Improvement of Asphalt Mix Design Technology for Pakistan

Strategic Pavement Research Study (SPRS) Phase-1

June - 2022

Highway Research and Training Centre (HRTC)
National Highway Authority (NHA)
Pakistan

Strat

Improvement of Asphalt Mix Design Technology for Pakistan

Strategic Pavement Research Study (SPRS) Phase-1

June - 2022

Highway Research and Training Centre (HRTC)
National Highway Authority (NHA)
Pakistan

MESSAGE

National Highway Authority (NHA) is committed to its role of 'Service Provider' and invested trillions of rupees on expansion and up-gradation of roads in Pakistan, over the past 3 decades. However, currently a rather bigger challenge is to maintain and preserve the new; as well as existing (ageing) network of more than 13000 kilometers of country's major highways; at optimal performance level. For this purpose, NHA is developing a national level Highway Research & Training Centre (HRTC). The Centre provides a platform to road engineers, practitioners, academicians and industry to develop optimized solutions for the growing needs of road infrastructure industry, in Pakistan.

I would like to thank the principal investigators, research associates and research assistants from National Institute of Transportation (NIT); National University of Sciences & Technology (NUST), Taxila Institute of Transportation Engineering (TITE); University of Engineering & Technology (UET) Taxila and Department of Transportation Engineering and Management, UET Lahore for their sincere and diligent efforts in successful execution of the study. My sincere thanks to the entire team at Highway Research & Training Centre for successfully implementing the Phase I of collaborative study of national importance i.e., "**Strategic Pavement Research Study (SPRS)**". The goal is to minimize premature failure of pavements and ensuring better value for money spent on Maintenance & Repair (M&R) of roads in Pakistan.

I would like to express my sincere gratitude to the National Highway Authority, Pakistan for continuous support and sponsoring of this study.

(Dr. Shafeeq Ahmad)
Executive Director (HRTC)

June – 2022

FOREWORD

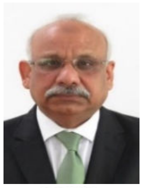
Pakistan's highway and motorway network is growing steadily. This expanding road network is exerting burden on pavement maintenance needs with growing backlog every year. To address the problem, National Highway Authority (NHA) through Highway Research & Training Centre (HRTC) has undertaken a phased project namely "Strategic Pavement Research Study" aiming to develop Pavement Design Systems and Standards for Pakistan. As part of a larger program, while recognizing the immediate need for sustainability and longevity of flexible pavement network to safeguard against premature distress, this research documents Phase 1 of the project i.e., "Improvement of Asphalt Mix Design Technology for Pakistan". The research described herein looks into laboratory characterization of indigenous pavement materials using selected aggregate gradations for asphalt concrete mixtures practiced in Pakistan and in different parts of the world. The research has identified performance and ranking of selected asphalt concrete mixtures over a range of temperature and stress regimes. The evaluated asphalt mixtures will be employed in controlled and instrumented test sections planned at new HRTC campus alongside Motorway M-1 at Burhan. With a view to get tangible and immediate results, the research helped in recommending asphalt mixtures to evaluate their performance on in-service, un-controlled test sections across road network in different temperature zones of Pakistan. Furthermore, the research also identified gaps/improvements to NHA's General Specifications of 1998. The improvements target aggregates, binder and asphalt concrete mixtures specifications.

This research also marks a concerted effort both by the industry and academia by "bridging the gap". I would like to thank our partnering academic institutions particularly the Principal Investigators (Academia) whose persistent input towards the research project made it more valuable. The efforts of the research assistants who pursued MSc and PhD degrees during this research are also appreciated. This research has played a great role towards industry – academia linkage program which is likely to continue to achieve the goal through laid out road map.

(Dr. Shahab Khanzada)
Chief Research Officer (HRTC) (2010-2016)
Consultant (Roads)

June – 2022

RESEARCH TEAM ACADEMIA



Dr. M. A. Kamal
Principal Investigator – UET Taxila
Member,
Prime Minister's Inspection Commission
Tel.: +92-51-9213934

Dr. Muhammad Irfan
Principal Investigator – NUST
Professor / Dean, Civil Engineering
NUST Campus, Risalpur
Tel. (Mob.): +92-300-9652413

Dr. Sarfraz Ahmed
Co-investigator
Professor / Head, Transportation & Geotechnical Engineering Department
NUST Campus, Risalpur
Tel. (Mob.): +92-322-3200036

Dr. Imran Hafeez
Co-investigator
Professor, Civil Engineering Department
UET Taxila
Tel. (Mob.): +92-333-4228382

Dr. Naveed Ahmad
Co-investigator
Professor, Civil Engineering Department
UET Taxila
Tel. +92-51-9047644

Dr. Ammad Hassan Khan
Principal Investigator – UET Lahore
Professor & Chairman, Department of Transportation Eng. & Management, UET Lahore
Tel. (Mob.): +92-300-4153570

Sabahat Hussan (PhD Scholar)
Research Associate
UET Taxila
Tel. (Mob.): +92-315-7255515

Mujassim Ali Rizvi (PhD Scholar)
Research Associate
UET Lahore
Tel. (Mob.): +92-334-4004393

RESEARCH TEAM HRTC

Dr. Shafeeq Ahmad
Executive Director,
HRTC
Tel. (Mob.): +92-334-5544456

Dr. Shahab Khanzada
Chief Research Officer,
HRTC (2010-2016)
Tel. (Mob.): +92-300-5019346

Dr. Aasim Inam
Director,
HRTC
Tel. (Mob.): +92-345-5055335

Engr. Attaullah
Material Engineer,
HRTC
Tel. (Mob.): +92-342-5000186

Engr. Abdul Rauf
Coordination Officer,
HRTC
Tel. (Mob.): +92-322-5046150

The research topics of MSc & PhD Theses, so far completed as part of Phase I of SPRS are as below:

PhD:

Sr. No.	Title	Name	Institute	Year
1	Permanent Deformation Evaluation of Asphalt Mixtures for the Development of Rutting Shift Functions	Sabahat Hussan (Research Associate)	University of Engineering and Technology, Taxila, Pakistan	2018
2	Influence of Performance Based Binder Properties on the Distress Behavior of Asphalt Mixtures	Mujasim Ali Rizvi (Research Associate)	University of Engineering and Technology, Lahore, Pakistan	2022

MSc:

Sr. No.	Title	Name	Institute	Year
1	Characterization of conventional HMA mixtures using simple performance testing protocols	Muhammad Junaid (Research Assistant)	National University of Sciences and Technology, Islamabad Pakistan	2016
2	Aggregate Packing Characteristics of Different Aggregate Gradations in Asphaltic mixtures	Muhammad Aakif Ishaq (Research Assistant)	University of Engineering and Technology, Taxila, Pakistan	2015
3	Laboratory characterization of HMA mixes subjected to indirect tensile fatigue test	Muhammad Aniq Gul (Research Assistant)	National University of Sciences and Technology, Islamabad Pakistan	2015
4	Durability assessment of bituminous mixtures for Pakistan	Muhammad Mohsin Bhutta (Research Assistant)	University of Engineering and Technology, Taxila, Pakistan	2014
5	Laboratory characterization of asphalt concrete mixtures using dynamic modulus test	Yasir Ali (Research Assistant)	National University of Sciences and Technology, Islamabad Pakistan	2014
6	Characterization of various HMA mixtures using Resilient modulus test	Muhammad Zeeshan (Research Assistant)	National University of Sciences and Technology, Islamabad Pakistan	2014
7	Durability assessment of bituminous mixtures for Pakistan	Salman Maqbool (Research Assistant)	University of Engineering and Technology, Lahore, Pakistan	2012

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

LIST OF ACRONYMS

AASHTO	American Association of State Highway and Transportation Officials
AC	Asphalt Concrete
AHN	Asian Highway Network
AMPT	Asphalt Mixture Performance Tester
ANN	Artificial Neural Networking
ANOVA	Analysis of Variance
APA	Asphalt Pavement Analyzer
ARL	Attock Refinery Limited
ASTM	American Society for Testing and Materials
AWC	Asphalt Wearing Course
BBR	Bending Beam Rheometer
BS	British Standard
CBR	California Bearing Ratio
CDI	Compaction Densification Index
COC	Cleveland Open Cup
COV	Coefficient of Variation
CPEC	China-Pakistan Economic Corridor
CWTT	Cooper Wheel Tracking Test
DBM	Dense Bitumen Macadam
DDT	Direct Tension Tester
DS	Dynamic Stability
DSR	Dynamic Shear Rheometer
DTT	Direct Tension Test
ESAL	Equivalent Single Axle Load
FN	Flow Number
FT	Flow Time
FWD	Falling Weight Deflectometer
HMA	Hot Mix Asphalt
HMB	High Modulus Base
HRB	Highway Research Board
HRTC	Highway Research & Training Centre

HWTD	Hamburg Wheel Tracking Device
IDT	Indirect Tensile
ITFT	Indirect Tensile Fatigue Test
LEA	Linear Elastic Analysis
LOE	Line of Equality
LVDT	Linear Variable Differential Transformer
MAPE	Mean Absolute Percentage Error
MDL	Maximum Density Line
ME	Mechanistic Empirical
MEPDG	Mechanistic Empirical Pavement Design Guide
MR	Resilient Modulus
NAT	Nottingham Asphalt Tester
NCHRP	National Cooperative Highway Research Program
NHA	National Highway Authority
NMAS	Nominal Maximum Aggregate Size
NRL	National Refinery Limited
NTEC	Nottingham Transportation Engineering Centre
NUST	National University of Sciences and Technology
OBC	Optimum Bitumen Content
OD	Oven Dried
OGFC	Open Graded Friction Course
PATTI	Pneumatic Adhesion Tensile Testing Instrument
PAV	Pressure Aging Vessel
PG	Performance Grading
PI	Penetration Index
PRD	Percentage Refusal Density
PSI	Pavement Survivability Index
PTCT	Partial Triaxial Compression Test
QA	Quality Assurance
QC	Quality Control
RAP	Reclaimed Asphalt Pavement
RLAT	Repeated Load Axial Test
RTFO	Rotating Thin Film Oven

RV	Rotational Viscometer
SATS	Saturation Ageing Tensile Stiffness
SGC	Superpave Gyratory Compactor
SHRP	Strategic Highway Research Program
SMA	Stone Mastic Asphalt
SPT	Simple Performance Tester
SSD	Saturated Surface Dried
TDI	Traffic Densification Index
TSR	Tensile Strength Ratio
UTM	Universal Testing Machine
VA	Air Voids
VFA	Void Filled with Asphalt
VFB	Void Filled with Bitumen
VMA	Voids in Mineral Aggregate
WT	Wheel Tracker

LIST OF SYMBOLS/ UNITS

hr.	Hour
H_Z	Hertz
in.	Inch
km	Meter X 10^3
mm	Meter X 10^{-3}
kN	Newton X 10^3
kPa	Pascal X 10^3
Pen	Penetration in m X 10^{-4}
PSI	Pounds per square inch
VB	Binder Content, percentage by volume
VV	Air void content, percentage by volume
°C	Degree Celsius

Summary

This research study report is concerned with improving asphalt mix design technology for Pakistan. The research described herein looks into the laboratory characterization of selected aggregate gradations for asphalt concrete mixtures practiced in Pakistan and in different parts of the world. The research has identified performance and ranking of selected asphalt concrete mixtures over a range of temperature and stress regimes. A detailed literature review covering behaviour of asphalt concrete mixtures and related test methods is given in Chapter 2.

Chapter 3 describes the methodology adopted for the research towards asphalt concrete mix preparation. The mixtures selected for laboratory tests comprised aggregate sources of Margalla and Ubhan Shah in combination with penetration grade binders i.e., 40/50, 60/70 from National Oil Refinery, Karachi and 60/70 pen grade from Attock Oil Refinery, Rawalpindi. Marshal method of mix design was employed for preparation of asphalt concrete mixture. Superpave Gyratory compactor was also employed for specimen fabrication for performance testing along with Slab Compactor.

Chapter 4 describes Bailey's methodology to characterize aggregate to achieve maximum achievable packing density using the local aggregates. The method of controlling the aggregate gradation volumetric parameters using voids in coarse aggregate and bulk specific gravity as developed by Bailey works fairly well on the local aggregates. The trends of relationships among the volumetric parameters in aggregate gradations with different nominal maximum aggregate sizes are in line with various studies as reported in this paper. The defined Bailey limits of different ratios can be adopted for developing the target aggregate gradation in Pakistan and the asphalt mixtures prepared within the defined limits perform better than the conventional asphalt mixtures.

Chapter 5 investigates the Dynamic modulus $|E^*|$ testing of different asphalt concrete mixtures at various temperature and frequency regimes. Results show that test temperature and loading frequency have significant impact on $|E^*|$ and phase angle, whereas; nominal maximum aggregate size (NMAS) is significant in wearing course and insignificant in base course mixtures. The laboratory determination of dynamic modulus values would facilitate the implementation of performance based mechanistic-empirical structural design and analysis approach in Pakistan. Further, Flow Number (FN) and Flow Time (FT) tests were also carried to characterize deformation behaviour of selected asphalt concrete mixtures. FN and FT test results would facilitate the pavement design engineers and practitioners to select suitable rut-resistant mixtures and help in performance prediction of AC mixtures.

Chapter 6 determines the Resilient Modulus testing on selected AC mixtures with different penetration grades of binder from National Refinery and Attock Refinery. Percentage decrease in resilient modulus values due to increase in temperature was relatively more in the mixtures prepared with NRL 40/50 than ARL 60/70. Effect of specimen diameter change on the resilient modulus values of different mixtures was found relatively similar at temperature 25°C and 40°C. Among the five main factors temperature was the most significant factor affecting the resilient modulus followed by load duration, bitumen type, nominal maximum size of aggregate and specimen diameter. The comparison of $|E^*|$ with Resilient Modulus (MR) showed that $|E^*|$ at 5 Hz for 25 °C

is strongly correlated with MR at same temperature (25°C) and loading frequency of 300ms; this correlation showed R^2 of 0.85.

Chapter 7 evaluates the deformation behaviour i.e., rutting in asphalt concrete mixtures on basis of terminal permanent deformation values and rutting/ permanent strain slope plots. Permanent strain evaluation plots indicated asphalt mixtures with coarser gradations stabilize with increasing loading cycles at high temperature conditions, hence, the impact of temperature variable is more dominant in case of finer graded mixtures. The ranking of different wearing course mixtures was found to be nearly consistent for ranking obtained by different combinations of laboratory rutting performance tests by Cooper Wheel Tracking Test (CWTT), Asphalt Pavement Analyzer (APA) test and Repeated Load Axial (RLA) Test. Ranking of relative suitability of each performance test has been done by comparing the time required to achieve one millimeter of rut depth. Rutting shift function / prediction models based on two statistical techniques are developed.

Chapter 8 outlines limited fatigue testing on selected asphalt concrete mixtures. Fatigue curves were derived and characterized by slope and coefficient of determination, suggesting that comparatively finer mixture performs relatively better among the tested mixtures for a given set of bitumen grades. A non-linear functional formulation was adopted to express fatigue resistance as a function of the initial strain, the viscosity of bitumen, the percentage of optimum bitumen content and the resilient modulus. The developed model performed reasonably well in predicting the fatigue life of the tested mixtures. This study would serve as the basis for selection of AC mixtures depending upon the fatigue life and would facilitate the implementation of M-E design procedure in Pakistan.

Chapter 9 describes Durability and Moisture susceptibility of asphalt mixtures, based on limited laboratory testing on selected asphalt concrete wearing and base course mixtures with a view to evaluating their susceptibility mainly towards moisture damage and durability in general. These tests include Hamburg Wheel Tracking, Modified Lottman, Loss of Marshall Stability, and Indirect Tensile Strength tests. Permeability rate in asphalt mixtures is sensitive to the type of aggregate gradation and Nominal Maximum Aggregate Size (NMAS) of the aggregate gradation. Permeability not only depends on the total amount of air voids present in the asphalt mixture, but also the interconnectivity of air voids. A study based on laboratory experimentation concludes that the maximum recommended permeability value in wearing course asphalt mixture may not exceed 1.5×10^{-2} mm/s.

Discussion and recommendations are presented in Chapter 10.

1 Introduction

1.1 BACKGROUND

The National Highway and Motorway network constitutes 5 to 6% of the total road network and carries 90 percent of Pakistan total traffic. Pakistan is gifted with a geo-strategic location with peripherals of South Asia on one side, and Central Asia on the other. In the south, the Arabian Sea forms a gateway to the vast Eurasian hinterland. This ideal location makes Pakistan one of the most attractive and shortest routes for transit to the Central Asian Republics (CARs). This fact has been recognized by the United Nations Economic & Social Commission for Asia and Pacific (UNESCAP) by designating different routes passing through various Asian countries as the Asian Highway Network (AHN). A number of inter-provincial national highways of Pakistan are a component of AHN. With the China-Pakistan Economic Corridor (CPEC) the road infrastructure and its maintenance needs are expected to grow given scarce materials and budgetary constraints.

The present national highway and motorway network is under strain due to rising traffic flows to cater local needs, heavy & uncontrolled axle loads and a slow pace of increase in road infrastructure capacity. Consolidation, preservation and improvement of existing highways & motorways, in addition to enforcement of axle loading regime, needs to be addressed immediately since precious road asset has been; and is being lost due to premature pavement distresses manifested as fatigue cracking and rutting in the asphalt concrete bound layers. This factor, which is costing Pakistan dearly, is primarily on account of non-availability of pavement structural and most importantly deficient asphalt mix designs, suiting local conditions. As a result, billions of rupees are being spent on maintenance of roads. With expanding infrastructure, huge maintenance backlog due to financial constraints has compounded the problem.

To address the issue, Highway Research & Training Centre (HRTC) launched a phased “Strategic Pavement Research Study” with Phase 1 focusing on “Improvement of Asphalt Mix Design Technology for Pakistan”. This sets the foundations of long-term project which couples two objectives i.e., to devise a pavement structural design methodology for Pakistan and to mitigate shortcomings and deficiencies in current asphalt mix design procedures through performance-based laboratory and field evaluation of existing and new asphalt mix gradations. This report addresses the performance-based laboratory research outcomes on selected indigenous materials included in Phase 1, jointly undertaken by HRTC in collaboration with Academic Partners.

1.2 PROBLEM STATEMENT

Research into the behavior of bituminous materials in pavements has attracted a great deal of world-wide attention for the past 30 years. A lot of progress has been made during this time in the understanding and modelling of the distresses in bituminous mixtures. The prediction of rut depth, for example, is an extremely complicated business, especially so when one compares it with the simple task of measuring the rut depth using a straight edge and a steel rule. The problem is not only limited to material characterization, but also to the accurate assessment of environmental conditions and calculations of the appropriate stress distribution within the pavement structure. Due to increasing traffic (both in terms of volume and axle loads) and associated road maintenance costs, it is significantly important to have reasonably accurate distress models for behavior of different pavement materials. However, the characterization of bituminous mixtures, still remains a challenge to be conquered.

Currently, in Pakistan, "AASHTO Guide for Design of Pavement Structure" and the "Marshall Mix Design" procedures are widely used for Asphalt Concrete (AC) pavement structural and bituminous mixture designs, respectively. These procedures are empirical and were developed almost half a century ago for much lighter loads and lower tyre pressures compared to that existent in Pakistan today. Therefore, these procedures need improvements for providing adequate and reliable designs for the heavy axle loads and tyre pressures existent in Pakistan.

The use of AASHTO procedure for designing AC pavements in Pakistan has been under question since the start of asphalt pavement construction where over the last three (03) decades there has been a mix of "success" and "failures". The mechanistic approach for pavement structural designs is promising but it still needs to be evaluated firstly, for its tendency to yield thicker asphalt concrete layers which needs to be addressed through a well-defined research process in view of local climatic, axle loading conditions and other cost-effective solutions such as the South African pavement design models of thin asphalt overlays. Secondly, adequate and cost-effective asphalt mix designs with appropriate equipment, local distress prediction models and its field validation have become absolutely essential. In order to proceed in direction where our large investments in highways, motorways and expressways are at stake, it is justified that asphalt mix designs are rated through performance-based testing. This will indeed save our huge investments and stop the vicious cycle where newly constructed pavements deteriorate at a much faster rate than expected.

"Strategic Pavement Research Study" is envisioned to have far reaching implications on the maintenance costs coupled with conservation of resources which presently are not being considered seriously. The study has been undertaken with a "Problem Oriented Outlook" with its mitigation through indigenous solutions where "Globally Tried and Tested Models" will be evaluated and implemented through well-defined research objectives.

1.3 OBJECTIVES & CONDUCT OF RESEARCH PROJECT

The objectives of this research were to experimentally investigate and characterize behavior of selected local aggregate gradations and binders with regard to their performance ranking based on laboratory testing. The research is part of a larger program which initially focuses on Laboratory Experiments including evaluation, modification and adoption of the bituminous mixture design methodology for designing rut and fatigue resistant pavements. The pavement research project namely "Strategic Pavement Research Study" research program is envisaged to be conducted in three phases:-

Phase I: Characterization of asphalt mixtures through Performance based Laboratory Experiments – "Improvement of Asphalt Mix Design Technology for Pakistan".

Phase II: Construction of Road Test Sections (including Instrumentation), Implementation of rut & fatigue resistant bituminous mixture designs on ongoing projects for field verification.

Phase III: Field Data Collection, Data Analysis and development of Pavement Design System for Pakistan.

This research report covers the tasks carried out in Phase I where characterization of selected asphalt concrete mixtures has been carried out through performance based laboratory experiments. The selected asphalt concrete gradations comprise aggregates and bitumen materials commonly used in local road construction industry.

1.3.1 Collaboration with Academia

To "pave the gap" between the road industry and the academia and to put in a concerted effort, HRTC has taken aboard University of Engineering & Technology Taxila, University of Engineering & Technology Lahore and National University of Sciences & Technology (NUST) Islamabad for Phase 1. The research study program is aimed towards improvement of asphalt mix design technology considering local materials, hot climatic conditions, heavy axle loads and high tyre pressures existent in Pakistan. HRTC recognizes that the industry would only benefit from close collaboration with the academia. Therefore, towards paving the gap, HRTC sponsored research students with specified area of research while pursuing their master and doctorate degrees as part of Phase I as given below:

- Development of Laboratory Rutting Transfer/Shift Functions of Bituminous Mixtures for Pakistan (PhD, UET Taxila)
- Development of Laboratory Fatigue Transfer/Shift Functions for Pakistan (MSc, NUST)
- Influence of Performance Based Binder Properties on the Distress Behavior of Asphalt Mixtures (PhD, UET Lahore)
- Mechanistic Characterization of Hot Mix Asphalt (HMA) Mixtures Using Resilient Modulus Test (MSc, NUST)
- Mechanistic Characterization of HMA Mixtures Using Dynamic Modulus Test (MSc, NUST)
- Durability Assessment of Bituminous Mixtures for Pakistan (MSc, UET Lahore)
- Durability Assessment of Bituminous Mixtures for Pakistan (MSc, UET Taxila)
- Packing Characteristics of Different Aggregate Gradations for Pakistan (MSc, UET Taxila)
- Rutting Evaluation of Hot Mix Asphalt Mixtures Using Static Creep and Repeated Load Tests (MSc, NUST)

1.4 THE RESEARCH PROJECT – (PHASE 1)

With a view to highlight the improvement needed towards asphalt paving mixtures mixture designs, the research project comprised local gradations of NHA as specified in NHA General Specifications of 1998 for both asphaltic wearing course and asphaltic base course. In addition, other gradations were adopted from Superpave and Asphalt Institute manual series which are being used around the globe. The material encompassed in the experimental design included aggregate from Margalla and Ubhan Shah quarry, and three bitumen sources of NRL 40/50, NRL 60/70 and ARL 60/70.

This thesis is organized into 10 chapters. Chapter 2 reviews the literature concerning the behavior and composition of bituminous materials. It presents a brief commentary on visco-elastic & stress-strain properties, distress in asphalt pavements, tests defining physical and mechanical properties of binders, aggregates. Bituminous mix preparation and processes involved to provide a background on the mixture research described herein. In addition, a brief on pavement design, axle loading and performance testing of asphalt concrete mixture is also presented. Chapter 3 describes aggregate, binder and mix characterization undertaken to formulate optimum asphalt mixture designs. It also comprises dynamic mechanical properties of the binders used in this project. Chapter 4 describes a method of grading analysis that takes into account the packing characteristics of individual aggregates providing a quantified criterion that can be used to control mixture properties such a workability, segregation and compactibility known as the Bailey's method. The method has been employed in particular mix designs employed for limited testing of selected asphalt mixtures. Chapter 5 describes study of dynamic modulus (|E*|) tests conducted at a range of temperatures and frequencies using asphalt mixture performance tester (AMPT). This research study also presents the catalogue for default dynamic modulus values for mixtures tested at temperatures ranges and frequencies by generating the master curves, which in turn provide the basis for the implementation of mechanistic-empirical analysis and design - an approach which may be more appropriate for heavy axle loads/ tyre pressures and climatic conditions prevalent in Pakistan. Furthermore, asphalt wearing and base course mixtures were subjected to repeated and static creep loading using Flow Number and Flow Time tests respectively for rutting susceptibility. Chapter 6 attempts to characterize different asphalt concrete mixtures using Resilient Modulus (MR) test. A comparison of resilient modulus with dynamic modulus values from the past research on similar experimental design was carried out in which a strong relation was found between the dynamic modulus values at 5 Hz load frequency with the resilient modulus values at 25°C temperature while at 40°C temperature the resilient modulus values showed a close agreement with that of dynamic modulus values at 1 Hz load frequency.

Chapter 7 describes the evaluation and ranking of selected representative asphalt mixtures with regard to their rutting susceptibility through various performance tests. A statistical analysis of the factors influencing the rutting susceptibility of asphalt mixtures has been carried towards development and validation of rutting shift functions based on laboratory performance data. Chapter 8 studies the effect of different aggregate gradations on the fatigue life using Indirect Tensile Fatigue Test (ITFT) and analyses the differential effect of penetration grade on the fatigue life. Under different stress levels, the effect of different factors including the bitumen type, the bitumen content, the resilient modulus, and the initial strain on the number of cycles to fatigue failure is investigated. The study comprises of four wearing course gradations, two bitumen sources 40/50 (NRL) and 60/70 (ARL), and single limestone aggregate source (Margalla). Chapter 9 describes limited laboratory testing on selected asphalt concrete wearing and base course mixtures with a view to evaluate their susceptibility mainly towards moisture damage and durability in general. These tests include Hamburg Wheel Tracking, Modified Lottman, Loss of Marshall Stability, and Indirect Tensile Strength tests and limited number of laboratory permeability tests. Bitumen from two refineries i.e., NRL and ARL, pertaining to penetration grade 40/50 & 60/70 respectively, has been used. Finally, a detailed discussion on the outcome of research with recommendations is presented in chapter 10.

2 Literature Review

2.1 INTRODUCTION

Asphalt concrete mainly consists of aggregates and asphalt binder. By varying the aggregates, the properties of asphalt concrete also vary and same is the case with change in asphalt binder. It has been generally observed that a pavement fails prematurely during its service life due to several factors manifesting in Asphalt Pavements in the form of distresses such as rutting, fatigue cracking, and stripping etc. A brief review of the asphalt concrete pavements, followed by individual material (aggregates and bitumen) properties, and the asphalt mixture performance evaluation is discussed in this chapter.

Primarily, the objective while designing an asphalt concrete pavement is to guard against vertical compressive stress on top of the subgrade and limiting horizontal tensile strains at bottom of the asphalt concrete layer. Both of these parameters are regarded as design criterions for structural deformation (rutting) and fatigue cracking, respectively. Recent developments in mechanistic analysis computer programs tend to increase the asphalt concrete thickness with increase in the design load applications, which is strictly based on the concept to mitigate bottom-up cracking. However, by increasing the asphalt concrete thickness greater than that required to keep strain levels at the bottom of asphalt concrete layer below the endurance limit, may not necessarily provide more life to the pavement but instead tend to over budget the project (NCHRP 646). There are many factors that may affect the longevity of asphalt concrete e.g., critical mix design parameters and endurance against the elements such as climatic conditions and moisture porosity etc. Long term performance of the flexible pavements in general is dependent on variable factors, therefore, in-depth understanding of the local pavement materials carries a lot of significance for the development of indigenous mix designs and pavement design solutions.

2.2 ASPHALT CONCRETE PAVEMENTS

Asphalt concrete is the main constituent of the modern flexible pavements. The design of flexible pavements has progressively evolved from an art to a complicated science, but the empirical approach still carries an important role in pavement design. In earlier days up to 1920s, thickness of pavement was determined on the basis of experience or certain thumb rules and the same thickness was used throughout a particular section irrespective of the varying underlying soil conditions. As time passed, experience gained by different agencies resulted in development of various methods to determine thickness requirement for a particular set of conditions.

Flexible pavements respond to wheel loading elastically, distributing the load to the underneath within a layered system (Lenz 2011). The load applied on top of the flexible pavement causes stresses and strains throughout the layers from top to the bottom. As the wheel load gets distributed over a larger area of each underlying layer, the magnitude of the resulting forces (stresses and strains) decreases from top to bottom. The typical cross section of the layered system in a flexible pavement is shown in Figure 2.1.

The design of flexible pavements is classified into five different categories including the empirical method with or without the soil strength, the limiting shear failure method, limiting deflection method, and regression method based on the road test or pavement performance and mechanistic-empirical method (Huang). The mechanistic-empirical method is based on the mechanical properties of the materials. A relationship can be developed between the input, in form of loading, and the output, in form of stresses and strains in the pavement. The stress and strain response values are used to determine the distress using the data obtained from laboratory and field tests. This approach is much better as the performance cannot be determined by theory alone. In 1960 Saal and Pell for the very first time recommended the use of horizontal tensile strains at the bottom of the asphaltic layer to minimize the effect of fatigue cracking, resulting in bottom-up cracking.

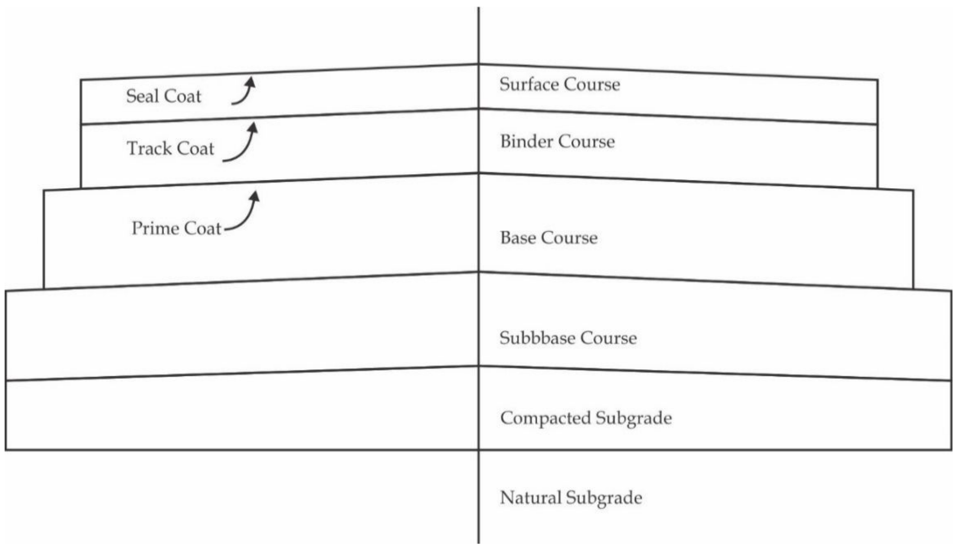


Figure 2.1: Typical Cross Section of the Layered System in Conventional Flexible Pavement

2.2.1 Viscoelastic Properties of Asphalt Concrete Pavement

The materials which store some of the mechanical energy and dissipate the rest in response to any mechanical stress are defined as viscoelastic materials. Robert. N. H, (2000), described bitumen as a viscoelastic material and this behaviour is dependent on both temperature and the time of loading. Bitumen at low temperature and shorter loading times behaves as an elastic solid, whereas at higher temperatures and loading times it behaves as a viscous material. At intermediate temperatures this material has a very complex nature and behaves as a viscoelastic material.

As asphalt binder (bitumen) is one of the main constituents of asphalt concrete, flexible pavements to greater extent exhibits the same viscoelastic behaviour. Figure 2.2 shows the response of elastic, viscous and visco-elastic material under same stress. In the elastic materials the strains produced in the material are proportional to the stresses applied, and once the load is removed the material recovers completely to its original position. Strain in viscous materials increases with time under constant loading, and once loading is removed the material stays in the deformed state. The behaviour of viscoelastic materials is a mix of both viscous and elastic materials, the constant stress increases the strain and once the load is removed the material recovers while some deformation is retained by the material which is permanent.

