, P and Q are larger than 2.5, the result of Group O is close to 5. Therefore the moisture susceptibilities of these groups are the worst in terms of ultimate strain ratio. This result confirms those in CSS and SCSM analysis.

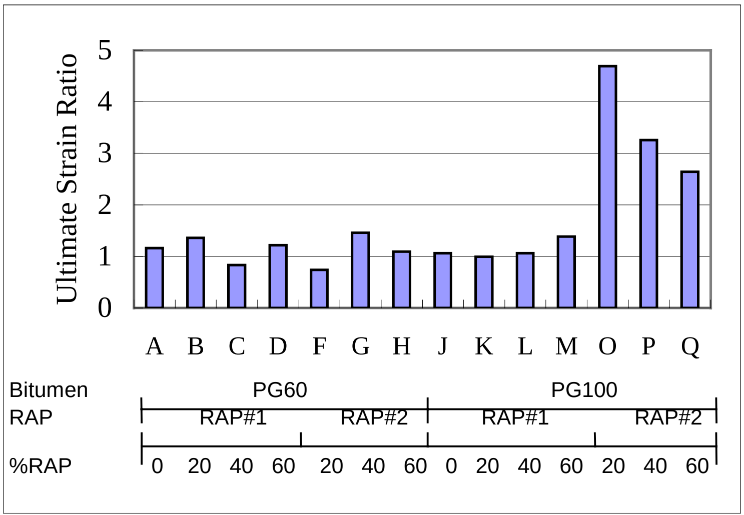


Fig.7 10 Ultimate strain ratios of 14 FA group (at 35 ^{0}C)

It can be inferred that severely aged RAP material (such as RAP #2) can help FA mixes to improve their susceptibility to permanent deformation in dry condition. However, moisture adversely affects performance of FA mixes containing more aged RAP material, and moisture susceptibility of these mixes to permanent deformation is very poor.

7.5.3 Fatigue Properties

After ITFT and ITST, fatigue testing results of 8 FA groups (Groups B, D, F, H, K, M, O and Q) under the soaked and the freeze-thaw conditions were obtained and listed in Table 7.6 and Table 7.7, including maximum tensile stresses and maximum tensile strains at the center of the specimen, vertical deformations, indirect tensile stiffness moduli, cycles to failure. The maximum tensile strains (ε_t)

were calculated by Eq. 6.6.

Table 7. 6 ITFT and ITST results of 8 FA mixes under the soaked condition

Group	Spec. No.	Stress (KPa)	Vert. Deform.(mm)	Stiff. M.(MPa)	Cycles to failure	Strain (ε_t) (microstrain)
B	B30-50	50	0.50	2704	3390	35
	B29-70	70	0.50	2121	2950	74
	B22-100	100	0.70	1332	2010	251
	B23-110	110	0.90	2382	290	144
	B31-130	130	1.10	1290	140	206
	B25-150	150	1.00	780	170	588
D	D21-100	100	0.70	3224	261450	87
	D26-130	130	1.00	1158	1300	368
	D22-150	150	1.20	2044	8480	224
	D23-170	170	1.30	1225	260	428
	D25-180	180	1.30	1369	470	400
F	F21-70	70	0.60	868	2380	132
	F22-90	90	0.70	2795	1510	68
	F23-110	110	0.80	2209	3530	146
	F24-130	130	1.00	886	240	458
	F25-150	150	1.10	1259	330	364
	F26-170	170	1.20	1302	390	434
	F27-190	190	1.30	825	180	708
H	H21-50	50	0.50	1102	8590	91
	H22-70	70	0.60	1080	5240	203
	H23-90	90	0.70	951	2670	303
	H24-110	110	0.90	756	150	445
K	K22-50	50	0.50	2647	5340	40
	K24-90	90	0.70	2993	5770	165
	K25-110	110	0.80	1963	2890	198
	K26-130	130	0.90	2109	3520	324
	K27-150	150	1.10	2022	1280	438
	K28-170	170	1.20	1458	410	433
M	M27-70	70	0.50	2226	5510	64
	M21-90	90	0.70	2816	3730	65
	M22-110	110	0.70	2546	4540	137
	M23-130	130	1.00	1648	3360	241
	M24-150	150	1.10	1088	570	421
	M25-170	170	1.30	1614	160	321
O	O24-70	70	0.50	3300	6510	42
	O21-90	90	0.60	2870	5000	65
	O26-110	110	0.80	3432	8170	101
	O27-130	130	0.90	2910	4270	139
	O28-150	150	1.00	841	680	571
	O29-170	170	1.20	2279	1080	226
Q	Q29-70	70	0.50	1662	2210	92
	Q22-90	90	0.70	2145	2690	85
	Q23-110	110	0.80	2945	4670	114
	Q24-130	130	1.00	1251	240	317
	Q25-150	150	1.10	1237	440	373

Table 7. 7 ITFT and ITST results of 8 FA mixes under the freeze-thaw condition

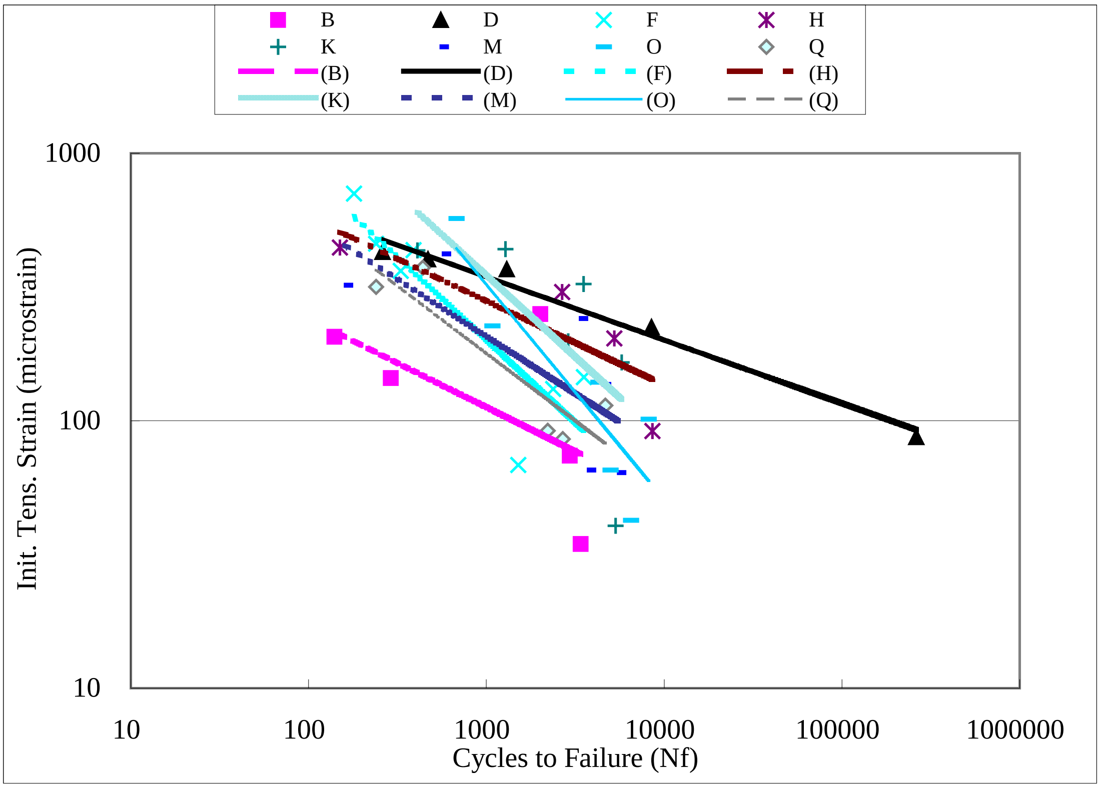
Group	Spec. No.	Stress (KPa)	Vert. Deform.(mm)	Stiff. M.(MPa)	Cycles to failure	Strain (ε_t) (microstrain)
B	B36-60	60	0.4	753.5	17660	144
	B35-70	70	0.5	560.5	3070	281
	B34-80	80	0.5	931	1930	172
	B12-90	90	0.6	796.5	760	237
	B9-100	100	0.7	504.5	560	411
	B37-110	110	0.7	1159	1570	216
	B38-120	120	0.8	465	310	575
D	D11-70	70	0.6	1246.5	53300	115
	D37-80	80	0.5	903.5	3570	169
	D38-90	90	0.6	1242	3180	189
	D36-100	100	0.7	584.5	310	365
	D33-110	110	0.8	1142	810	202
	D34-120	120	0.8	780	350	312
	D35-130	130	0.9	694	130	399
F	F37-70	70	0.5	903.5	11300	144
	F36-80	80	0.6	698.5	810	207
	F35-90	90	0.6	910	1210	245
	F32-100	100	0.7	742.5	1020	276
	F33-110	110	0.7	1024	1120	229
	F34-120	120	0.8	720	90	333
	F39-130	130	0.9	672	130	385
H	H34-70	70	0.5	258	250	615
	H7-80	80	0.6	516.5	410	303
	H33-90	90	0.7	731	760	262
	H9-100	100	0.7	553	130	370
	H28-110	110	0.8	240	50	890
K	K36-70	70	0.6	459.5	310	340
	k35-80	80	0.5	803	370	214
	k34-90	90	0.6	593	300	316
	K7-100	100	0.7	542.5	490	397
	K30-110	110	0.8	747	360	314
	K2-120	120	1	1081	450	203
	K31-130	130	0.9	534.5	30	507
M	M35-60	60	0.4	1587.5	94070	80
	M34-70	70	0.5	892.5	78560	176
	M33-80	80	0.6	864.5	3790	176
	M32-90	90	0.6	825	4270	194
	M31-100	100	0.7	1341	1610	160
	M29-110	110	0.8	719.5	350	278
	M30-120	120	0.9	687	340	355
	M38-130	130	0.9	668	210	464
O	O38-70	70	0.5	1581.5	25170	102
	O37-80	80	0.6	856.5	1760	200
	O34-90	90	0.6	1601	1610	118
	O33-100	100	0.7	1221	980	166
	O35-110	110	0.7	1082.5	1550	224
	O31-120	120	0.8	1309.5	850	202
	O32-130	130	0.9	1397	380	179
	O36-140	140	0.9	571.5	120	493
Q	Q36-50	50	0.4	823.5	8250	133
	Q35-60	60	0.4	497	1200	246
	Q32-70	70	0.5	1230	2330	102
	Q34-80	80	0.5	909.5	810	170
	Q38-90	90	0.7	964.5	330	194
	Q31-100	100	0.8	560.5	310	382
	Q5-110	110	0.8	780.5	700	277

The maximum fatigue stresses of each group range from 110 to 190 KPa for the soaked specimens, from 110 to 140 kPa for the freeze-thaw specimens. They are lower than those of dry specimens, which range from 250 to 310 kPa.

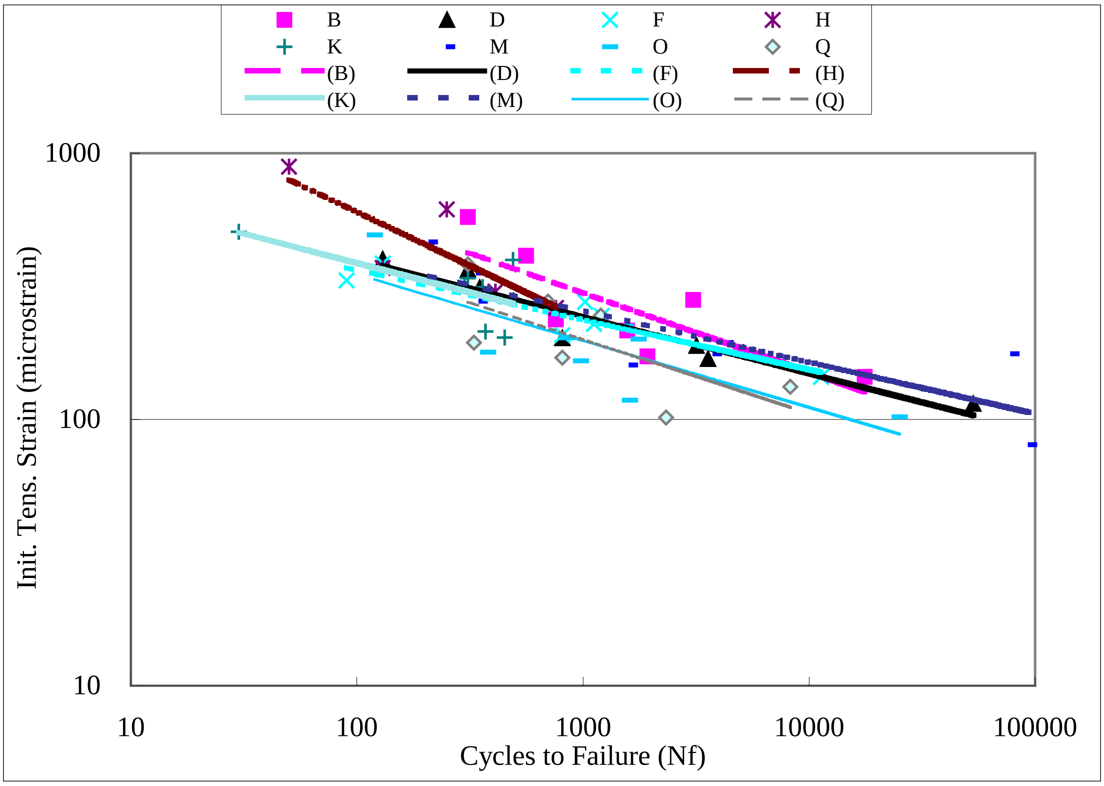
7.5.3.1 Fatigue equations

The stress-fatigue equation and the strain-fatigue equation, shown in Eq. 6.7 and Eq. 6.8, were also used to fit the fatigue data of FA mixes under the soaked and the freeze-thaw conditions. For each FA mix, fatigue equation's material parameters were established using the same method as used in the dry condition; i.e.: strains (ε_t) and cycles to failure (N_f) were firstly depicted in the log-log plot; then the least square fitting analysis was carried out for data of $\log_{10}(\varepsilon_t)$ and $\log_{10}(N_f)$.

The regressed fatigue lines of 8 FA mixes under the soaked and the freeze-thaw conditions are demonstrated in Fig.7.11. This figure shows fatigue lines of the soaked FA mixes (i.e. FA mixes under the soaked condition) deviate significantly from each other. Fatigue lines of the freeze-thaw FA mixes (i.e. FA mixes under the freeze-thaw condition) are closer to each other.



(a) Soaked



(b) Freeze-thaw

Fig.7 11 ITFT measurements for 8 FA mixes under the two moisture conditions

The material parameters of stress-fatigue equations and strain-fatigue equations of 8 FA mixes were obtained based on regression. Results, including k1, k2, k3, k4 and coefficients of determination (R-squires), are listed in Table 7.8. Group All denotes all data of 8 FA mixes.

Table 7. 8 Material parameters of strain-fatigue and stress-fatigue equations for 8 FA mixes at two moisture conditions

Group	$N_f=k1(\varepsilon_t)^{k2}$			$N_f=k3(\sigma_t)^{k4}$		
	k1	k2	R^2	k3	k4	R^2
Soaked condition						
B	2.98E+07	-2.123	0.5018	1.03E+11	-4.120	0.7775
D	5.39E+13	-4.228	0.9794	2.73E+32	-13.423	0.7769
F	5.27E+06	-1.617	0.7691	2.23E+12	-4.570	0.5705
H	6.33E+10	-3.187	0.7061	8.21E+14	-6.154	0.7491
K	1.49E+07	-1.641	0.4716	2.82E+09	-2.987	0.587
M	2.96E+08	-2.355	0.5734	1.17E+14	-5.233	0.6817
O	1.32E+06	-1.244	0.789	5.63E+11	-4.011	0.5947
Q	2.93E+07	-1.984	0.819	1.08E+16	-6.386	0.4374
All	4.40E+09	-2.837	0.4478	2.11E+18	-7.407	0.3289
Freeze-thaw condition						
B	2.02E+11	-3.356	0.668	8.27E+14	-6.035	0.7826
D	1.29E+14	-4.655	0.9136	5.20E+22	-9.843	0.8677
F	4.90E+15	-5.345	0.8649	3.61E+19	-8.375	0.7423
H	7.46E+08	-2.473	0.6902	1.25E+21	-9.625	0.3732
K	5.55E+13	-4.539	0.4379	2.59E+21	-9.542	0.2092
M	2.95E+15	-5.179	0.7212	2.24E+21	-9.107	0.9407
O	1.53E+12	-4.002	0.6474	1.84E+17	-7.057	0.8047
Q	1.98E+11	-3.608	0.5009	1.31E+12	-4.817	0.7167
All	2.02E+13	-4.329	0.654	8.40E+22	-10.142	0.4897

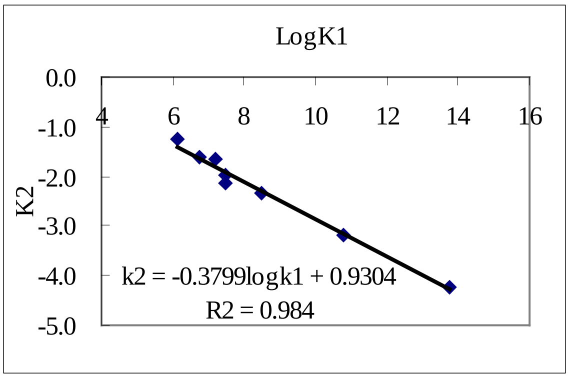
For Group All under each condition, strain-fatigue equation had higher R-square than stress-fatigue equation (see Table 7.8). Although these two R^2 are low, they can at least distinguish the good-of-fitness of the two equations. Hence the strain-fatigue equation (Eq. 6.7) had better good-of-fitness than the stress-fatigue equation, and was more suitable to describe the fatigue relationships of FA mixes than the latter equation. In this study, the strain-fatigue equation was adopted to characterize the fatigue properties of 8 FA mixes under the two moisture treatments. Table 7.8 also shows that R-square of Group All under the freeze-thaw condition is 0.654, higher than R-square (0.4478) under the soaked condition.

Under the soaked condition, k1 ranges from 1.23E+6 to 5.39E+13; k2 ranges from –4.228 to –1.244. Under the freeze-thaw condition, k1 and k2 vary from 7.45E+8 to 4.90E+15, and –5.345 to –2.473 respectively. Comparison of material parameters of 8 FA mixes under the two moisture conditions indicates that values of k1 of the freeze-thaw FA mixes are substantially larger than those of the soaked

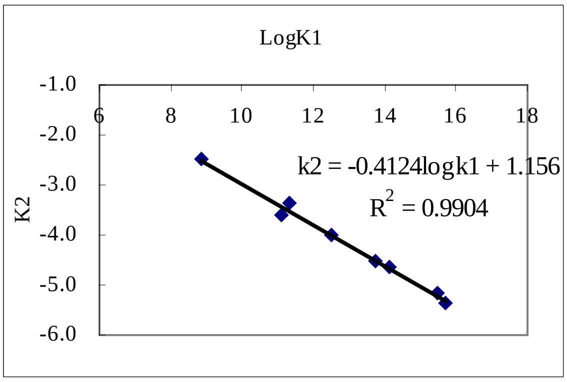
FA mixes; most of k2 of the freeze-thaw FA mixes are smaller than those of the soaked FA mixes.

Except for Group H, the freeze-thaw FA mixes have larger intercepts and sharper slopes (in the strain-fatigue scale plot, strain denotes horizontal coordinate, and fatigue denotes vertical coordinate) than the soaked FA mixes.

There are strong linear relationships between logk1 and k2 for the both conditions, shown in Eq. 7.10 and Eq. 7.11 respectively. Fig.7.12 demonstrates these two relationships.



(a) Soaked



(b) Freeze-thaw

Fig.7 12 Logk1 vs. k2 under the two moisture conditions

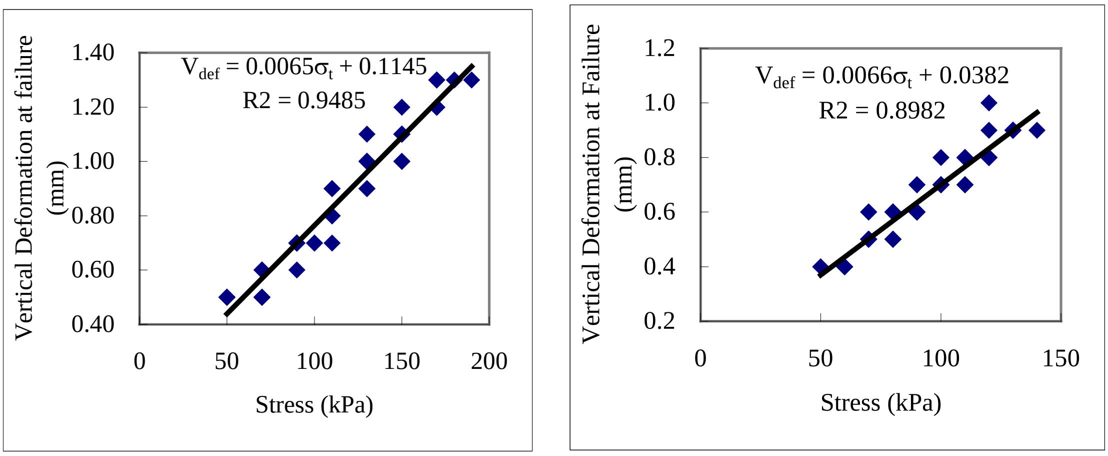
$$k2 = -0.3799 \cdot \log k1 + 0.9304$$ (R^2=0.984, for Soaked condition) (7.10)

$$k2 = -0.4124 \cdot \log k1 + 1.156$$ (R^2=0.9904, for Freeze-thaw condition) (7.11)

7.5.3.2. Deformation

Vertical deformations of specimens during ITFT also showed a very small increase rate as shown in the dry ITFT. Results of the maximum vertical deformation at failure of each FA mix range from 0.5 to 1.3 mm for the soaked specimens, and range from 0.4 to 1.0 mm for the freeze-thaw specimens (see Table 7.6 and Table 7.7).

The relationships between the vertical deformation at failure and fatigue stress under the two moisture conditions are shown in Fig. 7.13; and can be expressed with Eq. 7.12 and Eq. 7.13 respectively. There is a strong linear relationship (R-squares larger than 0.8982) between the vertical deformation at failure and the fatigue stress.



(a) Soaked (b) Freeze-thaw

Fig.7 13 Relation between vertical deformation at failure and stress under two moisture conditions

$$V_{Def} = 0.0065\sigma_t + 0.1145 \quad (R^2\text{=0.9485, for Soaked condition}) \quad (7.12)$$

$$V_{Def} = 0.0066\sigma_t + 0.0382 \quad (R^2\text{=0. 8982, for Freeze-thaw condition}) \quad (7.13)$$

where V_{Def} is the vertical deformation at failure (mm),

σ_t is the fatigue stress (kPa).

Relationships between the vertical deformation and cycles to failure (N_f) of 8 FA mixes under the moisture conditions are shown in Eq. 7.14 and Eq. 7.15. However coefficients of determination (R-square=0.1633 for the soaked condition and =0.4795 for the freeze-thaw condition) are low, and the variance between the vertical deformation and N_f is very large. These two equations are quite different from the result of the dry condition (linear equation, see Chapter 6); the relationship of the soaked specimens is the logarithmic, and the relationship of the freeze-thaw specimens is the power.

$$V_{Def} = -0.0667 \ln N_f + 1.3315 \quad (R^2\text{=0.1633 for Soaked condition}) \quad (7.14)$$

$$V_{Def} = 1.2635 N_f^{-0.0968} \quad (R^2\text{=0.4759 for Freeze-thaw condition}) \quad (7.15)$$

7.5.3.3 Nonlinear characteristics of the conditioned FA mixes

Stresses (σ_t) and initial strains (ε_t) of FA mixes obtained from the ITFT under the two moisture conditions were analyzed by the same method as used for the dry ITFT. To find the best relationships between stresses and strains, four methods, i.e., linear, logarithmic, power and exponential, were used to fit data of each group. R-squares of 8 FA mixes are shown in Table 7.9.

Table 7. 9 Fitting results for stresses and initial strains of FA specimens under two moisture conditions

Group	R-squares of Fitting				Equation adopted	
	Linear	Logarithmic	Power	Exponential		
Soaked Condition					σ_t=k7.ε_t^{k8}	
					k7	k8
B	0.7017	0.8804	0.8855	0.6107	13.548	0.3896
D	0.655	0.6862	0.7308	0.6786	26.584	0.3022
F	0.8185	0.7788	0.7502	0.7588	16.276	0.366
H	0.9944	0.9635	0.9925	0.969	5.1058	0.5009
K	0.956	0.9154	0.9808	0.9047	8.043	0.4876
M	0.8181	0.8882	0.8942	0.7981	15.94	0.3878
O	0.4507	0.7686	0.7814	0.4435	24.646	0.3157
Q	0.8323	0.8468	0.7843	0.7599	15.704	0.3766
All	0.6247	0.6793	0.6799	0.5739	16.018	0.3686
Freeze-thaw Condition					σ_t=k7'Ln(ε_t)-k8'	
					k7'	k8'
B	0.5021	0.5114	0.4948	0.4656	31.928	87.786
D	0.6919	0.739	0.7256	0.666	46.387	154.44
F	0.8481	0.8367	0.8503	0.8354	61.265	238.04
H	0.1373	0.0844	0.0512	0.0954	8.984	-35.362
K	0.1239	0.0677	0.0567	0.105	17.396	-0.0358
M	0.8261	0.8357	0.8272	0.7674	41.449	126.32
O	0.4975	0.578	0.5476	0.4463	39.244	100.64
Q	0.4164	0.4102	0.3718	0.3785	30.661	82.024
All	0.234	0.3272	0.3268	0.2248	26.142	48.758

For four fittings, most groups under the soaked condition have highest R-squares (in the blue shaded cells) when the power equation is used. Hence the power equation is the best for characterizing the stress-strain relationship of the soaked FA mixes in the ITFT. However, half of FA mixes under the freeze-thaw condition have the highest R-squares when the logarithmic fitting is used, and two out of eight R-squares of the logarithmic fitting are slightly lower than the largest R-squares of the power (or linear) fitting. For all data (Group All), the logarithmic fitting has the largest R-square. Hence, the logarithmic equation is most suitable to characterize the stress-strain relationships of the freeze-thaw FA mixes. Results under the two moisture conditions are shown in Eq. 7.16 and Eq.7.17.

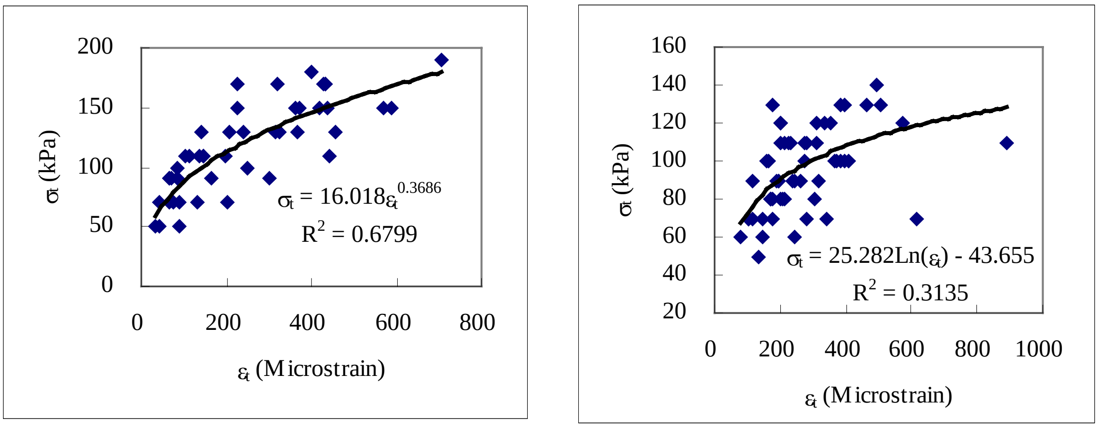
$$\sigma_t = k7\varepsilon_t^{k8} \qquad \text{(for soaked condition)} \qquad (7.16)$$

$$\sigma_t = k7' Ln(\varepsilon_t) - k8' \qquad \text{(for freeze-thaw condition)} \qquad (7.17)$$

where k7, k8, k7' and k8' are coefficients, their values are listed in Table 7.9. σ_t and ε_t are stress and initial strain. Ln denotes the natural logarithm.

For the soaked FA mixes, k8 varies from 0.26 to 0.54, and has the same magnitude as that of the dry FA mixes. However, k7 varies from 5.10 to 26.58, and is smaller than that of the dry FA mixes on average. Therefore the stress-strain relationships of the three conditioned FA mixes are different, this may be due to moisture conditioning methods.

Fig. 7.14 shows scatter plots of all stresses and strains of 8 FA mixes and their regressed curves. The nonlinear characteristics between stresses and strains are clear. R-square of power equation of FA mixes under the soaked condition is 0.6799, larger than that (0.3135) of logarithmic equation of FA mixes under the freeze-thaw condition. For the soaked FA mixes, when σ_t is lower than 200 kPa, the regressed curve is apparently nonlinear, this finding is the same as that of the dry FA mixes (see Chapter 6). For the freeze-thaw FA mixes, there is a clear nonlinearity when stress is lower than 400 kPa.



(a) Soaked (b) Freeze-thaw

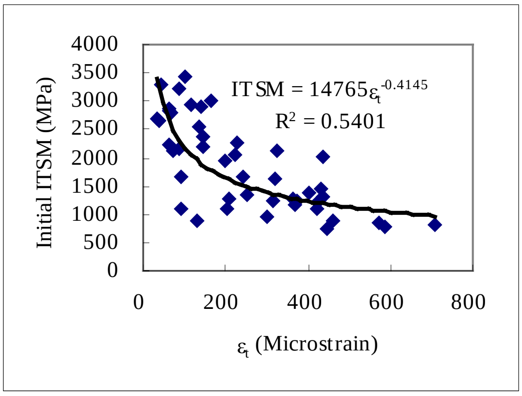
Fig.7 14 Relationships between stresses and initial strains under the two moisture conditions

ANOVA (α=0.05) was conducted in order to investigate the effects of bitumen type, RAP type and RAP content on the non-linearity of the FA mixes. Results are shown in Table 7.10. The responses are k7, k8, k7' and k8'. All probabilities of three factors in four responses' models are larger than 0.05. Hence three factors have no significant evidence of effects on nonlinear equations of the FA mixes under the two moisture conditions

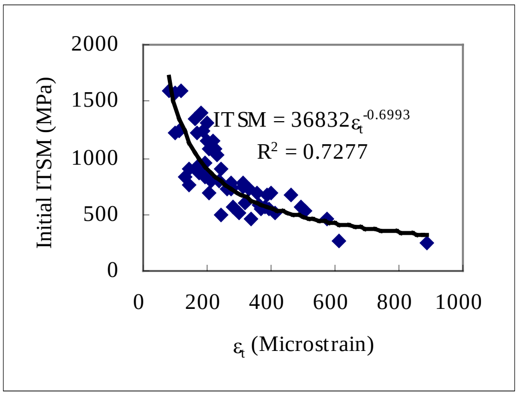
Relationships between the initial ITSMs and initial fatigue tensile strains of FA mixes under the two moisture conditions were fitted by four regression methods. The power regression was found to be most suitable to fit the data (see Fig 7.15). The R-squares are 0.5401 for the soaked FA mixes and 0.7277 for the freeze-thaw FA mixes.