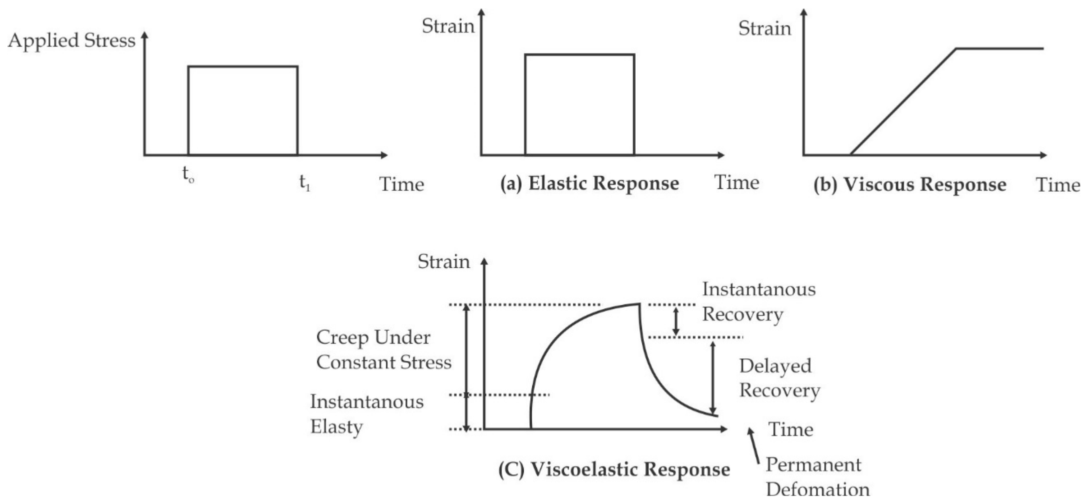


Figure 2.2: Response of material under constant stress loading (Van der Poel, 1954)

2.2.2 Stress & Strain in Asphalt Concrete Pavement

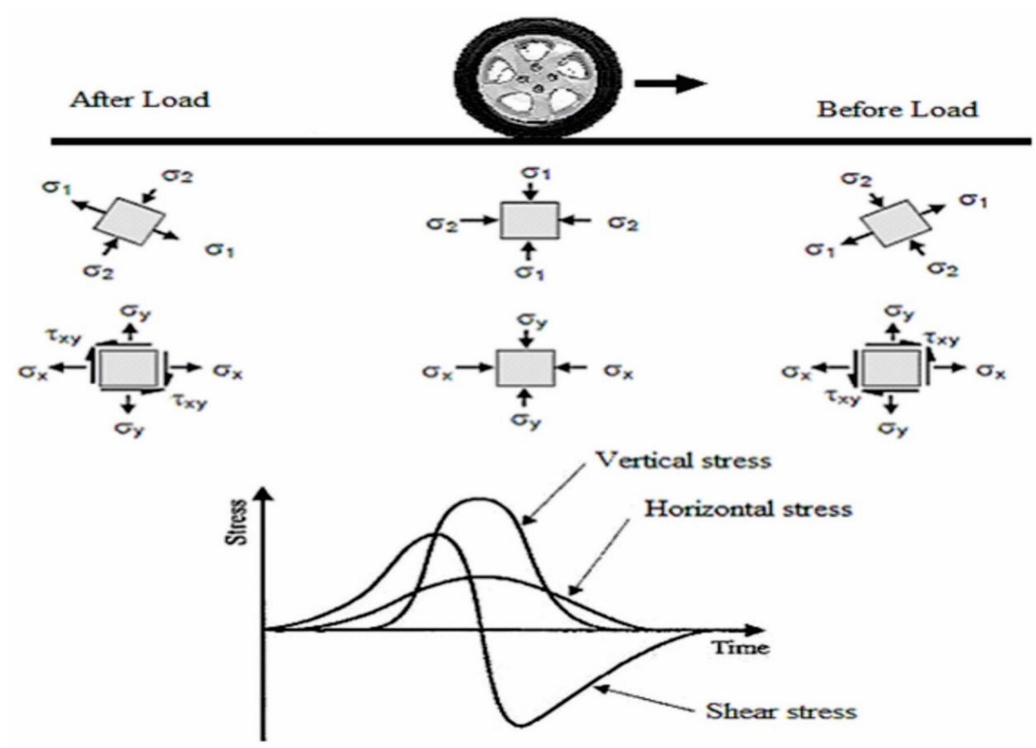


Figure 2.3: Principal stresses under the rolling wheel load (Shaw, 1980)

The structure of the flexible pavement generally consists of an asphaltic layer on top (asphaltic wearing course and asphaltic base course) with underlying crushed stone base and sub base layer. These upper layer configurations are provided with a compacted foundation i.e., subgrade as shown in Figure 2.1. The top layer of the pavement needs to be smooth to provide a comfortable ride and at the same time strong enough to resist the stresses applied to it by a moving wheel load.

Presence of viscoelastic binder in asphalt concrete pavements make them flex and return to original state under a moving load. Asphalt concrete pavements should have a balance of stiffness and flexibility so that they are capable of resisting permanent deformation, and also has sufficient tensile strength to resist fatigue cracking at the bottom of the bound layer after many load repetitions. Figure 2.3 shows the orientation of principal stresses (vertical, horizontal and shear stresses) induced in the flexible pavement with the rolling wheel passing over it.

2.2.3 Distresses in Asphalt Concrete Pavements

The distresses in pavement are classified into three major groups named as cracking, deformation and surface defects (Miller and Bellinger, 2011). Distress is the most important factor to be considered in the pavement design as it is directly related to the pavement performance. Among the various distress types, most of them are caused due to deficient materials, poor construction, and improper maintenance. Therefore, premature distress may not be due to in-adequate pavement design. Distress evaluation in pavements is an important part of the pavement management system. If a pavement is evaluated properly and a strategy is developed according to standard methods, maintenance and rehabilitation measures adopted prove to be effective.

Typical pattern observed during deterioration of asphalt concrete pavements is rutting, which develops rapidly in the initial years. Fatigue cracking usually does not occur until several repeated loadings but once it initiates, spreads rapidly.

2.2.4 Desirable Properties of Asphalt Concrete Pavements

A durable and well performing asphalt concrete mix exhibits various properties such as stability, fatigue resistance along with tendency of self-healing, skid resistance and workability. Stability is the ability of asphalt paving mixtures to resist the load associated deformation, through frictional and interlocking resistance of aggregates in the mix. Stability is related to the stiffness of the mix imparted by the asphalt and aggregate matrix. This property of the asphalt mix towards resistance of permanent deformation has an important role at higher temperature regimes.

Fatigue or crack resistance is the ability of asphalt bound layers within a pavement system to withstand crack initiation due to repeated load application over a period of time. Stiffer mixtures desirable for rut resistance may not provide desirable fatigue resistance, since stiffness of the mix is inversely related to the fatigue resistance.

Durability is the property of asphalt paving mixtures to resist the detrimental effect of water, air, temperature, and traffic. Therefore, the mix should contain sufficient bitumen to ensure an adequate film thickness around the aggregate particles. It helps to minimize the aging and hardening of bitumen while in service. The compacted asphalt mixtures should not have very high air voids to allow water penetration through the aggregate-asphalt matrix. Keeping the air voids in range reduces permeability and aging.

Skid resistance is the ability of asphalt pavement surface to offer resistance against slippery conditions. This property of the pavement surface is particularly important in wet conditions. It is directly related to vehicle's braking performance against skidding.

Another property of the asphalt mix which may affect its in-service performance is workability. The mix must possess smooth applicability during paving operation and to be compacted with reasonable effort. When more effort is required for placement and compaction, it means that the mix has very low workability which is not desirable for proper laying of asphalt paving mixtures.

2.2.5 Material Properties

The Asphalt Institute procedure which is based on elastic theory characterizes the properties of pavement component materials through elastic modulus i.e., resilient modulus and a Poisson's ratio. The elastic modulus of bituminous mixture varies with the loading time (due to the viscoelastic nature of bitumen); therefore, resilient modulus specific to corresponding material layer is selected for analysis and design of flexible pavements.

2.2.6 Pavement Design Parameters

The major design considerations for the structural design of flexible pavements includes the selection of design input values, i.e., traffic and loading, material properties and environmental conditions (MS-1 Asphalt Institute). In addition to the mentioned design parameters, pavement design guide (AASHTO 2002) takes pavement distresses as another input parameter.

2.2.7 Traffic Loading

Traffic loading takes into account number of standard axle repetitions. The entire spectrum of traffic with various axle configurations are converted into Equivalent Single Axle Load (ESAL) which is defined as a standard 80-kN (18-kip) single-axle load. This concept was originally devised by AASHO road test to convert the entire traffic load spectrum to a standard loading. Therefore, the design of pavement is based on number of ESAL repetitions during its design life. The principal criterions for flexible pavement design are limiting the vertical compressive stress on top of the subgrade against structural rutting and horizontal tensile strains at the bottom of asphalt concrete layer to mitigate load induced fatigue cracking.

Viscoelastic nature of asphalt concrete mix is directly related to the time of loading i.e., load duration. With the application of load, the bituminous material exhibit deformation that is time dependent. The applied load duration in the asphalt layer depends on the speed of the vehicle. Therefore, in the elastic theory of pavement analysis and design, resilient modulus value for each pavement layer should be selected according to the vehicle speed. In calculating resilient modulus of asphalt whether for field or laboratory analysis, the duration of load pulse is the function of vehicle speed. Loading time of higher speed vehicles is low and it results in smaller deformation and large resilient modulus.

2.2.8 Environment

The major environmental factors in pavement design are temperature, moisture levels and frost penetration. Temperature significantly affects the resilient modulus of asphalt concrete layer making it susceptible to shear flow under wheel loads causing non-structural rutting. Conversely, extremely low temperatures cause the asphalt concrete to crack especially with low penetration grade binders. Moisture content is another factor that affects the resilient modulus of subgrade soil and upper pavement layer configuration. On the other hand, frost penetration and related thawing phenomenon during the winter and spring seasons affects the entire pavement system.

2.2.9 Failure Criteria

As discussed earlier major flexible pavement failure criterions are rutting and fatigue cracking. However, many design guides also take into consideration some functional parameters such as ride quality. For example, AASHTO 1993 pavement design guide uses Present Serviceability Index (PSI) as the main functional parameter. This document however mainly focuses on the structural performance evaluation. A relatively recent advancement in this field is development of mechanistic-empirical (ME) pavement design approach which has become an adequate alternative and has been made part of the new pavement design guide (AASHTO 2002). The ME pavement design method also comprise the failure criterions as rutting, fatigue cracking in addition to low temperature cracking.

2.3 Material Properties for Asphalt Concrete

2.3.1 Aggregate Evaluation

"Aggregate" is a collective term for the mineral materials such as sand, gravel and crushed stone that are used with a binding medium (bitumen in the case of flexible pavements) to form asphalt concrete. Aggregates generally account for 92 to 96 percent of asphalt concrete by volume.

Aggregates can either be natural or manufactured. Natural aggregates are generally extracted from larger rock formations. The rocks extracted are typically reduced to a usable size by means of mechanical crushing.

Most of the engineering properties of asphalt mixture such as stiffness, stability, permeability, workability, fatigue resistance and resistance to moisture damage are defined by the aggregate gradation.

2.3.1.1 Mineral Properties

The mineral composition of an aggregate largely determines its physical characteristics and its behaviour as a pavement material. Therefore, when selecting an aggregate source, knowledge of the quarry rock's mineral properties can provide valuable information regarding the suitability of the resulting aggregate.

2.3.1.2 Chemical Properties

The chemical properties of aggregates are an important factor in pavement-aggregate material system. In asphalt concrete, aggregate surface chemistry can determine how well an asphalt cement binder will adhere to an aggregate surface. Poor adherence, commonly referred to as stripping, can cause premature structural failure. Some aggregate chemical properties can change over time, especially after the aggregate is crushed. A newly crushed aggregate may exhibit a different affinity for water than the same aggregate that has been crushed and left in a stockpile for a year.

2.3.1.3 Physical Properties

The physical properties of the aggregates are the most readily apparent and they have the most significant effect as material constituent in a pavement system. The most common measured physical aggregate properties are, Gradation and size, Toughness and abrasion resistance, Durability and soundness, Particle shape and surface texture, Specific gravity, Cleanliness and deleterious materials moisture absorption.

Aggregate gradation and aggregate shape properties or morphology of aggregate materials have been recognized by the Strategic Highway Research Program (SHRP) among the top factors that influence the stability of asphalt concrete mixtures. In dense graded asphalt mixtures, coarse aggregate size and shape properties are believed to enhance the in-service performance in high temperature and loading conditions.

2.3.2 Effects of Aggregate Gradation and Size

Selection of aggregate gradation for use in asphalt concrete pavement is important to pavement performance. Most of the performance of asphalt mixture mainly depends on aggregate properties and its composition i.e., aggregate gradation. Traditional method of gradation design follows maximum density line (MDL), along with some deviations from MDL to fulfil the minimum Voids in Mineral Aggregate (VMA) criteria (Asphalt institute, 2001). Adopting the maximum density line in conventional practice sometime does not produce a packed aggregate skeleton, which may be due to particles shapes and texture.

The conventional techniques for aggregate blending lacks accommodating the effect of aggregate shape and texture, resulting common pavement failures such as fatigue cracking and rutting. Actually, MDL targets densest packing in which coarse aggregates and fine aggregates combines in such a way that spaces created by coarse aggregates are filled with fine aggregates leaving minimum voids spaces. Also, the densest packing results in reducing the binder content. The amount of a particular size required to produce a densest aggregate gradation depends upon the physical characteristics of a particle. Packing characteristics commonly depends upon aggregate shape, texture and strength. Therefore, a single methodology of blending the aggregate does not work on all types of aggregates. Also, the conventional gradation does not include aggregate source properties; therefore, targeting midline alone may not guarantee the desired durability of asphalt mixtures.

In Pakistan, aggregate gradations are being designed using a conventional method that targets density line. This concept may lead to erroneous results, since the shape of particles do affect the distribution of voids. Conventional method of aggregate gradation is a trial-and-error method, which also depends upon the designer experience and may vary from person to person. To avoid this problem a new definition of coarse and fine aggregate has been developed by Bailey. In this method the concept of packing of aggregate to each particular size has been used. Arrangement of solids in a specified volume is termed as packing. It is desirable to attain the maximum possible packing of material, while blending the granular materials with binder. For asphalt mixtures, it is critical to obtain the strength from bounded material (Hafeez et al., 2014).

Aggregate proportions are approximately 95% of the volume of asphalt concrete mixture (TRB, 2012), and the distribution of aggregate played a critical role in a specific packing. It controls both volumetrics and aggregate interlock of a packed structure (Vavrik, W.R. et al., 2002). Dense graded as well as gap graded asphalt mixture's strength and stability depends on aggregate contact characteristics and interlocking of aggregates. Both the properties affect the load bearing capacity of mixtures (Hafeez et al., 2014). In pavement engineering, traditional asphalt mixture's design methods aim to design an aggregate skeleton to achieve densest packing for the purpose of higher strength and reduced amount of binder (Roberts, F.I. et al., 1996). The aggregate size distribution or overall aggregate gradation is one of the primary characteristics to assess an overall aggregate blend behaviour in the pavement. Almost all-important properties of an asphalt mixture, such as susceptibility to moisture damage, resistance against fatigue cracking and rut, tenderness of mix, stiffness and frictional resistance depends on packing of aggregate gradation (Shen, S. and Yu, H., 2011; Anthony, D.S. et al, 2003; Haritonovs, V. et al 2013; Vasconcelos, K.I. et al 2011). Pavement distresses can be reduced by improving the aggregate gradation or distribution of aggregate sizes (Hafeez et al., 2010). Different approaches have been used to improve the aggregate gradations, but to date Bailey method of aggregate packing proved to be one of the better techniques.

Bailey defined the coarse and fine aggregate using the packing theory, which is a more systematic approach to control the air voids in an aggregate blending. Bailey used three controlling factors of gradation named as Coarse Aggregate Ratio (CA ratio), Fine Aggregate Ratio of Coarse Portion (FAc ratio) and Fine Aggregate Ratio of Fine Portion (FAf ratio). These controlling factors are linked with voids in mineral aggregate (VMA), air voids (Va) and behaviour of mix during the compaction. Bailey method provides a set of tools by utilizing the aggregate size distribution and can be used to develop aggregate blends for any type of mix design method. This technique can be utilized in Marshall Method as well as in Superpave Method of mixture design (Botasso et al. 2012). Volumetric properties in the asphalt mixture can be controlled empirically using different ratios. The basic purpose of developing this method was to improve the rut resistance of an asphalt mixture [TRB, 2012]. Fine graded asphalt mixtures developed by Bailey method showed better durability against moisture (Al-Qadi et al. 2013; Goh et al. 2013). Bailey method improved packing of aggregates and other parameters, which are directly linked with VMA, Va and workability. This section discusses in detail the adoption of Bailey method in the improvement of local asphalt mixtures. Limits of different controlling factors have been proposed as findings of the current study.

Recent guidance for asphalt concrete gradation has been defined in terms of the Superpave restricted zone. This zone is located along the maximum density line between the 0.300 mm and 4.75 mm or 2.36 mm particle sizes (depending on the maximum nominal size of the aggregate). Avoiding this zone was intended to limit the amount of rounded natural sand that can contribute to mixture instability. Some research suggests that aggregate gradations plotting below this zone produce more rut-resistant mixtures.

Comparing asphalt concrete pavement performance based only on different aggregate gradations is not a simple matter. The inter-relationships among aggregate gradation, aggregate characteristics, and asphalt concrete volumetric properties are complex. In general, dense-graded asphalt concrete mixtures with adequate Voids in the Mineral Aggregate (VMA) provide improved resistance against degradation and fatigue cracking when used in thick pavements (Kandhal and Parker, 1998).

Observations concerning the effects of aggregate gradation and size on asphalt concrete mixture properties and performances as reported in the literature are as follows:

- Asphalt concrete mixtures with gradations passing through the restricted zone exhibit higher bulk density and lower air voids than mixtures with gradation plotting below and above the zone (Chowdhury et al. 2000).
- Fine-graded asphalt concrete mixtures have better fatigue performance than more coarsely graded mixtures (Sousa et al. 1996).
- Triaxial test results indicate that fine-graded asphalt concrete mixtures have greater shear strength than those with coarser gradations (Haddock et al. 1999).

The performance of asphalt concrete mixtures is also influenced by the maximum aggregate size. Asphalt concrete mixtures with larger maximum aggregate sizes were reported to exhibit better rutting performance than those with smaller maximum aggregate sizes (Stuart, Romero and Mogawer, 1999; Brown and Basset, 1990). Khedaywi and Tons (1998), also found that smaller coarse aggregate particles provided more aggregate interlocking and resulted in increased asphalt concrete shear strength.

Standard test techniques are used for proper characterization of aggregate material. Test results are an important part of mix design and can help predict pavement quality. Aggregate (coarse and fine) testing is carried out in accordance with ASTM standards.

The list of tests generally carried on the aggregate material is given in Table 2.1 and 2.2.

Table 2.1: Description of Quality Tests for Coarse and Fine Aggregates

Sr. No.	Quality Tests	Test Standards
	Coarse Aggregate	
1	Fractured Particles	ASTM D 5821
2	Flat and Elongated Particles	ASTM D 4791
3	Resistance to Degradation	ASTM C 131
4	Durability and Soundness	ASTM C 88
5	Deleterious Materials	ASTM C 142
	Fine Aggregate	
6	Un-compacted Voids	ASTM D 1252
7	Sand Equivalent	ASTM D 2419
8	Durability and Soundness	ASTM C 88
9	Deleterious Materials	ASTM C 142
	Coarse Aggregate	
1	Water Absorption	ASTM 127
2	Bulk Specific Gravity	ASTM 127
3	SSD Specific Gravity	ASTM 127
4	Apparent Specific Gravity	ASTM 127
5	Gradation	ASTM 136
6	Unit Weight, Loose	ASTM 29
7	Unit Weight, Rodded	ASTM 29

Sr. No.	Quality Tests	Test Standards
	Fine Aggregate	
9	Water Absorption	ASTM 127
10	Bulk Specific Gravity	ASTM 127
11	SSD Specific Gravity	ASTM 127
12	Apparent Specific Gravity	ASTM 127
13	Gradation	ASTM 136
14	Unit Weight, Loose	ASTM 29
15	Unit Weight, Rodded	ASTM 29

2.4 Asphalt Characterization

Asphalt or Bitumen is known to be a dark brownish to blackish coloured viscous hydrocarbon that is produced from the distillation of residues of petroleum. This distillation either occurs naturally in lakes, or in a petroleum refinery from the residue left after distillation of crude oil. The physical properties of asphalt largely influence the performance of asphalt concrete mixtures. Properties of asphalt change over time and its age hardening is an important factor to predict useful service life. The state-of-the-art Superpave testing system, also known as the Performance Grading (PG) system, developed in SHRP research program measures the physical properties of asphalt that simulates the field conditions. The Performance Grading (PG) system emphasizes strongly to control the viscosity at low temperature, however in Pakistan, major concern lies with the asphalt mixture performance under high temperature conditions.

It is necessary to check the suitability of both aggregates and asphalt for the preparation of asphalt concrete mixtures. Selected binder materials are characterized using different tests which include property tests and performance tests. Typical tests that are carried out on asphalt are shown in Table 2.2.

Table 2.2: Description of Property and Performance Tests for Asphalt

Sr. No.	Binder Tests	Test Standards
	Property Tests	
1	Flash and Fire point	ASTM D 92
2	Penetration	ASTM D 5
3	Ductility	ASTM D 113
4	Softening Point	ASTM C 36
	Performance Tests	
6	Rotational Viscometer (RV)	AASHTO T 316
7	Rollin Thin Film Oven (RTFO)	ASTM D 2872
8	Pressure Aging Vessel (PAV)	ASTM D 6521
9	Bending Beam Rheometer (BBR)	ASTM D 6648

It is necessary to perform tests according to the specifications to verify the acceptability of asphalt cement satisfying the desired characteristics including consistency, purity and safety. Different tests including property tests and performance tests must be conducted on the asphalt cement before asphalt concrete mixture preparation.

2.5 Types of Bituminous Mixtures

The bituminous paving mixtures are divided into three different categories, depending basically on the gradation of aggregate used in the mix. These three types are; dense graded, open graded, and gap graded asphalt mixtures (MS-2 Asphalt Institute).

Dense graded bituminous mixtures consist mainly of well graded aggregates i.e., all sizes of coarse aggregate, fine aggregate, and filler mixed with asphalt cement binder. The dense graded mixtures are usually referred by their nominal maximum aggregate size. These mixtures work well for the structural, patching, friction, and leveling needs. The open graded bituminous mixtures usually consist of large quantity of coarse aggregates and small number of fine aggregates mixed with bitumen. The use of these mixtures is to provide an open surface texture that will allow the water to drain into the mix. The mix design procedure of the open graded mixtures is different from dense graded bituminous mixtures due to the lack of fines in the mix. Also, the quantity of bitumen is less in open graded mixtures as compared to the dense graded mixtures.

A gap graded asphalt mixture is usually same as an open graded mixture but the amount of fine aggregates added into these mixtures is usually greater than the amount used in open graded mixtures. The materials for gap graded mixtures are crushed stone and gravels with bitumen and the manufactured sand. The intermediate size aggregates that are between # 4 and # 30 sieves are missing or present only in very small quantity.

2.5.1 Bituminous Mix Preparation

The standard method for preparation of bituminous paving mixtures is by using Marshall Procedure (ASTM, D6926). The laboratory preparation of bituminous paving requires batching of aggregates, mixing in proper amount of bitumen (after trials), heating the mixture to proper temperature followed by compaction of specimens. Bitumen and aggregates are thoroughly mixed at designated mixing temperatures. Temperature must be similar to temperature of asphalt mixing plant. Mechanical mixer is recommended for laboratory bituminous mixture preparation since mixing large quantity of material by hand is difficult. Mixing must be continued until the bitumen is uniformly distributed over the aggregate. The mix is placed in a pre-heated mould and compacted by a rammer with blows on either side depending upon the traffic condition. The weight of mixed aggregates taken for the preparation of the specimen may be suitably altered to obtain a compacted thickness of 2.5-inch. Approximately 4000 gm of aggregates are sufficient for preparation of specimens of thickness 6-inch and 3.75-inch diameter (MS-2 Asphalt Institute).

2.5.2 Compaction

The standard method for preparation of bituminous paving mixtures (ASTM D6926), recommends three kinds of Marshall compaction apparatus i.e., Compaction hammers with a manually held handle, Compaction hammers with a fixed hammer handle, and compaction pedestal

Manually held hammer usually has a flat, circular compaction foot with spring loaded swivel and 4.54 kg sliding mass with a free fall of 457-mm. Compaction hammer with a fixed hammer handle is mechanically operated. It comprises a surcharge on top of handle which is constantly rotating the base. It has a slanted, circular tamping face and a 4.54kg sliding weight with a free fall of 457.2-mm. A rotating mechanism is incorporated in the base. The base rotation rate and hammer blow rate are 18 to 30 rpm and 64 blows per minute, respectively. The compaction Pedestal consist of a nominal 8 by 8-inch wooden post approximately 18-inch long, capped with a steel plate approximately 12 by 12-inch and 1-inch thick. The wooden post may be oak, yellow pine, or other wood having an average dry density of 42 to 48 lb/ft^3.

2.5.3 Volumetric Analysis

The volumetric properties of compacted bituminous paving mixtures provide some indication of pavement service performance (MS-2 Asphalt Institute). The volumetrics include determining the volume of air voids, voids in mineral aggregate, and the voids filled with asphalt within an Asphalt Concrete Mixture. Bulk specific gravity of combine aggregate gradation, compacted mix, and the maximum theoretical specific gravity of the loose mixes are the preamble for volumetrics.

Air voids (VA) in compacted paving mixture consist of the small air spaces between the coated aggregate particles. Voids in mineral aggregate (VMA) are the intergranular void space between the aggregate particles in a compacted asphalt paving mixture that include the air voids and the effective bitumen content, expressed as a percent of total volume. The VMA is calculated on the basis of the bulk specific gravity of the aggregate and is expressed as percentage of the bulk volume of compacted paving mixture. VMA significantly affects the performance of asphalt mixtures because if it is too low, the mix may suffer durability problems, and if it is too high, the mix may show stability problems. The volume of bitumen in the mix along with the aggregate determines the thickness of the bitumen film around each aggregate particle. Without adequate film thickness, the bitumen oxidizes faster while thin coating is more easily penetrated by moisture, and tensile strength of the mix is badly affected. The VMA for any given mix must be sufficiently high to make sure that there is room for the asphalt plus the air voids.

Voids filled with asphalt (VFA) is the percentage of the intergranular void spaces between the aggregate particles (VMA) that is filled with asphalt.

2.5.4 Stability and Flow

Marshall stability and flow parameters are used in evaluation of asphalt mix design (ASTM D6927). These parameters are used to monitor the homogenous production of asphalt paving mixtures in the batching plant, as well. Tests are performed using compression testing machine. The stability of the mix measures the maximum load that is supported by test specimen at the constant loading rate of about 2-inch/minute. Basically, the load is increased until it reaches the maximum. When the load just starts to decrease, the loading is stopped and the maximum load is recorded. During the loading, an attached dial gauge to the machine, measures the specimen's flow as a result of the loading. The flow value is recorded in 0.01-inch increments at the same time the load is recorded.

2.6 Performance Evaluation of Asphalt Concrete Mixtures

2.6.1 Permanent Deformation

Rutting or permanent deformation is considered one of the major distress mechanisms in flexible pavements, which mostly occurs in asphalt bound layers due to lateral or shear flow of asphalt concrete under the wheel load. This is also the most common type of distress in Pakistan. Such deformation is load and temperature dependent which may include initial densification due to traffic compaction. Permanent deformation in asphalt (flexible) pavements, commonly referred to rutting, usually consists of longitudinal depressions in the wheel paths, which are an accumulation of small amounts of irrecoverable deformation caused by each load application as shown in Figure 2.4 (Asphalt Institute,1996). If an asphalt mixture ruts, it is normally because the mixture has insufficient shear strength to support the stresses to which it is subjected. (Sousa et al. 1991)

Research has shown that rutting in the asphalt concrete layer will generally occur within the top 3 to 5-in. If a poor quality asphalt mixture is being used, increasing the thickness will not decrease rutting. In fact, improving the material properties and mix characteristics will significantly decrease the rut depth (Kennedy, et al, 1996).

Mechanistic-Empirical Pavement Design Guide (MEPDG) has defined three distinct stages for the permanent deformation behaviour of asphalt materials under a given set of loading and environmental conditions. Primary stage has high initial level of rutting, with a decreasing rate of plastic deformations, predominantly associated with volumetric change. Secondary stage has small rate of rutting exhibiting a constant rate of change of rutting that is also associated with volumetric changes; however, shear deformations increase at increasing rate. While the tertiary stage has a high level of rutting predominantly associated with plastic (shear) deformations under no volume change conditions as shown in Figure 2.5 (AASHTO Design Guide, 2002).

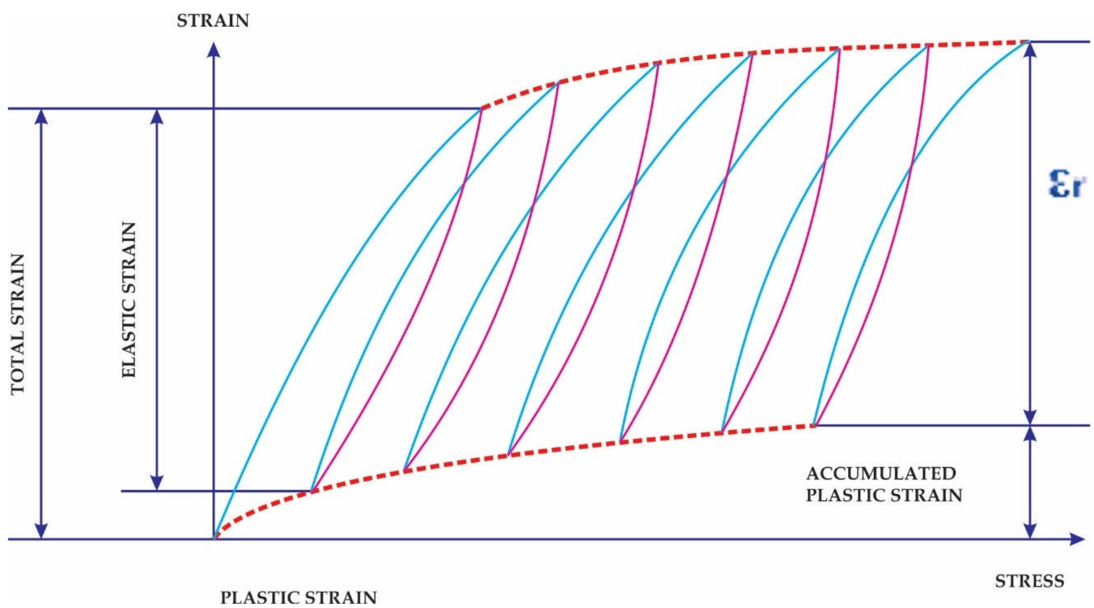


Figure 2.4: Accumulated Plastic Strains in Pavements (After Asphalt Institute, 1996)

If an asphalt material is loaded with a stress that is above the flow strength of the material, at that temperature the material will start to deform (Stumpf, 2007). First the material will deform rapidly, then, after some strain hardening has taken place, the material gets to a stage with a lower creep rate as shown in Figure 2.5. This stage is known as secondary creep, or steady state creep. In the third stage the material becomes unstable and results in rapid collapse.