Table 7. 10 ANOVA for nonlinear stress-strain equations under the two moisture conditions

Source	Degree of Freedom	Sum Squares	Mean Squares	F-value	Probability
(a) The soaked condition					
General Linear Model: k7 versus Bitumen, RAP Type, RAP Content					
Bitumen	1	0.99	0.99	0.01	0.923
RAP	1	0.71	0.71	0.01	0.935
%RAP	1	0.08	0.08	0	0.977
Error	4	372.58	93.15		
Total	7				
General Linear Model: k8 versus Bitumen, RAP Type, RAP Content					
Bitumen	1	0.00001	0.00001	0	0.975
RAP	1	0.000008	0.000008	0	0.978
%RAP	1	0.000009	0.000009	0	0.976
Error	4	0.035782	0.008945		
Total	7				
(b) The freeze-thaw condition					
General Linear Model: k7' versus Bitumen, RAP Type, RAP Content					
Bitumen	1	49.1	49.1	0.11	0.757
RAP	1	1.1	1.1	0	0.962
%RAP	1	62.5	62.5	0.14	0.727
Error	4	1780.6	445.1		
Total	7				
General Linear Model: k8' versus Bitumen, RAP Type, RAP Content					
Bitumen	1	2310	2310	0.19	0.682
RAP	1	35	35	0	0.959
%RAP	1	1225	1225	0.1	0.765
Error	4	47677	11919		
Total	7				



(a) Soaked



(b) Freeze-thaw

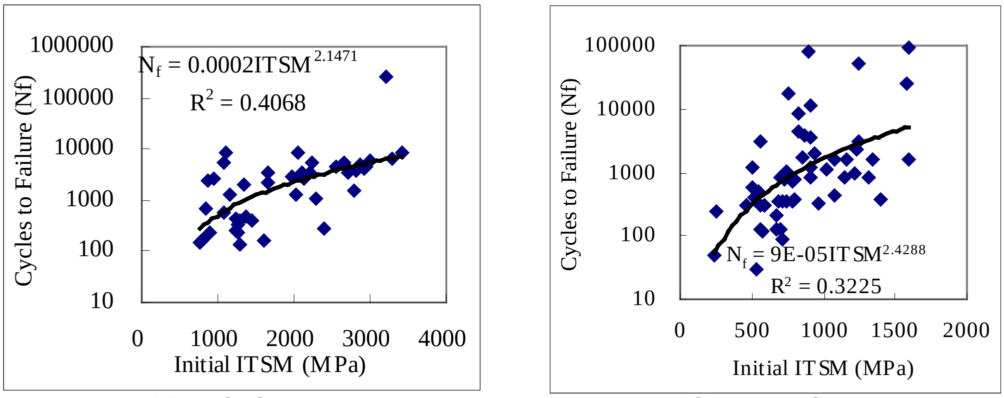
Fig.7 15 Relationships between initial ITSMs and fatigue tensile strains under the two moisture conditions

Eq.7.18 and Eq. 7.19 are the regressed power equations of the two moisture-conditioned FA mixes. Among the three moisture conditions, the dry FA mixes has the highest coefficient (77242, see Eq. 6.18), and the soaked FA mixes has the smallest result (14765). Under the dry and the freeze-thaw conditions, values of the power of the fitted equations are almost the same.

$$ITSM = 14765\varepsilon_t^{-0.4145} \quad (R^2=0.5401 \quad \text{for soaked condition}) \quad (7.18)$$

$$ITSM = 36832\varepsilon_t^{-0.6993} \quad (R^2=0.7277 \quad \text{for freeze-thaw condition}) \quad (7.19)$$

The relationships between cycles to failure and initial ITSMs are demonstrated in Fig.7.16. These relationships can be described using the power equation shown as Eq. 7.20 and Eq. 7.21. It is clear that fatigue life increases with an increasing of initial ITSM. Hence FA mix with higher initial ITSM may have long fatigue life. This finding is consistent to the result from the analysis of dry ITFT.



(a) Soaked (b) Freeze-thaw

Fig.7 16 Cycles to Failure vs. Initial ITSM under the two moisture conditions

$$N_f = 0.0002 ITSM^{2.1471} \quad (R^2=0.4068 \quad \text{for soaked condition}) \quad (7.20)$$

$$N_f = 9 \times 10^{-5} ITSM^{2.4288} \quad (R^2=0.3225 \quad \text{for freeze-thaw condition}) \quad (7.21)$$

7.5.3.4 Effects of Materials on fatigue properties under the moisture conditions

1. Statistical analysis

ANOVA was carried out using general linear model (GLM) to see if there were effects of factors on intercept and slope of the fatigue equations caused by variations from the RAP content, RAP type and bitumen type. The RAP content included two levels: i.e., 20% and 60%. RAP type had two levels: RAP#1 and RAP#2. Bitumen type included two levels: Shell 60 and Shell 100. The responses were the two material parameters: logk1 and k2. The α level of 0.05 was selected.

Table 7.11 shows the results of ANOVA. Under the freeze-thaw condition all probabilities in k2 and logk1 models are larger than the selected α. Hence the three factors and their interactions have no significant effects on the fatigue equation's material parameters of the freeze-thaw conditioned FA mixes.

Table 7. 11 ANOVA for K2 and logk1 of fatigue equations

Source	Degree of Freedom	Sum Squares	Mean Squares	F-value	Probability
(a) Soaked condition					
General Linear Model: k2 versus Bitumen, RAP and %RAP					
Bitumen	1	1.9316	1.9316	53.86	0.018
RAP	1	0.6699	0.6699	18.68	0.05
%RAP	1	3.2883	3.2883	91.69	0.011
Bitumen *%RAP	1	0.6166	0.6166	17.19	0.054
Bitumen *RAP	1	0.0759	0.0759	2.12	0.283
Error	2	0.0717	0.0359		
Total	7				
General Linear Model: Logk1 versus Bitumen, RAP and %RAP					
Bitumen	1	11.2744	11.2744	19.01	0.049
RAP	1	4.1179	4.1179	6.94	0.119
%RAP	1	21.0645	21.0645	35.51	0.027
Bitumen *%RAP	1	7.3969	7.3969	12.47	0.072
Bitumen *RAP	1	0.3303	0.3303	0.56	0.533
Error	2	1.1863	0.5931		
Total	7				
(b) Freeze-thaw condition					
General Linear Model: k2 versus Bitumen, RAP and %RAP					
Bitumen	1	0.281	0.281	0.2	0.674
RAP Type	1	0.662	0.662	0.48	0.526
%RAP	1	0.22	0.22	0.16	0.709
Error	4	5.489	1.372		
Total	7				
General Linear Model: Logk1 versus Bitumen, RAP and %RAP					
Bitumen	1	1.018	1.018	0.13	0.734
RAP	1	5.199	5.199	0.68	0.457
%RAP	1	1.74	1.74	0.23	0.659
Error	4	30.774	7.693		
Total	7				

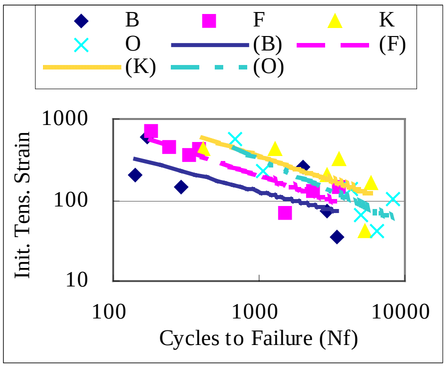
Note: %RAP = percentage of RAP added; * = interaction of factors; Probability less than 0.05 is considered significant.

Under the soaked condition, the three factors significantly affects the slope of the fatigue equation because their probabilities of F-test are smaller than 0.05; bitumen and RAP content significantly affects the fatigue equation’s intercept.

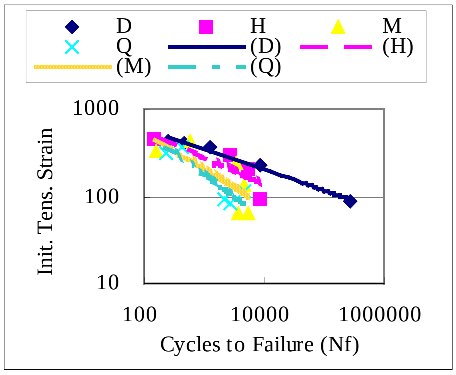
2. Discussion

ITFT data under the two conditions are further presented graphically in Fig. 7.17 and Fig. 7.18 in terms of the influencing factors

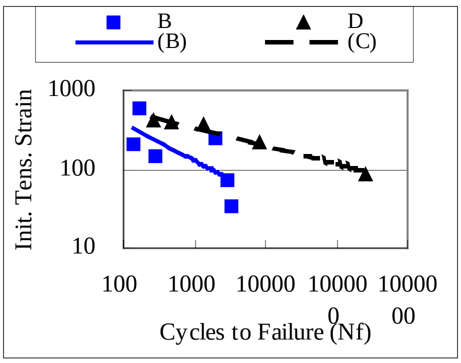
Under the soaked condition, when 60% RAP material was added into the FA mixes, mixes stabilized by Shell 60 (Groups D and H) had better fatigue properties than those stabilized by Shell 100 (Groups M and Q) (see Fig. 7.17 (b)). Results are opposite when 20% RAP materials was added into the FA mixes, i.e. fatigue properties of mixes stabilized by Shell 100 (Groups K and O) were better than those of mixes stabilized by Shell 60 (Groups B and F) (see Fig. 7.17 (a)). Hence, when Shell 60 bitumen is used to treat FA mixes, mixes containing 60% RAP have better fatigue properties under the soaked condition. This is maybe due to higher adhesion of the matrix mortar provided by Shell 60 bitumen than Shell 100 bitumen and higher interlock among the mixture provided by more aged RAP materials. Fig. 7.17 (c) to Fig.7.17 (f) further confirm the above explanation.



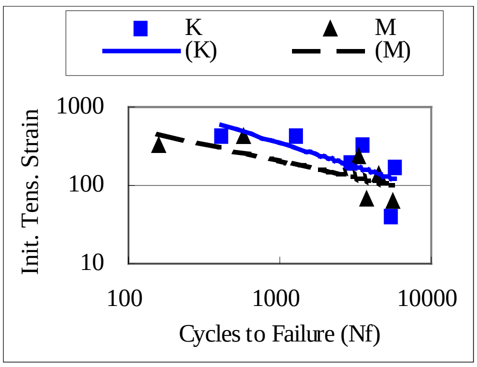
(a) 20% RAP



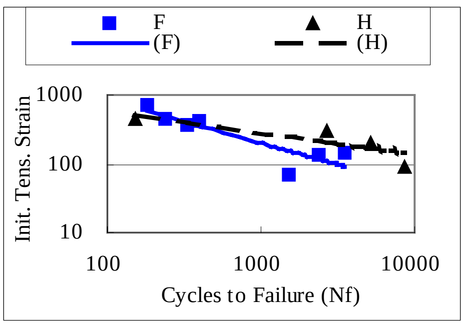
(b) 60% RAP



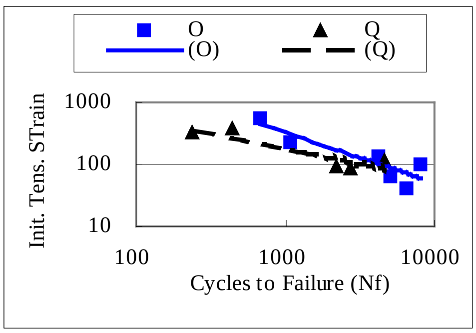
(c) Shell 60 +RAP#1



(d) Shell 100 + RAP#1



(e) Shell 60 +RAP#2

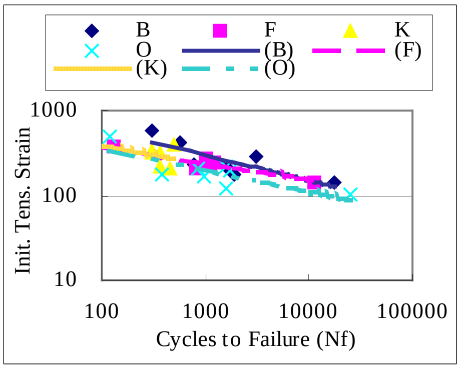


(f) Shell 100 + RAP#2

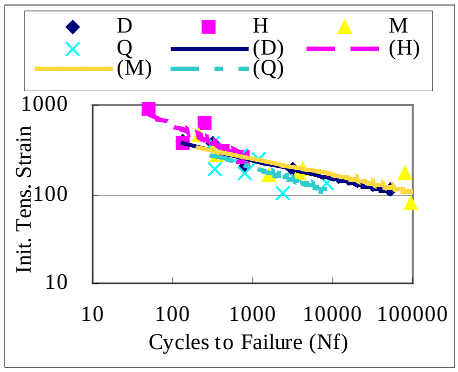
Fig.7 17 Effects of materials on fatigue lines (Soaked condition)

Above findings indicate that fatigue properties of the soaked FA mixes are affected by bitumen type, RAP type and RAP content. When RAP content is small, bitumen with larger penetration grade (e.g. Shell 100) is advantageous to fatigue properties of FA mixes; on the contrary, bitumen with smaller penetration grade (e.g. Shell 60) is advantageous to FA mixes' fatigue properties when RAP content is larger. This result is completely different from the conventional engineering judgment, i.e.: softer bitumen is advantageous to the HMA's fatigue properties under dry condition (SHRP-A-404 1994, Matthews et al 1993, Tayebali et al (1994). Therefore, under the rainy condition in southern China, Shell 60 bitumen is more advantageous to the fatigue properties of FA mixes than Shell 100 bitumen when more RAP materials are used.

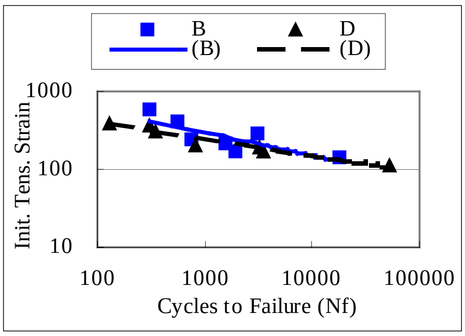
In Fig.7.18, fatigue lines cross over or lap over each other. It is difficult to find a clear relationship between factors and fatigue lines from this figure.



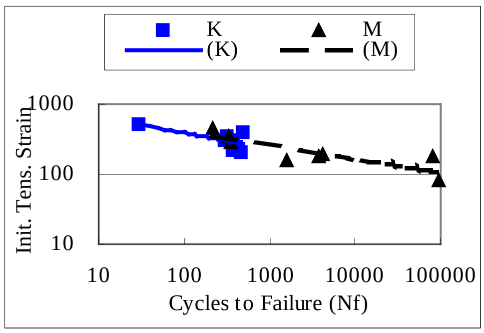
(a) 20% RAP



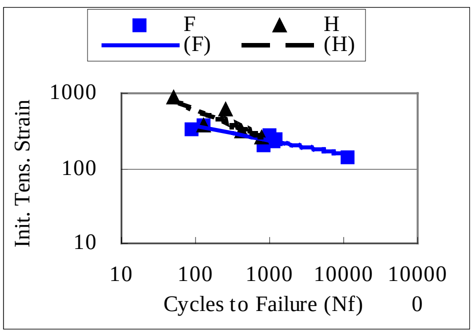
(b) 60% RAP



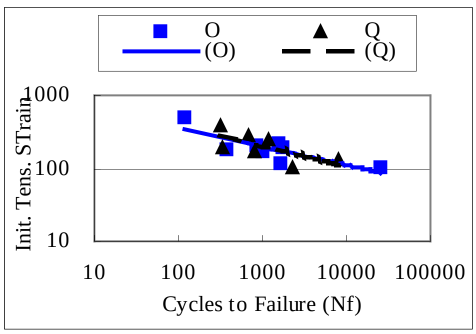
(c) Shell 60 +RAP#1



(d) Shell 100 + RAP#1



(e) Shell 60 +RAP#2



(f) Shell 100 + RAP#2

Fig.7 18 Effects of materials on fatigue lines (Freeze-thaw condition)

7.5.3.5 Effects of conditioning methods on fatigue properties

1. Fatigue lives at 100 microstrains

Generally, fatigue life at 100 microstrains (N_{f100}) can be used as a performance indicator for road base materials. Fig. 7.19 demonstrates fatigue lines of 8 FA mixes under the three conditions, which are drawn based on the material parameters listed in Table 6.5 and Table 7.8. However, it is difficult to compare 8 FA mixes due to many lines in one plot and crossover of these lines.