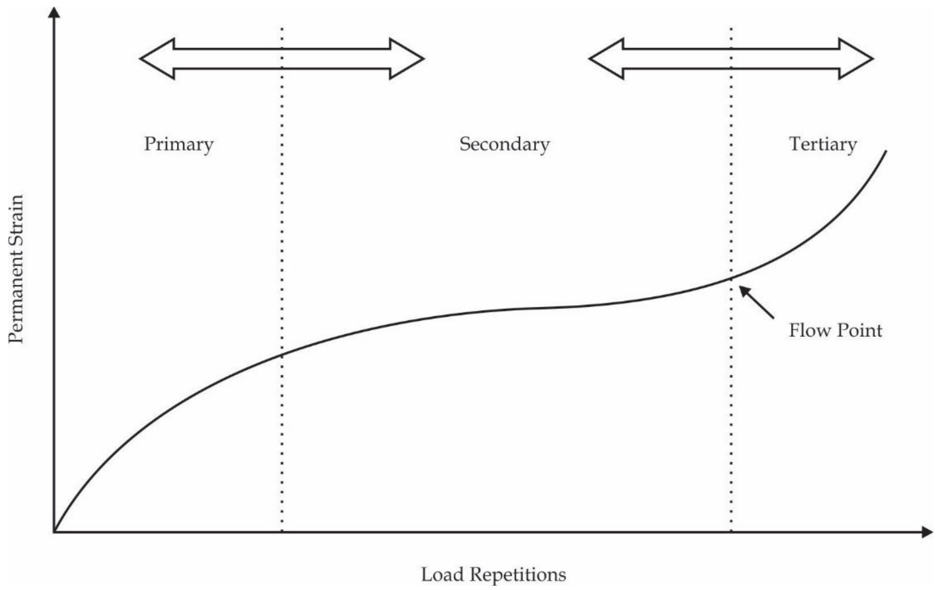


Figure 2.5: Typical Repeated Load Permanent Deformation Behavior of Pavement

The proportion of permanent deformation as shown in Figure 2.5 in different creep phases is important. The critical rut depth is generally set at 10 - 12 mm, if this depth is reached in the primary phase or in the first part of the secondary phase, the functional life of asphalt concrete layer is reduced drastically. In secondary phase, the rate of deformation slows down considerably. The dominant mode of deformation is caused by shear stress that overcomes flow strength of the material. The flow strength consists of two components, i.e., friction and cohesion. The deformation takes place in small iterations with each load application. Eventually, the road condition and the level of permanent strain will cause the asphalt mix to enter the tertiary phase and rapid unstable shear failure occurs (Carpenter, 1993).

2.6.1.1 Rutting Susceptibility

Rutting is recognized by a continuous depression parallel to traffic direction along the wheel paths. Asphalt is dependent on the loading time and temperature. it is normally observed at locations where heavy traffic travels at slow speed, such as intersections, bus bays, turning lanes etc. In Pakistan, rutting is a predominantly observed distress due to high pavement temperatures coupled with high tire pressures and unregulated heavy truck traffic.

Rutting susceptibility is dependent on material quality, irrespective of layer thickness (Parker, 1990). Rutting in flexible pavements is further dependent on various factors such as aggregate gradation, types of aggregates, binder type and its properties and extent of applied compaction efforts. Factors related to loading pattern includes: types of vehicles, tire material, tire pressure, speed of vehicles & axle load. Similarly, environmental factors, such as climatic conditions of the area and pavement temperature also affect the type and extent of rutting. Gu et al., (2015) have reported that low vehicle speed and overloading significantly increases the rutting potential of a pavement. Sel et al., (2014), conducted statistical analyses and concluded that there is a significant effect of test temperature and binder type on the Hamburg wheel tracker test results. A study by Xiao et al., (2007), showed that as air voids in the modified mixtures decrease, the rut depth from the Asphalt Pavement Analyzer (APA) test also decreases.

For determining simulated rutting resistance of recycled asphalt mixtures, Hamburg Wheel Tracking Test (HWTT) was used by Hafeez et al., (2014). Zhang et al., (2015), have reported that the response of asphalt mixtures to various test conditions by Partial Triaxial Compression Test (PTCT) is in accordance with that of typical triaxial compression tests. Doyle and Howard, (2013), have reported that PURWheel (Purdue University Wheel Tracker)-dry and APA (Asphalt Pavement Analyzer) were observed to be equally good indicators of rutting potential. Al-Hadidy and Yi-Qiu, (2011), & Chandra and Choudhary, (2013), used wheel tracking test to study the effect of modifiers in asphalt mixtures. The Asphalt Pavement Analyzer (APA) and repeated simple shear test at constant height (RSST-CH) were validated as useful Accelerated Pavement Testing (APT) tools by Martin and Park, (2003). Choubane et al., (2000), Rushing et al., (2012) & Kim et al., (2009), inferred that the APA may be used as an effective tool to rank asphalt mixtures in terms of their respective rut performance.

Rushing and Little (2014), presented results from a laboratory study to identify a performance test for accepting hot asphalt mixtures for constructing airport pavements. Statistical analyses performed on the results indicated that the rate of increase in permanent strain and the flow time value determined from triaxial static creep testing provides the strongest correlation to APA rutting. Chen et al. (2007), concluded that a strong correlation exists between the French Loaded Wheel Tracker (FLWT), Dynamic Stability (DS) and APA rut depth for the mixtures studied. Ahmed & Erlingsson (2014), carried out two different types of tests: an extra-large wheel-tracking (ELWT) test and a full-scale accelerated pavement test using a heavy vehicle simulator (HVS) to compute model parameters for Mechanistic Empirical Pavement Design Guide (MEPDG) model. Rutting was modeled as a function of traffic, temperature, and mix characteristics, including voids filled with asphalt, asphalt content, in-place air voids, surface area, and the densification slope by Archilla (2006). Hu et al., (2011), presented a new mechanistic-empirical M-E rutting model developed for hot-mixed asphalt asphalt concrete overlay thickness design and analysis. A research described by Suh and Cho (2014), explained the development of a model for estimating the rutting performance of dense graded asphalt concrete pavement under various temperatures and loads.

2.6.1.2 Repeated Load Axial Test

The performance of asphalt concrete pavements is considerably reduced with manifestation of rutting (Permanent deformation) which is load and temperature associated distress. Over the years, pavement researchers have discovered various means towards understanding of this complex behaviour. Flow time (FT) test also known as static creep test is the recommended tests in NCHRP Project 9-19 to determine fundamental properties of asphalt paving mixtures subjected to rutting performance (Witczak and Kaloush, 2002). Visco-elastic behavior of asphalt mixtures under a static stress level can be measured by this test. FT test is helpful in understanding the material's response by providing sufficient information about instantaneous elastic (recoverable) and plastic (irrecoverable) components as well as the viscoelastic and visco-plastic components. Total compliance at any given point in time can be calculated as ratio of measured strain to applied stress. The total compliance is distributed into three zones namely primary, secondary and tertiary zone. The primary zone refers to portion in which strain rate decreases with loading cycle while in secondary zone, strain rate is constant with load cycle. However, the tertiary zone indicates that strain rate again increases with load cycles. The Flow Number (FN) test is dynamic type of creep test where a haversine type of loading with 0.1 seconds loading time and 0.9 seconds of rest period. The FN test has ability to evaluate fundamental properties of asphalt mixtures which can correlate field rutting performance for different stress levels (Mohammed et al. 2006). Therefore, the FN and FT are important tests which help in characterization and selection of best available rut resistant asphalt mixtures.

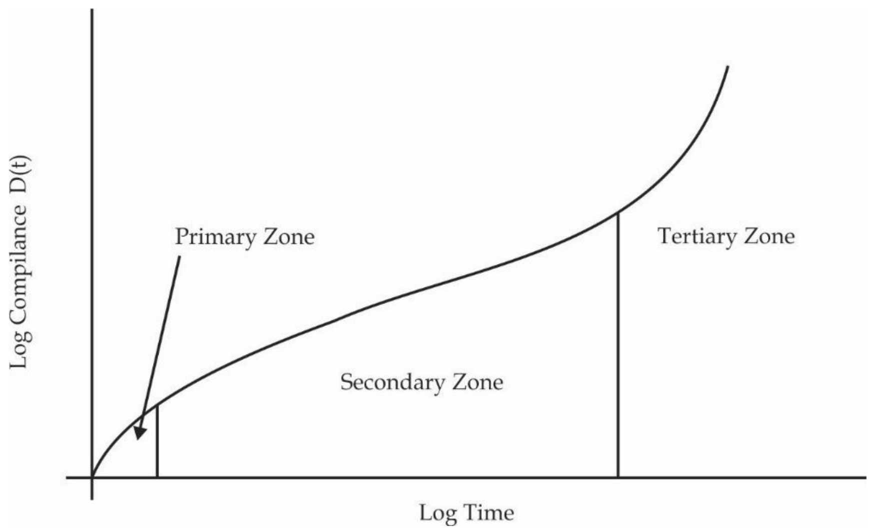


Figure 2.6: Compliance verses Time curve on Log Scale (Witczak, 2005)

Kaloush (2001), conducted a study for evaluation of permanent deformation and concluded that various factors have significant influence on rutting which includes but not limited to effective asphalt content and air voids, binder viscosity, and testing temperature. Another study illustrated factors like binder grade, binder viscosity, asphalt content, testing temperature, nominal maximum aggregate size, voids in the mineral aggregate, percentage passing sieves (No.4, No.16, No.200), and number of gyrations affecting rutting (Kvasnak et al, 2007). Rodeznò et al. (2010), developed regression model for predicting rutting behavior in laboratory having 12 parameters like maximum shear stress, normal stress, testing temperature, binder viscosity, aggregate gradation (percent retained on 3/4-in., 3/8-in., and No. 4 sieves), air voids, binder content, effective binder content, voids in the mineral aggregate, and voids filled with asphalt. Gandomi et al. (2011), developed non-linear model for prediction of FN of asphalt mixtures. The proposed model correlates the FN of Marshall specimens to the coarse and fine aggregate contents, percentage of air voids, percentage of voids in mineral aggregate, Marshall stability flow.

A study conducted by National Cooperative Highway Research Program (NCHRP) concluded that FN test results can also be surrogate for FT test (NCHRP, 2008). Apeagyei

(2014), developed FN predictive models using mix volumetric parameters and binder properties. First order multiple regression was carried out incorporating mixtures volumetric parameter (air voids, Voids in mineral aggregate, Voids filled with asphalt.) and Superpave rutting parameter. Apeagyei (2011), developed FN model as function of dynamic modulus and gradation. This study concluded that FN test strongly correlates the dynamic modulus at 38°C temperature for a range of material. Also, veracity of developed model was checked by predicting FN for 12 mixtures. This study concluded that the dynamic modulus and gradation could be considered as potential rutting specification parameters for QC/QA purposes in the field. Archilla et al. (2007), proposed new method to determine the flow number in bituminous mixtures from repeated axial load tests. They modelled permanent strains versus load cycles curve as a continuous function. The proposed method yields sufficient evidence of reduction in FN and the obtained data was further employed to develop the power model. The laboratory FN test results indicated a substantial degree of variability and reported coefficient of variation (COV) between ranges of 22 to 58% (Mohammed et al. 2006). In similar study conducted by Bhasin et al. [13] on 12 asphalt mixtures reported COV for FN between 4 to 81%. Zhang et al. (2004), carried out comparison of FN, dynamic modulus, and repeated load tests for evaluation of AC permanent deformation. The main objective of this research study was to characterize asphalt mixtures using aforesaid test method and develop correlation for substitution of one test method by another. The FN index concept was introduced in contrast to the classical FN (cycles) parameter, revealed a better correlation with results of both the dynamic modulus and repeated load tests.

Zulkati et al., (2012), reported that a repetitive uniaxial compressive load on cylindrical specimens of asphalt-concrete mixtures provides a reasonable simulation of asphalt pavement subjected to repetitive axle loads. Zhang et al. (2015), reported that the response of asphalt mixtures to various test conditions by Partial Triaxial Compression Test (PTCT) is in accordance with that of typical triaxial compression tests. Repeated load axial tests or dynamic creep tests were used by Shafiei and Namin (2014), Al-Khateeb et al., (2013) and Chen et al., (2011), to simulate the effect of various fillers and modifiers on rutting susceptibility of asphalt mixtures. Rushing and Little (2014), presented results from a laboratory study to identify a performance test for accepting hot asphalt mixtures for constructing airport pavements. Statistical analyses performed on the results indicated that the rate of increase in permanent strain and the flow time value determined from triaxial static creep testing provides the strongest correlation to APA rutting.

A rutting transfer function or prediction model is an equation that is used to predict pavement life in terms of number of repetitions (or loading cycles) to failure. Apeagyei (2011), developed multiple regression models to describe the relationship among flow number (FN), dynamic modulus, and gradation. The prediction model between laboratory rut depth and the dynamic stability of each asphaltic layer, and the longitudinal grade was developed by Wang et al., (2009) using multiple, nonlinear regression analysis. Tapkin et al., (2009), presented an application of Artificial Neural Networking (ANN) technique to predict permanent strain of polypropylene modified asphalt mixtures in a repeated load axial test. Mirzahosseini et al., (2013), developed an ANN based rutting propensity model for asphalt mixtures; percentages of coarse aggregate, filler, bitumen, air voids, voids in mineral aggregate, and Marshall Quotient were employed as the predictor variables using flow number test. Archilla and Madanat (2000), developed a non-linear rutting regression model based on observations of AASHO Road test between numbers of ESALs along with thawing index as an environmental parameter.

2.6.2 Dynamic Modulus

Dynamic modulus |E*| is norm value of complex modulus and considered as stiffness property of material (Di Benedetto et al. 2001). The field performance of asphalt concrete is correlated with dynamic modulus test which complements the mix design criteria, and considered as a key material characterization input parameter in the Mechanistic-Empirical (M-E) design of pavement structures. Therefore, various researches are underway to establish default dynamic modulus values for different regions and implement mechanistic-empirical pavement design. The ensuing paragraphs presents brief overview of the researches carried out on development of stress dependent master curves to serve as the default dynamic modulus values for different asphalt concrete mixtures across the globe.

Lee et al. (2002), investigated the effect of granite on asphalt mixture and developed master curves based on the laboratory evaluated dynamic modulus and compared with the dynamic modulus predictive equations presented in NCHRP 1-37A (NCHRP, 2004). Various researchers have considered the development of master curves as a precursor for implementation of M-EPD (Hassan et al. 2011; Cao et al. 2013). The dynamic modulus test is costly and difficult to perform in laboratory. Hence, various research attempts made to develop dynamic modulus predictive models namely Witczak (NCHRP, 2004), Hirsch (Chirstensen et al. 2003) and new Witczak model (Witczak & Fonseca, 2007; Gedafa et al. 2009). Zhang et al. (2012), investigated permenant deformation and fracture resistance using dynamic modulus test and concluded that dynamic modulus and phase angle of the asphalt mixtures remained constant with the load cycles. Zhao et al. (2012), constructed the triaxial master curves for three asphalt mixtures and proposed a model of vertical shift factor which was a function of reduced frequency and confining pressure.

A research carried out in India for implementation of mechanistic-empirical design approach using AASHTOWARE software, dynamic modulus master curves were developed at varying temperatures and frequencies and used for pavement design. This study concluded that most of highway cross-sections failed to comply with accurate design analysis and proof checking. Further for design life of eight years, the thickness calculated by the software was much higher than proposed by the Indian roads congress (Ghosh et al. 2013). Shahadan et al. (2013), reported that highest performance in terms of rutting and fatigue factors can be achieved when frequency of cumulative traffic loading was between 15 to 20 Hz.

Dynamic modulus is a basic design parameter in 2002 AASHTO mechanistic empirical pavement design guide (MEPDG) which is an outcome of NCHRP project 1-37A (NCHRP, 2004). A number of past research studies evaluated the dynamic modulus test results and discussed various factors affecting the dynamic modulus (stiffness parameter) and phase angle (visco-elastic response). Flintsch et al. (2007), conducted dynamic modulus test and identified various factors including: aggregate; asphalt content; and reclaimed asphalt pavement RAP percentage which had significant influence on dynamic response. Clyne et al. (2003), reported that asphalt binder is one of factors affecting dynamic modulus and concluded that softer the asphalt binder, lower would be the dynamic modulus and vice versa.

Shu and Huang (2008), developed micromechanical model for |E*|, and based on proposed model, closed-form equations were derived to predict |E*| of asphalt mixtures. The results indicated that predicted dynamic moduli were reasonably close to measured ones at high frequencies. This study also concluded that use of stiff asphalt binder, low asphalt content and air voids significantly increase |E*| values. In a subsequent study, Shu and Huang (2009), predicted the dynamic modulus of asphalt mixtures using differential method incorporating viscoelastic effect, aggregate gradation, and air voids as input variable and concluded that this method closely predicts the laboratory obtained results. Cho et al. (2010), evaluated the dynamic modulus of mixtures used in Korea and developed the |E*| predictive equation for Korean M-EPDG and concluded that developed predictive equation was well correlated with measured values. Contreras et al. (2010), reported that dynamic modulus was affected by the loading frequency. Gedafa et al. (2010), compared the dynamic modulus test results of field extracted core with laboratory prepared specimens and concluded that laboratory dynamic modulus could be compared with field measured dynamic modulus at 4°C.

Bayat and Knight (2010), related the laboratory dynamic modulus with field response and results indicated that laboratory obtained dynamic modulus was inversely proportional to the field measured pavement response of the asphalt longitudinal strains. Apeagyei (2011), reported that dynamic modulus and gradation is one of the parameters considered for rutting potential. Yu and Shen (2011), conducted the dynamic modulus test, developed 3D discrete element model, and reported that aggregate packing had significant influence on dynamic modulus. Temperature is one of the factors affecting the dynamic modulus reported in several researches (Apeagyei et al. 2012). Abu Abdo (2012), carried sensitivity analysis using tornado charts and extreme tail analysis and concluded that dynamic shear modulus of binder and air voids has significant effect on predicted |E*|. A research study evaluated |E*| of 20 dense graded asphalt mixtures of Northeast US region and compared with |E*| values predicted by Witczak, Hirsch models and ENTPE transformation. This study concluded that binder grade, air voids, and presence of Reclaimed Asphalt Pavement (RAP) significantly influence the |E*| values (Li et al. 2012).

El-Badawy et al. (2012), in a research study also calibrated the predictive equations for Idaho state mixtures. Hossain et al. (2013), conducted |E*| test on loose asphalt mixtures procured from field and developed rut prediction model using |E*| test data, along with the actual vehicular traffic and environmental data and concluded a reasonable agreement in field measured and predicted rut values. A study was carried out in Saudi Arabia for implementation of AASHTOW are-Pavement design and evaluated two models namely NCHRP 1-37A and 1-40D, and concluded that NCHRP 1-37A |E*| model showed accurate and unbiased results (Khateeb et al. 2014). Biligiri and Way (2014), predicted the |E*| of asphalt mixture using Hirsch model, NCHRP 1-37A model and NCHRP 1-40D models at two aging levels: original and RTFO aged conditions. The results revealed that predicted |E*| for RTFO mixtures were 40% higher than original. This study concluded that NCHRP 1-40D gave best prediction followed by Hirsch and NCHRP 1-37A model.

2.6.3 Resilient Modulus

Resilient modulus is also called the elastic modulus and is defined as the ratio of deviator stress and recoverable strain under the repeated loads:

$$\mathrm{MR} = \frac{\sigma_d}{\varepsilon_r} \quad (2\text{-}1)$$

Where;

MR	=	Resilient Modulus
σ_d	=	Deviator Stress
ε_r	=	Recoverable Strain

The resilient modulus is the elastic modulus which is used in the layered elastic theory. It is obvious that the most paving materials are not elastic and produce permanent deformation subsequently with each load application. However, if the repeated load is small associated with the strength of the material, the deformation under each load repetition is almost recoverable which can be considered elastic. There is a substantial permanent deformation at the early stage of load applications, characterized as plastic strain. As the number of repetitions increases, the plastic strain due to each load repetition reduces. The strain will be completely recoverable after 100 load repetitions (Huang, 2007).

There are numerous factors which affect the resilient modulus of asphalt concrete mixtures, when the resilient modulus test is performed on the specimens using indirect diametral tension test arrangement. Major factors are temperature, load waveform and pulse duration applied to the specimens, thickness and diameter of specimen, and nominal maximum size of aggregate.

Bassett et al. (1990), performed a laboratory analysis to evaluate the effect of changing the maximum aggregate size of a particular gradation on rut development and on other material properties of asphalt aggregate mixes. A 4-inch and 6-inch diameter frame was used to prepare/compact the specimens for mix design as well as the assessment of mixture properties. Test results showed that mixtures having larger aggregate were comparatively stronger than mixtures having smaller aggregate having similar air voids of 4%. Almudaiheem et al. (1991), concluded that the percentage of indirect tensile strength of specific asphalt concrete mixture used as a cyclic load, affects the resilient modulus value. Tests were performed on the specimens having cyclic load ranging from 10 to 30% of indirect tensile strength of similar specimen having identical mixture properties. They concluded that the degree of load in resilient modulus test should be large as it provides a lesser resilient modulus value, to obtain more conventional design. A total of 4% difference in resilient modulus values was found for the samples having 4% asphalt content by weight of the mix at loading regime of 1000 and 2700 N.

Lim et al. (1995), prepared asphalt concrete specimens in the laboratory having three different diameters including 4-inch, 5-inch and 6-inch to examine the sample size influence on the resilient modulus and indirect tensile strength. It was recommended that 5-inch and 6-inch diameter samples would contribute more stiffness and tensile strength for mixtures having large stones. It was concluded that better characterization of the asphalt mixtures could be achieved from resilient modulus and the indirect tension test using large diameter samples. Pan et al. (2005), measured the resilient modulus for asphalt mixtures to investigate their elastic properties using indirect diametral tension test setup according to ASTM D 4123. They examined the effects of coarse aggregate morphology which was the main factor, and other material properties on the resilient modulus asphalt mixtures. They observed that by using coarse aggregates having uneven morphologies enhanced the resilient modulus values obtained at a temperature of 25°C of different asphalt combinations. The variations in aggregate gradation did not considerably affect the association between the coarse aggregate morphology and the resilient modulus. But reducing the nominal maximum size of aggregate from 19 mm to 9.5 mm showed an increased progressive effect of aggregate morphology on the resilient modulus of asphalt mixtures.

Saleh et al. (2006), investigated different factors influencing the resilient modulus of asphalt concrete mixtures. These factors were: the compaction methodology, diameter and thickness of specimen, duration and form of load pulse and the nominal maximum size of aggregate. Two kinds of asphalt concrete mixtures with unlike maximum aggregate sizes (10 mm and 14 mm) were considered. Marshall and Gyratory compaction methodologies were practiced for specimen preparation. Sinusoidal and triangular load pulse arrangements were used to quantify the resilient modulus. This study also involved the examination of different interrelated factors which influenced the resilient modulus. Full factorial design of experiments disclosed that the nominal maximum size of aggregate was the utmost significant factor upsetting the resilient modulus, followed by the load pulse duration, and the specimen’s geometry including thickness and diameter.

Loulizi et al. (2006), performed stiffness tests at five different temperatures on two representative mixtures. It was examined that at each testing temperature the diameter of the specimen affected the value of resilient modulus. The values acquired using 100-mm-diameter specimens were greater than those found using 150-mm-diameter specimens. A robust relation among the dynamic modulus values obtained at 5 Hz frequency and the resilient modulus values was observed. Jahromi et al. (2009), examined various factors affecting the resilient modulus of asphalt concrete mixtures. Two level factorial analysis of experimentation was carried out incorporating five different factors. These factors include maximum nominal size of aggregate, diameter and thickness of specimen, and type and period of the load pulse. Using factorial analysis technique, it was concluded that the maximum nominal aggregate size was the most significant factor influencing the resilient modulus followed by the load pulse duration and the specimen shape (diameter and thickness).

Khan et al. (2012), studied the influence of four factors comprising percentage bitumen content, specimen’s diameter, test temperature and load pulse length on resilient modulus of asphalt concrete mixtures. The specimens having 4-inch and 6-inch diameter were made by using Marshall compaction technique with 4% and 5% asphalt content. Tests were carried out at temperatures of 25°C and 40°C in UTM-25 machine with indirect diametral tension test arrangement. Load pulse of haversine-shaped having the load time duration of 100ms and 300ms was applied to simulate the actual fast and slow truck traffic speed. It was observed that all four factors have an inverse effect on resilient modulus of asphalt concrete mixtures and mainly temperature was the most influencing factor affecting the resilient modulus followed by load pulse period and diameter of specimen.

Tjan et al. (2013), compared values found in laboratory testing with the estimated values of the resilient modulus applying Asphalt Institute method, for unique asphalt mixture used in Indonesia. Indirect diametral tension test setup was used to obtain the resilient modulus values. It was observed that for resilient modulus values less than 2000 MPa, values obtained in the laboratory were in-between 0.7 to 1.1 times of the predicted values. For resilient modulus values greater than 2000 MPa, values obtained in the laboratory were in-between 1.19 to 1.6 times of the predicted values. It was determined that the deviation of estimated modulus values from the real obtained values is within an acceptable range and they can be used practically.

Temperature is the most important aspect for the performance of the pavement structure as temperature of the asphaltic layer influences the resilient modulus of asphalt concrete, fatigue properties of bitumen and the plastic strains. For temperature beyond 20°C, the resilient modulus of asphalt concrete decreases quickly and reaches to questionable low values at 40°C. Therefore, this temperature range is critical for asphalt concrete layer (Per Ullidtz 1987). Stroup et al. (1997), conducted wide investigation on the effect of temperature and load duration on asphalt concrete resilient modulus. The load ranges of 0.1 and 1.0 sec at various temperatures including -18, 1, 25, and 40°C were evaluated. It was found that with intensification of the load duration, the resilient modulus reduced at all temperatures excluding -18°C. At -18°C, marginal increase in the resilient modulus was observed. Ziari et al. (2005), concluded in their investigation that the resilient modulus drastically reduces with increasing temperature. Kamal et al. (2005), investigated the resilient performance of asphalt concrete mixture by varying temperature and reported reduction of almost 85% in resilient modulus values from 25°C to 40°C.

Almudaiheem et al. (1991), concluded that the percentage of indirect tensile strength of specific asphalt concrete mixture used as a cyclic load affects the resilient modulus value. Tests were performed on the specimens having cyclic loading ranging from 10 to 30% of indirect tensile strength of similar specimen having identical mixture properties. Almost 4% difference in resilient modulus values was noted for samples having 4% asphalt content at loading conditions of 1000 and 2700 N. Loulizi et al. (2002), concluded that the loading time affects the properties of asphalt concrete being a viscoelastic material, therefore, it was suggested that the load cycle time for asphalt concrete dynamic tests should be 0.03s to simulate loading times of moving trucks at an average speed. Saleh et al. (2006), found that the load pulse length had substantial effect on resilient modulus values as the resilient modulus reduced with the increase in the load pulse period. They reported the development of high strains due to longer loading time whereas the load pulse form and strain level had insignificant effect on the resilient modulus of asphalt concrete mixtures.

A 4-inch or 6-inch diameter specimen having thickness ranging from 1.5 to 2.5 inch can be used for determination of resilient modulus of asphalt concrete mixtures using the indirect diametral tension test arrangement. The test specimens can be fabricated in the laboratory or obtained from field coring. The resilient modulus and indirect tension testing (diametral testing) on various diameter specimens were performed by Lim et al. (1995). Specimen with 4, 5, and 6-inch diameter were made, having identical diameter/height ratio of 1.6. It was found that with the similar aggregate gradation and bitumen content, the resilient modulus reduced with the increase of specimen diameter. Their study concluded that the specimen diameter, i.e., geometry of specimen influences the resilient modulus.

Lim et al. (1995), studied the influential effect of specimen diameter to maximum nominal stone size fraction on the resilient modulus. It was observed that the resilient modulus reduces as the ratio of specimen diameter to maximum nominal aggregate size improved. Therefore, it was concluded that greater resilient modulus values would be obtained using a small diameter specimen with large top stone size. Pan et al. (2005), performed laboratory testing to study the influential effect of material's properties on the resilient modulus of asphalt concrete and found that the coarse aggregate morphology is the chief factor that affects the resilient modulus. They observed that by using coarse aggregates having uneven morphologies enhanced the resilient modulus values obtained at a temperature of 25°C of different asphalt combinations. It was also noted that the different aggregate gradation did not considerably affect the correlation between the coarse aggregate morphology and the resilient modulus of asphalt concrete mixture. Saleh et al. (2006), conducted a research to relate different factors that influences the resilient modulus. Resilient modulus testing was carried out using 4-inch and 6-inch laboratory compacted specimens. It was found that the most significant factor that influences the resilient modulus was the nominal maximum aggregate size. Higher resilient modulus values of asphalt concrete mixtures were observed with coarser gradations due to the fact that in coarser aggregate arrangement the large particles have better interlocking. Jahromi et al. (2009), found that the maximum nominal size was the most substantial factor influencing the resilient modulus.

2.6.4 Fatigue Life

Fatigue cracking in flexible pavements is a series of interconnected cracks caused by the fatigue failure of asphalt concrete subjected to repeated loading. Fatigue is generally defined as "the phenomenon of fracture under repeated or fluctuating stress having a maximum value generally less than the tensile strength of the material" (Pell, P.S., 1971). The definition has been applied to the pavements with the assumption that the only mechanism producing stresses and strains in the pavements are the wheel load. The definition of fatigue was further improved and defined as "the phenomenon in flexible pavements causing cracking, consisting of two phases of crack initiation and crack propagation, and caused by tensile strains generated in the pavement not only by traffic loading but also temperature variation and construction practices" (Wu, F., 1992).

2.6.4.1 Indirect Tensile Fatigue Test

The asphalt concrete pavement structures require a more economic and suitable design as compared to other engineering structures. The major distresses related to the asphalt concrete pavements that reduce the service life and increase the cost of maintenance are rutting, high temperature deformation, and fatigue cracking, which are either load related or due to low temperature (Hamed, 2010). The distresses in pavement are classified into three major groups namely; cracking, deformation and surface defects according to Miller and Bellinger, (2003).

Fatigue in asphalt concrete is generally defined as the phenomenon causing cracking, consisting of crack initiation and propagation phase, caused due to the tensile strains generated in pavements due to traffic loading, temperature variation and the construction practices combined (Wu, 1992). The damage in the structure of asphalt concrete pavements, cracking and deformation, were due to environment and loading which were further narrowed down to the frequency and magnitude of loading, loading time and variation in temperature. The fatigue characteristics of any asphalt concrete mix can be expressed as a relation between number of cycles to failure and the initial strain (Monismith, et. al. 1985). Lytton, et. al. (1994), explained the two phases of fatigue cracking including the crack initiation followed by crack propagation. The primary material properties affecting the fatigue life of a particular mix are the bitumen content, stiffness and the air voids in the mix, while the aggregate gradations, shape angularity and type have an indirect effect on the mixture by changing the bitumen content.

Significant amount of work has been carried out by Cooper and Pell (1974), to check the significance of material variables on fatigue life of asphalt concrete mixtures. The decrease in the stiffness of the mix increases the fatigue life, also the mixture variable and mode of testing have a significant effect on the fatigue behaviour of asphaltic mixtures (Adedimila and Kennedy, 1976). Al-Khateeb and Ghuzlan (2014), verified that the relation between number of cycles to failure and initial strain are best represented by exponential functional form. Further, Khattak and Baladi (2001), reported the use of controlled strain and controlled stress mode of loading. Ghuzlan (2001), documented that the fatigue behaviour of asphalt concrete mixtures can be predicted using the energy approach. Chen (2002), further explained that the dissipated and stored energies in the viscoelastic materials can be a reason of fatigue damage. The fatigue in asphaltic pavement develops gradually through three phases of crack initiation, stable crack growth and unstable crack propagation. Marasteanu et. al. (2004), experimented and concluded that many different test methods based of fracture mechanics including Disk-Shaped Compact Tension Test, Semi-circular Bending Test and Bending Beam Test can be used to determine the fatigue characteristics of asphalt concrete mixes.

The indirect tensile fatigue test (ITFT) due to its simplicity compared to other alternative tests is very appealing. In the early seventies, work was carried out to evaluate the performance of ITFT and it was concluded that the test presented much shorter fatigue life to failure than the bending beam test (Kennedy, 1977). Adedare et. al. (1975), carried out research to study the fatigue characteristics of asphalt mixtures by repeated indirect tensile test at a range of temperatures (10, 24 and 38°C), and developed equations to predict fatigue life of different asphalt mixtures. Rodrigues (2000), developed a model for predicting fatigue cracking in asphaltic pavements on the bases of mixture bonding energy to analyse the propagation of crack. The effect of mixture compaction on the indirect tensile fatigue was studied and it was reported that rolling compactor best simulates field compaction (Hartman, et. al. 2001).

Al-Qadi and Nassar (2003), studied four different types of approaches including traffic wander and stress state to calculate shift factors to predict fatigue performance of in-service asphalt concrete mixtures. The fatigue life of asphalt concrete was estimated in a research using ultrasonic method for testing and concluded that by adding the parameter of ultrasonic analysis in conventional fatigue model, fatigue life can be predicted without destroying specimens (Marasteanu, et. al., 2004). Further, Awanti et.

al., (2007), studied the influence of the rest period on fatigue characteristics of bituminous mixtures modified using Styrene Butadine Styrene (SBS) polymer compared to straight run 80/100 penetration grade bitumen. Chatti and Mohtar (2004), studied the effect on the fatigue life of asphaltic mixtures under different axle configurations. Salami and Chatti (2011), evaluated the rutting and fatigue prediction model for asphalt concrete pavements under multiple axles loading and concluded that the process of calibration of fatigue and rutting model using MEPDG does not improve the damage calculation due to multiple axles and that is underestimated.