Table 7.12 tabulates fatigue lives at 100 microstrains of the three conditioned FA mixes, which were predicted based on strain-fatigue equations. For each condition, fatigue lives at 100 microstrains of 8 FA mixes were ranked. Ranking results are as follows:

For dry ITFT: F>H>O>D>K>M>Q>B;
For soaked ITFT: D>H>K>M>O>Q>F>B;
For freeze-thaw ITFT: M>F>D>K>B>O>Q>H.

Three rankings are different. However, the dry ITFT's ranking is somewhat identical to the soaked ITFT's. The freeze-thaw ITFT' ranking is apparently different from results of the dry ITFT and the soaked ITFT.

For each group, ratios of fatigue lives at 100 microstrains under the soaked and the freeze-thaw conditions to those under the dry condition were calculated and listed in Table 7.12. The higher the fatigue life ratio is, the better the moisture susceptibility to fatigue is.

Under the freeze-thaw condition, Groups F, H, O and Q have the shortest fatigue life ratios among 8 FA mixes, which are 108.72%, 11.22%, 31.33% and 76.98% respectively. Hence FA mixes containing RAP#2 have poor moisture susceptibility to fatigue under freeze-thaw condition whichever Shell 60 or Shell 100 is used as the foamed bitumen.

Under the soaked condition, fatigue lives of Groups B, F, O and Q are respectively 17.24%, 3.34%, 8.88% and 20.10% of the corresponding dry groups. These four groups have the shortest fatigue life ratios among 8 FA mixes. Three fourths of mixes containing 20%RAP materials have the shortest fatigue life ratios. It implies that under soaked condition FA mixes containing 60% RAP materials have longer fatigue lives than those mixes containing 20% RAP materials.

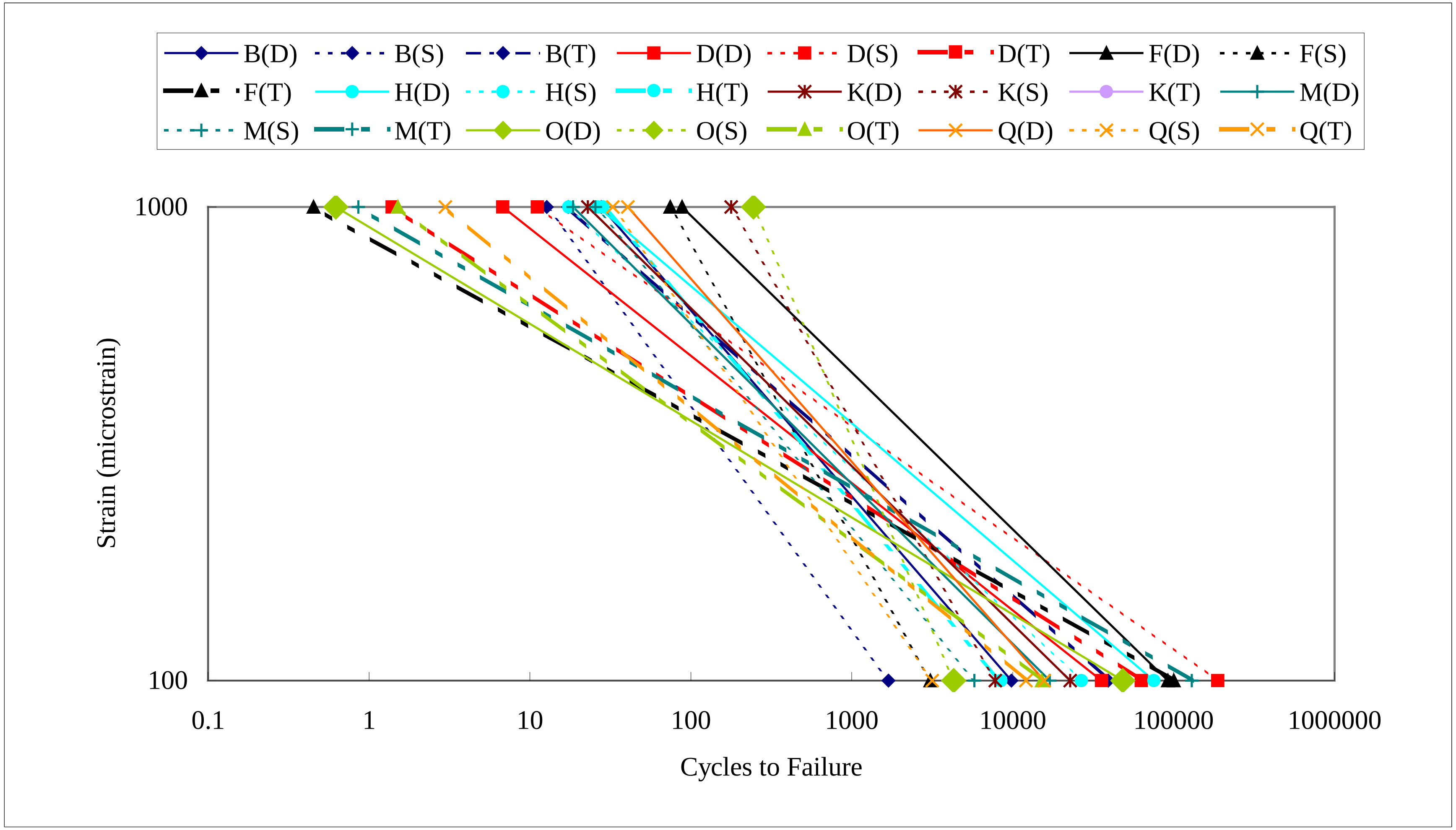


Fig.7 19 Fatigue lines of 8 FA mixes under the three conditions

Note: Letter "D", "S", "T" in the parentheses denote the dry condition, soaked condition and freeze-thaw condition respectively.

Table 7. 12 Fatigue lives ($Nf_{100)}$ at 100 microstrains for 8 FA mixes under the three conditions

Group	Dry ITFT				Soaked ITFT					Freeze-thaw ITFT				
	$N_f=k1(\varepsilon_t)^{k2}$			Cycles at 100 μms	$N_f=k1(\varepsilon_t)^{k2}$			Cycles at 100 μms	FLR_s (Ratio of soaked to dry) (%)	$N_f=k1(\varepsilon_t)^{k2}$			Cycles at 100 μms	FLR_f (Ratio of freeze-thaw to dry) (%)
	k1	k2	R^2		k1	k2	R^2			k1	k2	R^2		
B	1.23E+09	-2.548	0.7485	9.81E+03	2.98E+07	-2.123	0.8631	1.69E+03	17.24	2.02E+11	-3.356	0.668	3.93E+04	400.48
D	9.82E+11	-3.720	0.9196	3.56E+04	5.39E+13	-4.228	0.7106	1.88E+05	529.15	1.29E+14	-4.655	0.9136	6.31E+04	177.11
F	1.00E+11	-3.018	0.8193	9.21E+04	5.27E+06	-1.617	0.64	3.08E+03	3.34	4.90E+15	-5.345	0.8649	1.00E+05	108.72
H	6.10E+11	-3.454	0.8844	7.54E+04	6.33E+10	-3.187	0.5852	2.68E+04	35.52	7.46E+08	-2.473	0.6902	8.45E+03	11.22
K	2.25E+10	-2.998	0.9838	2.28E+04	1.49E+07	-1.641	0.9096	7.80E+03	34.25	5.55E+13	-4.539	0.4379	4.63E+04	203.29
M	1.42E+10	-2.962	0.8025	1.70E+04	2.96E+08	-2.355	0.7484	5.78E+03	34.04	2.95E+15	-5.179	0.7212	1.29E+05	762.95
O	2.91E+14	-4.890	0.8392	4.84E+04	1.32E+06	-1.244	0.8705	4.29E+03	8.88	1.53E+12	-4.002	0.6474	1.52E+04	31.33
Q	2.34E+09	-2.587	0.6717	1.57E+04	2.93E+07	-1.984	0.5385	3.16E+03	20.10	1.98E+11	-3.608	0.5009	1.21E+04	76.98
Ranking(N_{f100} at 100μms)	F>H>O>D>K>M>Q>B				D>H>K>M>O>Q>F>B					M>F>D>K>B>O>Q>H				

In Table 7.12, ratios of the moisture-conditioned to the unconditioned (dry) in the shaded cells are larger than 100%. Under freeze-thaw condition, the fatigue ratios of Groups B, D, F, K and M are larger than 100%, ranging from 108.72% to 762.95%. Under the soaked condition, only Group D's fatigue ratio (529.15%) is larger than 100%. Hence Group D has good moisture susceptibility to fatigue under the two moisture conditions.

Under the soaked condition, all group's fatigue lives significantly reduce except for Group D. Under the freeze-thaw condition, however, five groups out of eight have higher fatigue lives than the corresponding groups under the dry condition. Group M has the highest ratio, up to about 760%. Hence there is a huge difference between the soaked ITFT and the freeze-thaw ITFT.

Comparison between material parameters of fatigue equations under the two moisture conditions indicates that intercepts of the freeze-thaw fatigue equations are higher than those of the soaked fatigue equations. This may be due to the pore water pressure caused by the freeze-thaw treatment. It has been argued that Lottman procedure was too severe because the warm water soaking of the vacuum saturated and frozen specimen could develop internal water pressure (Hunter and Ksaibati 2002).

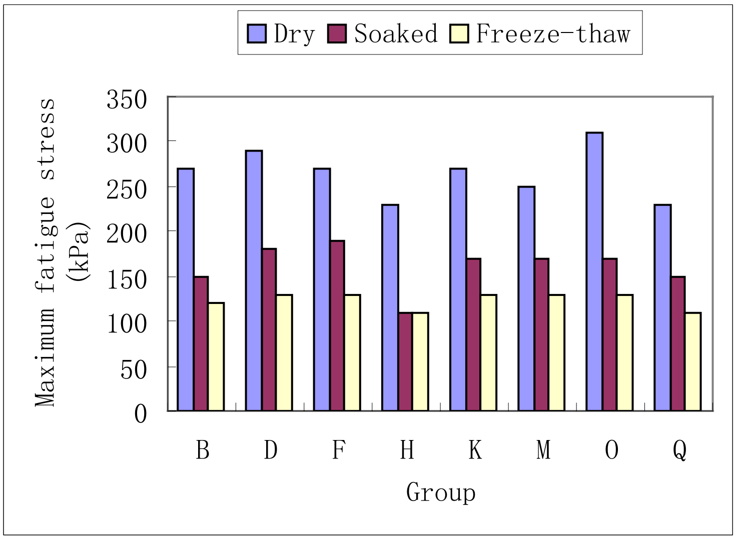
A conclusion can be drawn from above analysis: the soaked ITFT is better than the freeze-thaw ITFT because the former can efficiently distinguish the moisture susceptibility of FA mixes to fatigue and isn't apt to be affected by the internal water pressure. From the view of the operation, the procedure of the former is easier than that of the freeze-thaw ITFT. Therefore, the soaked ITFT is recommended for determining the moisture susceptibility of FA mixes to fatigue.

2. Maximum fatigue stresses

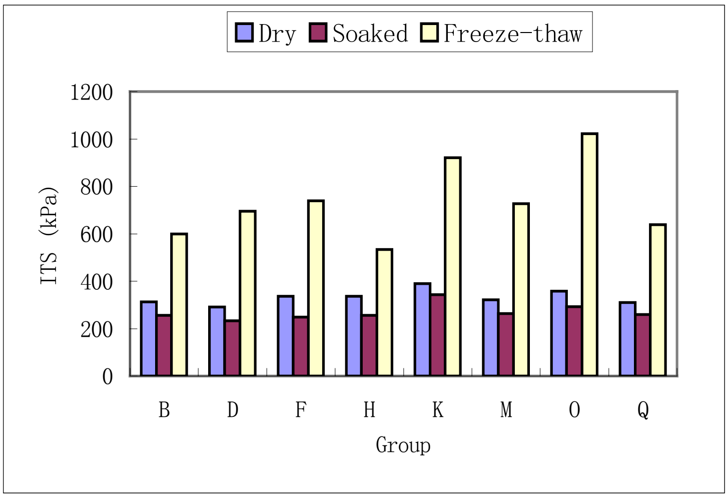
Table 7.13 summarizes the maximum fatigue stresses of FA mixes under the three conditions. In order to compare ITS results of FA mixes under three conditions, the ITS of FA mixes under freeze-thaw condition was conducted. ITS results under three conditions are also listed in Table 7.13.

Table 7. 13 Results of maximum fatigue stresses and ITSs under the three conditions

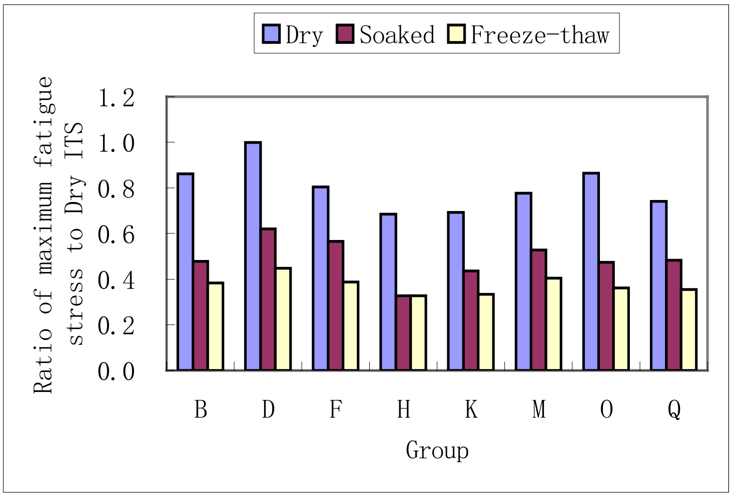
Group	ITS (kPa)			Maximum fatigue stress in ITFT (kPa)			Ratio of Maximum fatigue stress to ITS		
	Dry	Soaked	Freeze-thaw	Dry	Soaked	Freeze-thaw	Dry	Soaked	Freeze-thaw
(a)	(b)	(c)	(d)	(e)	(f)	(g)	(h)=(e)/(b)	(i)=(f)/(b)	(j)=(g)/(b)
B	313.14	256.51	598.74	270	150	120	0.862	0.479	0.383
D	290.34	233.34	695.50	290	180	130	0.999	0.620	0.448
F	335.38	249.03	739.36	270	190	130	0.805	0.567	0.388
H	335.4	256.35	534.27	230	110	110	0.686	0.328	0.328
K	389.66	343.02	920.33	270	170	130	0.693	0.436	0.334
M	321.35	263.23	726.95	250	170	130	0.778	0.529	0.405
O	358.46	292.82	1023.17	310	170	130	0.865	0.474	0.363
Q	310.54	258.94	638.34	230	150	110	0.741	0.483	0.354



(a) Maximum fatigue stresses under the three conditions



(b) ITSs under the three conditions



(c) Ratios of maximum fatigue stresses to dry ITSs

Fig.7 20 Maximum fatigue stresses and ITSs under the three conditions

These results are also demonstrated in Fig. 7.20. Under the dry condition, the maximum fatigue stresses of FA mixes ranged from 230 to 310 kPa; under the soaked condition, the stresses reduced, ranging from 110 to 190 kPa. Under the freeze-thaw condition, the maximum fatigue stresses were the smallest, ranging from 110 to 130 kPa. It is clear that moisture conditioning will lower maximum fatigue stresses which FA mixes can bear. The harsher the conditioning is, the lower the maximum fatigue stresses are (see Fig. 7.20 (a)).

Fig.7.20 (b) illustrates ITS results under three conditions. Compared with ITSs under the dry condition, ITSs of FA mixes reduced after soaking treatment; however, it's unexpected that ITSs of FA mixes under the freeze-thaw condition increased significantly after the freeze-thaw treatment, which are higher than those under dry condition. This is maybe due to the water pore pressure caused by a cycle of freeze-thaw treatment, because air voids of FA mixes are very large (ranging from 10% to 15%).