2.6.5 Durability/Moisture Susceptibility

2.6.5.1 Introduction

In general, durability of asphaltic mixtures can be defined as the resistance against the embrittlement of bitumen due to ageing and damage induced due to water that accelerates the failure processes, e.g., fatigue cracking, rutting, stripping etc. However, various agencies and researchers define the term durability in various perspectives, e.g., Whiteoak (1990), stated that the durability is actually the ability to maintain satisfactory adhesion, cohesion and rheology in long term service. Al-Jassar (2003), emphasized that the durability of asphaltic concrete must ensure that a pavement layer maintains its desired properties like resistance against permanent deformation, cracking and bleeding. The Asphalt Institute Manual Series – II (1988) highlights only the interaction of water when discussing the durability of bituminous mixtures. Two major factors affecting the durability are ageing and moisture. Ageing has two types, i.e., short term and long-term ageing. Rolling Thin Film Oven Test (RTFOT) simulates short term ageing of bitumen as per AASHTO T 240 while Pressure Ageing Vessel (PAV) simulates long term ageing of bitumen for 7-10 years period of time as per AASHTO R 28.

2.6.5.2 Moisture Damage

The presence of water (or moisture) often results in premature failure of pavements in the form of isolated distress caused by debonding of the bitumen film from the aggregate surface or early rutting/fatigue cracking due to reduced mixture strength (Lu and Harvey, 2007). Moisture sensitivity has long been recognized as an important mixture design consideration. Francis Hveem (1940), realized the importance of water resistance and identified it as a critical engineering property that needs to be determined in the selection of quality material for pavement construction (Santucci, 2002).

Moisture sensitivity is primarily concerned with the potential for loss of adhesion between the binder and aggregate in the presence of moisture, commonly called stripping. Critical situations leading to stripping largely relate to the extent of moisture saturation in the asphalt mixture and level of traffic stress, which can be avoided by considering the pavement design, mixture design and construction factors that can lead to this moisture saturation. Sensitivity to moisture damage in a particular asphalt mixture is further influenced by the characteristics of component materials, particularly the type and proportion of aggregates, filler and binder. The detrimental effects of moisture damage are required to be minimized to achieve the goal of perpetual pavements.

The susceptibility of road material (aggregate & bitumen) to moisture damage is dependent on the interfacial characteristics of the material and is determined by using the surface energy characteristics of the material (Airey et al., 2007).

2.6.5.3 Sources of Moisture

An asphalt pavement is exposed to several cycles of precipitation during its service life. Sources of moisture in an asphalt pavement can be either internal or external.

Moisture is left inside the pavement before construction in the form of inadequately dried aggregate (Santucci, 2002). Also, if water comes in contact with hot bitumen it is converted into steam and its volume increases which can result in foaming and boil-over of hot bitumen (Read and Whiteoak, 2003).

The following are the three major external sources of moisture:

- Moisture entering the pavement from surface because of poor drainage, poor construction (compaction) or mixture design having high air voids and thus more permeability.
- Moisture entering from sides because of poorly constructed shoulders and poor side drainage.
- Moisture from beneath the subgrade because of high water table and poor drainage characteristics of base and subbase material.

Figure 2.7 below illustrates different sources of moisture in a pavement structure.

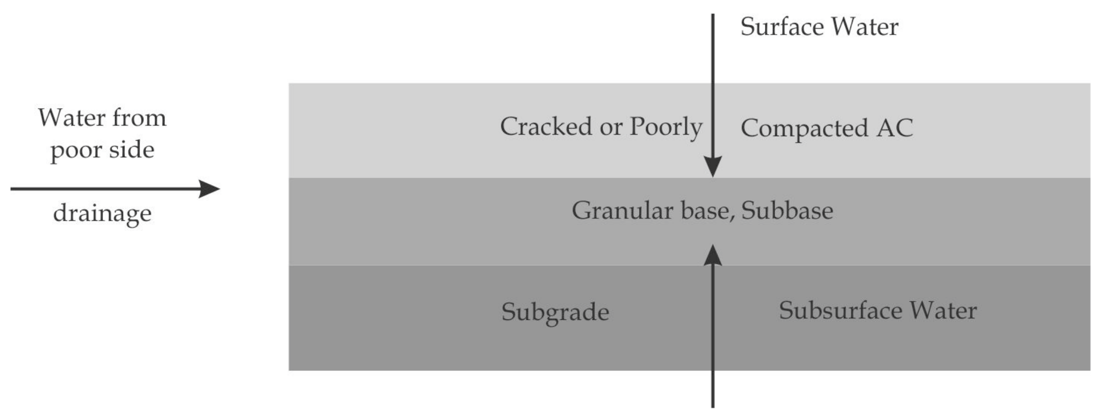


Figure 2.7: Sources of Water in an Asphalt Pavement (Santucci, 2002)

2.6.5.4 Types of Moisture Damage

Presence of moisture either causes certain types of pavement distresses and/or increases the severity of already existing distresses.

Pavement distresses that are directly caused by moisture damage include:

- Stripping

Adhesive failure between the bitumen film and aggregate surface results in debonding which, in an advanced state, is identified as “stripping”. Stripping converts a high strength asphalt treated pavement layer to a much weaker untreated aggregate section. When it occurs in isolated spots throughout the pavement, it can rapidly develop into potholes.

- Corrugations
- Raveling and weathering
- Water pumping

Moisture increases the severity of the following already existing damages:

- Potholes
- Premature Fatigue Cracking/Rutting

Over more extensive areas, premature fatigue cracking or rutting may develop due to the reduced support strength of the overall pavement structure.

Some of the common moisture induced damages are shown in Figure 2.8.

Fatigue Cracking

Stripping

Pothole

Rutting

Figure 2.8: Pavement Distresses (Ahmad, 2011)

Debonding is not always necessary in a moisture sensitive situation. Water in the pavement may simply weaken the asphalt mixture by softening or partially emulsifying the bitumen film without removing it from the aggregate surface. During this weakened state, the asphalt pavement layer is subjected to accelerated damage from applied traffic.

2.6.5.5 Effect of Moisture Damage

Probably the most damaging and often hidden effect of moisture damage is associated with reduced pavement strength. The higher vertical compressive stress in the moisture-damaged pavement can result in overstressing the underlying pavement layers and ultimately can create excessive permanent deformation or rutting in the wheel paths on the pavement surface. Higher tensile (bending) strains at the bottom of the treated pavement layer can translate into earlier than expected fatigue failure as shown in Figure 2.9. The higher bending strain, ε_2, associated with the moisture damaged pavement produces a much lower predicted fatigue life, N2, than the bending strain, ε_1, associated with the dry asphalt pavement structure.

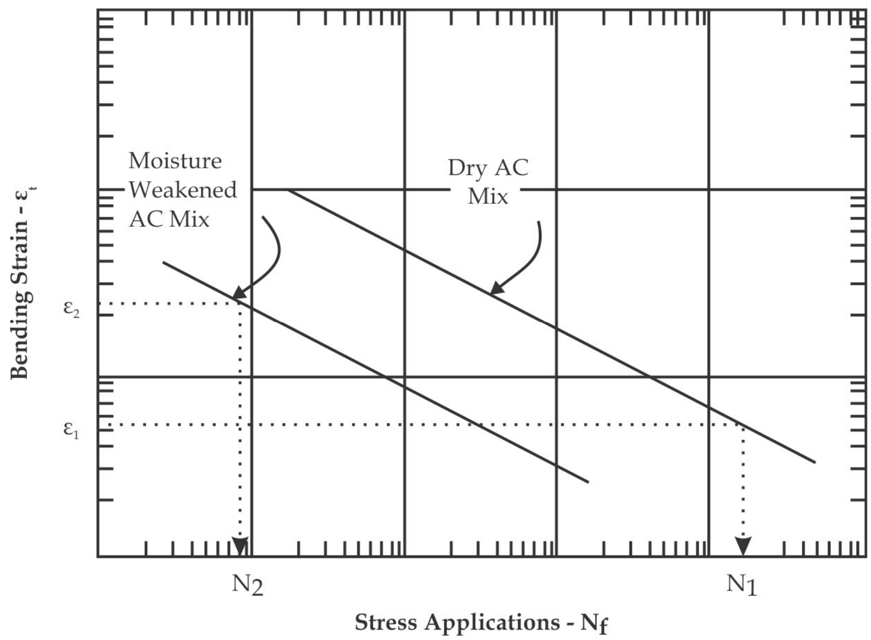


Figure 2.9: Moisture Effect on Fatigue Response (Santucci, 2002)

2.6.5.6 Factors Effecting Moisture Sensitivity

Bitumen and aggregates are the main constituents of an asphalt mixture. Aggregates make up roughly 95% by mass of a dense graded asphalt mixture with the binder being the remaining 5%. Thus, the type/source of bitumen and aggregates used, and their surface characteristics play a significant role in an asphalt mixture's resistance to water action. Asphalt mixture moisture susceptibility can be influenced by factors like physical properties of the materials, design considerations and construction quality.

2.6.5.6.1 Aggregate Physical Properties

Physical properties of the aggregate, such as shape, surface texture and gradation, influence the bitumen content of the mixture and hence the bitumen film thickness. Thick films of bitumen resist the action of water better than thin films (Santucci, 2002). Rough surface textured aggregates help promote better mechanical adhesion at the bitumen-aggregate interface.

2.6.5.6.2 Aggregate Chemistry

Aggregates normally consist of inorganic (mineral origin) polar compounds and their properties vary depending upon the source and type of rock with which they are formed. Surface chemistry of the aggregate is also important. Aggregates range from basic (limestone) to acidic (quartzite) (Asphalt Institute MS-24, 2007). Bitumen, on the other hand, has relatively few basic ingredients, but can have an acidic tendency depending on the source. Therefore, bitumen would be expected to adhere better to alkaline (basic) aggregates (opposite charges attract) such as limestone than to acidic siliceous aggregates. More silica contents are an indication of acidic nature while more carbonate contents represent basic nature.

2.6.5.6.3 Presence of Clay

Clay, either in the form of fine aggregate or as a thin coating over the larger aggregate particles, can create a major problem with moisture sensitivity. Clay expands in the presence of moisture and acts as an effective barrier to the adhesion of bitumen to the aggregate surface.

2.6.5.6.4 Bitumen Properties

Bitumen consists of organic compounds (contains carbon) that are mostly hydrocarbons. They are non-polar materials but contain certain amount of polar organic compounds. These polar organic compounds can have an acidic or basic nature. The examples of acidic compound in bitumen are carboxylic acids. The acidic value of bitumen normally ranges from 1.5-5 mg KOH/g while the basic value ranges from 0-1 mg KOH/g (Asphalt Institute MS-24, 2007). This is an indication that bitumen is mostly acidic.

Bitumen properties also play a role in the moisture sensitivity of asphalt pavements. Complete coating of the aggregate surface during mixing is critical and is affected by the viscosity of the bitumen, which, in turn, is controlled by the mixing temperature used in a hot mix plant. Bitumen film thickness is influenced by bitumen viscosity as well as the use of additives such as polymers or rubber. The source of the bitumen (or how it is produced in a refinery) can have some effect on its moisture sensitivity in an asphalt mixture.

2.6.5.6.5 Bitumen-Aggregate Interactions

As bitumen is mostly acidic in nature, it adheres well with the aggregates having basic characteristics like limestone. The calcium carbonate in the limestone reacts with carboxylic acid in bitumen and forms a strong bond. On the other hand, as siliceous aggregate also has acidic nature, no chemical bond is formed between the bitumen and aggregate. Water, being polar in nature, can easily replace the bitumen from the aggregate surface.

The above explanation, however, is not always true as the material surface properties may be different than its bulk chemistry (Kim, 2009). Also, because of the complex chemistry of the bitumen it is not possible to get similar results with even similar types of binders. In order to fully characterize the moisture susceptibility of a certain material, a lot of surface energy testing in conjunction with mechanical moisture sensitivity assessment is required to be done. It is also important to study the chemistry of the materials in detail.

2.6.5.6.6 Compaction

The primary construction issue that needs to be addressed in making asphalt pavements more moisture resistant is adequate compaction during construction. Better compaction of dense graded asphalt mixture leads to lower air void content and lower permeability of the completed pavement. Both factors reduce the ability of external moisture from entering the pavement.

2.6.5.7 Moisture Damage Mechanisms

Several theories have been proposed to describe the mechanisms of moisture damage. An asphalt pavement is exposed to moisture throughout its service life especially in the areas which have large amounts of rainfall throughout the year. Kringos and Scarpas (2005), have explained the following three main phenomenon of moisture damage.

2.6.5.7.1 Advective flow through an Asphalt Mixture

This is a short-term phenomenon in which the direct contact of moving water with an asphalt mixture causes desorption of the outer layer of bitumen film. Water washes away the bitumen film layer by layer.

2.6.5.7.2 Diffusion Leading to Bitumen-Aggregate Interface Failure

Diffusion is the homogenization of the chemical components of a phase at an atomic or molecular level. Water reaches the bitumen-aggregate interface through this phenomenon and replaces the bitumen film from the aggregate surface. This adhesive failure at bitumen-aggregate interface largely depends on the chemical characteristics of the two materials.

2.6.5.7.3 Diffusion Causing Dispersion of the Bitumen Film

Amount of diffusion of water in the bitumen film increases with the passage of time until the bitumen particles loose cohesion and dispersion of the bitumen film occurs. This phenomenon largely depends on the individual properties of the bitumen and the level of exposure to water.

Following are some other mechanisms that cause the moisture damage to an asphalt pavement.

2.6.5.7.4 Emulsification

Emulsification of the bitumen film can occur in a pavement due to the presence of emulsifying agents in the aggregate such as clay particles. Traffic provides the action needed to promote emulsification. The resulting emulsion may migrate to the pavement surface and produce localized fat spots (Asphalt Institute MS-24, 2007).

2.6.5.7.5 Pore Pressures

Pore pressures can build up in an asphalt pavement due to the action of traffic in the presence of moisture. These pressures alternating between compression and tension can result in a debonding of the bitumen from the aggregate or raveling of the pavement surface. Freezing of water present in the pavement also has the same effect as expansion of entrapped water and can seriously damage the pavement (Asphalt Institute MS-24, 2007).

2.6.5.8 Moisture Sensitivity Assessment

Durability of an asphalt pavement decreases with the passage of time because of ageing and moisture exposure. A moisture damaged pavement cannot efficiently support the traffic induced stresses and strains because of reduction in its strength. Numerous laboratory tests have been developed over the years in an effort to predict the moisture sensitivity of asphalt mixtures.

In order to avoid premature failure of road pavements, related to the bitumen-aggregate adhesion characteristics, it is very important to devise a laboratory test which could predict the moisture sensitivity of an asphalt mixture.

Airey and Choi (2002), carried out an extensive literature review on moisture sensitivity test methods for bituminous pavement materials. Moisture sensitivity tests are carried out on loose coated aggregates or asphalt mixtures. Different conditioning processes are used to simulate the field exposure conditions and the conditioned specimens are then inspected visually or mechanically for moisture sensitivity evaluation.

A summary of different bitumen-aggregates adhesion/moisture sensitivity assessment testing techniques is provided below.

2.6.5.8.1 Tests on Loose Coated Aggregate

These tests involve the following steps:

- Immersion of loose compacted mixture in water or chemical solution
- Immersion is done for specified time at room or elevated temperature
- Separation of bitumen from aggregate is assessed visually

Correlation with field: little information is available to correlate data with field performance.

2.6.5.8.1.1 Types of Tests

- Static Immersion Test (AASHTO T182, ASTM D1664)
- Dynamic Immersion Test
- Chemical Immersion Test (by use of sodium carbonate)
- Rolling Bottle Method
- Boiling Water Test (ASTM D3625)
- Ancona Stripping Test
- Boiling Water Stripping Test
- Ultrasonic Method
- Net Adsorption Test (SHRP, M001)
- Modified Net Adsorption Test

Summary of some of the commonly used techniques is provided as follows;

2.6.5.8.1.2 Static Immersion Tests

In this test aggregates are coated with bitumen and are then immersed in water. After a certain period of time, visual inspection is made to assess the degree of stripping. It is considered as the simplest test. The example is coating and stripping of bitumen-aggregate mixtures, [AASHTO T182 (ASTM D1664)].

2.6.5.8.1.3 Dynamic Immersion Tests

The only difference between a static and dynamic immersion test is that the sample is agitated mechanically by shaking or kneading in the later type.

2.6.5.8.1.4 Chemical Immersion Tests

Aggregates coated with bitumen, are boiled in solutions of sodium carbonate with varying concentrations of the chemical. The strength of solution which first causes stripping is related to the adhesion strength between the two materials.

2.6.5.8.1.5 Boiling Water Tests

This test involves the visual observation of the loss of adhesion in un-compacted bituminous coated aggregate mixtures due to the action of boiling water. The example is "Effect of water on bituminous-coated aggregate using boiling water" (ASTM D3625).

2.6.5.8.2 Tests on Compacted Asphalt Mixtures

These involve the following major steps:

- Samples are prepared in laboratory or cored from the field
- Conditioning of samples to simulate in-situ condition
- Tests are performed to calculate strength or stiffness
- Assessment of moisture damage is made by calculating ratio of conditioned to unconditioned/wet to dry strength

Correlation with field: sometimes provides good correlation between laboratory tests and field results, but not always reliable (Airey and Choi, 2002).

These types of tests are further divided into three major categories;

2.6.5.8.2.1 Immersion Mechanical Tests

Change in mechanical property of a mixture after immersion in water is measured. Measured mechanical properties are usually indirect tensile stiffness and indirect tensile strength.

Tests Types

- Texas Freeze-Thaw Pedestal Test
- Immersion Compression Test (AASHTO T165, ASTM D1075)
- Marshall Stability Test (AASHTO T245)
- Duriez Test (NFP 98-251-1)
- Lottman Test
- Tunnicliff and Root Procedure
- Modified Lottman Procedure (AASHTO T283)
- LINK Bitutest Water Sensitivity Protocol

2.6.5.8.2.2 Immersion Wheel Tracking Tests

These tests incorporate aspects of repeated traffic loading in their procedures.

Tests Types

- Immersion Wheel Tracking Test
- Hamburg Wheel Tracking

A summary of the above-mentioned test techniques is provided below:

2.6.5.8.2.3 Immersion Mechanical Test Methodologies

A compacted asphalt mixture specimen is immersed in water and is tested for a specific mechanical property. The results are compared to the original mechanical property, in normal conditions. Any change in the mechanical property after immersion in water is related to the stripping potential of the material. The ratio of the two results, expressed in percentage, is used as an indirect measure of stripping.

The mechanical properties that can be measured include shear strength, flexural strength and compressive strength. The two most common are known as the retained Marshall Stability test (AASHTO T245) and the retained stiffness test using Nottingham Asphalt Tester (NAT). Other examples include immersion compression test [AASHTO T165 (ASTM D1075)], indirect tension test with moisture saturation only (ASTM D4867); and indirect tension test with moisture saturation and one freeze-thaw cycle (AASHTO T283).

A new adhesion testing fixture has been developed by Fini et al., (2006), of the Illinois Center for Transportation to assess bituminous crack sealant-aggregate adhesion, utilizing the Direct Tension Tester (DTT). The briquette assembly consists of two half-cylinders of aggregates having 25mm diameter and 12mm length. The assembly (shown in Figure 2.10) has a half cylinder mold, open at the upper part. Prior to pouring the sealant, the assembly is heated to facilitate sealant flow and to ensure a uniform bonding area. The aspect ratio of the sealant is maintained as 1. After 1h of curing, the specimen is trimmed and kept in the DTT cooling bath for 30 minutes before testing.

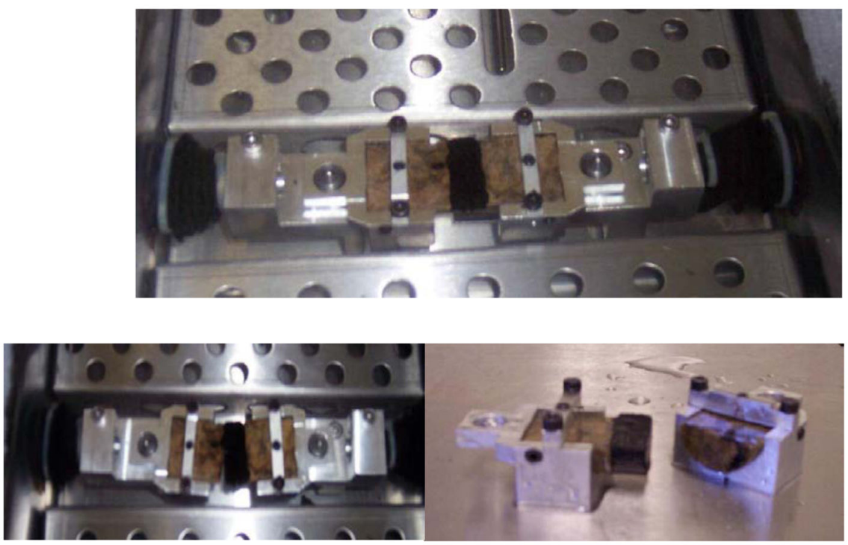


Figure 2.10: Adhesion Test Fixture (Fini et al., 2006)

2.6.5.8.2.4 Immersion Trafficking Tests

The main problem with most of the above-mentioned tests is that they do not consider the effect of trafficking on stripping. Immersion wheel tracking test (AASHTO T324) has been developed to overcome this problem. In this test, a specimen is immersed in a water bath and is tested by a loaded solid rubber tyre. The tyre forms a rut on the surface of specimen. Development of the rut is measured until stripping starts. The most popular type of wheel trafficking equipment is the Hamburg wheel tracking device and is shown in Figure 2.11.

Figure 2.11: Hamburg Wheel Tracker

2.6.5.8.2.5 Environmental Conditioning System (AASHTO TP 34)

An environmental conditioning system consists of three main components;

- Fluid conditioning system
- Loading System
- Environmental conditioning system

Fluid conditioning system is used to condition the specimen and determine their permeability. The loading system consists of a tri-axial cell which serves as a loading frame and the samples are tested in an un-confined tri-axial configuration. The resilient modulus and permeability values obtained in dry conditions are compared to the values after conditioning. Results obtained from this test do not correlate well with the results obtained from field cores (Airey and Choi, 2002).

The shortcomings of most of the above mentioned methods is that only one discrete point/result is obtained from each test. In order to get a more realistic picture of the overall trend of failure, a large number of tests are required to be performed for a range of conditions. Conditioning of samples is done to get a desired saturation level for better field simulation. The freeze-thaw cycle method is used in some of the tests and, because of its severity, is considered to better simulate field conditions but it may cause film rupture damage and thus may not be a true representation of the original condition (Airey and Choi, 2002).

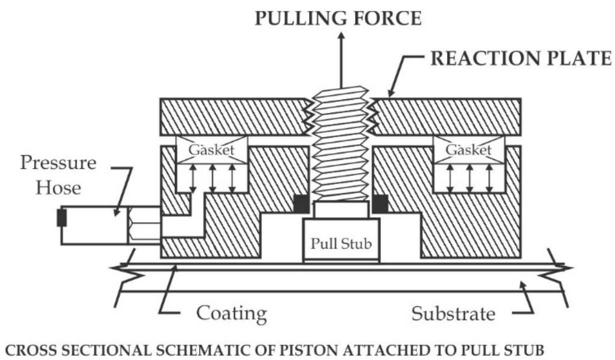
2.6.5.8.3 Latest Advancements

2.6.5.8.3.1 Pull-off Tests

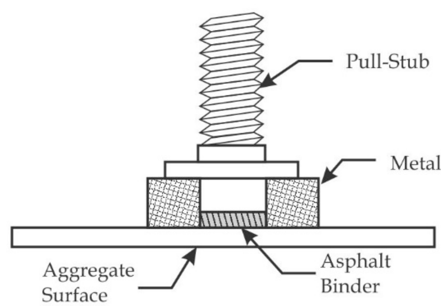
Bitumen adhesion is also assessed by using different types of pull-off tests. The Instron pull-off test is used to remove the aggregate specimens from the bitumen under controlled laboratory conditions by using an Instron apparatus. Similarly, the Limpet pull-off test is used to measure the bond strength between the aggregate of a surface dressing and the under lying adhesive by using a Limpet apparatus (Read and Whiteoak, 2003).

A modified version of the pull-off test method specified in pull-off strength of coatings using portable adhesion testers (ASTM D4541) is utilized to study the moisture susceptibility of binders (Kanitpong and Bahia, 2005). The procedure quantitatively measures the bond strength of binders or mastics applied to a substrate using the Pneumatic Adhesion Tensile Testing Instrument (PATTI). A schematic diagram of this device is shown in Figure 2.12(a). The bitumen is applied to a pull stub, which is then attached to the aggregate surface as shown in Figure 2.13(b). The film thickness of bitumen is controlled by using two pieces of 1/4 x 1/4 x 2½ inch metal blocks put under the pull stub. The space underneath the pull-stub and aggregate surface is the film thickness of bitumen specimen. The PATTI transmits the air pressure to the piston, which is placed over the pull stub and screwed on the reaction plate (Figure 2.12(a)). The air pressure induces an airtight seal formed between the piston gasket and the aggregate surface. When the pull stub is debonded from the aggregate surface, the pressure at which the cohesive or the adhesive failure occurs is measured and converted to the pull-off strength (kPa).

The pull-off strength can be used as an indicator of the adhesive bond strength of the bitumen.



a) PATTI device



b) Specimen preparation for adhesion or pull-off tensile strength test

Figure 2.12: Pneumatic Adhesion Tensile Testing Instrument (Kanitpong and Bahia, 2005)

2.6.5.8.3.2 Blister Test

This test consists of coating of bitumen over a flat aggregate surface. A hole, drilled through the aggregate under the coating, permits the application of steady or varying fluid pressure to the underside of the coating as shown in Figure 2.13. Adhesion is measured by the force required to displace the film from the aggregate (Fini et al., 2007).

It is not always possible to have the best bitumen-aggregate combinations, as it depends a lot on the availability of type of road aggregates on the site. Different treatment techniques are being used by the industry in order to improve the bitumen-aggregate adhesion characteristics. Irradiation is supposed to increase the wettability of the aggregates and could be used as an aggregate treatment technique in order to improve the bitumen-aggregate adhesion.

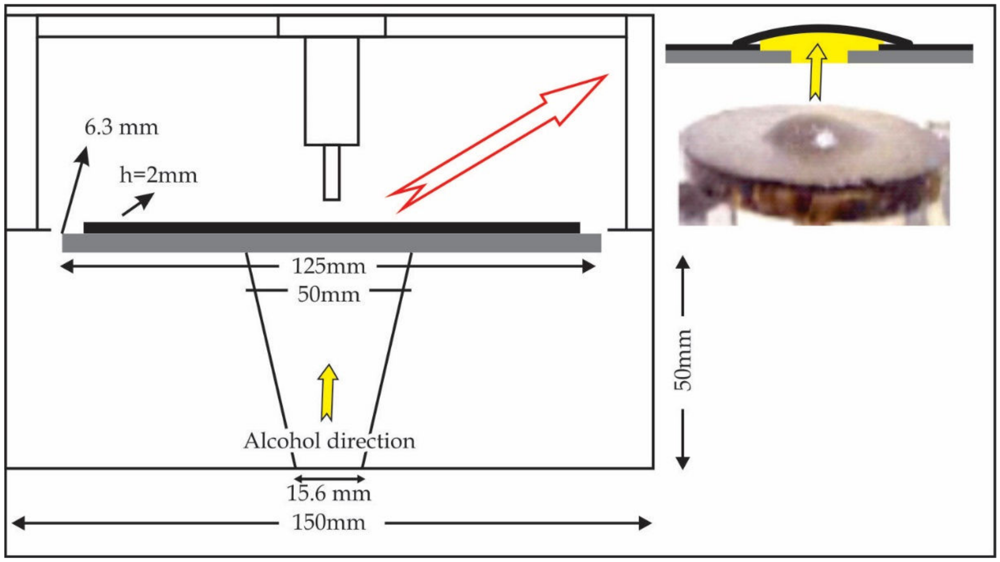


Figure 2.13: A Schematic of Blister Apparatus (Fini et al., 2007)

2.6.5.8.3.3 Beam Fatigue Test

Lu and Harvey (2007), developed a fatigue-based test procedure for moisture sensitivity assessment of different asphalt mixtures. A controlled-strain flexural beam fatigue test is used to study these moisture effects. Tests are performed at 10 Hz loading frequency, and 200$\mu\varepsilon$ strain level with specimens pre-saturated at 635 mm-Hg vacuum for 30 minutes.

The following parameters are used for sample preconditioning:

- Moisture Content: low (20-30 percent saturation) and high (50-70 percent saturation).
- Conditioning Temperature: 25°C and 60°C
- Conditioning Duration: one day and ten days

Lu and Harvey (2007) compared the results from this fatigue test with the results obtained from both the TSR (Tensile Strength Ratio) test and the HWTD (Hamburg Wheel Tracking Device) test. Test results show that the fatigue test procedure can distinguish mixtures on basis of their moisture sensitivities. The performance of the mixtures was found to be consistent with the prior experience for these mixtures.

2.6.5.8.3.4 Saturation Ageing Tensile Stiffness (SATS) Test

A new conditioning protocol and testing procedure has been developed by the Nottingham Transportation Engineering Centre (NTEC) known as the Saturation Ageing Tensile Stiffness (SATS) Test, shown in Figure 2.14, to measure the moisture sensitivity of asphalt mixtures.

Airey et al. (2006), used SATS and AASHTO T283 to test high modulus base (HMB) material samples. The relative performance of the two tests was compared in terms of measured retained stiffness modulus of the specimens. The values obtained from AASHTO T283 were approximately double than that produced from SATS for same saturation level. This gives an indication that SATS was more severe on the samples. The drop in the stiffness modulus for samples tested with SATS was due to the combined effect of moisture and air (ageing). The values obtained from SATS testing were comparable to the stiffness modulus values observed on a trial site where HMB material was placed. This gives an indication that SATS probably better simulates the field conditions as compared to AASHTO T283, at least for the set of mixtures that were analyzed.

Figure 2.14: SATS (Ahmad, 2011)

SATS measures the combined effects of ageing and moisture using the following steps:

- Initial saturation of specimen
- Ageing and saturation in a high temperature and pressure environment for extended period
- Calculation of retained saturation
- Calculation of retained stiffness modulus as:

$$\text{Retained stiffness modulus} = \frac{\text{Stiffness modulus after test}}{\text{Stiffness modulus before test}} \qquad \text{2-2}$$

The SATS and AASHTO T283 procedures provide moisture sensitivity/susceptibility of different mixtures and can be used for correlation with field performance but as they do not assess the fundamental material properties like cohesion and adhesion, they cannot explain the cause of poor and good performance and do not provide any feedback for redesigning poorly performing mixtures (Airey et al., 2007).

The followings are maybe desirable for an asphalt mixture to provide better resistance to moisture damage.

- Basic aggregate
- Low air voids
- High binder content to have greater film thickness
- Active filler

The above-mentioned properties and the mechanical properties like fracture, healing and viscoelastic properties are required for complete characterization of moisture damage (Airey et al., 2007). These mechanical properties are influenced by binder-aggregate adhesive bond energy along with physical properties like air voids, aggregate shape, etc.

2.6.5.9 Durability of Asphalt Paving Mixtures - Summary

According to Gorkem and Sengoz (2009), the environmental factors, i.e., air, water and temperature have a significant effect on the durability of asphaltic mixtures. In moderate climatic conditions, a large portion of the deterioration could be due to traffic loading, as a result, distresses manifests as rutting, fatigue cracking and raveling. However, due to severe climatic conditions, these stresses increase dramatically with traffic and water. Water may enter across the bitumen film through diffusion and removes the bitumen film from the aggregate surface.

Thus, one can say that stripping is the rupture of asphalt texture in asphalt mixtures under the action of traffic loading and presence of water at the same time. Mehrara and Khodaii (2012), stated that the durability is adversely affected by stripping because it is one of the most important types of distresses. A thorough understanding of the stripping process may lead towards the improvement of asphalt mix design methods.

According to Roberts (2009), stripping is a process that could be influenced by moisture and will result in a loss of strength through the weakening of the bond between the aggregate and the asphalt cement. Kennedy (1995), concluded that the structural integrity of bitumen-aggregate mixtures can be degraded through loss of cohesion. This can result in the reduction of stiffness and strength of the mixture and can cause a reduction of pavement ability to support stresses and strains induced due to traffic loading. According to Scholz (1995), the durability of asphaltic mixtures is dependent on various properties. The study explained the effects of water on bitumen hardening and bitumen aggregate bonding.

Ekblad et al., (2013), stated that durability represents an important property of road pavements and is related to their ability to withstand deterioration due to mechanical loading, climate and time. An important aspect of durability is water susceptibility, i.e., the ability of the asphalt to resist deficiencies in the aggregate-binder interface and/or reduction of the cohesion of the mastic (i.e., fines and bitumen) or the bituminous binder itself in the presence of water/moisture.