Ratios of the maximum fatigue stresses under the three conditions to ITSs under the dry condition were also calculated and listed in Table 7.13. Fig.7.20 (c) clearly illustrates the effect of the moisture condition on the maximum fatigue stress. After moisture treatment, the maximum stresses which FA mixes can support reduce; ratios of the maximum fatigue stress to dry ITS are lower than 0.62 for the soaked condition, and lower than 0.45 for the freeze-thaw condition. The maximum fatigue stress which FA mixes can bear under the freeze-thaw condition is the lowest. This result explains why pavement is apt to fail after the freeze-thaw condition in the spring season. Hence the pavement state under this situation is defined as a worst environment in some countries, e.g. China (DOC JTJ014 1997).

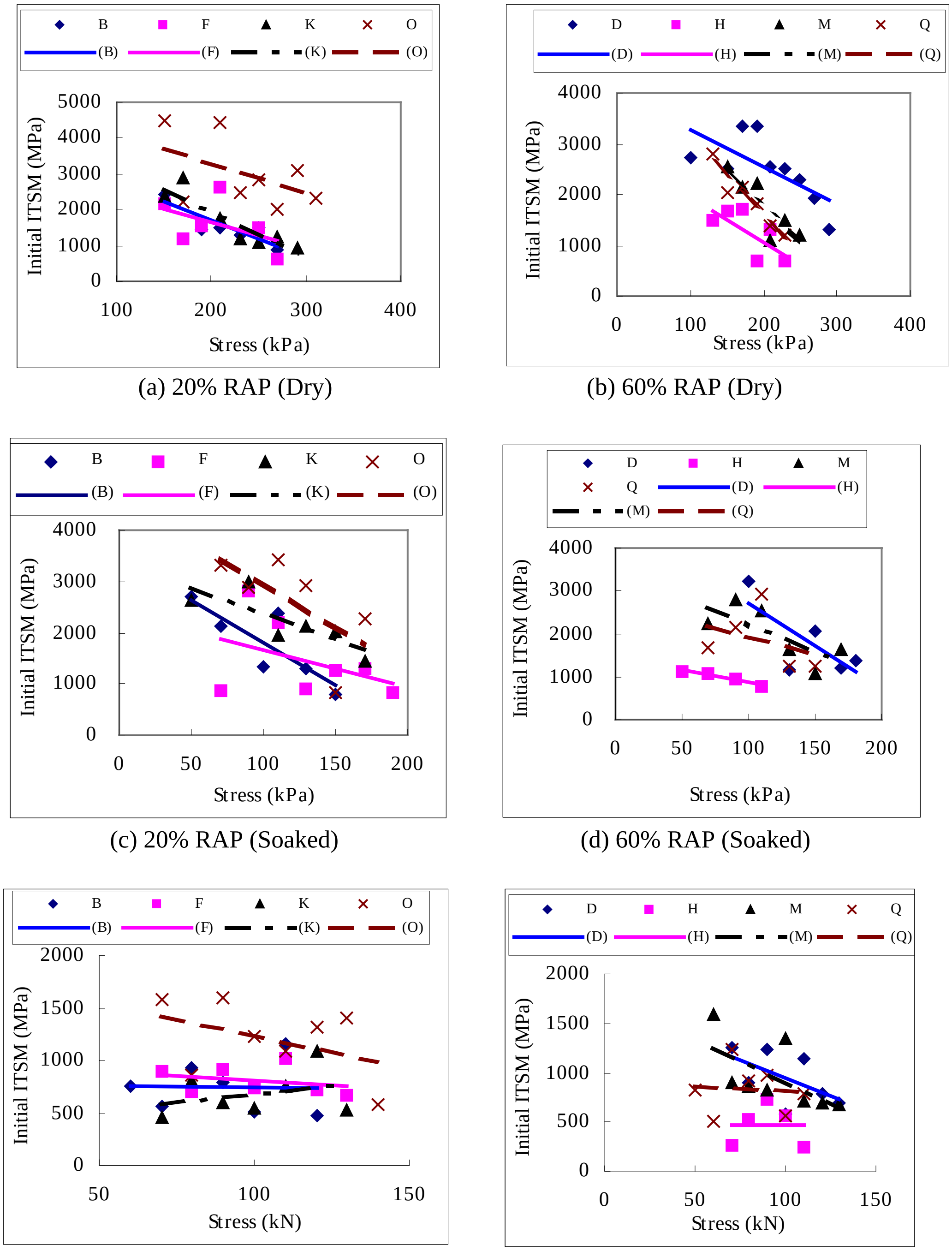
Among 8 FA groups, Groups H and K have the smallest ratios of maximum fatigue stresses to dry ITSs.

3. Initial ITSMs

The initial ITSM, which reflects the basic mechanic property of FA mixes, was tested using indirect tensile stiffness test (ITST) before the ITFT and used to calculate the initial strain of FA mixes during ITFT.

ITST data of 8 FA mixes under the three conditions were depicted in the scatter plots. Each FA mix had at least 5 initial ITSMs (see stiffness moduli in Tables 6.4, 7.6 and 7.7). ITST data of each mix were fitted by regression method. It's found that the best good-of-fitness was obtained when linear regression was used.

Fig.7.21 demonstrates regressed line of each mix under the three conditions. When 20% RAP materials is added into the FA mixes, Group O has the highest initial ITSMs under the three conditions. When 60% RAP materials is added into the FA mixes, Group D achieves the highest initial ITSMs under the three conditions, on the contrary, Group H has the lowest ITSMs.



(a) 20% RAP (Dry)

(b) 60% RAP (Dry)

(c) 20% RAP (Soaked)

(d) 60% RAP (Soaked)

(e) 20% RAP (Freeze-thaw)

(f) 60% RAP (Freeze-thaw)

Fig.7 21 Initial ITSM vs. fatigue stress under three conditions

It is not clear why Group O, which contains 20% RAP#2 and is stabilized by Shell 100, can obtain high ITSM, while Group H, which contains 60% RAP#2 and is stabilized by Shell 60, only obtain the low ITSM. However Group D, which contains 60% RAP#1 and is stabilized by Shell 60, has high ITSM too. At this time, there is still no good explanation for this scenario.

For each condition, all data of 8 FA mixes were fitted by linear regression and depicted in the Fig.7.22. The following findings can be obtained from this figure:

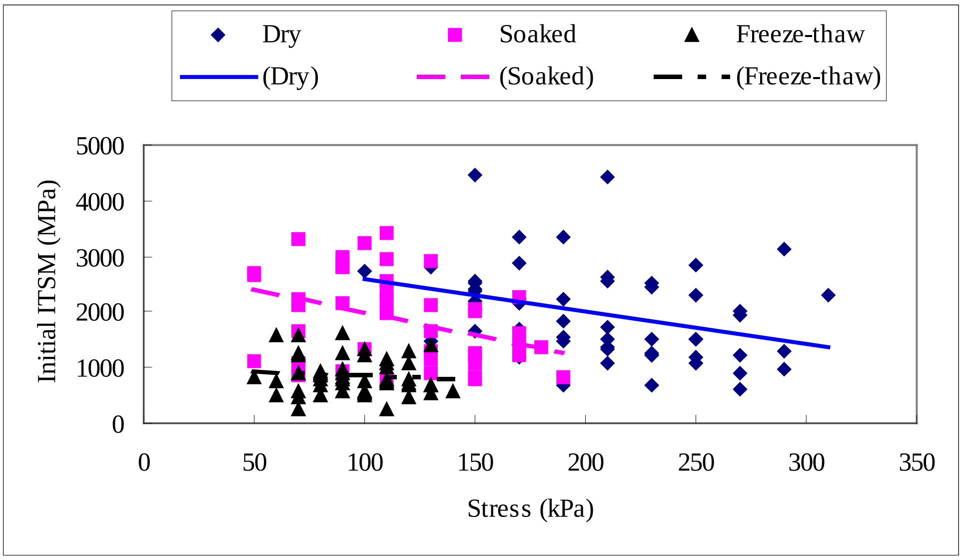


Fig.7 22 Fitted ITSM lines under the three conditions

(1) Slopes of fitted lines under the dry and the soaked conditions are larger than that under the freeze-thaw condition, that is to say the dry and the soaked conditions cause larger effect of the stress on decreasing rate of the initial ITSMs than the freeze-thaw condition.

(2) The dry FA mixes have the largest range of the fatigue stresses followed by the soaked FA mixes. The range of the fatigue stresses under the freeze-thaw condition is the smallest.

On average, the harsher the conditioning is, the smaller the initial ITSM is. This is why the maximum fatigue stresses which FA mixes can bear are the smallest for the freeze-thaw condition, the second smallest for the soaked condition, and the highest for the dry condition.

The relationships between initial ITSM and stress under the three conditions can be express by Eq. 7.22 to Eq. 7.24. R-squares of these three equations are very small, especially for the freeze-thaw condition. Hence there is large variation of the initial ITSMs caused by the stresses.

$$ITSM = -5.9263\sigma_t + 3196.5 \quad (R^2=0.116 \text{ for dry condition}) \quad (7.22)$$

$$ITSM = -8.1075\sigma_t + 2806.8 \quad (R^2=0.1503 \text{ for soaked condition}) \quad (7.23)$$

$$ITSM = -1.6155\sigma_t + 1001.6 \quad (R^2=0.0124 \text{ for freeze-thaw condition}) \quad (7.24)$$

7.5.4 Summary

Analysis results of indirect tensile testing, dynamic creep testing and indirect tensile fatigue testing are summarized and listed in Table 7.14. ITS results of 14 FA groups are larger than 0.7, the cutoff of the moisture susceptibility. Group C has the highest ITSR value (0.91) and Group F has the lowest ITSR value (0.74). Hence all of the FA mixes meet the requirement of the moisture susceptibility to indirect tensile strength.

In fact, there is not much strength to “loose” for the hot mixes. FA mixes loose more strength than the hot mixes. This is due to the adhesion between aggregates and not fully coating of the larger aggregate. FA mix is binded by mortar. Theoretically, strength loose depends on many factors, e.g. mixing temperature, bitumen content, aggregate property, anti-stripping agent. For the hot mixes, strength will also greatly lose when the aggregate is inappropriately mixed or compacted.

Table 7. 14 Summary of moisture susceptibility parameters

	ITS	Permanent Deformation			ITFT				
							Ratio of the maximum fatigue stress to dry ITS		
Group	ITSR (%)	CSSR (%)	SCSMR (%)	USR (%)	FLR_s (Ratio of soaked to dry) (%)	FLR_f (Ratio of freeze-thaw to dry) (%)	Dry ITFT	Soaked ITFT	Freeze-thaw ITFT
A	0.75	1.126	0.743	1.16					
B	0.82	1.636	0.547	1.36	17.24	400.48	0.862	0.479	0.383
C	0.91**	1.136	0.494	0.83					
D	0.8	0.936	1.681	1.22	529.15	177.11	0.999	0.620	0.448
F	0.74**	2.254	2.242	0.74	3.34	108.72	0.805	0.567	0.388
G	0.83	1.733	0.394	1.46					
H	0.76	2.151	0.43	1.09	35.52	11.22	0.686	0.328	0.328
J	0.76	1.368	0.712	1.06					
K	0.88	1.434	0.62	0.99	34.25	203.29	0.693	0.436	0.334
L	0.83	1.45	0.639	1.06					
M	0.82	4.726	0.185	1.38	34.04	762.95	0.778	0.529	0.405
O	0.82	3.368	0.262	4.69	8.88	31.33	0.865	0.474	0.363
P	0.88	3.232	0.362	3.26					
Q	0.83	4.839	0.236	2.64	20.1	76.98	0.741	0.483	0.354

Note: Values in the pink-shaded cells represent good results; values in the turquoise-shaded cells represent poor results.

Table 7.14 shows that Group D has the smallest CSSR (0.936) followed by Group C. While Groups M, O, P and Q have larger CSSR. Group F has the largest SCSMR value followed by Group D. Groups M, O, P and Q have the smallest SCSMR values. The USRs of Groups C and Group F are the smallest, and results of Groups O, P and Q are the highest. USR of Group D is not very large among 14 FA groups.

Results of CSSR, SCSMR and USR indicate that Groups C, D and F have better moisture susceptibility to permanent deformation, whilst Groups O, P and Q have the poorest moisture susceptibility. It implies that except for Groups F and M, FA mixes containing RAP#1 material have better moisture susceptibility to permanent deformation than those mixes containing RAP#2 material. Among 14 FA groups, those mixes containing RAP#2 material and stabilized by Shell 100 bitumen have the poorest moisture susceptibility to permanent deformation.

Results of ITFT reflect that soaked FLR_s of Groups F and O are the smallest, and soaked FLR_s of Group D is the largest. Under the freeze-thaw condition, Groups H and O have the smallest FLR_f; whilst FLR_f results of Groups B, D, F, K and M are larger than 100%.

Table 7.14 shows that Group D has the highest ratios of the maximum fatigue stress to dry ITS under the dry, soaked and freeze-thaw conditions. Whilst, ratios of the maximum fatigue stress to dry ITS of Group H are the lowest under these three conditions. These results reflect that Group D has the strongest capacity to bear the fatigue stress; while Groups H has the poorest capacity to bear the fatigue stress.

Therefore, Group D has the longest fatigue life under the soaked condition and the strongest capacity to bear fatigue stress. The fatigue lives of Group O under the two moisture conditions (soaked and freeze-thaw) are short. Group H has the poor fatigue properties under the two moisture conditions.

In summary, all 14 FA groups meet the requirements of the moisture susceptibility if the Lottman's criterion is used to evaluate the FA mixes. FA mixes containing RAP#1 material have better moisture susceptibility to permanent deformation than those mixes containing RAP#2 material. Group D, which contains 60% RAP#1 and is stabilized by Shell 60, has the best fatigue properties under the soaked and freeze-thaw conditions. Due to the best overall performance under the moisture conditions, Group D can be recommended to use used in paving in the trial road in order to investigate its field performance.

7.6 CONCLUSION

In this study, a 24-hour-soaking method was selected to evaluate moisture susceptibility of FA mixes to strength and permanent deformation. ITS and permanent deformation results between the dry FA mixes and the soaked FA mixes were analyzed.

Two kinds of moisture-conditioning methods, i.e., freeze-thaw conditioning and 24-hour soaking conditioning, were selected to investigate effects of moisture and conditioning methods on fatigue properties. Fatigue equations of the soaked and the freeze-thaw FA mixes were established based on the ITFT; deformation and the nonlinear characteristics of the moisture-conditioned FA mixes were discussed; effects of the bitumen types, RAP types and RAP content on the fatigue properties under the two moisture conditions, and effects of moisture conditioning methods on fatigue properties of FA mixes were analyzed.

Based on results presented in this chapter, the following conclusions are given:

1. Indirect Tensile Strength

(1) Bitumen types and RAP content significantly affect the dry ITS and the soaked ITS. RAP type, however, insignificantly affects the dry and the soaked ITS.

(2) For both dry and soaked conditions, FA mixes stabilized by Shell 100 have higher ITS results than those stabilized by Shell 60.

(3) RAP material can help FA mixes to improve their moisture susceptibility to ITS. FA mixes have less moisture susceptibility to ITS than PA10, and have poorer result than AC20.

(4) All of 14 FA mixes meet the requirement of the moisture susceptibility to ITS in light of Lottman's criterion.

2. Permanent Deformation

(1) In dry condition, bitumen significantly affects permanent deformation of FA mixes. RAP content has significant effect on the soaked CSS, and the interaction of bitumen type and RAP type (or RAP content) also affects the dry US. In dry condition, SCSM is affected by three factors and their interactions.

(2) Under the dry condition, softer bitumen would result in less susceptibility of FA mixes to permanent deformation. Under the soaked condition, however, harder bitumen (Shell 60) is statistically superior to the softer bitumen (Shell 100) in reducing the moisture susceptibility of FA mixes to permanent deformation.

(3) Under the dry condition, more aged RAP material (RAP#2) would result in less susceptibility of FA mixes to permanent deformation. However, moisture severely affects the susceptibility of those mixes containing RAP#2 and stabilized by Shell 100 bitumen, their moisture susceptibilities to permanent deformation are very poor, although they have best resistance to permanent deformation in dry condition.

(4) On average, FA mixes containing RAP#1 material have better moisture susceptibility to permanent deformation than those mixes containing RAP#2 material.

3. Fatigue

(1) Fatigue relationships of the moisture-conditioned FA mixes can be characterized by the strain-fatigue equation. For the soaked FA mixes, bitumen types, RAP types and RAP content significantly affect the slope of the fatigue equation of the soaked FA mixes; bitumen types and RAP content have significant effect on the fatigue equation's intercept. For the freeze-thaw FA mixes, three factors and their interactions have no significant effect on the fatigue equation's material parameters.

(2) For the soaked and the freeze-thaw FA mixes, there is a clear nonlinear relationship between the stress and the strain. However, the moisture conditions affect the material parameters and types of the nonlinear stress-strain equations.

(3) The soaked ITFT is better than the freeze-thaw ITFT because the former can efficiently distinguish the moisture susceptibility of the moisture-conditioned FA mixes and its procedure is easier to operate.

(4) The capacity of 8 FA mixes to bear fatigue loading reduces when the moisture condition becomes harsher. The dry FA mixes have the largest fatigue stress range followed by the soaked FA mixes. The freeze-thaw conditioned FA mixes have the smallest fatigue stress range.

(5) Group D has the longest fatigue life under the soaked condition and the strongest capacity to bear fatigue stress. The fatigue lives of Group O under the two moisture conditions (soaked and freeze-thaw) are short. Group H has the poor fatigue properties under the two moisture conditions.

In summary, Group D, which contains 60% RAP#1 material and is stabilized by Shell 60, has the best fatigue properties under the moisture conditions, has the best moisture susceptibility to permanent deformation, its moisture susceptibility to tensile strength also meets the Lottman's requirement. Therefore, under moisture condition in southern China, this kind of FA mix can be recommended as the road base. It is recommended to use less RAP material (e.g. RAP#1) and harder bitumen (e.g. Shell 60) in this area in order to obtain better engineering properties of FA mix.

CHAPTER EIGHT SUMMARY, CONCLUSIONS AND RECOMMENDATION

8.1 INTRODUCTION

There were limited studies worldwide on Foamed Asphalt (FA) as a paving material. Combining recycled asphalt materials with FA not only is important to Hong Kong, but also has global significance in terms of preserving the environment and saving public money. This dissertation focuses on the application of Foamed Asphalt (FA) to roadway paving, and associated laboratory and analytical analysis in order to establish a recommendation for its usage as a base layer under the load carry surface layer. This chapter is organized in three parts. First of all, the background, framework, methodologies, as well as the creative efforts, of this study are summarized. Conclusions of the various analyses are given, including foamability of the bitumens, FA mix design, testing and evaluation of FA mixes. Finally recommendations are provided related to mix design, in-situ performance evaluation, experimental work, effects of the hydraulic materials, curing time and stockpile time on the FA mixes' properties. Research needs for further study are also discussed.

8.2 SUMMARY

Since Professor Csanyi proposed the bitumen's foaming method in the mid 1950's, the foamed bitumen process has been in development for more than 40 years. Foamed bitumen can be used as a stabilizing agent with a variety of materials ranging form good quality crushed stone, marginal gravels with relatively high plasticity to RAP material. It has been successfully applied on low traffic road, especially with marginal material in Australia and South Africa. Due to its poor abrasion and large voids, the cold-recycled mix stabilized by foamed bitumen is commonly used as base course or road base covered by a hot-mixed surfacing course.

FA mixes have many advantages. Cold recycling by foamed bitumen is a sustainable and economic way. Therefore, it provides a feasible method to solve Hong Kong's RAP problem of needing landfill areas when land is getting scare, especially for recycling a large amount of the RAP materials produced from the road rehabilitation.

So far, FA mix has never been applied in Hong Kong. Before using the RAP material by the cold-recycling method in the paving project, it is necessary to propose a FA mix design method and investigate the properties of FA mixes. The study is launched under this situation.

In Hong Kong, rainfalls are frequent and heavy in almost half year, and temperature is higher than 30 ^{0}C in summer. Fatigue properties, permanent deformation and moisture susceptibilities are critical to FA mixes used as the road base layer in Hong Kong. However, studies on fatigue properties, permanent deformation and moisture susceptibilities of FA mixes were not yet reported extensively. Therefore, FA mixes' properties should be investigated in detail for the benefit of immediate application in Hong Kong.

This study is aimed at proposing a FA mix design method and investigating properties of the cold-recycled RAP mixes stabilized by foamed bitumen. Tests of permanent deformation, fatigue property and moisture susceptibility were limited to only the laboratory due to time and resources limitation of conducting in-situ studies. The low-temperature properties and effects of curing time on FA mixes' properties were also excluded in this study.

A systematical study of the bitumen foamability was carried out. Various testing conditions were considered in the foaming test for two penetration-grade bitumens, Shell 60 and Shell 100. After tests and analysis, the most important foaming parameters, half-life and the maximum expansion ratio, for producing FA mixes were determined. Mechanism analysis of foamability difference between the two bitumens was performed, and effect of water content on this difference was discussed.

Decay lines of the two bitumens under each condition were analyzed. A four-parameter power function and a three-parameter exponential function were established to fit decay lines of Shell 60 and 100 respectively based on non-linear fitting method.

The gradation of the FA mixes was determined in view of the size of RAP materials and requirement of the bituminous mix design used in Hong Kong. The OMC was determined using the modified AASHTO OMC testing. The DBC was obtained in terms of soaked ITS by Marshall design method. Volumetric properties and mechanical properties (e.g. ITSM and ITS) of 16 FA mixes were also tested.

In this study, permanent deformation of FA mixes was evaluated in laboratory by repeated load axial creep (dynamic creep) test. In addition to the ultimate strain, the CSS, and SCSM were initially used to investigate susceptibility of FA mixes to permanent deformation. Effects of bitumen grades, RAP content and RAP types on resistance of FA mixes to permanent deformation were studied.

Based on the ITFT results, fatigue equations of FA mixes were established, fatigue lives between FA mixes and the hot-rolled mixes were compared, characteristics of the fatigue failure and effects of materials on the fatigue properties were analyzed, and nonlinear characteristics of FA mixes was discussed.

Moisture susceptibility tests, including the soaked ITS test, the soaked dynamic creep test, the soaked ITFT test, and the freeze-thaw ITFT test, were conducted in order to investigate the moisture susceptibility of FA mixes. Fatigue and permanent deformation between the dry and the moisture conditions were compared. Moisture susceptibilities of FA mixes to the permanent deformation, ITS and fatigue were evaluated.

Chapters 3, 5, 6 and 7 are dedicated to the foaming properties of the two bitumens, properties of FA mixes, including permanent deformation, fatigue and moisture susceptibility. The follows are the creative efforts of this study:

1. To minimize the human error on recording of the half-life results, digital video recorder was used to record the whole foaming process in this study. This method can obtain not only the accurate half-life of the foamed bitumen, but also its decay line.

2. Difference of the foaming properties between the two bitumens, and effects of bitumen viscosity and water content on the bitumen's foaming properties were analyzed based on mechanism of the foaming bubble. This analysis helps engineers and researchers to understand the bitumen's foamability in detail.

3. The decay functions of the two bitumens were established using the non-linear fitting method and best subset regression analysis. The mechanism difference of the two decay functions was discussed. Decay functions reveal how temperature and water content affect the foaming properties. ERMax and half-life of Shell 60 and Shell 100 under various conditions can be predicted by the two decay functions.

4. The design gradation (WC 20) of FA mixes was proposed in view of requirement of the Hong Kong's bituminous mixture design and size of RAP materials. It meets the requirement of the preferable gradings of the FA mixes well. This gradation can be used as a guide for recycling RAP materials by foamed bitumen in road rehabilitation and maintenance in Hong Kong.

5. Susceptibility of FA mixes to permanent deformation was evaluated using information of the linear portion of log-log dynamic creep curve. This information excludes the initial strain. CSS and SGSM were used to evaluate susceptibility of FA mixes to permanent deformation only caused by the repeated loading. The densification and total depth of the permanent deformation were evaluated by Intercept and Ultimate strain respectively.

6. Temperature of the dynamic creep testing was determined based on Hong Kong's meteorological data. It is more reasonable for application of FA mixes in Hong Kong.

7. The ITFT was firstly adopted to evaluate fatigue properties for FA mixes using the NAT-10.

Fatigue failure characteristics and effects of materials on the fatigue properties of FA mixes were analyzed, nonlinear characteristics of these mixes were discussed. These efforts help researchers and engineers to have comprehensive understanding fatigue properties of these recycled materials.

8. A 24-hour-soaking conditioning was selected to evaluate moisture susceptibility of FA mixes to ITS and permanent deformation. Freeze-thaw conditioning and 24-hour soaking conditioning were selected to test moisture susceptibility of FA mixes and investigate the effect of moisture conditioning methods on fatigue of FA mixes. Moisture susceptibility of FA mixes was firstly and comprehensively analyzed. This study can also help researchers and engineers to understand how FA mixes behave under the dry and moisture conditions.

This study is comprehensive and significance for future research and application of FA mixes. Foamability study provides a clear explanation on the mechanism of foaming and decaying of bitumen. A gradation (WC 20) is catered to the application of FA mixes in Hong Kong. In light of the temperature and moisture situation in Hong Kong, temperature of dynamic creep testing is determined; susceptibility of FA mixes to permanent deformation, fatigue properties and moisture susceptibility of these mixes have been systematically investigated. These efforts can be used to guide the mix design and evaluation of FA mixes.

8.3 CONCLUSIONS

Main conclusions in this dissertation are given chapter by chapter as follows.

A literature review in Chapter 2 "**COLD RECYCLING BY FOAMED BITUMEN: THE STATE-OF-ART AND APPROACHES OF THE STUDY**" yields the following conclusions:

RAP recycling is widely accepted as a feasible alternative to most highway rehabilitation and reconstruction methods. Recycling RAP materials by the foamed bitumen is an economical and innovative cold-recycling technology; it is sustainable and environmentally friendly.

Due to environmental protection and governmental encouragement, the cold in-place recycling using foamed bitumen provides a promising way to solve Hong Kong's RAP problem, especially to re-use a large amount of the RAP materials produced from road rehabilitation.

From Chapter 3 "**FOAMABILITIES OF BITUMENS**", the following conclusions are drawn:

1. Effects of foaming condition

Air pressure has little effect on the decay line, whilst water content significantly affects the decay line. ERMax of the two bitumens increases with an increase of water content. Water content higher than 2% adversely affects half-life; half-lives seldom exceed 5 seconds when water content is higher than 2%.

2. Effects of bitumen viscosity

Viscosity has an important impact on foaming properties of the two bitumens. Bubble of Shell 60 is most likely to collapse in the first mode, whilst bubble of Shell 100 in the second mode. Almost all of ERMaxs of the lower-viscosity Shell 100 are smaller than those of the higher-viscosity Shell 60. On the contrary, most of half-lives of Shell 100 are larger than those of Shell 60. ERMax of Shell 60 occurs at about 0 to 5 seconds, and ERMax of Shell 100 occurs at about 0 second.

3. ERMax vs. half-life

Shell 60 exhibits a clear inverse relationship between ERMax and half-life. However Shell 100 does not show a clear relationship as Shell 60 does.

4. Decay functions

Decay lines of Shell 60 and Shell 100 can be well fitted by a 4-parameter power function and a three-parameter exponential function respectively. Decay property also reveals that water content has direct influence on the foaming properties of the foamed bitumen.

5. Optimum foaming condition (OFC)

The condition of 5 bar pressure, 170 ^{0}C and 1.7% water content is determined as the OFC for Shell 60. The OFC of Shell 100 is the combination of 3% water content, 5 bar pressure and 160 ^{0}C.

In Chapter 4 “**MIX DESIGN**”, two kinds of penetration-grade bitumens and two different RAP materials were selected, gradation of FA mix was proposed. 4 RAP contents were considered in the mix design. OMCs and DBCs were determined for each FA mix. Air voids, ITSs and ITSMs of FA mixes were tested. The following conclusions are drawn:

1. OMC

OMC tests were performed in accordance with the modified AASHTO OMC method for 8 FA mixes (4 RAP contents X 2 RAP types). For the two RAP materials, OMCs and dry unit weights decrease as RAP content increases.

2. Design bitumen content (DBC)

The DBC is considered as the bitumen at which the soaked ITS is maximum. DBCs of 14 FA mixes are in the range of 2.5 to 3.5%. Whichever RAP#1 or RAP#2 is added, DBCs of the FA mixes treated by the two bitumens are 3.5% for the lower RAP content (0% and 20%), and 3.0% for the higher RAP content (40% and 60%).

3. Volumetric properties

Whichever RAP#1 or RAP#2 is added or whichever Shell 60 or Shell 100 is used, dry densities and MTSGs decrease with an increase of bitumen content. There is a concave second order polynomial relationship between air void and bitumen content. The air void reaches the minimum at 2.5% to 3.5% of bitumen content, at which the highest soaked ITS can be obtained.

4. Mechanics properties

The dry and soaked ITSs of 14 FA mixes are higher than 200 kPa and 100 kPa respectively. FA mixes containing RAP#1 have higher soaked ITSs than those mixes containing RAP#2.

There is a convex second order polynomial relationship between soaked ITS and bitumen content for each FA mix, which corresponds to the concave second polynomial relationship between air void and bitumen content, i.e.: at 3.0% to 3.5% of BC, the soaked ITS reaches the maximum, and air void reaches the minimum.

ITSM decrease with an increase of RAP content for FA mixes containing RAP#1. However, for those containing RAP#2, the minimum ITSM appears at 20% of RAP, the maximum ITSM appears at 0% of RAP, not at minimum air void.

With respect to the "**PROPERTIES OF PERMANENT DEFORMATION**" discussed in Chapter 5, dynamic creep test was selected to evaluate characteristics of permanent deformation for FA mixes. Testing results were analyzed by CSS, Intercept and SCSM. The following conclusions are drawn:

1. Failure

No failure of FA mixes occurs in the dynamic creep test after 1,800 load cycles with an axial stress of 100 KPa at 35 ^{0}C.

2. Effects of materials

RAP content has no significant effect on CSS. There is a significant effect of bitumen grade on CSS; CSSs of FA mixes stabilized by Shell 100 are smaller than those mixes stabilized by Shell 60. Therefore, high bitumen grade may help FA mixes to reduce their susceptibility to permanent deformation. Ageing of RAP and RAP content insignificantly affect susceptibility of FA mixes to permanent deformation.

3. CSS vs. SCSM

There is a good correlation between CSS and SCSM. CSS decreases when SCSM increases, i.e. FA mix with high creep strength will have less susceptibility to permanent deformation.

4. Variance

CSSs, Intercepts, SCSMs of FA mixes exhibit high variance except for air voids and ultimate axial strains. It is difficult to establish correlation between CSS and air void.

5. Comparison between FA mixes and hot asphalt mixes

Test results for FA mixes and hot asphalt mixes were compared. Susceptibilities and creep strengths of FA mixes are better than those of AC-20 and PA-10. Initial axial strains of FA mixes are between the results of AC-20 and PA-10.

In Chapter 6 "**FATIGUE PROPERTIES**", the ITFT was selected to evaluate the fatigue properties for FA mixes. The fatigue properties of FA mixes were comprehensively analyzed. The following conclusions are drawn:

1. Fatigue equations

The strain-fatigue equation can satisfactorily characterize the fatigue data of FA mixes. There is a strong linear relationship between the intercepts (logk1) and slopes (k2) of the fatigue equation.

2. Comparison of fatigue lives

Fatigue lives at 100 microstrains of FA mixes are far smaller than those of the hot-rolled mixes. This may be due to the much higher air void and relatively lower adhesion of the FA mixes than those of the hot-rolled mixes.

3. Vertical deformation in the ITFT

Vertical deformations at failure of FA mixes range 0.6 to 2.0 mm, far smaller than those of the hot-rolled mixes. For FA mixes, linear relationship between the failed vertical deformations and stresses is very strong; while linear relationship between the failed vertical deformations and cycles to failure (Nf) is poor.

4. Failure characteristics

Unlike the hot-rolled mixes, FA mixes show an apparently violent facture at failure in the ITFT; no plastic and viscoelastic deformation occur in the fatigue tests. Only ideal and single cleft failures were found when FA mixes failed. Therefore the FA mixes substantially differ from the hot-rolled mixes in fatigue failure characteristics.