Qudais and Shweily (2005), determined the effects of environmental damage on asphalt mixtures. The effects of aggregate gradation, type of asphalt cement and antistripping additives were studied to evaluate the creep and environmental behavior of asphalt concrete.

According to Dehnad et al., (2013), moisture has adverse effects on rutting. Moreover, dynamic creep test was also performed on saturated and dry asphalt mixtures. Khodaii et al., (2013), stated that the aggregate gradation has an influence on the moisture damage potential, increasing the bitumen would decrease the stripping potential and proved it by calculating TSR. Liu et al., (2017), concluded that nominal maximum aggregate size (NMAS) of aggregate gradations effect the permeability of asphalt mixtures. Xu et al., (2016) established that open graded asphalt mixtures exhibit higher susceptibility to moisture damage. Lottman (1982), presented the main causes of stripping mechanism as follows:

- Wheel loading repetitions produce pore water pressure in the mixture; temperature differences cause expansion contraction, i.e., formation of ice and thermal shocks; or a combination of these factors.
- At high temperature, asphalt film removal by water.
- Interaction of water vapors with mastic (bitumen and filler) and larger aggregate interfaces.
- Presence and interaction of clay minerals with water.

Air void contents play a significant role in defining the moisture resistance of an asphalt mixture. It is pertinent to mention that most of the recent research studies (Liu et al. 2017, Xu et al. 2016, Khan 2017, and Tarefder & Ahmad 2017) on durability/moisture susceptibility of asphalt concrete emphasize on controlling air voids in the asphalt mixture for improved water resistance and longevity of flexible pavements.

3 Aggregate, Binder & Mix Characterization

3.1 Introduction

This chapter outlines the base line tests carried out to characterize aggregate and bitumen material. Apart from empirical tests, bitumen was subjected to performance based testing criteria, relevant to Strategic Highway Research Program (SHRP) procedure. Furthermore, selection of aggregate gradations and laboratory test performed have been discussed in detail. Laboratory samples of different gradations were prepared for both asphaltic wearing and base course mixtures. For preparation of asphalt concrete mix design, Marshal Mix design method was used for evaluation of the optimum bitumen content. Marshall tests pertaining to flow, stability etc., along with volumetric evaluation of Void in Mineral Aggregates (VMA), Voids Filled with Asphalt (VFA) and Air Void (Va) content were determined according to procedures outlined in Asphalt Institute Manual Series -2 (MS-2). Based on the optimum bitumen content evaluated using Marshall Mix Method, core specimens were fabricated using Superpave gyratory compactor (SGC) while roller compactor was used for slab specimens respectively.

3.2 Aggregate Selection & Characterization

Aggregates can either be natural or manufactured. Natural aggregates are mostly mined or excavated from quarry with large rock formations and crushed to varying sizes as per requirement. Aggregates provide the skeletal structure of the asphalt concrete for load bearing capability of a pavement layer. It is therefore important to select aggregate type which meets the requirements as set out in standard specifications. The basic strength properties of asphalt concrete are greatly influenced by the characteristics of aggregate such as the size, gradation, surface texture and its shape. Aggregates for asphaltic concrete are generally required to be durable, hard, tough, strong, properly graded with low porosity, and to have clean, rough and hydrophobic surfaces. Laboratory characterization of aggregates from two (02) main quarries i.e., Margalla (Punjab North) and Ubhan Shah (Sindh South) has been carried out. Tests performed on aggregates are presented in Table 3.1 below. Standard tests were carried out for both fine and coarse aggregates to determine their suitability.

Table 3.1: Test standards for the Quality and Property Tests of Aggregates

QUALITY TESTS		
1	Fractured Particles	ASTM D 5821
2	Flat and Elongated Particles	ASTM D 4791
3	Resistance to Degradation	ASTM C 131
4	Durability and Soundness	ASTM C 88
5	Deleterious Materials	ASTM C 142
6	Un-compacted Voids	ASTM C 1252
7	Sand Equivalent	ASTM D 2419

QUALITY TESTS		
PROPERTY TESTS		
1	Water Absorption Bulk Specific Gravity SSD Specific Gravity Apparent Specific Gravity	ASTM 127
2	Gradation	ASTM 136
3	Unit Weight, Loose & Rodded	ASTM 29

3.2.1 Quality & Property Test

The proceeding sections define various quality tests along with their results for the selected aggregate sources:

3.2.1.1 Fractured Particles (ASTM D5821)

Fractured particle is a particle of aggregate having the least number of fractured faces as specified, and a fractured face is termed as that surface of an aggregate particle that is rough, angular or has been broken due to crushing, by nature or by any artificial means. This test method determines the percentage, by count or by mass, of a coarse aggregate sample that consist of a fractured particle meeting the above mentioned requirements. The purpose of the requirement is to provide maximum shear strength to the bound or unbound aggregate matrix by increasing the friction between the particles. Tests were conducted according to ASTM D 5821 for coarse aggregates only where the minimum requirement to pass the test i.e., more than 90% was met as shown in Table 3.2 below:

Table 3.2: Results of Fractured Particle Test

Sieve Size (inch)	Margalla	Ubhan Shah
1 ½ to 1 1 to ¾ ¾ to ½ ½ to 3/8 3/8 to No. 4	100% Crushed Aggregate	100% Crushed Aggregate
Minimum Requirement	90% Min.	

3.2.1.2 Flat and Elongated Particles (ASTM D4791)

Aggregate shape has greater influence on the properties of bound or unbound aggregate matric and may also affect the compaction and consolidation of a particular material. This test provides a compliance of the coarse aggregate with the specific characteristics of its relative shape. The test method comprises of the percentage of elongated particles, flat particles or both flat and elongated particles. There are two procedures that can be followed. The first procedure or Method A gives the reflection of the original process developed which is intended for all non-Superpave applications. While the other process or Method B compares the maximum particle dimension to the minimum and is intended to be used for Superpave specification. Flat or elongated particles have a tendency to lock up more readily during the compaction process which makes compaction difficult as compaction requires re-orientation of the aggregate particles, and consequently they also tend to break during compaction making the aggregate gradation finer and possibly to cause a lower VMA value than expected. The test has been performed for both the Margalla and Ubhan Shah coarse aggregates only. Table 3.3 shows the results of the average of three replicate tests. According to the standard ASTM D 4791 the percentage elongated and flat should be less than or equal to 15%.

Table 3.3: Results of Flat and Elongated Particle Test

Sieve Sizes (inch)	Margalla			Ubhan Shah		
	Wt. of 100 Particles (g)	Wt. of Flat Particles (g)	Wt. of Elongated Particles (g)	Wt. of 100 Particles (g)	Wt. of Flat Particles (g)	Wt. of Elongated Particles (g)
1 to ¾	1791.0	259.0	78.0	1950.0	294.5	126.5
¾ to ½	616.0	44.0	14.0	630.0	67.3	87.5
½ to 3/8	223.0	47.0	0	280.0	92.4	81.0
3/8 to No. 4	100.0	21.0	15.0	160.0	43.8	70.4
Total Mass	2726.0	371.0	106.0	3020.0	498.0	365.4
Percentage	100%	13.6%	3.9%	100%	16.5%	12.1%

3.2.1.3 Resistance to Degradation (ASTM C131)

The resistance to degradation of an aggregate is checked using the Los Angles Abrasion test (Figure 3.1). This test evaluates the toughness and also the abrasion characteristic of the aggregate. The test involves placing a certain weight of coarse aggregates (above sieve No. 12) specified number of steel balls and defined standard weights in a rolling mill/drum which is then subjected to specific rotations/revolutions. Once the revolutions are complete the material is passed through sieve # 12 and the percentage loss in the retained material is evaluated. According to the NHA specifications an abrasion value of 30% or less is satisfactory for the coarse aggregates. The test has been performed following ASTM C 131 on both Margalla and Ubhan Shah aggregate and the results are shown in Table 3.4 below:

Table 3.4: Results of Los Angeles Abrasion Test

Sr. No.	**Margalla**				**Ubhan Shah**			
	Total Mass	Retained #12	Passing #12	Resistance to Degradation	Total Mass	Retained #12	Passing #12	Resistance to Degradation
	(gm)	(gm)	(gm)	(%)	(gm)	(gm)	(gm)	(%)
1	5008.0	3653.0	1372.0	27.4	5000.0	3867.6	1132.3	22.6
2	5000.0	3672.0	1328.0	26.6	5000.0	3975.0	1025.0	20.5
3	5005.0	3598.0	1407.0	28.1	5000.0	4000.0	1000.0	20.0
Avg.	5004.0	3635.0	1369.0	27.4	5000.0	3947.5	1052.4	21.0

Figure 3.1: Los Angeles Abrasion Test Equipment

3.2.1.4 Durability and Soundness (ASTM C88)

The aggregates that are resistant to the action of weathering are much more durable and less prone to degradation in the field to cause a premature failure of the pavement. Therefore, the soundness test is carried out to check the resistance of aggregate to disintegration and fragmentation by the action of weathering, due to the freeze thaw cycles. In this test the aggregates are repeatedly submerged in magnesium sulphate or sodium sulphate solution for 24 hours causing the solution to penetrate into the aggregate pores. Once the absorbed solution starts to crystalize this simulates the action of formation of ice crystals in the freeze thaw cycles. On completion of cycles the aggregates are washed with barium chloride to remove the salt solution from the pores of the aggregates. The aggregates are then passed through particular set of sieves mentioned in the standard and the change in mass is recorded as percentage degradation. Table 3.5 shows the test results for soundness of aggregates tested:

Table 3.5: Results of Durability and Soundness Test

Sieve Size	Margalla	Ubhan Shah
Coarse Aggregate 1 ½ to No. 4	9.69 %	0.17 %
Fine Aggregate No. 4 to No. 30	4.7 %	2.44 %
Specification Requirement	12% Maximum	

3.2.1.5 Deleterious Material (ASTM C142)

The presence of excess amount of silt and clay, organic particles or any other substance that absorbs water reduces durability, water tightness and strength of concrete. The primary purpose of this test is to determine the amount of clay lumps present in the aggregates. This test is an approximate method to determine the clay and other friable particles might cause loss of bonding between binder and the aggregate in asphalt mix. The Table 3.6 shows the average result of the tests performed in accordance with ASTM C 142 on the Margalla aggregate using three trials each:

Table 3.6: Results of Deleterious Materials Test

	Margalla	
	Coarse Aggregate	Fine Aggregate
Wt. Before Washing (gm)	5000.0	500.0
Wt. After Washing (gm)	4976.0	485.7
Percentage Clay (%)	0.481	2.867

3.2.1.6 Un-compacted Voids (ASTM C1252)

The test for un-compacted voids provides a comparative estimate of the sphericity, angularity and the surface texture of the aggregates. Once the void content of the fine aggregate is measured it can indicate what effect the fine aggregate will have on the workability of a particular mixture. There are three methods to measure the voids, out of which two of the methods use the graded fine aggregate on the whole while one method uses different size fractions. The test is performed according to ASTM C 1252 for fine aggregates and the average result of the three trials is shown in Table 3.7 for both the Margalla and Ubhan Shah aggregate respectively:

Table 3.7: Results of Un-Compacted Voids in Fine Aggregate

Test Parameter	Margalla	Ubhan Shah
Void Content (%)	39.3	41.0

3.2.1.7 Sand Equivalent

Similar to the test for the deleterious materials in the aggregate, sand equivalent test determines the undesirable soil particles that coat the fine aggregates and hinder the proper binding between the bitumen and aggregate. In this test fine aggregates passing No. 4 sieve are poured into a graduated cylinder along with small amount of flocculating agent. These are mixed or agitated so as to lose the bond between the clay and sand particles. After the sedimentation time mentioned in the standard sand equivalent is determined as the ratio of height of sand to clay. More the sand equivalent value the lesser the clay particles present. The test was conducted for both Margalla and Ubhan Shah aggregates according to ASTM D 2419 and the result is shown in Table 3.8.

Table 3.8: Results for Sand Equivalent Test

	Margalla	Ubhan Shah
Sand Equivalent (%)	76.6	82.0

3.2.1.8 Water Absorption and Specific Gravity

The rate of water absorption and specific gravity of aggregates is regularly used by the engineers and practitioners for the design and construction of pavements. In asphalt mix water absorption and specific gravity of both coarse and fine aggregates is considered to be important. The most important factors in the quality control of bituminous mixtures, voids in mineral aggregates and the amount of bitumen absorbed, are evaluated on the basis of bulk specific gravity. The test includes determination of the average density of the coarse particles ignoring the voids that are present in between the particles, specific gravity also termed as the relative density, as well as the percentage of absorption in the aggregates. The densities are termed as Oven Dried (OD), Saturated Surface Dried (SSD) or apparent density depending on the procedure that is being used for the test. The specific gravity is generally the ratio of the density of a particular material to the density of water at a constant temperature, and the basic measurements to be taken for the calculation are the volume and the mass of the particles.

Bulk Specific Gravity is also referred as Bulk Dry Specific Gravity (Gsb). This includes the volume of aggregates as well as the air voids in between the particles, while on the other hand the mass is only the mass of the solid aggregate particles as the air present in the voids has no mass.

Saturated SurfaceDry Specific Gravity involves the measurement of volume of the aggregate as well as the volume of the permeable voids filled with water. Same as the volume the mass measured include the mass of the aggregate and the mass of water filled in the permeable voids of the aggregate.

Apparent Specific Gravity measures only the mass and volume of just the aggregate particles, not even the volume of the permeable voids. This is intended to provide the specific gravity of only the solid.

Tests were performed according to ASTM C 127 and ASTM C 128, and the result is shown in Table 3.9 for Margalla and Ubhan Shah aggregates.

Table 3.9: Results of the Specific Gravity

Size of Aggregate	Specific Gravities			
	Margalla		Ubhan Shah	
	Bulk	Absorption (%)	Bulk	Absorption (%)
20 – 38 mm	2.640	1.14	2.681	0.359
10 – 20 mm	2.630		2.678	
5 – 10 mm	2.620		2.682	
0 – 5 mm	2.590	2.27	2.584	1.586

3.2.1.9 Gradation

The test is performed to determine the particle size distribution of aggregates. The distribution of particle sizes is one of the most important quality of aggregates that effect the composition of asphalt concrete. The particle size distribution not only effect the volumetric but also the workability and permeability of the asphalt concrete. The test is performed by measuring the percent mass of the dry aggregate on each progressive sieve once the aggregate sample is passed through a nest of sieves. The resultant graph of percentage passing versus sieve size is known as the gradation chart. The test is performed according to ASTM 136 for both the aggregate sources.

3.2.1.10 Unit Weight

The test covers the method to determine the bulk density of aggregate in both loose and compacted conditions, further the voids between the particles either fine, coarse or mixed together. This particular test can only be performed for aggregate sizes not exceeding a nominal maximum size of 5 inches (125mm). The bulk density measured in this test can also be useful in purchasing of bulk materials. Unit weight is a term traditionally used to describe the property determined by the weight over unit volume concept. The test is performed in accordance to ASTM 29 standard. The results are shown in the summary Table 3.10.

3.2.2 Summary of Aggregate Characterization

Table 3.10: Summary of the Results of Aggregate Tests

<table>
<tr><th colspan="2">Standard Test Method</th><th colspan="4">ASTM C-127 & 128</th><th>C-131</th><th>C-88</th><th>C-142</th><th>D-4791</th><th>D-5821</th><th>D-4791</th><th>D-2419</th><th>C-1252</th><th colspan="2">C-29</th></tr>
<tr><td rowspan="2">Quarry Source</td><td rowspan="2">Size of Aggregate (mm)</td><td colspan="4">Specific Gravities</td><td>Wear by L.A Abrasion %</td><td rowspan="2">Soundness %</td><td rowspan="2">Clay Lumps %</td><td rowspan="2">Elongation %</td><td rowspan="2">Fractional Faces %</td><td rowspan="2">Flakiness %</td><td rowspan="2">Sand Equivalent %</td><td rowspan="2">Void Content %</td><td colspan="2">Unit Weight Kg/m3</td></tr>
<tr><td>Bulk</td><td>SSD</td><td>App.</td><td>Abs %</td><td>Class A</td><td>Loose</td><td>Rodded</td></tr>
<tr><td rowspan="4">Margalla Aggregate</td><td>20~30</td><td>2.640</td><td>-</td><td>-</td><td rowspan="3">114.0</td><td rowspan="4">27.4</td><td rowspan="3">9.69</td><td rowspan="3">0.48</td><td rowspan="4">3.58</td><td rowspan="4">100% Crushed</td><td rowspan="4">13</td><td rowspan="4">76.6</td><td rowspan="3">-</td><td rowspan="3">1543</td><td rowspan="3">1625</td></tr>
<tr><td>10~20</td><td>2.630</td><td>-</td><td>-</td></tr>
<tr><td>10~05</td><td>2.620</td><td>-</td><td>-</td></tr>
<tr><td>5~0</td><td>2.590</td><td>-</td><td>-</td><td>2.270</td><td>4.7</td><td>2.87</td><td>39.3</td><td>1570</td><td>1780</td></tr>
<tr><td rowspan="4">Ubhan Shah Aggregate</td><td>20~30</td><td>2.681</td><td>-</td><td>2.704</td><td rowspan="3">0.359</td><td rowspan="4">21.0</td><td rowspan="3">0.17</td><td rowspan="3">0.94</td><td rowspan="4">16.6</td><td rowspan="4">100% Crushed</td><td rowspan="4">12</td><td rowspan="4">82.0</td><td rowspan="3">-</td><td rowspan="3">1361</td><td rowspan="3">1496</td></tr>
<tr><td>10~20</td><td>2.678</td><td>-</td><td>2.704</td></tr>
<tr><td>10~05</td><td>2.682</td><td>-</td><td>2.711</td></tr>
<tr><td>5~0</td><td>2.584</td><td>-</td><td>2.694</td><td>-</td><td>2.44</td><td>-</td><td>41</td><td>-</td><td>1790</td></tr>
</table>

3.2.3 Selection of Gradation

Table 3.10 shows the summarized results for the aggregate characterization. The detailed results are shown previously.

The choice and effect of gradation on asphalt concrete performance has long been recognized. Different agencies indicated different gradations for asphalt concrete mixtures keeping in view the maximum aggregate size. Asphalt mix prepared from aggregates of same source (quarry) with unvarying physical and chemical properties with same percentage of asphalt content but with changed gradation will result different properties and will perform contrarily under the same loading and environmental conditions.

The appropriate proportion of aggregate in gradation is very important for proper asphalt concrete mixture. Five (05) different types of gradations are used for asphalt wearing course and four (04) different gradations for asphalt base course mixtures considered in the research matrix. Gradations considered for asphalt wearing course are NHA – A, NHA – B, Superpave – 1, Superpave – 2 and Asphalt Institute MS-2 while for asphalt base course mixture the gradations evaluated are NHA – A, NHA – B, Superpave – 2 and Dense Bituminous Macadam (DBM). Table 3.11 shows the optimum (mid-point) gradation for both asphalt wearing and base course mixtures. From above gradation table, gradation charts are generated for both wearing and base course mixtures. Figure 3.2 shows optimum (mid-point) curves for all wearing course gradations. Similarly Figure 3.3 shows optimum (midpoint) curves for all base course gradations.

Table 3.11: Gradations for Wearing and Base Course Mixtures

Sieve Size	**Asphalt Wearing Course Gradations**					**Asphalt Base Course Gradations**			
	Cumulative Percentage Passing (%)					Cumulative Percentage Passing (%)			
	NHA-A	NHA-B	SP-1	SP-2	MS-2	NHA-A	NHA-B	SP-2	DBM
37.5 mm	100	100	100	100	100	95	100	100	100
25.4 mm	100	100	100	100	100	77	82.5	94	95
19 mm	95	100	100	100	100	65.5	72.5	86	83
12.5 mm	76	82	94	95	95	52.5	62.5	73	70
9.0 mm	63	70	87	84	82	44	52.5	65	63
6.4 mm	51.5	59	74	57	69	37	44	53	57
4.75 mm	42.5	50	65	45	59	31.5	37.5	44	52
2.36 mm	29	30	37	30	43	22.5	25	25	39
1.18 mm	20	20	21	20	30	15.5	18	16	28
0.6 mm	13	15	14	15	20	10.5	13.5	11	20
0.3 mm	8.5	10	9	10	13	7	10	7	14
0.15 mm	6	7	7	6	8.5	5.5	7	5	9
0.075mm	5	5	5	4	6	4.5	4.5	4	5.5
Pan	0	0	0	0	0	0	0	0	0

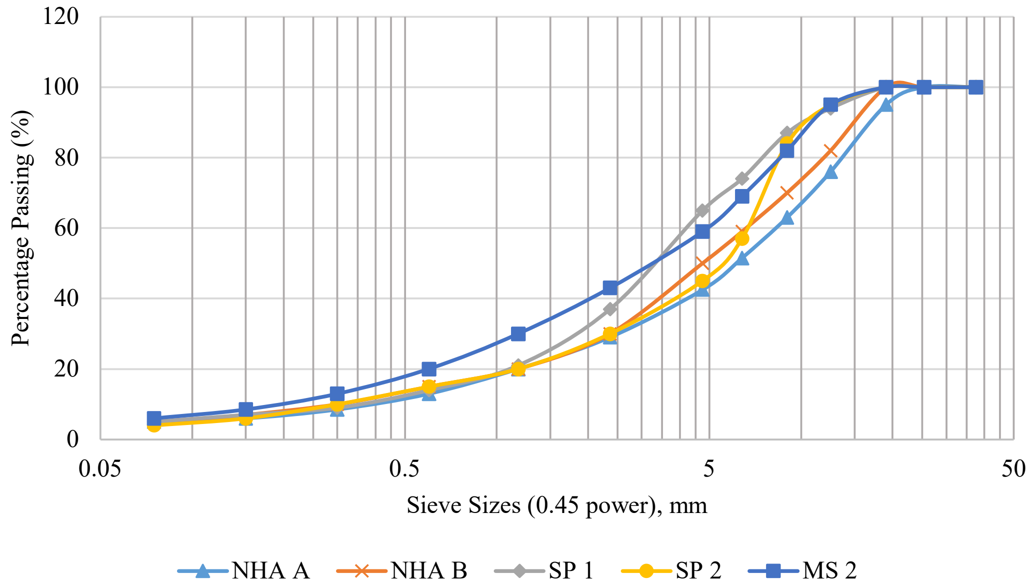


Figure 3.2: Optimum (Centre line) Gradation Curve for Asphaltic Wearing Course

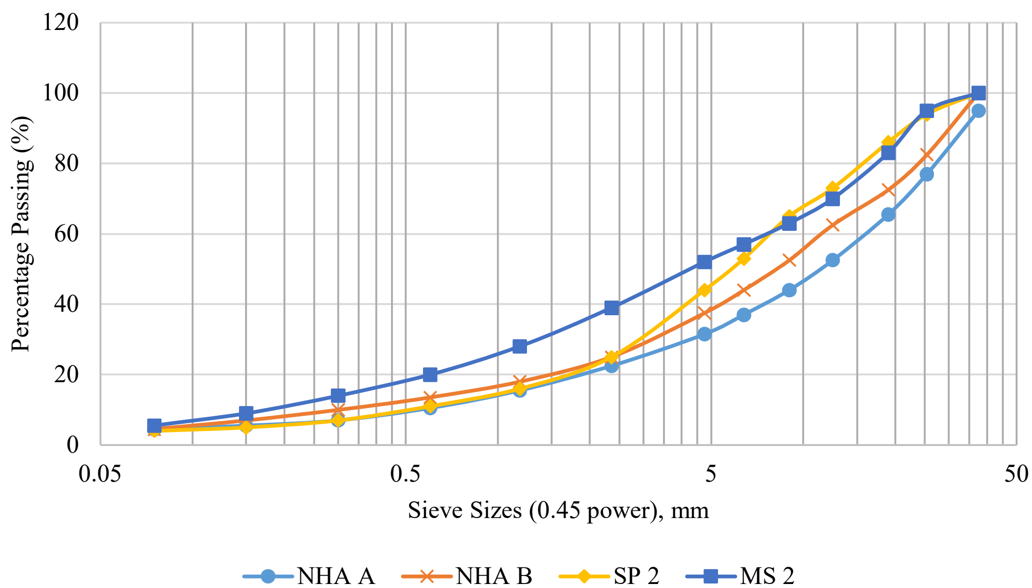


Figure 3.3: Optimum (Centre line) Gradation Curve for Asphaltic Base Course

3.3 Penetration Grading System

The Penetration grade classification was established as early in the year 1900 to represent the consistency of semi-solid asphalt. In this system, ASTM D 946 specifications classify un-aged asphalt binders into grades according to the penetration value measured at 77°F (25°C). The basic assumption of penetration grading system is that, the less glutinous the asphalt the deeper the needle will penetrate. Asphalt binder performance is empirically linked with the depth of penetration. Therefore, softer (higher penetration) asphalt binders are used for cold climates and harder (lesser penetration) are used for warm climates. In Pakistan, a standard 60/70 penetration grade is mostly used for construction of flexible pavements. However, the refineries of Pakistan also produce 40/50 and 80/100 penetration grades. The test only provides the comparative consistency of asphalt binder at a definite temperature, which can be used as a sign of vulnerability of asphalt binder to rutting and fatigue cracking.

3.4 Asphalt Binder Selection & Characterization

The physical properties of asphalt binders largely influence the performance of asphalt concrete mixtures. Properties of asphalt change over time and its age is an important factor in prediction of its performance behavior. The new Superpave tests, also known as the Performance Grading (PG) system, developed through SHRP research program measures the physical properties of asphalt that directly relate to field performance. The Performance Grading (PG) system emphasis strongly to control the viscosity at low temperature, but this is not an issue in Pakistan to greater extents due to the climatic conditions and prolonged summers. The check on suitability of both the aggregate and bitumen for the asphalt concrete preparation is necessary. There are a variety of tests including property and performance test which need to be performed before the bitumen is used for asphalt concrete preparation. Three different bitumen grades are tested including NRL 40/50, NRL 60/70 and ARL 40/50.

3.4.1 Asphalt Grading System

Commonly there are three grading systems used to characterize asphalt binders. The traditional grading systems include penetration and viscosity grading system. In early 1990s as a result of the Strategic Research Program (SHRP) in United States, a performance-based asphalt grading system known as Superpave performance grade (PG) evolved.

3.4.1.1 Viscosity Grading System

A superior asphalt grading system was developed in early 1960s that included coherent scientific viscosity test. This precise test replaced the practical penetration test for the categorization of asphalt binders as the viscosity is the fundamental property of asphalt. ASTM D 3381 specifications established for asphalt binders to test them at 140°F (60°C) and 275°F (135°C) which matches to the representative maximum temperature and temperature at the time of laying of asphalt in the field, respectively. Viscosity grading can be done on original/virgin as well as aged residue samples of asphalt binder. Viscosity grading is not yet established in Pakistan. The test fails to characterize the binder at low temperatures to reduce the cause of thermal cracking. Two asphalt binders meeting the specifications of penetration and viscosity may behave in a different way at other temperatures.

3.4.1.2 Property and Performance Test

It is necessary to perform tests according to the specifications to verify the acceptability of asphalt cement satisfying the desired characteristics including consistency, purity and safety. Different tests including property tests and performance tests must be conducted on the asphalt cement before asphalt concrete mixture preparation. Tests shown in Table 3.12 are performed on the different sources of bitumen to assess particular properties of asphalt binders.

Table 3.12: Test Standards for Property and Performance Test of Asphalt Binder

PROPERTY TESTS		
1	Penetration	ASTM D5
2	Flash and Fire Point	ASTM D92
3	Ductility	ASTM D113
4	Softening Point	ASTM C36
PERFORMANCE TESTS		
1	Rolling Thin Film Oven (RTFO)	ASTM D2872
2	Pressure Ageing Vessel (PAV)	ASTM D 6521
3	Rotational Viscometer (RV)	AASHTO T316
4	Bending Beam Rheometer (BBR)	ASTM D6648
5	Dynamic Shear Rheometer (DSR)	ASTM D7175

3.4.1.3 Penetration

Penetration test has been one of the oldest tests to measure the consistency of the asphalt binder, and the test is performed using the ASTM D 5 test. To perform the penetration test binder is heated to a suitable temperature to help it flow, but not too much that the properties of the binder are affected, and poured into a test container. The samples are brought to the standard test temperature of 25°C using temperature controlled water bath. The containers are then placed in the penetrometer equipment and a total load of 100g is applied to the needle for 5 seconds to penetrate into the binder. Penetration test was performed using two specimens of each binder and the reading were taken at five points in each sample. The detailed results of the penetration test are shown in Table 3.13 which show satisfactory results.

Table 3.13: Results for Penetration Test of Asphalt Binder

Binder	NRL 40/50		NRL 60/70		ARL 60/70	
	Penetration (0.1mm)					
	Sample 1	Sample 2	Sample 1	Sample 2	Sample 1	Sample 2
1	40	`49	65	70	61	65
2	42	46	61	70	65	60
3	41	40	64	62	60	61
4	40	45	67	65	68	68
5	43	40	69	64	62	64
Average	43		66		63	

3.4.1.4 Flash and Fire Point

The flash point of an asphalt binder is that temperature at which the sample suddenly flashes due to the presence of an open flame, on the other hand the point at which the binder gives a constant flame that temperature is termed as the fire point. Flash and fire point test are performed according to ASTM D 92 and an apparatus known as the Cleveland Open Cup (COC) is used in the test. The procedure involves filling a brass cup with asphalt binder, up to a certain volume, and heat the brass cup at a constant rate passing a test flare above the cup at definite intervals. Once the above described conditions are achieved the temperature for the flash and fire point are recorded. Flash and fire point tests were conducted using three trials for each binder. The flash point according to the specifications should be greater than 232°C. Table 3.14 shows the results for flash and fire point testing.

Table 3.14: Results of Flash & Fire Point Test of Asphalt Binder

Binder	Sample No.	Flash Point (°C)	Fire Point (°C)
NRL 40/50	1	336	360
	2	334	360
	3	336	358
	Average	335	359
NRL 60/70	1	330	356
	2	328	364
	3	328	360
	Average	329	360
ARL 60/70	1	330	364
	2	326	360
	3	328	362
	Average	328	362

3.4.1.5 Ductility

The physical property of asphalt binder, ductility, is considered to be an important characteristic. This particular test measures the ductility of asphalt binder by elongating a standard sized piece, dog bone like shaped, of asphalt binder to a point it is broken or has passed the criteria of ASTM D 113 standard test specification. The test like penetration test is performed at 25°C in a constant water bath. Once placed in the assembly the specimen is pulled out at a speed of 5 centimeter per minute until the sample breaks. The samples having a ductility value greater or equal to 100 cm are considered satisfactory. The test has been performed according to ASTM D 113 and the results are shown in the summarized Table 3.15.

Table 3.15: Results for Ductility Test of Asphalt Binder

Binder	NRL 40/50	NRL 60/70	ARL 60/70
	Ductility (cm)		
1	100	100	100
2	100	100	100
3	100	100	100

3.4.1.6 Softening Point

The temperature at which the bitumen specimen does not sustain the weight of a 3.5 g steel ball is known as the softening point of bitumen. To perform this test a ring and ball apparatus is used according to the standard of ASTM D36. First of all, the binder is heated to a certain temperature, to make it flow but keep the properties unchanged, and poured into a mold to form horizontal disks of bitumen. Once in the apparatus the steel balls are placed on the disks and the temperature is increased up to a point the disks soften enough to let the ball fall a distance of 25mm. The temperature recorded at the point is the softening point of the bitumen. Softening point test were conducted using three specimens of each binder and the results are shown in the Table 3.16.

Table 3.16: Results of Softening Point Test of Asphalt Binder

Binder	Sample No.	Softening Point (°C)			
		Right	Left	Difference	Average
NRL 40/50	1	51.5	52.0	0.5	51.8
	2	52.0	51.0	1.0	51.5
	3	52.0	51.0	1.0	51.5
	Average	51.8	51.3	0.8	51.6
NRL 60/70	1	44.5	45.0	0.5	44.8
	2	48.0	47.0	1.0	47.5
	3	45.0	44.5	0.5	44.8
	Average	45.8	45.5	0.7	45.7
ARL 60/70	1	47.5	48.0	0.5	47.8
	2	48.0	49.0	1.0	48.5
	3	48.5	48.0	0.5	48.3
	Average	48.0	48.3	0.7	48.2

The summary of results for base line fundamental testing of the binders used in the research is presentenced in table below:

Table 3.17: Summary of Results of the Property Tests for Asphalt Binder

Property Tests		Bitumen Type		
		NRL 40/50	NRL 60/70	ARL 60/70
1	Penetration Value	43	66	63
2	Flash Point (°C)	335	329	328
3	Fire Point (°C)	359	360	362
4	Ductility (cm)	≥100	≥100	≥100
5	Softening Point (°C)	51.6	45.7	48.2

3.4.1.7 Asphalt Binder Ageing Using RTFO and PAV

Asphalt binder becomes stiffer, its viscosity increases, primarily due to oxidation which occurs rapidly during construction of bituminous mixtures but more slowly when the mixture is in service. The reaction with atmospheric oxygen is identified as probably being the major cause of age hardening. Typically, there are two types of binder ageing, short term ageing that simulates the ageing during construction and can be replicated using Rolling Thin Film Oven (RTFO) (Figure 3.4) and long term ageing that simulates the ageing of pavement for 7 – 10 years and can be replicated in the Pressure Ageing Vessel (PAV) (Figure 3.5).

Rolling thin film oven test RTFO (AASHTO T240; ASTM D2872) is performed to pass the binder through short term ageing phenomenon. Binder to be aged is poured into the bottles that are part of RTFO apparatus. During the RTFO ageing, binder in un-agitated form is taken and heated until it begins to flow. Then the RTFO bottles are filled with 35 ± 0.5g of asphalt binder. The bottles are then turned upside down for homogeneity of the binder around the walls of the bottle. The bottles are fixed in the rack of the oven and rotated at 15 revolutions per minute for a total of 85 minutes at a test temperature of 163°C so the binder may be exposed to air and temperature. Once the rotations are complete all the bottles are to be taken out of the oven and residue of all the bottles is scrapped and collected in a single container.

Figure 3.4: Rolling Thin Film Oven (RTFO)

Figure 3.5: Pressure Ageing Vessel (PAV)

Long term ageing is achieved by passing the RTFO aged binder through the ageing phenomenon in pressure ageing vessel, PAV. 50 grams of RTFO aged binder is placed in pans which are loaded into the PAV. The binder is then subjected to an air pressure of 2100 ± 100 kPa for 20 hours at a temperature of 90, 100, or 110°C, depending on the mean maximum weekly pavement temperature. Normally ageing temperature of 100°C is required except when the binder will be used in a desert region. In this case high pavement design temperature is greater than 64°C. The performance based properties of the selected binders were determined by performing tests specified by ASTM D-6373 on binders using Dynamic Shear Rheometer (DSR), Rotational Viscometer (RV) and Bending Beam Rheometer (BBR). Testing on asphalt binders was carried out in three ageing conditions i.e., original, rolling thin film oven (RTFO) aged and pressure ageing vessel (PAV) aged condition. RV testing was performed on binders in original condition.

3.4.1.8 Rotational Viscometer (RV)

Rotational viscometer (Figure 3.6) is used to determine the viscosity of asphalt binder at higher temperatures, simulating temperatures at pumping, mixing and compaction. This test can be conducted at various temperatures according to the need, but generally it is conducted at 135°C as mentioned in AASHTO T 316. Temperature-viscosity graphs can be developed using the rotational viscometer for assessing the mixing and compaction temperatures used in the mix design. This test measures the torque of a cylinder-shaped shaft immersed in asphalt binder at a constant rotational speed of 20 rpm, and the torque is further processed to viscosity by the equipment. The viscometer can be used to measure the viscosity of asphalt binders in the range of 0.01 Pa.s (0.1 poise) to 200 Pa.s (2000 poise). Viscosity is measured within the temperature range of 100 to 260°C (212 to 500°F). Tests were performed on each bitumen grade using three replicates at two different temperatures. Table 3.18 shows the results for Rotational Viscometer (RV) testing. Figure 3.7 presents the trends for RV testing.

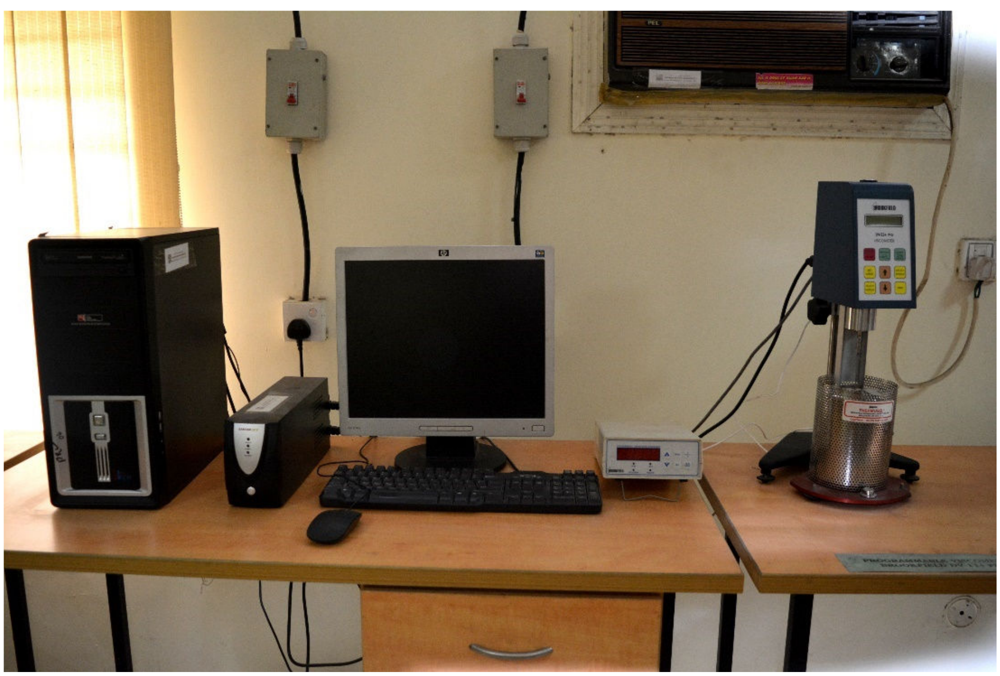


Figure 3.6: Rotational Viscometer (RV)

Table 3.18: Results of Rotational Viscosity Test of Asphalt Binder

Binder	Test Temperature (°C)	Torque (%)	Shear Stress	Shear Rate Sec^{-1}	Viscosity (cP)
NRL 40/50	124.60	6.50	55.25	6.80	812.50
	134.94	3.73	31.45	6.80	462.50
	144.75	2.28	19.55	6.80	287.50
	154.64	1.48	12.75	6.80	187.50
	164.69	1.03	8.50	6.80	125.00
	174.68	0.73	5.95	6.80	87.50
	184.78	0.52	4.25	6.80	62.50
NRL 60/70	124.70	4.63	39.53	6.80	581.25
	134.81	2.78	23.80	6.80	350.00
	144.79	1.73	14.45	6.80	212.50
	154.96	1.15	9.35	6.80	137.50
	164.58	0.80	6.80	6.80	100.00
	174.68	0.56	5.10	6.80	75.00
	184.71	0.39	3.40	6.80	50.00
ARL 60/70	125.03	2.79	23.80	6.80	350.00
	134.64	1.68	14.45	6.80	212.50
	145.04	1.06	8.93	6.80	131.25
	155.00	0.73	5.95	6.80	87.50
	164.84	0.51	4.25	6.80	62.50
	174.61	0.38	3.40	6.80	50.00
	185.00	0.24	1.70	6.80	25.00

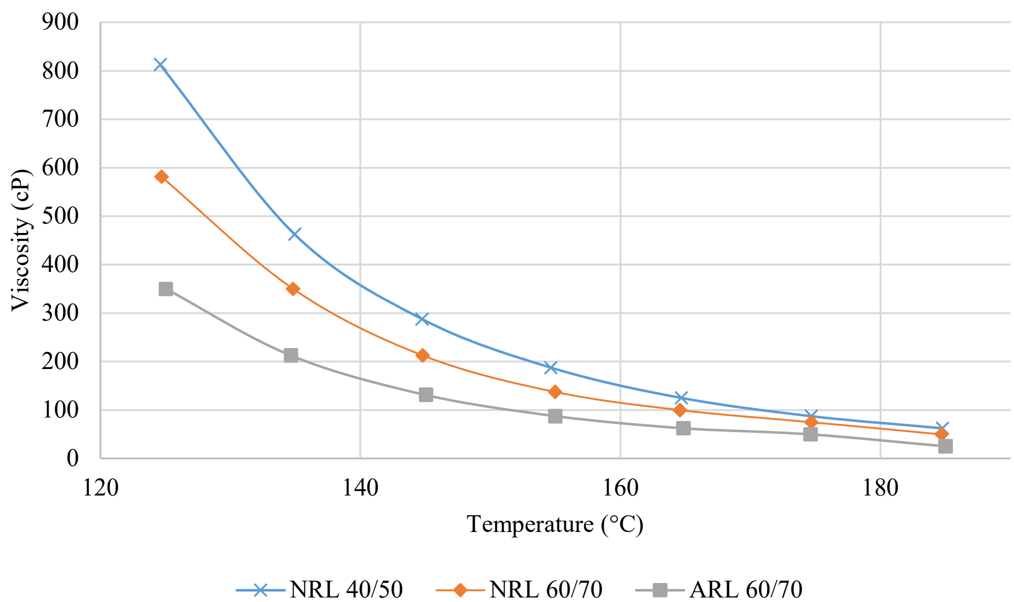


Figure 3.7: Rotational Viscosity Plot for Asphalt Binder

3.4.1.9 Bending Beam Rheometer (BBR)

The bending beam rheometer measures the low-temperature stiffness characteristics of the asphalt binder. The test equipment and test procedure used for this test is AASHTO TP1. A beam of asphalt binder 125 mm long, 12.5 mm wide and 6.25 mm thick is formed by pouring the asphalt binder into a mold and allowing it to cool. The beam is then placed in the cooling bath until it achieves the specified testing temperature. During testing, the loading platform and beam are submerged in a cooling bath. The bath liquid maintains the test temperature and at the same time provides buoyancy to the specimen. Buoyancy minimizes deflections caused by the mass of the beam. The beam is placed on supports, 100 mm apart, and is loaded at the mid span with a constant load of 95 to 100 g.

Figure 3.8: Bending Beam Rheometer (BBR)

Bending Beam Rheometer (BBR) tests were carried out on each bitumen type at three different temperatures including 0°C, -6°C, -12°C and -18°C. Tests were carried out on the aged samples of bitumen. Rotating Thin Film Oven (RTFO) and Pressure Ageing Vessel (PAV) had been used to age the bitumen before testing. Test values can be used to find the low temperature grade of bitumen which is a part of the performance grading (PG) system used in the Superpave mix design. Table 3.19 shows the results for Bending Beam Rheometer (BBR) testing and Figure 3.9 presents the trends for Bending Beam Rheometer testing.

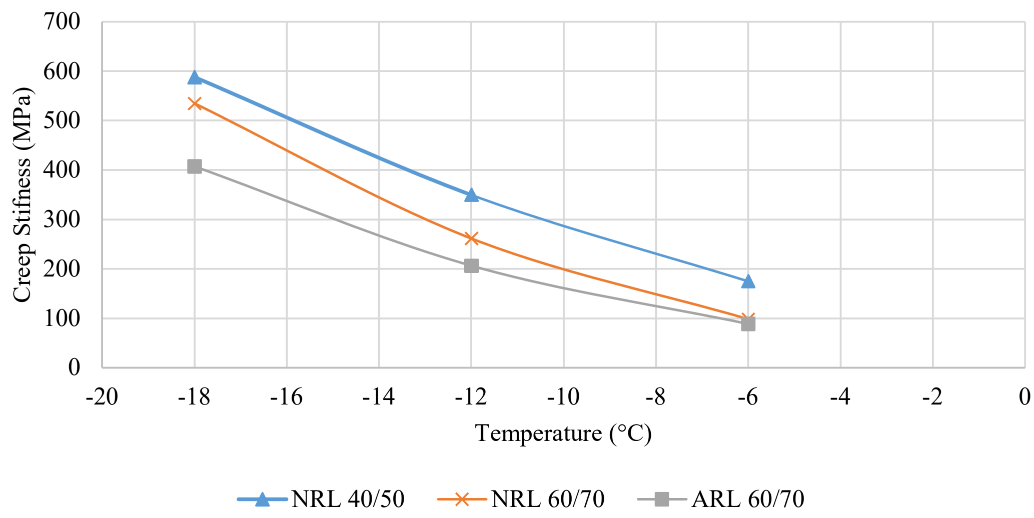


Figure 3.9: Temperature versus Stiffness Plot for PAV Aged Asphalt Binder

Table 3.19: Results for Bending Beam Rheometer (BBR) Test of Asphalt Binder

Binder	Test Temperature (°C)	Sample	Load (mN)	Deflection (mm)	Stiffness (MPa)	m-value
NRL 40/50	0	1	982.0	1.2433	64.2558	0.528938
		2	983.2	1.4661	54.7179	0.528696
		Average	982.6	1.3547	59.4869	0.528817
	-6	1	977.9	0.4353	183.3025	0.389046
		2	980.8	0.4802	166.6406	0.388455
		Average	979.35	0.45775	174.9716	0.388751
	-12	1	985.3	0.2396	335.4772	0.324154
		2	984.3	0.2205	364.1617	0.273689
		Average	984.8	0.23005	349.8195	0.298922
	-18	1	985	0.1365	588.6523	0.173744
		2	983.9	0.1367	587.2437	0.194908
		Average	984.45	0.1366	587.948	0.184326
NRL 60/70	0	1	984.9	2.4410	32.8618	0.536664
		2	990.5	2.6154	30.7371	0.585441
		Average	969.7	2.5282	31.7995	0.561052
	-6	1	980.1	0.8298	96.3698	0.445858
		2	986.8	0.8049	100.0134	0.439460
		Average	983.45	0.81735	98.1916	0.442659
	-12	1	983.9	0.3375	237.8023	0.314361
		2	983	0.2807	285.7533	0.309442
		Average	983.45	0.3091	261.7778	0.311902
	-18	1	987.5	0.1735	464.3510	0.212684
		2	982.2	0.1325	604.6065	0.228540
		Average	984.85	0.153	534.4788	0.220612
ARL 60/70	-6	1	979.2	0.8890	89.8589	0.382889
		2	981.4	0.9185	87.1743	0.359654
		Average	980.3	0.90375	88.5166	0.371272

Binder	Test Temperature (°C)	Sample	Load (mN)	Deflection (mm)	Stiffness (MPa)	m-value
	-12	1	985.4	0.3877	207.3689	0.338606
		2	979	0.3892	205.2467	0.332024
		Average	982.2	0.38845	206.3078	0.335315
	-18	1	985.1	0.2013	399.2605	0.215430
		2	981.8	0.1928	415.4200	0.247418
		Average	983.45	0.19705	407.3403	0.231424

Figure 3.9 show decrease in creep stiffness of all three binders with increase in temperature. Highest value of stiffness is observed for NRL 40-50 that ranges from 587 MPa to 174 MPa for temperature of -18°C to -6°C. Figure 3.10 shows increase in m-value of asphalt binders with the increase in temperature. ARL 60-70 shows highest m-values at -18°C that is 0.2314. From temperature -18°C to -6°C, m-value ranges from 0.18 to 0.4 for all three binders. Figure 3.11 shows increase in deflection in asphalt binders with the increase in temperature. Deflection varies from 0.13mm to 0.9037mm with in temperature range of -18°C to -6°C for all binders. ARL 60-70 shows highest deflection than NRL 40-50 and NRL 60-70.

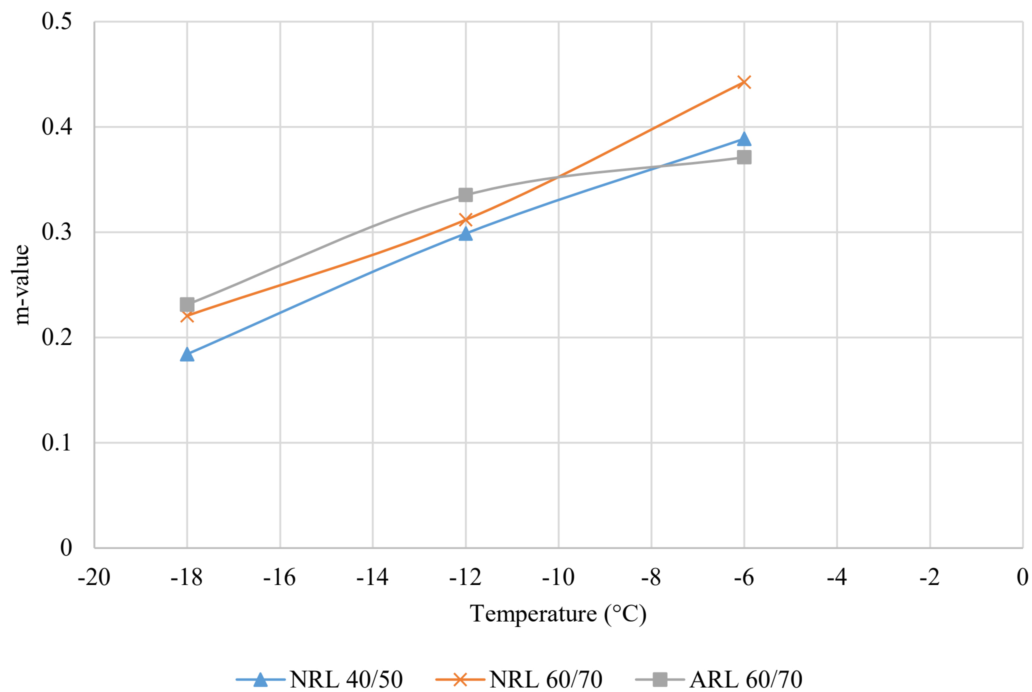


Figure 3.10: Temperature versus m-value Plots for PAV Aged Asphalt Binder

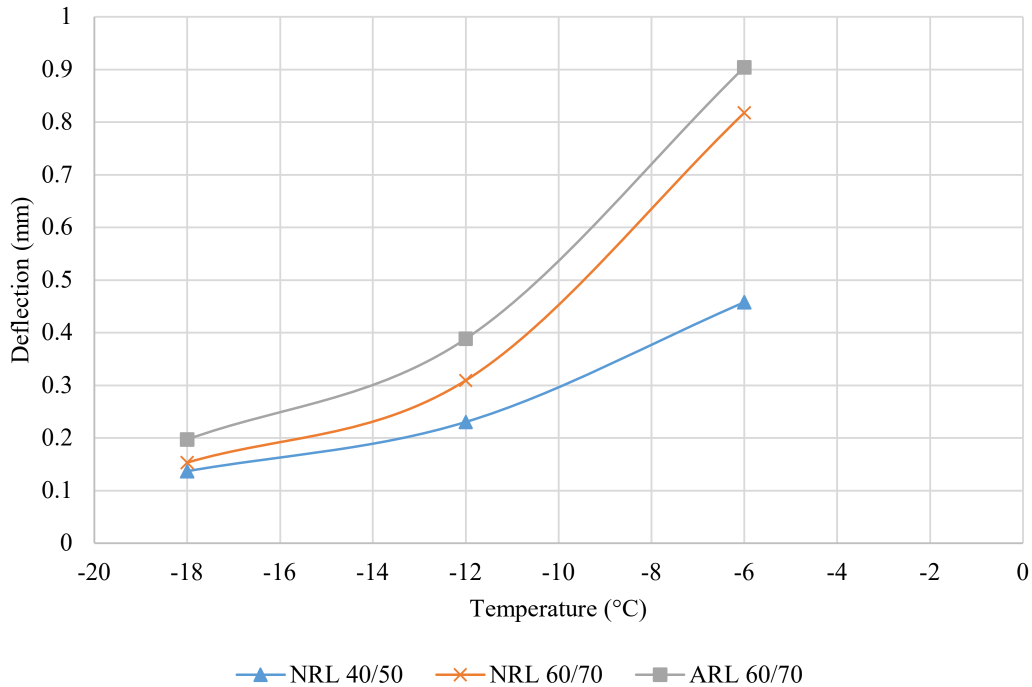


Figure 3.11: Temperature versus Deflection Plots for PAV Aged Asphalt Binder

3.4.1.10 Dynamic Shear Rheometer (DSR)

The dynamic shear rheometer is used to measure the linear viscoelastic moduli of asphalt binders in the sinusoidal loading. Measurements can be obtained at different temperatures, frequencies, stress and strain levels. Equation 3-1 is used to calculate the complex modulus, G*. The applied torque (τ) and angular rotation (θ) is measured using Equations given below:

$$G^* = \frac{\tau_m}{\gamma_m} \qquad \text{3-1}$$

Where;

G* = Complex Shear Stiffness Modulus, kPa

τ_m = Maximum Shear Stress, kPa

γ_m = Maximum Shear Strain

$$\tau = \frac{2T}{\pi r^3} \qquad \text{3-2}$$

$$\gamma = \frac{\theta r}{h} \qquad \text{3-3}$$

Where;

T = Applied Torque

r = Radius of Plate

Θ = Deflection or Angle of Rotation.

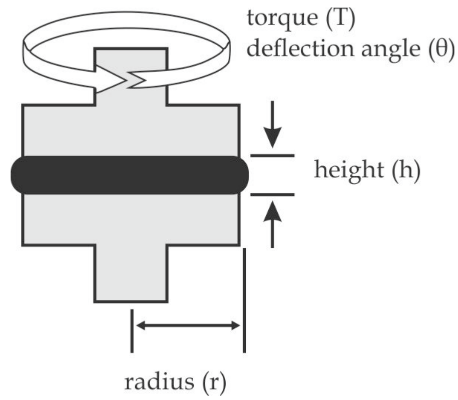
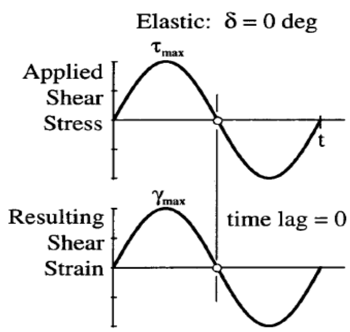
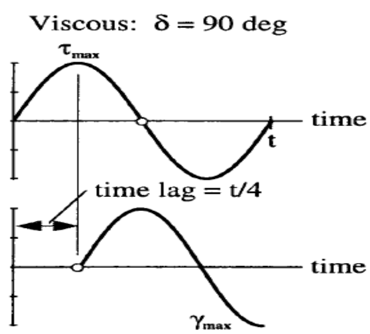


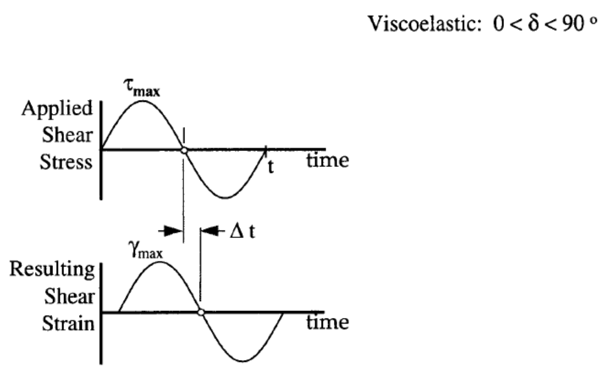
Figure 3.12: Principal of Operation of Dynamic Shear Rheometer (DSR)



(a)



(b)



(c)

(a) Elastic Behaviour (δ = 0°),
(b) Viscous Behaviour (δ = 90°) and
(c) Viscoelastic Behaviour (0° <δ< 90°)

Figure 3.13: Relationship of Shear Stress and Shear Strain

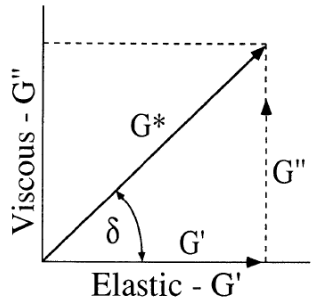


Figure 3.14: Graphical Representation of Components of Complex Shear

The values of G'' and G' can also be considered to be an estimate of the viscous component and the elastic components of the complex stiffness modulus G*. A perfectly elastic material exhibits a phase angle equal to zero, while a viscous material would exhibit an angle of 90°. Thus, an elastic material would exhibit maximum shear stress and maximum shear strain at the same time, while for a perfectly viscous material maximum shear stress would occur at the same time as minimum shear strain. Asphalt shows elastic behavior (δ = 0) at cold temperatures and viscous behavior (δ = 90°) at very high temperatures. In order to minimize rutting the stiffness value, G*/sin δ, of the asphalt binder after RTFO must be greater than 2.2 kPa at the maximum 7 – day average pavement design temperature. If the ageing does not occur during construction, then the stiffness value of the original unaged asphalt binder must be greater than 1.0 kPa at the same pavement temperature. On the other hand, to control fatigue the stiffness value, G* sin□, of the asphalt binder after RTFO and PAV ageing must be less than 5000 kPa at average pavement temperature.

Figure 3.15: Dynamic Shear Rheometer (DSR)

It is observed in Figure 3.16 with increase in temperature, the G* value is decreased. G* values of PAV aged binders are observed to be highest among original and RTFO aged binders. G* values of PAV aged binders have been observed from 60578kPa to 1349kPa in a temperature range of 7°C to 37°C whereas original and RTFO aged binders have been observed from 63.5 kPa to 0.075 kPa in a temperature range of 46°C to 82°C. According to previous research (Epps et al, 2001) found decrease in G* of binder with increase in temperature. It may be due to decrease in viscosity of binder with increase in temperature.

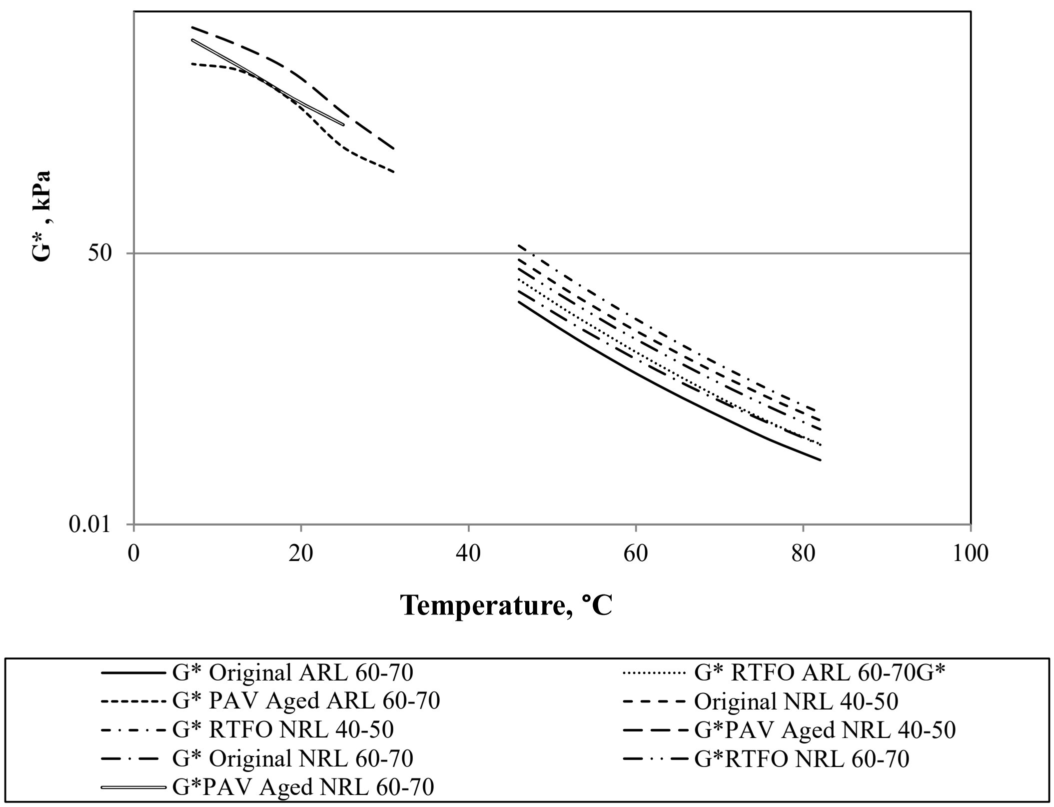


Figure 3.16: Effect of Temperature on G* of Asphalt Binders

Figure 3.17 shows an increase in phase angle value with increase in temperature. From 7°C to 31°C the phase angle value ranges between 30°C to 65°C whereas the phase angle value of original and RTFO aged binders varies between 80°C to 88°C in the temperature sweep of 46°C to 82°C. Gandhi in 2008 also found increase in phase angle with increase in temperature. Due to increase in temperature, viscosity of binder decreases and it takes more time to return to its original shape hence increasing the phase angle of binder.

According to Figure 3.17 value of G*/Sin δ decreases with the increase in temperature. From 7°C to 31°C values of G*/Sin δ ranges between 126630kPa to 1500kPa for PAV aged binders. Values for original and RTFO aged binders varies from 64kPa to 0.3kPa within temperature range of 46°C to 82°C. (Epps et al., 2001) indicated decrease in G*/Sin δ with increase in temperature. That may be due to decrease in viscosity of binder due to increase in temperature.