5. Effects of materials on fatigue properties

Only RAP type significantly affects the fatigue lives at 100 microstrains of FA mixes at the 90% confidence level.

FA mix containing RAP#2 has longer N_{f100} than mix containing RAP#1. The higher viscosity bitumen is more advantageous to N_{f100} of FA mix with/without RAP material than the lower-viscosity bitumen.

The higher viscosity bitumen is advantageous to the fatigue life, at the maximum fatigue stress level, of FA mix containing less aged RAP material. The lower viscosity bitumen is advantageous to fatigue life, at the maximum fatigue stress level, of FA mix containing more aged RAP material.

6. Prediction based on the basic properties

It is difficult to accurately predict the fatigue lives for FA mixes using responses of the basic property tests because there is poor correlation between material parameters of the fatigue equations and responses of the basic property tests.

7. Nonlinear characteristics in the ITFT

FA mixes exhibit nonlinear characteristics between stress and strain, which can be expressed by the power equation.

Moisture susceptibility of FA mixes was discussed in more detail in Chapter 7 "**Moisture Susceptibility**". The following conclusions are drawn based on the comprehensive analysis of testing data:

1. Indirect Tensile Strength

(1) Effects of materials

Bitumen type and RAP content significantly affect the dry ITS and the soaked ITS. RAP type, however, insignificantly affects the dry and the soaked ITS. For both dry and soaked conditions, FA mixes stabilized by Shell 100 have higher ITS results than those stabilized by Shell 60.

(2) Comparison between FA mixes and hot asphalt mixes

RAP material can help FA mixes to enhance their moisture susceptibility to ITS. FA mixes have better moisture susceptibility to ITS than PA10, have poor result than AC20.

(3) Evaluation

All of 14 FA mixes meet the requirement of the moisture susceptibility to indirect tensile strength in terms of the Lottman's criterion.

2. Permanent Deformation

In dry condition, bitumen significantly affects FA mixes' permanent deformation. RAP content has significant effect on the soaked CSS, and the interactions of bitumen type and RAP type (or RAP content) also affect the dry US. In dry condition, SCSM is affected by three factors and their interactions.

Under the dry condition, softer bitumen will result in less susceptibility of FA mixes to permanent deformation. Under the soaked condition, however, harder bitumen (Shell 60) is statistically superior to the softer bitumen (Shell 100) in reducing the moisture susceptibility of FA mixes to permanent deformation.

Under the dry condition, more aged RAP material (RAP#2) will result in less susceptibility of FA mix to permanent deformation. However, moisture severely affects the susceptibilities of those mixes containing RAP#2 and stabilized by Shell 100 bitumen. Their moisture susceptibilities to permanent deformation are very poor, although they have best resistance to permanent deformation in dry condition. In average, FA mixes containing RAP#1 have better moisture susceptibilities to permanent deformation than those mixes containing RAP#2.

3. Fatigue properties

(1) Fatigue equations

Fatigue relationship of the moisture-conditioned FA mix can be suitably characterized by the strain-fatigue equation. For the soaked FA mixes, bitumen type, RAP type and RAP content significantly affect the slope of the fatigue equations of the soaked FA mixes; bitumen type and RAP content have significant effect on the fatigue equations' intercept. For the freeze-thaw FA mixes, three factors and their interactions have no significant effect on the fatigue equations' material parameters.

(2) Nonlinear characteristics under the moisture conditions

Under the moisture conditions, there is a clear nonlinear relationship between the stress and the strain. Stress-strain relationship can be characterized by a power equation for the soaked FA mixes, by a logarithmic equation for the freeze-thaw FA mixes.

(3) Effects of RAP content and bitumen grade

For the soaked FA mixes, when RAP content is small, high penetration-grade bitumen (Shell 100) is advantageous to fatigue properties of FA mixes; on the contrary, low penetration-grade bitumen (Shell 60) is advantageous to FA mixes' fatigue properties.

(4) Comparison of conditioning methods

The soaked ITFT is better than the freeze-thaw ITFT because the former can efficiently distinguish the moisture susceptibilities of FA mixes and the soaking procedure is straightforward. Therefore, the soaked ITFT is recommended to investigate the moisture susceptibility of FA mixes to fatigue.

(5) Evaluation of moisture susceptibility

Under the soaked and the freeze-thaw conditions, FA mixes containing 60% RAP#2 have poor moisture susceptibilities to fatigue whichever Shell 60 or Shell 100 is used as the foamed bitumen; and FA mixes containing 20% RAP#2 and stabilized by Shell 100 has poor moisture susceptibilities to fatigue too.

(6) Capacity to bear the maximum fatigue stress

The capacity of FA mix to bear fatigue loading reduces when the moisture condition becomes harsher. The dry FA mixes have the largest fatigue stress range followed by the soaked FA mixes. The freeze-thaw conditioned FA mixes have the smallest fatigue stress range.

(7) Initial ITSM vs. loading stress

Under the dry and the soaked conditions, the initial ITSM decreases with an increase of stress. Under the freeze-thaw condition, however, the stress has little impact on initial ITSM.

In summary, Group D, which contains 60% RAP#1 and is stabilized by Shell 60, has the best fatigue properties under the moisture conditions, has the best moisture susceptibility to permanent deformation, its moisture susceptibility to tensile strength also meets the Lottman's requirement. Therefore, under moisture condition in southern China, this kind of FA mix can be recommended as the road base. It is recommended to use less aged RAP material (e.g. RAP#1) and harder bitumen (e.g. Shell 60) in this area in order to obtain better engineering properties of FA mix.

Group D contains 60% RAP#1 and is stabilized with 3% of Shell 60 bitumen (by aggregate mass). The bitumen is foamed at the combined condition of 1.7% water content added into the bitumen, 5 bar air pressure and 170 ^{0}C. This kind of FA mix is compacted with 6.07% water content added into the aggregate (by aggregate mass).

8.4 RECOMMENDATIONS

On the basis of the experimental test and analysis described in this dissertation, a number of recommendations are provided for further study of FA mixes.

1. With respect to foamability of bitumen, the following recommendations are made:

(1) To investigate the foamability of softer bitumens (e.g. penetration-grade 150) and effects of these bitumens on FA mixes' properties, foaming test and analysis should be further carried out for them.

(2) Can the foaming agent, which can improve the bitumen's foaming properties, have effects on FA mixes' properties? Property differences between FA mix with the foaming agent and mix without the foaming agent should be investigated.

2. Literature review and research into mix design of FA mixes lead to the following recommendations:

(1) The existing mix design for FA mix is to optimize the strength at the worst conditions, i.e. soaked conditions, by ITS test. The ITS test offers a convenient way to evaluate FA specimens in this manner. Other tests, such as resilient modulus, dynamic creep and mix volumetrics, can be further used to verify the selected optimum mix.

(2) The existing mix design based on the Marshall method is a type of experiential method. It is believed that in the long term, the experiential method will be replaced by the performance-related design method. Superpave's volumetrics design should be tested in the FA mix design.

(3) Literature review indicates that the half-warm foamed bitumen treatment can improve the property of FA mixes. This is a new process: aggregates are heated to temperatures above ambient but below 100 ^{0}C before the application of the foamed bitumen. The RAP materials show great potential for treatment with the half-warm foamed bitumen process. Hence analytical results and conclusions should be based on further research.

3. The experimental testing performed leads to the following recommendations:

(1) Dynamic creep testing and fatigue testing with confinement should be conducted in further study because laboratory based test only stimulates the real situation of pavement.

(2) More technologies, such as capillary rise, permeability and/or dielectric testing, for evaluating the moisture infiltration of FA mixes can be used in the further study.

(3) Gyratory compaction method for sample preparation should be further investigated and effect of this method should be compared with that of Marshall method.

4. Evaluation of the FA mixes' properties leads to the following recommendations:

(1) In this study, evaluation of the in-place performance for FA mix wasn't considered. Hence it should be investigated by testing the cored samples from the pavement or by testing the

pavement using the accelerated load facility. The information of in-place performance of FA mixes can verify and adjust the mix design in laboratory.

(2) Effects of the additives on FA mixes' properties or on the improvement of FA mixes' properties should be further studied, e.g. the effect of the Portland cement added on the rate at which FA mixes gain strength or on the fatigue properties of FA mixes, the effect of other filler types (hydrated lime, rock dust, milled blast furnace slag) on the fatigue properties of FA mixes, the effect of the crumbed rubber on the properties of FA mixes.

(3) The resistance of FA mixes to permanent deformation under heavy traffic should be further studied in order to apply these materials to the heavy traffic road.

(4) It is necessary to establish a set of criteria of property evaluation (including ITS, permanent deformation, ITSM and fatigue) for FA mixes containing RAP materials for the purpose of the extensive application of these materials.

5. The following recommendations are made for extensive application of FA mixes:

(1) The effect of curing time and temperature on the engineering properties of FA mixes should be further studied. It will help to apply FA mixes to the rapid maintenance of the pavement.

(2) The effect of the stockpile time on FA mixes' engineering properties should be further studied. This study can help to improve the construction efficiency.

As a field trial of FA, the rehabilitation project can be considered. Because there are a large amount of RAP materials produced. Choose a section which will be rehabilitated, and apply the FA technology to this section. Investigate the in-field properties, e.g. strength, stiffness, moisture susceptibility of the cored samples, fatigue property of the materials prepared in field.

The long-term pavement performance can also be investigated by FWD, ALF and other apparatus.

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APPENDIX A LABORATORY PROCEDURES FOR MIX DESIGN

A.1 Apparatus

The following laboratory equipment is required to carry out the design of foamed asphalt mixes:

- A laboratory foamed bitumen plant capable of producing foamed bitumen at a rate of between 50 g and 200 g per second. The method of production should closely simulate that of full-scale production of foamed bitumen. The apparatus should have a thermostatically controlled kettle capable of holding a mass of 10 kg of bitumen at between 150° C and 205° C, within a range of ± 5° C. In addition, a low-pressure compressed air supply of 0 - 800 kPa with an accuracy of ± 25 KPa should be included in the apparatus. The plant should have a system of adding cold water to the hot bitumen, variable from 0 percent to 5 percent by mass, with an accuracy of 0,2 percent. The plant should be designed so that the foam can be discharged directly into the mixing bowl of an electrically driven laboratory mixer with a capacity of at least 10 kg.

- Marshall compaction moulds, 101.6 ± 0.5 mm in diameter and 87.3 ±1mm high, with baseplate and extension collar to fit the moulds. A Marshall compaction hammer with a 98.5± 0.5 mm diameter flat face and a 4536 ± 5g sliding weight with a free fall of 457 ± 3 mm (ASTM D1557 2000). The use of a mechanical hammer is preferred. A compaction pedestal consisting of a 203 x 203 x 457 mm wooden post capped with a 305 mm square steel plate. The pedestal shall be so installed that the post is plumb and the cap is level and must be provided with a rigid vertical guide for the hammer. The wooden post must be secured to a solid concrete slab.

- A mould holder of suitable design to hold the mould in place during compaction.

- A specimen extractor of suitable design to remove the briquette from the mould without damage.

- A balance to weigh up to 5 kg accurate to 1 g.

- A spatula with a blade of approximately 150 mm in length.

- A compression testing machine capable of applying a load of at least 20 kN at a rate of 50,8 mm per minute, fitted with a load measuring device to measure a load of at least 15 kN and accurate to 0.1 kN.

• An air cabinet capable of maintaining a temperature of 25° C ±1° C.

• Two hardened steel loading strips, 13 ± 0.1 mm wide, each with a concave surface having a radius of curvature of 51 ± 1 mm and at least 70 mm long.

• The edges of the bearing surface should be rounded slightly to remove the sharp edge. The bearing strips should be mounted in a frame of suitable design to align the strips on the test specimen.

• A steel load-transfer plate, round or square, to transfer the load from the compression testing machine to the top bearing strips without deformation. Its dimensions should be such that it will cover at least the length of the specimen to be tested on the bearing strip.

• Callipers to measure the length and diameter of test specimens to the nearest 0.5 mm.

• Silicone grease or oil (such as stop-cock grease).

• A vacuum desiccator or other appropriate vessel and a vacuum pump capable of reducing pressure to less than 50 mm mercury, connected to a manometer. A thermometer capable of measuring a temperature between 0° C and 50° C ± 0.2° C.

A.2 Determination of Foaming Characteristics of the Bitumen

The objective is to determine the percentage of water which will optimize the foaming properties of a particular bitumen by maximizing the expansion ratio and half-life of the foamed bitumen. This is achieved as follows:

• Calibrate the bitumen and water flow rates. Regulate the bitumen discharge rate to 100 grams per second. Regulate the air supply pressure to 100 kPa.

• Select 4 to 5 temperatures (160 to 200 ^{0}C) at which the foaming characteristics are to be measured. Bring the temperature of the bitumen to the required temperature and maintain the bitumen for at least 15 minutes before commencing with foam production. Then, for each temperature, measure the foaming characteristics of five samples of foamed bitumen at water injection rates ranging from 1% to 4% by mass of the bitumen, in increments of 0.5%, as follows:

- for each sample, allow 500g of foam to discharge into a 20 litre steel drum.
- measure the maximum expansion ratio of the bitumen during foaming, using a ruler in the side of the drum. Using a stopwatch, measure the time in seconds which the foam takes to dissipate to half of its maximum volume. This is defined as the half-life.
- plot a graph of the expansion ratio and half-life versus moisture content for all samples on the

same set of axes, as shown in Fig. A.1. This will enable the foam water content to be optimized.

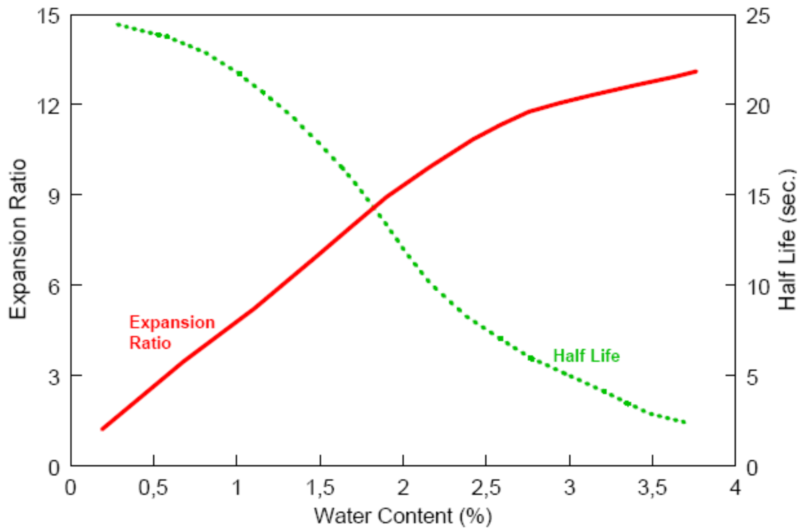

Fig.A. 1 Optimizing foaming characteristics

A.3 Aggregate Preparation

- Determine the gradation (see AASHTO T27 (1999)) and plasticity index (see AASHTO T90 (2000)) of the aggregates;
- In some cases blending of more than one aggregate may be undertaken to provide the required grading. At this stage cement, lime or other fillers may be added if required. Lime treatment must be conducted if the PI of the aggregates is greater than 12;
- Carry out a moisture/density relationship test, using the modified AASHTO method, so as to obtain the optimum moisture content (OMC) (see ASTM D1557 (2001));
- Oven-dry the aggregate to constant mass at 105° C. In the case of reclaimed bituminous materials, drying should be carried out at a lower temperature to prevent the particles from sticking together;
- Determine the bitumen content of reclaimed bituminous materials.
- When dry, riffle the sample and weigh into five batches, each 10 kg.