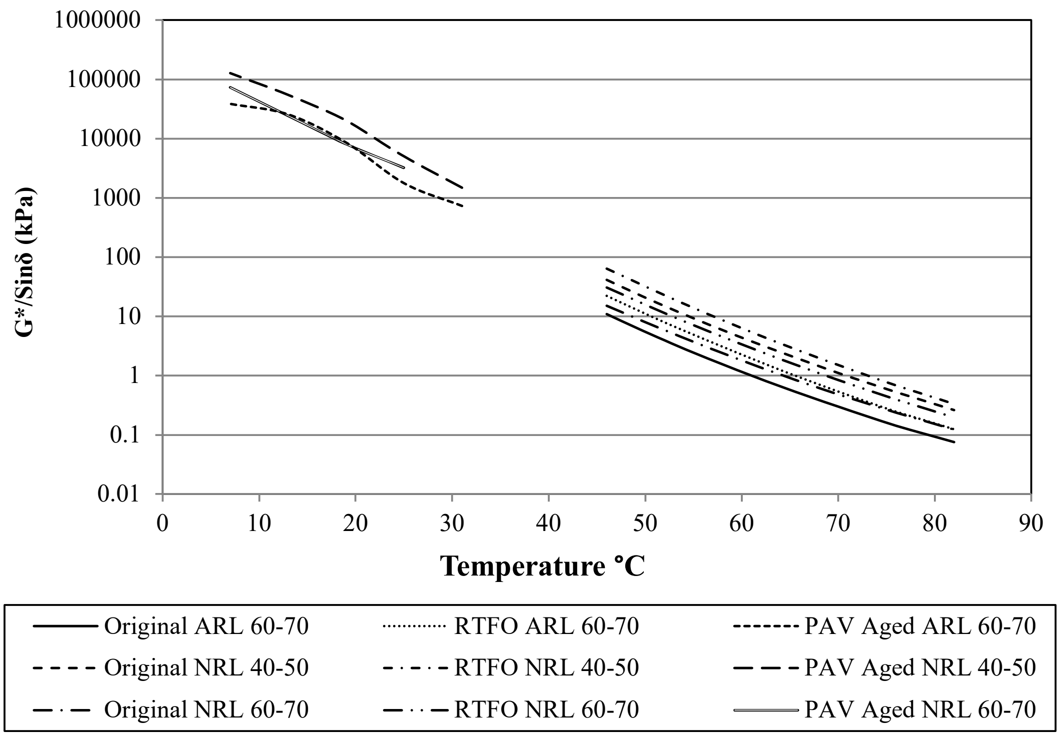


Figure 3.17: Effect of Temperature on G*/Sin δ of Asphalt Binders

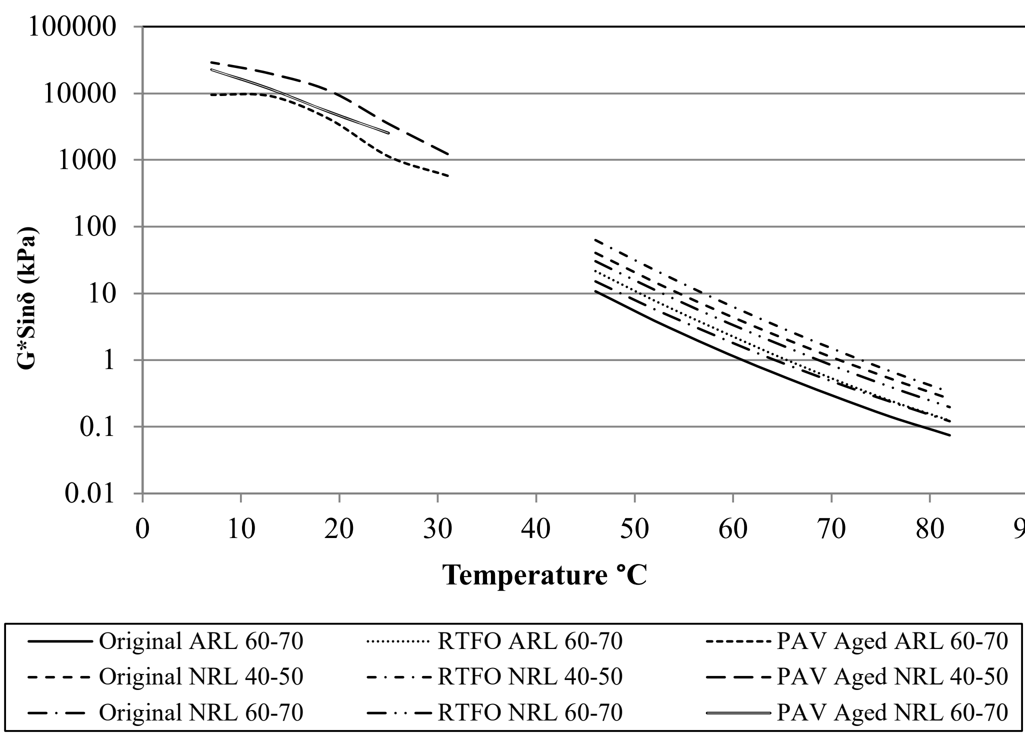


Figure 3.18: Effect of Temperature on G*Sin δ of Asphalt Binders

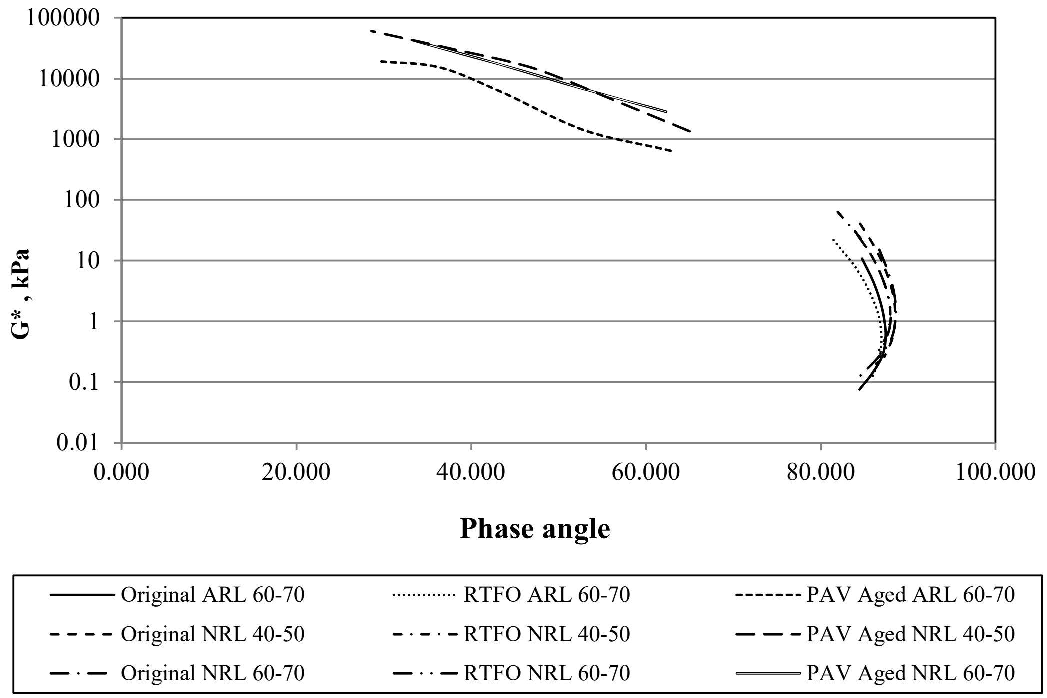


Figure 3.19: Variation in G* with change in Phase Angle for Asphalt Binders

In Figure 3.18 it can be seen that there is a decrease in G*Sin δ with the increase in temperature. For temperature 7°C to 37°C values of G*Sin δ ranges from 29000 kPa to 500kPa for PAV aged binders. Original and RTFO aged binders show comparatively lesser values and their values lie within range of 65 kPa to 0.075 kPa from temperature 46°C to 82°C. Figure 3.19 shows decrease in complex shear modulus with the increase in phase angle. G* varies from 40700kPa to 2900kPa from phase angle 30° to 65° for PAV aged binders. Original and RTFO aged binders show lower values of G* as compared to PAV aged binders. From phase angle 83° to 86° the original and RTFO aged binders show G* within the range of 31kPa to 0.1kPa.

It has been observed in above figures that the complex shear modulus of all binders decreases with increase in temperature. Also, it is observed that the complex modulus values are increased for the binders after RTFO ageing. The PAV aged binders have far high values of complex shear modulus as compared to Original and RTFO aged binders. Phase angle value tends to increase first with increase in temperature and then decrease slightly. The phase angle increases with the increase of test temperature from -20°C to 20°C, but for 40°C and 54°C the phase angle decreases with the increase of the test temperature which indicates aggregate interlocking effects.

3.4.2 Summary of Binder Characterization

The summary of all the conventional property tests performed on the three bitumen sources is shown in Table 3.20. The results show that all the three sources satisfy the penetration grade requirements according to the ASTM standards. The performance test results whereas show that NRL 40/50 is the most viscous grade of bitumen followed by NRL 60/70 and ARL 60/70 respectively.

Table 3.20: Summary of Property Test Results of Asphalt Binder

	Penetration Test			Flash & Fire Point Test			Softening Point Test			Ductility Test		
Standard	ASTM D5/ AASHTO T49			ASTM D92/ AASHTO T49			ASTM D36			ASTM D113		
Bitumen Source	NRL 40/50	NRL 60/70	ARL 60/70	NRL 40/50	NRL 60/70	ARL 60/70	NRL 40/50	NRL 60/70	ARL 60/70	NRL 40/50	NRL 60/70	ARL 60/70
Results	43	66	63	335°C & 359°C	329°C & 360°C	328°C & 362°C	52°C	46°C	48°C	Greater than 100 mm		

3.5 Bituminous Mix Preparation

The design of asphalt concrete bases on the concept of determining the best possible combination of aggregate and asphalt binder to give the pavement structure a long lasting performance. The aggregate structure is the main concern in asphalt mix design to prevent deformation, so the mix design should provide a stable mixture resistant to further densification under traffic. A well-designed mix needs to be constructed so that very little change takes place in the air voids after construction. The asphalt concrete mixture must not only have high shear strength to resist rutting but also have high tensile strength and flexibility to provide sufficient fatigue life to resist cracking. Many laboratory procedures have been developed to determine the necessary percentages of materials that are to be used in the asphalt concrete mix and these procedures include the determination of correct aggregate blend to produce proper gradation on mineral aggregates along with the type and amount of asphalt binder to be used. Among the many methods developed for the preparation of asphalt concrete mixtures Marshall Method is mostly preferred which was developed by Bruce G. Marshall in 1939 at the Mississippi Highway Department and the standard test specification followed is ASTM D 6926. The Marshall method is originally applicable to only the asphalt concrete paving mixtures which contain aggregates with a maximum size of lesser and equal to 1 inch (25 mm). For asphalt concrete mixtures containing aggregates with maximum sizes up to 1.5 inch (38 mm) the modified Marshall method is used.

The heavy axle loads present in Pakistan impose high stresses on the aggregate structure, and for that even the 75 number of blows in Marshall for wearing course may not be sufficient enough to produce a worthy structure. A good quality aggregate with suitable gradation can increase the shear strength of asphalt concrete mixture. The air voids in the mixture are one component for the selection of a good mix design. The purpose of adequate air voids is to ensure that the mix does not rut due to loss in shear strength at low air voids or has durability issue due to open structure with high voids. The correct air voids are subjected to the voids in mineral aggregate, the aggregate structure in terms of the gradation, nominal maximum aggregate size, amount of fines, compaction level and the bitumen content.

There are a series of test samples prepared for a range varying bitumen content in determining the design bitumen content for a particular gradation of aggregates so that well defined curves a developed for the data. Each of the test sample generally requires around 1200 g (2.7 lb) of aggregate, and to get an adequate amount of data at least three replicate test samples are to be prepared for every bitumen content. The standard test sample has a diameter of 4 inch (102 mm) and height of 2 ½ inch (64 mm), and for the modified test the samples have a diameter of 6 inch (152.4 mm) and a height of 3.75 inch (95.2 mm). The detail of the equipment to be used in the test for both the standard and the modified Marshall test are present in the Asphalt Institute Manual Series No. 2 (MS-2). The steps involved in the preparation of the test specimens are as following:

3.5.1 Number of Specimen

The number of specimens to be prepared were at least three replicate samples for each of the combination of aggregate gradation and bitumen content. For every combination 5 different percentages of bitumen were selected to create a data set which further could be used to determine the optimum bitumen content. A total of 17 samples were prepared for each combination of aggregate and bitumen, including 15 samples for the determining the optimum bitumen content and 2 samples to verify the determined optimum bitumen content. The summary different combinations of aggregate and bitumen and the number of specimens prepared in shown in Table 3.21 and Table 3.22.

Table 3.21: Number of Specimens for Wearing Course Gradation

Gradation	Aggregate	Bitumen	No. of Sample to Determine OBC	No. of Sample for Verification
NHA – A	Margalla	NRL 40/50	15	2
		ARL 60/70	15	2
	Ubhan Shah	NRL 40/50	15	2
		NRL 60/70	15	2
		ARL 60/70	15	2
NHA – B	Margalla	NRL 40/50	15	2
		ARL 60/70	15	2
	Ubhan Shah	NRL 40/50	15	2
		NRL 60/70	15	2
		ARL 60/70	15	2
SP – 1	Margalla	NRL 40/50	15	2
		ARL 60/70	15	2
	Ubhan Shah	NRL 40/50	15	2
		NRL 60/70	15	2
SP – 2	Margalla	NRL 40/50	15	2
		ARL 60/70	15	2
	Ubhan Shah	NRL 40/50	15	2
		NRL 60/70	15	2
MS – 2	Margalla	NRL 40/50	15	2
		ARL 60/70	15	2
	Ubhan Shah	NRL 40/50	15	2
		NRL 60/70	15	2
		ARL 60/70	15	2

Table 3.22: Number of Specimens for Base Course Gradation

Gradation	Aggregate	Bitumen	No. of Sample to Determine OBC	No. of Sample for Verification
NHA – A	Margalla	ARL 60/70	15	2
	Ubhan Shah	NRL 60/70	15	2
NHA – B	Margalla	ARL 60/70	15	2
	Ubhan Shah	NRL 60/70	15	2
SP – 2	Margalla	ARL 60/70	15	2
	Ubhan Shah	NRL 40/50	15	2
		NRL 60/70	15	2
		ARL 60/70	15	2
DBM	Margalla	ARL 60/70	15	2
	Ubhan Shah	NRL 40/50	15	2
		NRL 60/70	15	2
		ARL 60/70	15	2

3.5.2 Preparation of Aggregate & Bitumen

The aggregates are dried up to a constant temperature at 105°C to 110°C, and once dried then separate the aggregates into the required sizes using the dry sieving technique. The effect of gradation of the aggregate on the performance of asphalt concrete in an issue which many different agencies around the world cater by using different gradations keeping in consideration the maximum aggregates size. Even if the different asphalt concrete mixtures use the same source of aggregate, meaning that the physical and chemical properties remain the same, but by changing the gradation can alter the performance of the aggregate and bitumen blend under the same loading and environmental conditions. In order to study the effect of gradation four different wearing course gradations were used in the research that are shown in Table 3.11.

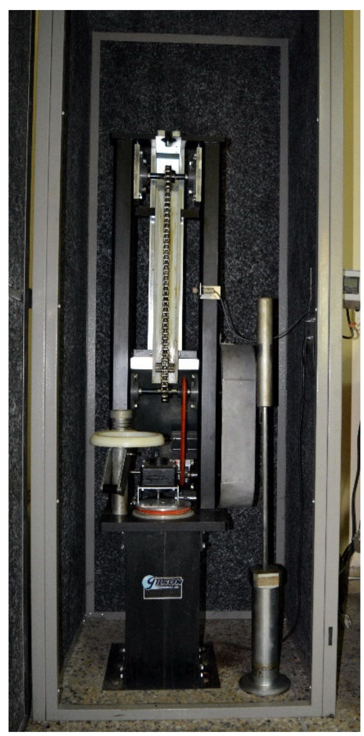
3.5.3 Mixing of Aggregate & Bitumen

The temperature versus viscosity plots for the bitumen being used in the mix are helpful to get the required temperature needed to get a viscosity of 170 ± 20 centistokes kinematic and 280 ± 30 centistokes kinematic for mixing and compaction respectively. Figure 3.20 shows the equipment used to mix the aggregate and bitumen to prepare a uniform blend. Generally, a trial sample is to be prepared to prepare the aggregate batch. The aggregates should be placed in the mixing bowl and dry mix thoroughly, once the aggregate blend and bitumen are within the limits of the mixing temperature mix the aggregate and bitumen as quickly and thoroughly as possible to achieve a uniform consistency.

Figure 3.20: Electric Mixer to blend Aggregate and Bitumen

3.5.4 Compaction of Specimen

(a)



(b)

Figure 3.21: Pedestal, Hammer (mechanical) and the Mold used for Marshall Specimen;

(a) For 4-inch Dia Sample, (b) For 6-inch Dia Sample

The sample mold assembly is thoroughly cleaned and heated to a temperature between 95 °C and 150 °C. The entire batch of aggregate and bitumen is placed in the mold in which paper disk has been placed and scoop the mixture with a spatula or trowel around the perimeter and in between to get a slightly rounded shape. Once placed the mold on the compaction assembly and apply 75 or 112 number of blows to wearing course gradation and base course gradation respectively for heavy traffic, once the blows are completed then repeat the sample compaction on reverse side of the mold also. Remove the sample using a jack or other compressive device after cooling. Pedestal, hammer (mechanical) and the mold used for preparing Marshal Samples are shown in Figure 3.21.

3.5.5 Volumetric Properties

The Marshall method subjects each and every compacted sample to test for the bulk specific gravity, the stability and flow of the sample and the density and air void analysis in order to determine the optimum bitumen content of a particular blend of aggregate and bitumen. The equipment required for the performance of the mentioned tests is a compression testing device known as the Marshall Testing Machine, this apparatus should conform to the ASTM D 1559 standard.

3.5.5.1 Bulk Specific Gravity

The bulk specific gravity test may be performed as soon as the freshly-compacted specimens have cooled to the room temperature according to the ASTM D 1188. The process of determining the bulk specific gravity of the particular compacted asphalt concrete specimen requires taking the weight of the dry sample, the weight of the sample submerged in the water for a time until the voids are filled with water and the weight of the samples using the saturated surface dry method.

3.5.5.2 Stability & Flow Test

Once completed for the test of bulk specific gravity, as the test is non-destructive the same samples are then subjected to stability and flow test, to determine the stability and flow values of the particular mixtures. The value got from the stability test is actually the maximum load, in Newton (Ib.), the standard Marshall Test sample can resist at a temperature of 60°C, and to achieve that the standard samples are dipped into a water bath at 60°C ± 1°C for almost 30 to 40 minutes before the test. The Marshall Testing Machine applies the load by increasing the load at a rate of 50.8 mm/minute until the maximum load is achieved, and the Marshall stability is the load recorded at the point when the load just starts to decrease. As the test is being performed a dial gauge is also attached to the frame in which the sample is placed and the deformation in the vertical direction is recorded in the increment of 0.25 mm. the deformation at the point the maximum load is noted is the flow value of the sample. The tests were performed for all the different types of mixtures mentioned earlier. The stability of any asphalt concrete is associated with the friction and cohesion between the aggregates and subsequently related to the resistance capability of asphalt concrete to rutting and shear stresses. The cohesion between the aggregates in an asphalt concrete is due to the biding force provided by bitumen and also the interlocking of aggregates with each other. Figure 3.22 shows the equipment used for the stability test of Marshall Samples.

Figure 3.22: Marshall Stability and Flow Test Equipment

3.5.5.3 Density & Void Analysis

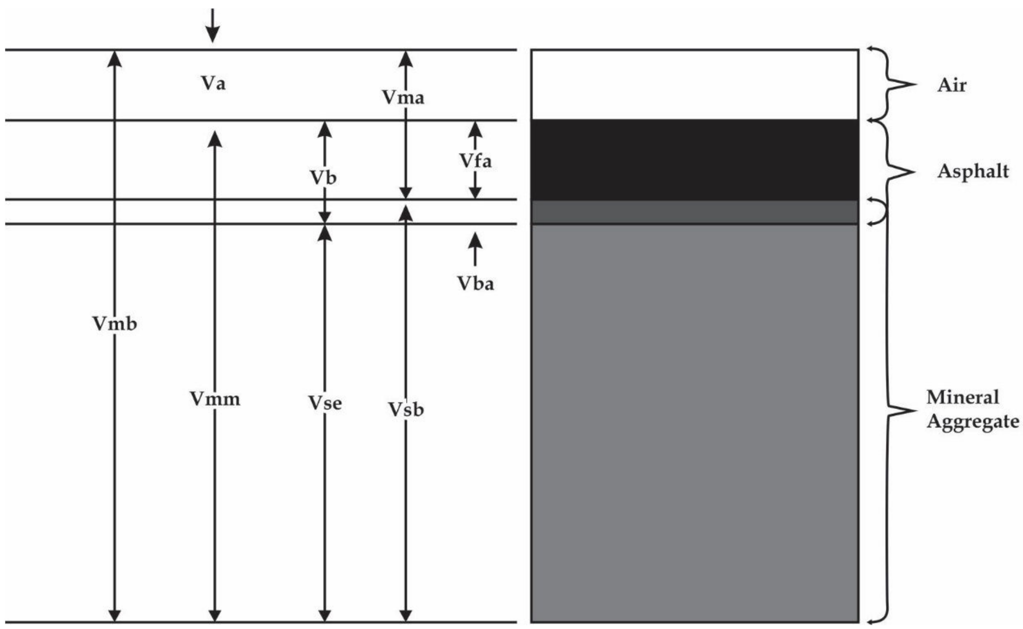


Figure 3.23: Schematic Drawing of Volumes in Compacted Asphalt Concrete Sample

The volumetric properties, or the density and void percentages, of a compacted asphalt concrete provide some indication of the mixture's probable pavement service performance. Aggregates are porous minerals and have the tendency to absorb water and bitumen up to a certain level, and this absorption varies with every different type of aggregate. Therefore, it is very important to determine these properties of aggregates on basis of three methods (bulk specific gravity, apparent specific gravity and effective specific gravity) that measure the specific gravity of aggregates differently defined by the volume of aggregate as the mass remains the same (MS – 2 Asphalt Institute). The Figure 3.23 shows the different volumes present in the compacted asphalt concrete including volume of voids in mineral aggregate (Vma), Bulk volume of compacted asphalt concrete mix (Vmb), volume of void less paving mixture (Vmm), volume of voids filled with bitumen (Vfa), volume of air (Va), volume of bitumen (Vb), volume of absorbed bitumen (Vba) and volume of aggregate (Vsb by using bulk specific gravity and Vse by using effective specific gravity).

3.5.6 Interpretation of Data to Determine Optimum Bitumen Content

There was a need to determine the optimum bitumen content (OBC) of the asphalt concrete mixtures before the specimens for the performance test of Indirect Tensile Fatigue test were prepared. In accordance to that the required volumetric properties, the stability & flow of the asphalt concrete mixtures was to be known including the percentage of air voids, voids in the mineral aggregates and the voids that are filled with asphalt. After the samples were subjected to rigorous testing to determine the density, voids and specific gravities the test results need to be compiled and organized so that meaningful data can be inferred from the results. The data of the tests performed earlier is used to develop graphical plots and best fit lines and curves were drawn according to the data set. The graphs plotted against the bitumen content to determine the optimum bitumen content of the selected mixtures are:

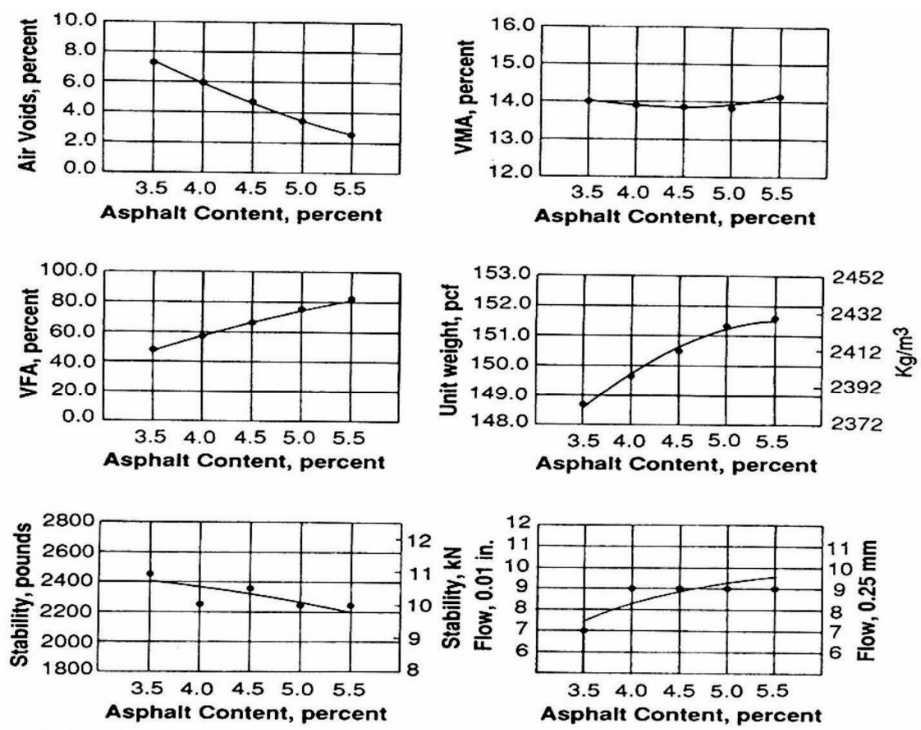


FIGURE 19-4 Test property curves for hot-mix design data by the Marshall method. (Courtesy the Asphalt Institute.)

Figure 3.24: Curves of the Property Test for Asphalt Concrete Design by Marshall Method

- Stability against Bitumen Content
- Flow against Bitumen Content
- Unit Weight of Total Mix against Bitumen Content
- Percentage of Air Voids (Va) against Bitumen Content
- Percentage of Voids Filled with Bitumen (VFA) against Bitumen Content

- Percentage of Voids in Mineral Aggregate (VMA) against Bitumen Content

The study of the plotted graphs can be helpful in determining the sensitivity of the mix to the bitumen content. The trends generally note that the stability increases with increase in bitumen content up to a maximum value and then starts to decrease, likewise the flow value increase consistently with increase in the bitumen content. The air voids and voids in the mineral aggregate decrease with increase in the bitumen content showing an inverse trend, on the other hand void filled with asphalt tend to increase with the increase in bitumen content. The trends can be seen in Figure 3.24, these are the representative curves for the blend of NHA – A wearing course gradation with Margalla aggregate and bitumen source of ARL 60/70. The criteria used to check, whether the results are satisfactory or not, are for heavy traffic as recommended by Asphalt Institute. The stability of the sample is to be more than 816 kg and a flow value in between 8 to 14. The percentage of air voids can vary from 3 to 5 percent, while the voids filled with asphalt (VFA) are to be from 65 to 75 percent. The optimum bitumen content is determined as the bitumen content that produces 4 percent air voids, the other properties are then read from the graph and the confirmatory test samples are prepared which are tested to be in range of the properties determined from the graphs. The Table 3.23 and Table 3.24 shows summary of the results for the optimum bitumen content and corresponding property values of different mixtures.

Table 3.23: Results for the Optimum Bitumen Content for Wearing Course Gradations

Aggregate Type & Gradation		Bitumen Type	OBC (%)	Stability Kg	Gmb	Gmm	Air Voids (%)	VMA (%)	VFA (%)
Margalla	NHA – A	NRL 40/50	3.88	1298	2.389	2.503	4.55	12.85	64.57
	NHA – B		4.43	1250	2.363	2.484	4.89	13.52	63.80
	SP – 1		5.60	1383	2.342	2.449	4.37	15.10	71.05
	SP – 2		4.50	1214	2.364	2.479	4.64	14.13	67.16
	MS – 2		4.73	1586	2.362	2.484	4.90	13.70	63.90
	NHA – A	ARL 60/70	4.00	1362	2.392	2.498	4.17	12.30	66.07
	NHA – B		4.10	1291	2.370	2.482	4.51	12.95	65.16
	SP – 1		5.00	1424	2.338	2.449	4.53	14.70	69.18
	SP – 2		4.40	1295	2.366	2.482	4.67	13.96	66.53
	MS – 2		4.80	1554	2.340	2.455	4.68	14.52	67.73
Ubhan Shah	NHA – A	NRL 40/50	3.78	1189	2.399	2.536	5.40	12.50	56.80
	NHA – B		4.60	1006	2.375	2.491	4.66	14.05	66.85
	SP – 1		5.50	1176	2.341	2.457	4.72	15.82	70.16
	SP – 2		4.44	1073	2.382	2.497	4.60	13.64	66.27
	MS – 2		4.81	1255	2.364	2.484	4.83	14.47	66.61
	NHA – A	NRL 60/70	3.93	1190	2.394	2.515	4.80	12.80	62.26
	NHA – B		4.65	1198	2.380	2.492	4.50	13.70	67.17
	SP – 1		5.40	1160	2.339	2.451	4.57	15.90	71.26
	SP – 2		4.55	1182	2.376	2.485	4.40	14.00	68.50
	MS – 2		4.70	1375	2.381	2.488	4.30	13.40	67.70

Table 3.24: Results for the Optimum Bitumen Content for Base Course Gradations

Aggregate Type & Gradation		Bitumen Type	OBC (%)	Stability Kg	Gmb	Gmm	Air Voids (%)	VMA (%)	VFA (%)
Margalla	NHA – A	ARL 60/70	3.3	2650	2.402	2.510	4.30	11.38	62.19
	NHA – B		3.7	2905	2.396	2.505	4.35	11.87	63.33
	SP – 2		3.6	2295	2.388	2..504	4.63	12.17	61.93
	DBM		3.9	3496	2.380	2.495	4.80	12.40	61.33
Ubhan Shah	NHA – A	NRL 60/70	3.08	2628	2.423	2.524	4.00	11.25	64.42
	NHA – B		3.25	2617	2.413	2.514	4.02	11.58	65.32
	SP – 2	NRL 40/50	3.88	2310	2.383	2.499	4.64	12.19	61.96
	DBM		3.95	3513	2.382	2.501	4.79	12.42	61.38
	SP – 2	ARL 60/70	3.93	2213	2.385	2.501	4.64	12.21	61.98
	DBM		4.00	3389	2.386	2.506	4.78	12.38	61.36

3.6 Summary

This chapter summarizes the methodology adopted for the research comprising asphalt concrete mix preparation. The mixtures selected for research and the laboratory tests used to determine the properties of the mixtures have been discussed. Aggregate sources of Margalla and Ubhan Shah were used for performance testing in combination with penetration grade binders procured from NRL 40/50, NRL 60/70 and ARL 60/70. Complete laboratory characterization of aggregates and binders was made. Detailed methodology for the preparation of mixtures by both Marshall method and Superpave Gyratory compactor has been discussed including the mixing, compaction and sample preparation. Marshall method was used to determine the optimum bitumen content for the different mixtures. Gyratory compactor and slab compactor were used for sample fabrication.

4 Aggregate Packing Characteristics

Targeting the midline of aggregate gradation envelope does not alone guarantee packing of aggregates. Source and consensus properties of aggregates need to be considered while developing a gradation for asphalt mixtures. This chapter presents a research study to improve the packing characteristic of NHA-Asphalt wearing coarse aggregate gradation using a Bailey Method. Experimental program composed of proposing criteria for limiting values of aggregate gradation with different sizes for Ubhan Shah Quarry. Asphalt mixtures were prepared using NHA-B and Baily improved gradations with same nominal maximum size. Performance tests were conducted on the asphalt mixtures to ascertain the rutting, permanent deformation, fatigue characteristics and asphalt mixture compatibility.

4.1 Introduction

Asphalt mixtures generally composed of asphalt binder, aggregates, sand and filler particles that may include modifiers such as polymers, hydrated lime or phosphoric acid. Performance of asphalt mixture mainly depends on aggregate properties and its composition called as aggregate gradation. Traditional method of gradation design follows maximum density line (MDL), but with some deviations from MDL to fulfill the minimum VMA (Voids in Mineral Aggregate) criteria (Asphalt institute, 2001). By using 0.45 power grading chart, aggregate structure and maximum density line can be controlled with sufficient voids in mineral aggregates to ensure a durable asphalt mixture. The conventional techniques for aggregate blending lacks accommodating the effect of aggregate shape and texture, resulting common pavement failures such as fatigue cracking and rutting.

Adopting the maximum density line in conventional practice sometime does not produce a packed aggregate skeleton, which may be due to particles shapes and texture. In Densest packing, coarse aggregates and fine aggregates combines in such a way that spaces created by coarse aggregates are filled with fine aggregates leaving minimum voids spaces. Also, the densest packing results in reducing the binder content. In Pakistan, aggregate gradations are being designed using a conventional method that targets density line. This practice may lead to erroneous results some time because the shape of particles may affect the distribution of voids. The amount of particular size required to produce a densest aggregate gradation depends upon the packing characteristics of that particle. Packing characteristics commonly depends upon the aggregate shape, texture and strength. Therefore, single methodology of blending the aggregate does not work on all types of aggregates. Also, the conventional gradation does not include aggregate source properties; therefore, targeting midline alone may not guarantee the desired durability of asphalt mixtures. Conventional method of aggregate gradation is a trial and error method, which also depend upon the designer experience and may vary from person to person. To avoid this problem new definition of coarse aggregate and fine aggregate has been developed by Bailey. In this method the concept of packing of aggregate to each particular size has been used.

Arrangement of solids in a specified volume is termed as packing. It is desirable to attain the maximum possible packing of material, while blending the granular materials with binder. For asphalt mixtures, it is critical to obtain the strength from bounded material (Hafeez et al., 2014). For asphalt mixtures, aggregate proportions approximately 95% of the volume of HMA mixture (TRB, 2009). The distribution of aggregate played a critical role in a specific packing. It controls both volumetric and aggregate interlock of a packed structure (Vavrik, et al., 2002). Dense graded as well as gap graded asphalt mixture's strength and stability depends on aggregate contact characteristics and interlocking of aggregates. Both the properties affect the load bearing ability capacity of mixtures (Hafeez, 2014). In pavement engineering, traditional asphalt mixture's design methods aim to design an aggregate structure to achieve densest packing for the purpose of higher strength and reduced amount of binder (Roberts F. L, et al). The aggregate size distribution or overall aggregate gradation is one of the primary characteristics in finding how an overall aggregate particle blend will behave as pavement material.

Almost all important properties of asphalt mixture such as sensitivity to moisture damage, resistance against fatigue cracking, rutting resistance, mixture's stability, and tenderness of mix, stiffness and frictional resistance etc. depend on packing of aggregate gradation (Shen. S, and Yu. H, 2011). Hafeez et al. (2010) stated that the pavement distresses can be controlled by the aggregate gradation or distribution of aggregate sizes (Hafeez et al., 2010). The primary factors that influence the development of aggregate skeleton and resist rutting of asphalt mixtures are the aggregate gradation, shape, texture and angularity. Resistance of asphalt mixtures against rutting mainly depends on aggregate gradation. Stability of asphalt mixtures, resistance to fatigue cracking and stiffness properties mainly depends on aggregate gradations [(Anthony, et al., 2003), (Haritonovs V., et al., 2013), (Vasconcelos K.L., et al., 2011) (Mishra D., Tutumuler E., 2012)].

Bailey defines the coarse and fine aggregate in a new way using the packing theory, which is a more systematic approach to make optimal aggregate blending. Bailey used three controlling factors of gradation named as CA ratio (coarse aggregate ratio), FAc ratio (fine aggregate ratio of fine portion) and FAf ratio (fine aggregate ratio of fine portion). These controlling factors are linked with VMA, Va and behavior of mix while compaction. Bailey method (TRB, 2002) provides a set of tools by utilizing the aggregate size distribution. This method can be used to develop aggregate blends for any type of mix design method. It can be used for Marshall as well as Superpave mixtures. Volumetric properties are controlled by empirical rules provided by this method. The basic purpose of developing this method was to improve the rutting resistance in asphalt mixtures (TRB, 2002). Botasso et al. (2012) concluded that asphalt mixtures can be designed in an adequate way by using Bailey method. Al-Qadi et al. (2013) reported fine graded asphalt mixtures developed by Bailey method proved good in durability and moisture. Goh et al. (2013) found that Bailey method results in significant improvement against rutting and fatigue cracking as compared to conventional Superpave mixtures. Bailey method focus on packing of aggregates and parameters directly linked with VMA, Va and workability. This chapter also discusses an approach to address the packing issue of aggregate blends used in asphalt mixtures for Pakistan. Limits of different controlling factors have been proposed.