A.4 Specimen Preparation

All five of the 10 kg batches are treated with foamed bitumen at different bitumen content, 1% apart. For each batch, carry out the following procedure:

- Place the entire batch (10 kg) into the bowl of the mechanical mixer (WLB10 laboratory foamed bitumen plant/mixer);
- Add the required percentage (by mass) of lime and or/cement as per the formula:

$$M_{Cement} = \frac{C_{Add}}{100} \times \left[M_{Sample} \times (1 + \frac{C_{Add}}{100}) \right] \quad \text{(A.1)}$$

where M_{Cement} = mass of lime or cement to be added (g)

M_{Sample}= dry mass of the sample (g)

C_{Add}= percentage of lime or cement required (% by mass)

The following application rates of lime or cement should be used as a guide:

-when the PI of the sample exceeds 10 but is less than 16, add 1% hydrated lime;

-when the PI is 16 or more, add 2% hydrated lime;

-when the PI is 10 or less, add 1% ordinary Portland cement.

• Estimate the moisture content by reducing the optimum moisture content to optimum mixing purposes according to Eq. (A.2) and estimate the water needed for the sample according to Eq. (A.3).

$$W_{Add} = W_{OMC} - W_{Moist} - W_{Reduc} \quad \text{(A.2)}$$

$$M_{Water} = \frac{W_{Add}}{100} \times (M_{Sample} + M_{Cement}) \quad \text{(A.3)}$$

where W_{Add} = water content to be added to sample (% by mass)

W_{OMC}= optimum moisture content (% by mass)

W_{Moist}= moisture content in sample (% by mass)

$W_{Reduc.}$=reduction in water content according to Fig. A.2 for the optimum mixing moisture content (0.3XW_{OMC}-0.6) (% by mass)

M_{Water} =mass of water to be added (g)

M_{Sample}=dry mass of the sample (g)

M_{Cement}=mass of lime or cement added. (g)

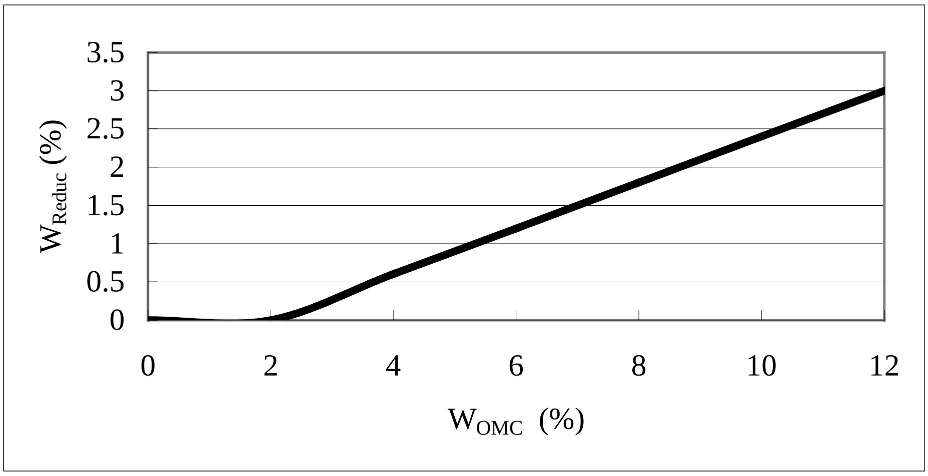

Fig.A. 2 Reduction of moisture content for optimum mixing moisture content

Before adding the foamed bitumen, put the aggregates into the mechanical mixing bowl, and add the needed water in the aggregate, mix the material for 60 seconds.

Note: when 2% hydrated lime is added to reduce plasticity (where the PI of the sample exceeds 16), mix thoroughly in the mechanical mixer after the addition of water until uniform. Then place the sample in a sealed container and allow to stand for 24 hours before proceeding.

A.5 Treatment with Foamed Bitumen

• add foamed bitumen to the prepared sample according to Eq. (A.4)

$$M_{Bitumen} = \frac{B_{Add}}{100} \times (M_{Sample} + M_{Cement}) \qquad \text{(A.4)}$$

where $M_{Bitumen}$=mass of foamed bitumen to be added (g)

M_{Add}= foamed bitumen content (% by mass)

M_{Sample}=dry mass of sample (g)

M_{Cement}=mass of lime or cement added (g)

• set the timer on the Wirtgen laboratory foamed bitumen plant according to Eq. (A.5)

$$T = factor \times \frac{M_{Bitumen}}{Q_{Bitumen}} \qquad \text{(A.5)}$$

where T = time to be set (s)

$M_{Bitumen}$=bitumen percentage to be added (% by mass)

$Q_{Bitumen}$= bitumen flow rate for the foamed bitumen plant (g/s)

factor = compensation for bitumen losses on mixing arm and bowl (experience shows that a factor of 1.25 gives acceptable results)

• position the mechanical mixer adjacent to the foaming unit so that the foamed bitumen can be discharged directly into the mixing bowl;

• mix the material and moisture in the mixing bowl unit uniform;

• without stopping the mixer, discharge the required mass of foamed bitumen into the mixing bowl;

• continue mixing for a further 30 seconds;

• transfer the foamed bitumen treated material into a sealed container;

• repeat this procedure to obtain five samples of foamed bitumen treated material at the required bitumen contents. These samplers are now ready for the manufacture of briquettes.

• determine the content of each sample treated with foamed bitumen.

A.6 Moisture and Bitumen Contents

• Take duplicate samples from each batch for moisture and bitumen content checks.

- Dry to constant mass at 105° C to 110° C and determine the moisture content of the material (see ASTM D2216 (1998)).
- Carry out a bitumen content determination (see ASTM D 2172 (2001))

A.7 Procedure for the Compaction of Specimens (Briquette)

- prepare the Marshall mould and hammer by cleaning the mould, collar, base-plate and face of the compaction hammer;
- place a round plastic or paper disc at the bottom of the mould;
- weigh enough material to achieve a compacted height of 63.5±1.5 mm (usually about 1.15 kg is sufficient). Poke the mixture with a spatula 15 times around the perimeter and poke the rest of the surface 10 times leaving the surface slightly rounded;
- compact the mixture by applying 75 blows with the compaction hammer. Care should be taken to ensure that the hammer can fall freely;
- remove the mould and collar from the pedestal, invert it, replace it and press it down so that it rests firmly on the baseplate;
- compact the briquette with another 75 blows.

Number of specimen needed at each bitumen content (1 batch):

three for dry ITS,
three for soaked ITS,
two for volumetric (only testing the theoretical maximum specific gravity)

A.8 Curing

- After compaction, remove the mould from the baseplate and allow the specimen to cure for 24 hours in the mould at ambient temperature before extruding it by means of an extrusion jack or other means;
- Place the specimens on a smooth flat tray and cure in a forced draft oven for a further 72 hours at 40 °C.

A.9 Dry Density

- The dry density of each briquette should be checked after they have cooled to ambient temperature. The dry density of specimens should be calculated using Eq. (A. 6):

$$D = \frac{100}{W_{Moist} + 100} \times \frac{4 \times M_{Briq}}{\pi \times d^2 \times h} 1000 \qquad \text{(A. 6)}$$

where D= dry density (kg/m^3)

W_{Moist}=moisture content of sample during compaction (% by mass)

M_{Briq}=mass of briquette immediately after compaction (g)

h= average height of briquette (cm)

d=diameter of briquette (cm)

Note $\frac{100}{(W_{Moist}+100)}$ is the factor used to convert the wet density to dry density.

- Exclude from further testing any briquette whose dry density differs from the mean dry density of the batch by more than 50 kg/m^3.

A.10 Determination of Indirect Tensile Strength

The standard indirect tensile strength test (see ASTM D4867 (2002)) is used to test compacted, cured foamed asphalt specimens under dry and soaked conditions. The indirect tensile strength is determined by measuring the ultimate load to failure of a specimen which is subjected to a constant deformation rate of 50.8 mm/minute on its diametrical axis. The procedure is as follows:

- Leave the cured briquette overnight at room temperature before testing;
- measure the height of each briquette at four evenly spaced places around the circumference and calculated the average height, L (m);
- measure the diameter of each specimen, D(m);
- place the briquettes in the air cabinet at 25 °C ± 1 °C for at least 1 hour but for not longer than 2 hours before testing;
- remove a specimen from the air cabinet and place it into the loading apparatus;
- position the sample such that the loading strips are parallel and centred on the vertical diametral plane;
- place the transfer plate on the top bearing strip and position the assembly centrally under the loading ram of the compression testing device;
- apply the load to the specimen without shock at a rate of advance of 50.8 mm per minute until the maximum load is reached;
- record this load, P, accurate to 0.1 kN.

In order to determine the indirect tensile strength of soaked specimens, use the following procedure prior to testing:

- place the cured specimen in the vacuum desiccator, cover with water at 25 °C ± 1 °C;
- apply a vacuum of 50 mm of mercury for 10 ± 1 minutes, with the timing period commencing once the required vacuum has been reached;
- remove the specimen, surface dry and test for the ultimate indirect tensile load as described in

the preceding paragraph.

Calculate the indirect tensile strength (ITS) for each specimen to the nearest 1 kPa using the following formula:

$$ITS = \frac{2 \times P}{\pi \times h \times d} \qquad \text{(A.7)}$$

where *ITS*=indirect tensile strength (kPa)

P=maximum applied load (kN)

h= average height of briquette (cm)

d=diameter of briquette (cm)

The retained ITS (ITSR), also called ITS ratio, is the relationship between the dry and soaked ITS for a specific bitumen content:

$$ITSR = \frac{SoakedITS}{DryITS} \qquad \text{(A.8)}$$

A.11 Determination of the Design Bitumen Content

Plot a graph of the measured indirect tensile strengths versus bitumen content for all the samples (soaked and dry tests) on the same set of axes. The bitumen content at which the soaked ITS is at its maximum is taken as the Design Bitumen Content for the foamed asphalt mix.

A.12 Determination of Additional Mix Properties at the Design Bitumen Content

When required, additional tests such as resilient modulus and dynamic creep tests may be performed on samples at the design bitumen content. The results would possibly be required for the structural design of the foamed asphalt layer. Additional samples would have to be compacted and cured for this purpose, as described above.

APPENDIX B LABORATORY PROCEDURES FOR OMC TEST

B.1 Method

Prepare at least four (preferably five) specimens having water contents such that they bracket the estimated optimum water content. A specimen having water content close to optimum should be prepared first by trial additions of water and mixing. Select water contents for the rest of the specimens to provide at least two specimens wet and two specimens dry of optimum, and water contents varying by about 2%, not exceeding 4%. At least two water contents are necessary on the wet and dry side of optimum to accurately define the dry unit weight compaction curve. Fig.B.1 shows the apparatus for preparation of the specimen.

Fig.B. 1 Apparatus for preparation of the specimen

B.2 Mixing

Use approximately 5 lbm (2.3kg) of the sieved aggregate for each specimen to be compacted. To obtain the specimen water contents selected, add or remove the required amounts of water. Thoroughly mix each specimen to ensure even distribution of water through out and then place in a separate covered container prior to compaction.

B.3 Compaction

After curing, if required, each specimen shall be compacted as follows:

- Determine and record the mass of the mold or mold and baseplate.
- Assemble and secure the mold and collar to the baseplate. The mold shall rest on a uniform

rigid foundation. Secure the baseplate to the rigid foundation.

• Compact the specimen in five layers. Compact each layer with 25 blows for the 4 in.(101.6mm) mold. Apply the blows at a uniform rate in such a manner as to provide complete, uniform coverage of the specimen surface. After compaction, each layer should be approximately equal in thickness.

• Following compaction of the last layer, remove the collar and baseplate from the mold. A knife may be used to trim the soil adjacent to the collar to loosen the soil from the collar before removal to avoid disrupting the soil below the top of the mold.

• Carefully trim the compacted specimen even with the top and bottom of the mold by means of the straight edge scraped across the top and bottom of the mold to form a plane surface. Fill any holes in either surface with unused or trimmed soil from the specimen, press in with the fingers, and again scrape the straightedge across the top and bottom of the mold. For very wet or dry soils, soil or water may be lost if the baseplate is removed. For these situations, leave the baseplate attached to the mold.

• Determine and record the mass of the specimen and mold to the nearest gram. When the baseplate is left attached, determine and record the mass of the specimen, mold and baseplate to the nearest gram.

• Remove the material from the mold. Obtain a specimen for water content by using either the whole specimen (preferred method) or a representative portion. When the entire specimen is used, break it up to facilitate drying. Otherwise, obtain a portion by slicing the compacted specimen axially through the center and removing about 500g of material from the cut faces. Obtain the water content in accordance with ASTM D2216 (1998).

Fig B.2 shows a mechanical compactor used in the determination of OMC in this research.

Fig.B. 2 Mechanical compactor

B.4 Calculation

Following compaction of the last specimen, calculate the dry unit weight and water content of compacted specimen as follows:

- Water content is calculated in accordance with ASTM D 2216.
- Calculate the moist density and dry unit weight:

$$\rho_m = \frac{(M_t - M_{md})}{1000V} \tag{B.1}$$

where ρ_m = moist density of compacted specimen, Mg/m^3,

M_t = mass of moist specimen and mold , kg,

M_{md} = mass of compaction mold, kg, and

V = volume of compaction mold, m^3.

$$\rho_d = \frac{\rho_m}{1 + \frac{w}{100}} \tag{B.2}$$

where ρ_d = dry density of compacted specimen, Mg/m^3, and

w = water content, %.

$$\gamma_d = 9.807\rho_d \quad \text{in kN/m}^3 \tag{B.3}$$

where γ_d = dry unit weight of compacted specimen.

- To calculate points for plotting the 100% saturation curve or zero air voids curve select values of dry unit weight, calculate corresponding values of water content corresponding to the condition of 100% saturation as follows:

$$w_{sat} = \frac{\gamma_w G_S - \gamma_d}{r_d G_S} \times 100 \tag{B.4}$$

where w_{sat} = water content for complete saturation, %,

γ_w = unit weight of water, 9.789 kN/m^3 at 20 ^{0}C,

γ_d = dry unit weight of aggregate, (kN/m^3), and

G_S = specific gravity of aggregate.

B.5 Correction of unit weight and water content for total sample

After all specimens have been compacted, correction of unit weight and water content for total sample must be done as following:

- Prepare the sample from which compaction test specimens are to be taken in accordance with above procedures. Determine the mass of the moist fine fraction of the sample and the mass of the moist oversize (plus 19-mm sieve) fraction of the total sample. Determine the water content of each of the two fractions in accordance with ASTM D2216. Calculate the mass of the dry finer fraction and the dry oversize fraction as follows:

$$M_D = M_M /(1+w) \qquad \text{(B.5)}$$

where: M_D = mass of the dry material (finer or oversize fraction), g,

M_M = mass of the moist material (finer or oversize fraction), g, and

w = water content of the respective finer or oversize fractions expressed as a decimal.

• Calculate the percentage of the finer fraction and of the oversize fraction of the sample by dry weight as follows:

$$P_F = 100M_{DF} /(M_{DF} + M_{DC}) \qquad \text{(B.6)}$$

and $$P_C = 100M_{DC} /(M_{DF} + M_{DC}) \qquad \text{(B.7)}$$

where P_F = percent of finer fraction by weight,

P_C = percent of oversize fraction by weight,

M_{DF} = mass of dry finer fraction, and

M_{DC} = mass of dry oversize fraction.

• Determine the bulk specific gravity (G_M) of the oversize fraction as set forth in ASTM C 127 (2001).

• Calculate the corrected water content and corrected dry unit weight of the total material (combined finer and oversize fractions), as follows:

$$C_W = (w_F P_F + w_C P_C) \qquad \text{(B.8)}$$

where C_w = corrected water of combined and oversize fractions,

w_F = water content of finer fraction expressed as a decimal,

w_C= water content of oversize fraction expressed as a decimal, and

$$C\delta_D = 100\delta_F G_M \delta_w /(\delta_F P_C + G_M \delta_w P_F) \qquad \text{(B.9)}$$

where $C\delta_D$ = corrected unit dry weight of the total material (combined finer and oversize fractions),

G_M= bulk specific gravity,

δ_F = dry unit weight of the finer fraction, and

δ_w = unit weight of water (9.802 kN/m^3).

B.6 Plotting

Plotting the wet unit weight and water content of each compacted specimen can be an aid in making the above evaluation. If the desired pattern is not obtained, additional compacted specimens will be required. Generally, one water content value wet of the water content defining the maximum wet unit weight is sufficient to ensure data on the wet side of optimum water content for the maximum dry unit weight.