4.2 Bailey's Methods of Gradation

Aggregate gradation in an asphalt mixture can be selected by using Bailey method as devised by Robert D. Bailey, which was further modified by Dr. Bill Pine from heritage research, Indiana. Modification in Bailey method now allows the designer to prepare gradation for any type of asphalt mixture with better volumetric control using locally available aggregate. The Bailey procedure is a systematic tool of aggregate blending that emphasis on the importance of aggregate interlock and uniform gradation in an asphaltic mixture. To create aggregate blends, Bailey method uses the relationship of aggregate gradation and volumetric so that desired mixture volumetric and the development of coarse aggregate interlock can be met. Aggregates have a random nature of shape and size; this type of nature does not allow filling the volume completely by any combination of aggregate particles. Whenever aggregate particles packs together, it results in some amount of gap between the particles. Several factors such as type and extent of competitive energy, shape and texture of aggregate particles, gradation and particle strength define packing of aggregate particles.

Combination of coarse and fine aggregate in an aggregate gradation at a given level of compaction defines packing and hence the stiffness of the asphalt mixture. Bailey method defined coarse aggregate as large particles that create voids in a unit volume and fine aggregates as particles filling these void spaces. Fine aggregate portion of the blend can further be subdivided into coarse portion and a fine portion known as course and fine fractions of fine aggregate, respectively. The break points separations for different segments are fixed using a set of control sieves for overall aggregate blend. Baily defined control sieves using a 2-dimensional and 3-dimensional analysis and called them as Primary, Secondary and Tertiary Sieve. A factor of 0.22 based on average shape criteria is known as the particle size ratio, which defines the voids created by each coarse aggregate particle in a blend and filled by a fine aggregate thereto. Primary control sieve (PCS) being a sieve with 0.22 times the Nominal Maximum Aggregate Size (NMAS), followed by secondary (SCS) and tertiary control sieves (TCS) as 0.22 times the primary and secondary sieve, respectively. Coarse portion of coarse aggregate is named as pluggers, while coarse aggregate's fine division is called interceptors. Separation points of these points are known as half sieve (HS). Following relationship may express the mathematical form of different sieves;

$$PCS = NMAS \times 0.22 \tag{4-1}$$

$$SCS = PCS \times 0.22 \tag{4-2}$$

$$TCS = PCS \times 0.22 \tag{4-3}$$

$$HS = PCS \times 0.5 \tag{4-4}$$

Accumulated material on TCS and passing through SCS is called "Coarse Fraction" of the fine portion of the fine aggregate; while material moves through TCS is called "Fine Fraction" of the fine portion of the fine aggregate of overall blend. The purpose of such division is to control size of aggregate within each fraction.

Bailey controls aggregate packing using apparent specific gravity (Gsa), bulk specific gravity (Gsb), rodded unit weight (RUW), and loose unit weight (LUW), which can be determined as per AASHTO standards. These properties help in determining voids variability in the aggregate blend as well as the amount of compaction efforts required to produce a desirable stiffness in the asphalt mixture. Such properties also help in controlling volumetric of asphalt mixtures. Desired level of coarse aggregate interlock in an as asphalt mixture as well as the percentage of voids created by coarse aggregate structure is totally depends on design unit weight. Lower level of coarse aggregate interlock is attained at the loose unit weight of the coarse aggregate. Aggregate interlock at this state is only due to shoveling or dropping of aggregate into a mold without any extra compactive effort employed. Upper level of coarse aggregate interlock is represented by the RUW state. Various researches set guidelines for the selection of design unit weight for the coarse aggregate. One of the guidelines indicates that selected unit weight should be nearest to lose unit weight of coarse aggregate.

Compatibility of mixtures linked with the chosen unit weight, when the chosen unit weight (CUW) is less than the LUW it will be more compactible and vice versa. Coarse aggregate interlock is not gained when CUW is less than the LUW because the finer particle pushing the coarse particles away. That is due to more percentages of fine material as compared to voids created by coarse aggregate interlock. Selection of chosen unit weight near the loose unit weight is an indication that some level of interlocking of coarse aggregate has been achieved, but the skeleton has relatively higher air void. On the other hand, it would be difficult to attain desired level of compaction, if design unit weight (DUW) is closer to the densest value. RUW of aggregate is selected as DUW of fine aggregate. Aggregates in rodded condition represent a relatively good-packing condition.

The requirement of packing depends on the type of aggregate, desired volumetric of the mixtures and desired function of the material. A designer has to select different parameters like voids in coarse aggregate (VCA) accordingly. Bailey method provides a detailed procedure to play with these parameters and define limits to each condition. Volumetric relationship for each coarse aggregate gives voids and can be calculated using the following relationship;

$$Voids = \left[1 - \frac{DUW}{Gsb\ \times Weight\ of\ Water}\right] \times\ Blend\ Percentage \qquad 4\text{-}5$$

Similarly, we can determine the contribution of each unit weight from each individual fine aggregate that will fill the voids spaces in coarse aggregates using the following relationship;

$$Amount\ Contributed = DUW\ \times Blend\%\ \times Total\ Void\ Space \qquad 4\text{-}6$$

Using design unit weight of each individual aggregate designer can determine blend percentages based on total unit weights. Bailey method makes blends by volume, it is desired to express blend into weight instead of blend percentages, because it seems more practical and accurate to blend aggregate source by weight than by volume. By adding contributions from each individual aggregate, we can calculate the initial blend percentages for each type of aggregate in terms of weights from each individual aggregate to the overall aggregate blend unit weight. The initial blend percentages at the end need to be corrected for the amount of fine aggregate contained in the coarse aggregate stockpiles and the amount of coarse aggregate contained in the fine aggregate stockpiles. The adjusted or corrected blend percentage for each individual stockpiles can be determined using the following relationship;

$$Adjusted\ \% = (Initial\ \%) + (FA\ in\ CA\ ) - \left[\frac{Initial\ \%\ *\ Sum\ CA\ in\ FA}{Total\ \%\ of\ CA}\right] \quad \text{4-7}$$

Similarly, blend percentages can be adjusted to account for coarse aggregate that is present in the fine aggregate using the following relationship;

$$Adjusted\ \% = (Initial\ \%) + (CA\ in\ FA\ ) - \left[\frac{Initial\ \%\ *\ Sum\ FA\ in\ CA}{Total\ \%\ of\ FA}\right] \quad \text{4-8}$$

Mineral Fill (MF) can be added to balance the amount of fines in overall aggregates blend. MF addition leads to the adjustments of the one of fine aggregate percentages, so that addition of mineral fill can be reflected. Depending on the requirements, sometime we add mineral filler to balance the amount of fines. The total requirement of mineral filler in the overall aggregate blend can be determined by using the following relationship;

$$\%MF\ Needed = \left[\frac{\%Fines\ Desired - \%Fines\ in\ Blend}{\%Fines\ in\ MF}\right] \quad \text{4-9}$$

Final blend percentages for the selected fine aggregate that is being partially replaced due to addition of mineral filler can be calculated at the end to correct the aggregate gradation. Bailey provides a check for Packing, while determining the requirement of different aggregate fraction by using three different ratios namely; CA ratio, FAc and FAf ratio. These ratio for final blend must be within the Bailey defined limits that can be found from the literature.

This chapter presents the utilization of Bailey method to develop limiting values for different aggregate gradations and investigating the adoption of Bailey method. An aggregate gradation was developed using the Bailey method equivalent to NHA Class-B (asphalt wearing course) that has been considered as a fine conventional aggregate gradation. The effect of improvement of aggregate gradation needs to be considered in the overall asphalt mixture performance.

4.3 Research Objectives

Followings were the main objectives of this research study:

- To investigate the possible relationship between the voids in coarse aggregate and maximum achievable packing (density) using the local aggregates.
- To establish limiting values of voids in coarse aggregate against achievable packing (density) at different nominal maximum aggregate sizes using the Bailey Method.
- To investigate the effect of packing characteristic on asphalt mixture performance
- To determine the optimum proportions of aggregate to be used in general framework using Bailey method and to investigate the effects of these optimum proportions on the performance of asphalt mixtures.

4.4 Research Methodology

The overall research approach has been illustrated in Figure 4.1. A Two phase study was planned to accomplish the study objectives: Phase-I comprises of selecting the aggregate source, determining the engineering properties of aggregate and defining limits of Gsb and VCA. Phase-II comprises the selection of two aggregate gradations with same NMAS, preparing the asphalt mixtures and conducting the performance tests.

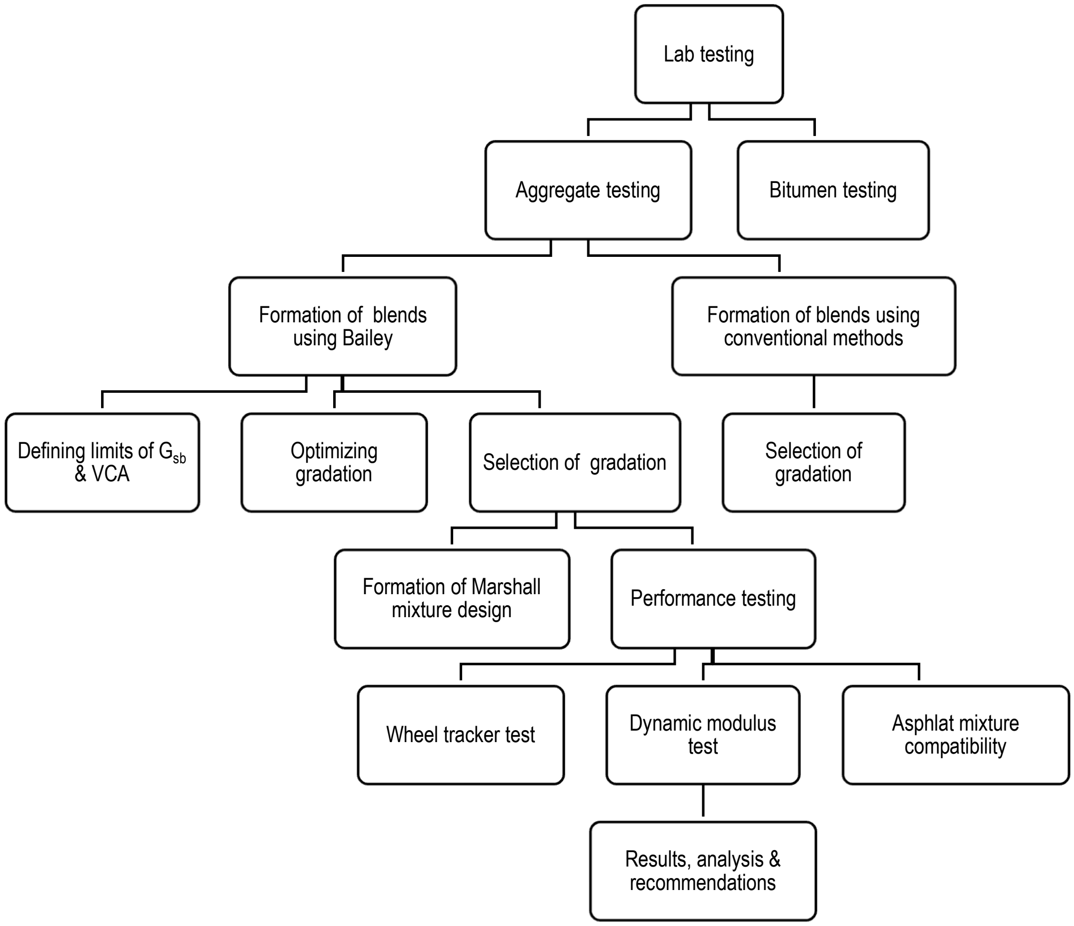


Figure 4.1: Scope of Research Study

Phase-I comprises of three sub-phases to obtain the information on the effect of Bailey parameters; to investigate the relationship between CA LUW, Gsb and VCA at each NMAS, to conduct an optimization task by searching the appropriate combination of the Bailey parameters to obtain a mixture that performs well for pavements and establishing the limiting values of Gsb and VCA against each NMAS.

Phase-II mainly involved preparing the asphalt mixtures and compared the mixture performances between gradation blends that were designed both with and without the Bailey method. This was to ascertain whether Bailey method could significantly improve the job mix gradation. Final aggregate gradation prepared using the Bailey method fulfill defined ranges of different ratios and the blend was within the envelope recommended by National Highway Authority of Pakistan.

4.5 Materials

Single aggregate source and asphalt binder source has been selected for this section of the study as discussed in this chapter. Properties of materials were determined and reported in the proceeding sections.

4.5.1 Aggregate

The aggregate used in the study was obtained from Ubhan Shah Quarry near Khairpur in the southern part of Pakistan. Table 4.1 summarizes the conventional properties of aggregates. Tests on aggregates were conducted in accordance with ASTM and BS specifications.

Table 4.1: Engineering Properties of Tested Aggregates

Test	Standard	Value (%)	Specification limits
Fractured Particles (Two faces)	ASTM 5821	100	90%(min)
Flakiness & Elongation	BS 812.108	12, 16	25% (max)
Aggregate Impact Value	ASTM D 5874	14	30% (max)
Water Absorption	ASTM C 127	0.85	2% (max)
Deleterious Materials	ASTM C 142	0.94	1% (max)
Soundness (Coarse)	ASTM C 88	0.18	8% (max)
Soundness (Fine)	ASTM C 88	2.48	8% (max)
Los Angeles Abrasion	ASTM C 131	22	30% (max)
Un compacted Voids (Fine)	ASTM C 1252	40	45% (min)
Sand Equivalent	ASTM D 2419	81	50% (min)
Plasticity Index	ASTM D 4318	Non-Plastic	6 % (max)

It may be noted from Table 4.1 that percentage absorption, soundness and loss Angeles abrasion value of different fractions of aggregates are within the prescribed specification limits, which is an indicator of relatively good quality materials. Table 4.2 reports properties of both coarse aggregate and fine fractions.

Table 4.2: Aggregate Properties for Voids Determination

Properties	CA#1	CA#2	CA#3	FA#1
LUW	1368	1368	1347	-
RUW	1487	1529	1471	1790
Bulk specific gravity	2.681	2.678	2.682	2.584
Apparent specific gravity	2.704	2.704	2.711	2.694
% Absorption	0.317	0.357	0.403	1.586

The properties of aggregate as reported in Table 4.2 has been used in the development of Bailey's improved gradation.

4.5.2 Asphalt Binder

Neat (straight run) asphalt binder of 60/70 penetration grade (equivalent to PG 64-22) for this study was procured from National Refinery Limited, Pakistan. The basic properties of asphalt binder were determined using conventional tests and results of asphalt binder conventional testing have been reported in Table 4.3.

Table 4.3: Physical Properties of Asphalt Binder

Test	Standard	Value
Penetration at 25 ºC, 100 g (0.1 mm)	ASTM D5	75
Softening point (ºC)	ASTM D36	46
Specific gravity at 25 ºC	ASTM D70	1.03
Ductility at 25ºC (cm)	ASTM D113	100+
Loss on heating at 163 ºC, 5 hrs, percent wt. (Max.)	ASTM D1754	0.2
Flash Point, Cleveland open cup, ºC (Min.)	ASTM D92	232

Frequency sweep test was also run on RTFO aged asphalt binder using a dynamic shear rheometer to ascertain time and temperature dependency. A frequency range of 0.1 to 100 Hz and a temperature range of 40 to 76 °C (40, 52, 64, and 76 °C) was chosen for the development of master curve. The stress to strain ratio and their corresponding phase difference in an oscillatory motion represents complex shear modulus and phase angle, respectively [16]. A master curve developed using a time and temperature superposition principle (TTS) allows us to ascertain the viscoelastic response of asphalt binder within a frequency domain. Master curve demonstrates asphalt binder rheological behavior over a wide range of temperature and frequency. Figure 4.2 shows the master curve plotted from DSR data between the complex shear modules and reduced frequency using sigmoidal curve fit parameters.

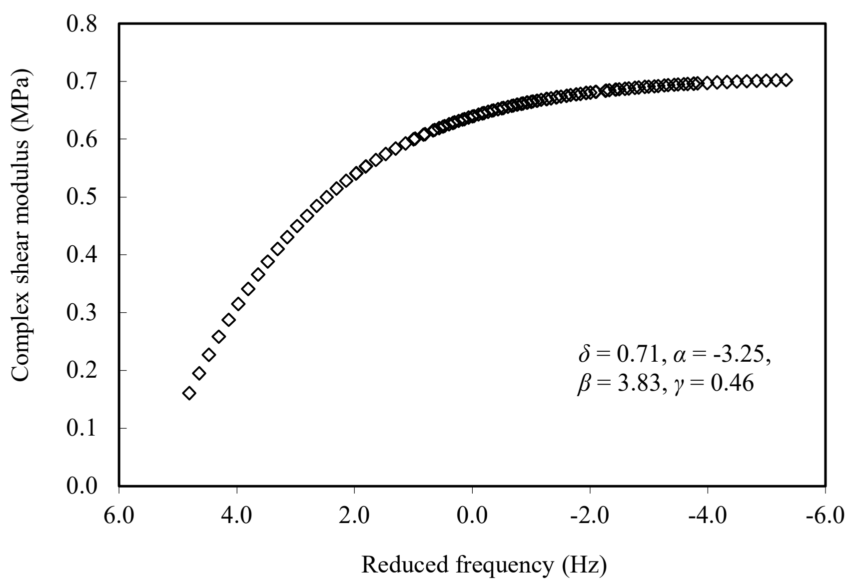


Figure 4.2: Developed Master Curve Using Time and Temperature Superposition Principle

Visco-elastic behavior of asphalt binder is dependent on the pavement temperatures and traffic loading, which has been predicted by developing the master curve from a frequency sweep test. Also, shift factors indicate the temperature sensitivity of asphalt binder.

4.6 Packing Characteristics

4.6.1 Relationship Between VCA & Gsb:

Bulk specific gravity (Gsb) of aggregate is an indicator of aggregate probable behavior in the asphalt mixture. Keeping the bulk specific gravity on higher side means coarser aggregate skeleton is in densest form. The volumetric properties of asphalt mixtures depend on Gsb. As Gsb increases, VMA and VFA also increases and vice versa. On the other hand, lowering the voids in coarse aggregates (VCA) values the packing of mixture improves and vice versa. One of the objectives of the present study was to optimize the Gsb and the VCA limits for the selected aggregate quarry.

This study found a reasonable relationship between the voids in coarse aggregate (VCA) and maximum achievable packing density (Gsb) using the local aggregates. The method of controlling the aggregate gradation volumetric parameters using voids in coarse aggregate and bulk specific gravity as developed by Bailey works fairly well on the local aggregates. The trends of relationships among the volumetric parameters in aggregate gradations with different nominal maximum aggregate sizes are in line with past studies, as reported in this paper. The criteria of optimizing the VCA and Gsb along with their boundary limits works fairly well and it minimizes the chances of segregation during the mass production.

The defined Bailey limits of different ratios can be adopted for developing the target aggregate gradation in Pakistan. The asphalt mixtures prepared within the defined limits perform better than the conventional asphalt mixtures. Trials were made at 95%, 100 % and 105 % as an initial value of LUW against NMAS of 12.5 mm, and possible relationships were found keeping the ratios within defined limits. Table 4.4 report optimized Bailey parameters at 100% LUW for an aggregate gradation with NMAS 12.5 mm. Several trials were made to obtain the acceptable range of packing parameters.

Table 4.4: Trials at 100% LUW for NMAS 12.5 mm

LUW	Gsb	VCA	CA	FA_c	FA_f
100	2.622	49.10	0.65	0.38	0.47
	2.622	49.10	0.63	0.38	0.47
	2.623	49.10	0.62	0.38	0.47
	2.623	49.10	0.61	0.38	0.47
	2.623	49.10	0.59	0.38	0.47
	2.623	49.10	0.58	0.38	0.47
	2.624	49.10	0.57	0.38	0.47
	2.624	49.10	0.55	0.38	0.47
	2.624	49.10	0.54	0.38	0.47
	2.624	49.10	0.53	0.38	0.48
	2.625	49.10	0.52	0.38	0.48
	2.625	49.10	0.51	0.39	0.48

Similarly, value of different parameters was obtained at 95 % and 105 % LUW, where first lowest value of VCA obtained against the mean Gsb value was selected. Table 4.5

reports the selected values of these parameters at 95, 100, and 105% LUW. It may be noted that Gsb value increases with an increase in %LUW. In contrast, VCA values decrease with an increase in %LUW.

Table 4.5: Optimized Bailey Parameters at NMAS 12.5 mm

CLUW	Gsb	VCA	CA	FA_c	FA_f
Ranges			(0.50-0.65)	(0.35-0.5)	(0.35-0.5)
95	2.622	51.6	0.58	0.38	0.47
100	2.623	49.1	0.58	0.38	0.47
105	2.627	46.5	0.51	0.39	0.48

Table 4.6 reports values for different NMAS as obtained under the similar procedure. It may be noted from Table 4.6 that CA ratios for the presented data are within the defined range for each NMAS.

Table 4.6: Optimized Bailey Parameters at Various NMAS

NMAS	LUW	Gsb	VCA	CA	FA_c	FA_f
25 mm	Ranges			(0.70-0.85)	(0.35-0.5)	(0.35-0.5)
	95	2.641	51.674	0.73	0.4383	0.4839
	100	2.643	49.147	0.77	0.4343	0.4844
	105	2.646	46.587	0.73	0.4335	0.485
19.5 mm	Ranges			(0.60-0.75)	(0.35-0.5)	(0.35-0.5)
	95	2.643	50.9	0.61	0.42	0.46
	100	2.646	48.4	0.66	0.42	0.46
	105	2.648	45.8	0.64	0.41	0.46
12.5 mm	Ranges			(0.50-0.65)	(0.35-0.5)	(0.35-0.5)
	95	2.622	51.6	0.58	0.38	0.47
	100	2.623	49.1	0.58	0.38	0.47
	105	2.627	46.5	0.51	0.39	0.48
9.5 mm	Ranges			(0.40-0.55)	(0.3-0.5)	(0.35-0.5)
	95	2.632	52.3	0.47	0.38	0.47
	100	2.635	49.8	0.44	0.38	0.47
	105	2.638	47.3	0.42	0.38	0.47

Similarly, FA_c ratio lies within range of 0.40, which is preferable, but FA_f ratio is at higher side. FA_c and FA_f ratio controls the fine portion. Higher values indicate denser configuration, which results in decrease of VMA. With an increase in % LUW value bulk specific gravity increases and VCA decreases. Similar trends were observed for aggregate gradation with different NMAS. Figure 4.3 shows a relationship between LUW, Gsb and VCA.

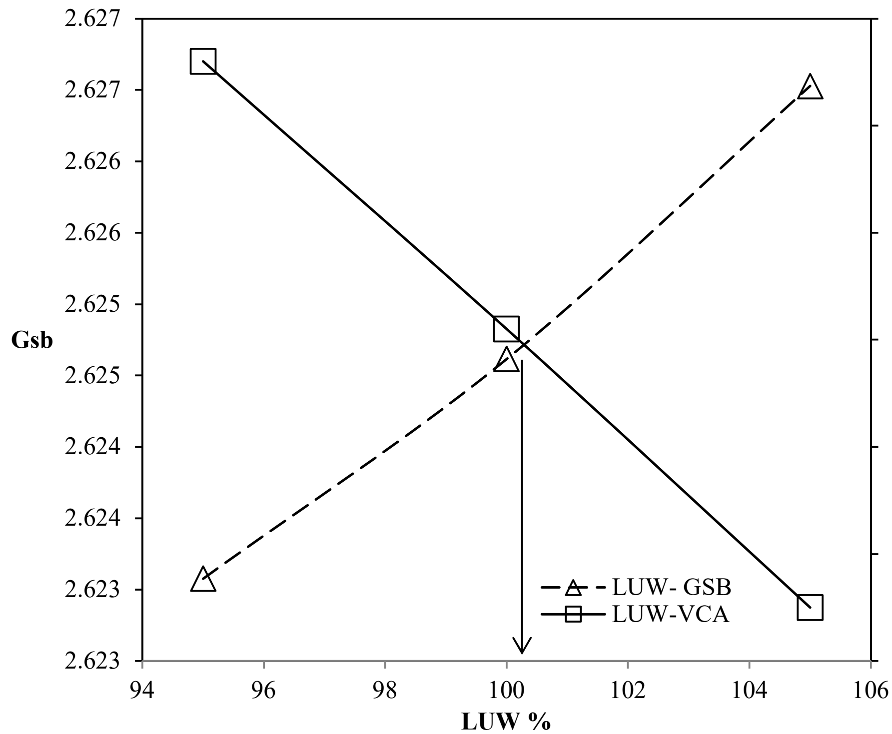


Figure 4.3: Optimization of Gradation using Relationship of LUW, VCA and Gsb

It may be noted from the Figure 4.3 that selected point would be the intersection of two plots which has been drawn among Gsb, VCA, and LUW. It may be noted that value of VCA decreases with an increase in LUW value, while VCA percentage decreases with an increase in % LUW. The point of intersection of VCA and Gsb was taken as optimized point for a given range of loose unit weight. The intercept of this point yield optimized value of Gsb, VCA and LUW. Similarly, optimized values of these parameters were obtained against each aggregate gradation. Upper and lower limits of packing factors were defined accordingly. This practice of defining the limits of packing factor was different than what have been adopted in the conventional methods. By adopting such practices, one can control variation in the volumetric properties of aggregate gradation. Table 4.7 provides limits of Gsb and VCA as set using the proposed criteria.

Table 4.7: Limits of Gsb and VCA of Various NMAS for Coarse Graded Mix

NMPS	Gsb		VCA	
Mm	Lower limit	Upper limit	Lower limit	Upper limit
37.5	2.651	2.66	46.55	51.71
25	2.641	2.646	46.58	51.7
19.5	2.643	2.649	45.8	52
12.5	2.621	2.627	46.5	51.7
9.5	2.632	2.638	47.3	52.3

Table 4.7 is representing the minimum and maximum values of Gsb that we can obtain using Bailey Method on Ubhan Shah Quarry keeping all the ratios within the Bailey defined limits.

4.7 Asphalt Mixture Design

An aggregate gradation with NMAS of 12.5mm was selected from NHA specification for asphalt wearing course. A gradation with same NMAS was prepared using Bailey method to investigate the effect of improvement. Targeted gradation from Bailey method was the optimized gradation. Various trials were run to meet the optimized parameters as discussed earlier in the previous sections. Aggregate gradation was chosen at an average value of loose unit weight, CA ratio, FA_c ratio and FA_f ratio. Both the gradations and their envelopes have been reported in Table 4.8.

Table 4.8: Aggregate Gradations Used in this Study

Sieve size	Class B midline	Bailey	Class -B lower	Class B upper
(mm)	(Passing %)	(Passing %)	(Passing %)	(Passing %)
19	100	100	100	100
12.5	82.5	77.4	90	75
9.5	70	64.1	80	60
4.75	50	39.9	60	40
2.36	30	27.7	40	20
1.18	19.5	16.6	27	12
0.6	13.5	10.4	19	8
0.3	10	8.1	15	5
0.15	7.5	5.7	11	4
0.075	5.5	4.1	8	3

Midline aggregate gradation from NHA and gradation developed using a Bailey method were selected for asphalt mixture preparations. Two asphalt mixtures were prepared with both the aggregate gradation and 60/70 penetration grade asphalt binder.

Conventional Marshall Method of mix design was utilized for optimization of asphalt contents as per ASTM D 1559. Table 4.9 reports selected aggregate gradations. Asphalt mixture's specimen was compacted using a Marshall compactor with 75 blows on each side. Samples were mixed at 153°C and compacted at 145°C. Optimum asphalt contents were calculated by taking an average of maximum unit weight, maximum stability and 4% air voids. The optimum asphalt contents were checked to satisfy the volumetrics of the specimens within the specified limits. The optimum value of asphalt content for both the conventional and Bailey gradation mixtures were 4.65% and 4.61%, respectively. Summary of Marshall Mix design parameters has been reported in Table 4.9.

Table 4.9: Marshall Mix Design Parameters

Design Parameters	Bailey mix	Conventional mix
Gsb	2.646	2.631
AC (%)	4.61	4.65
Stability (kg)	1009	1090
Gmb	2.4	2.398
Va	4.2	4
VMA	13.9	13.1
VFA	69.89	69.41

It may be noted from Table 4.9 that volumetric parameter of asphalt mixture prepared with Bailey's improved gradation were better as compared to the conventional mixture.

4.8 Performance Testing

The purpose of conducting the performance tests was to ascertain asphalt mixture stiffness and fatigue response under various types of loading and temperature conditions, with a view to capturing the effect of bailey method in improving the aggregate gradation and thus the asphalt mixture performance. Prior to performance testing, asphalt mixture's compactibility test was run to ascertain the compaction characteristics.

4.8.1 Asphalt Mixture Compactibility Test

Asphalt mixture compactibility test aims to measure the tendency of asphalt mixture to change its density with the application of compactive efforts. In other words, it exhibits the amount of energy stored during the compaction process. Asphalt mixtures prepared at 153±5°C were compacted using a SHRP Gyratory Compactor (SGC). Density and number of gyrations were recorded corresponding to at N=8 gyrations and at densities equivalent to 92%, & 98% of Gmm. Number of trials were run to target gyrations against 92% of Gmm as well as against N=98% of Gmm. Comparisons of both these mixtures were made and densification curves were plotted using number of gyrations as well as density data. Two types of indexes can be calculated to predict asphalt mixtures compactibility.

4.8.2 Compaction Densification Index (CDI)

The Compaction Densification Index (CDI) is termed as the area from the 8th gyration to 92% of Gmm in the densification curve as shown in Figure 4.4.

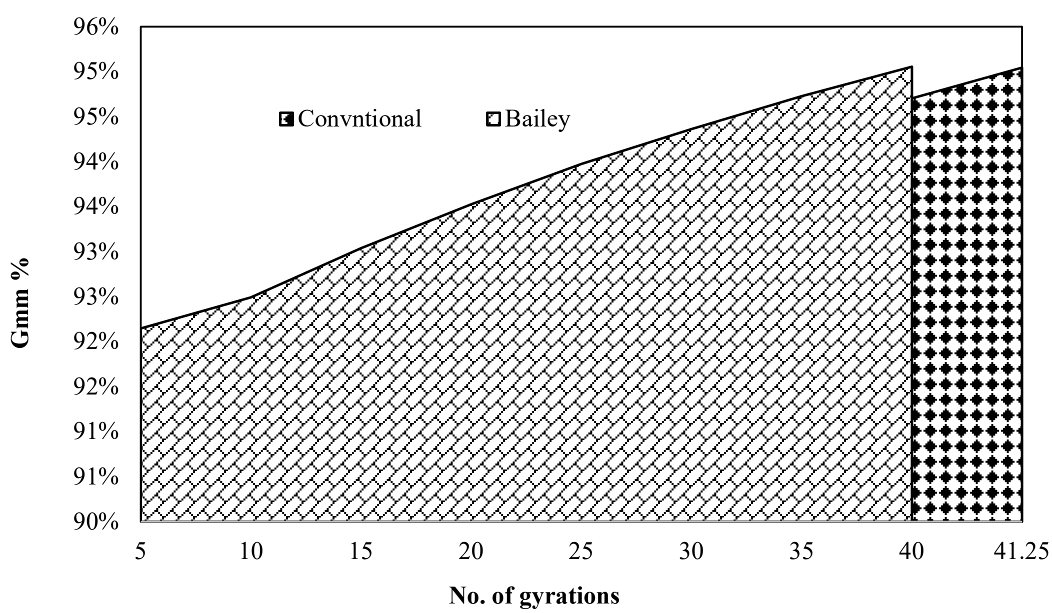


Figure 4.4: Compaction Densification Index of Conventional and Bailey Mix

Mathematically, CDI is a difference of number of gyration at 92% of Gmm and eighth gyration. This index reflects total amount of energy that may be applied by the roller to compact the mixture. The initial compaction made by the paver itself can be represented by 8th gyrations of SGC, while 92% of Gmm represents the density of asphalt mixture at the end of finished rolling. Compaction densification index represents the asphalt mixtures compactibility. Higher value of CDI indicates that the mixture is difficult to compact and a greater number of passes of roller are required to obtain the desired density. A low value CDI is an indication of tenderness of mixtures that must be avoided. The obtained CDI for Bailey and conventional mixtures were 33.5 and 28.5, respectively. Higher index means more compaction energy required to compact the asphalt mixture in case of Bailey's mixture.

4.8.3 Traffic Densification Index (TDI)

Traffic Densification Index (TDI) may be defined as the area from 92%, through 96%, to 98% of Gmm in the densification curve as shown in Figure 4.5.

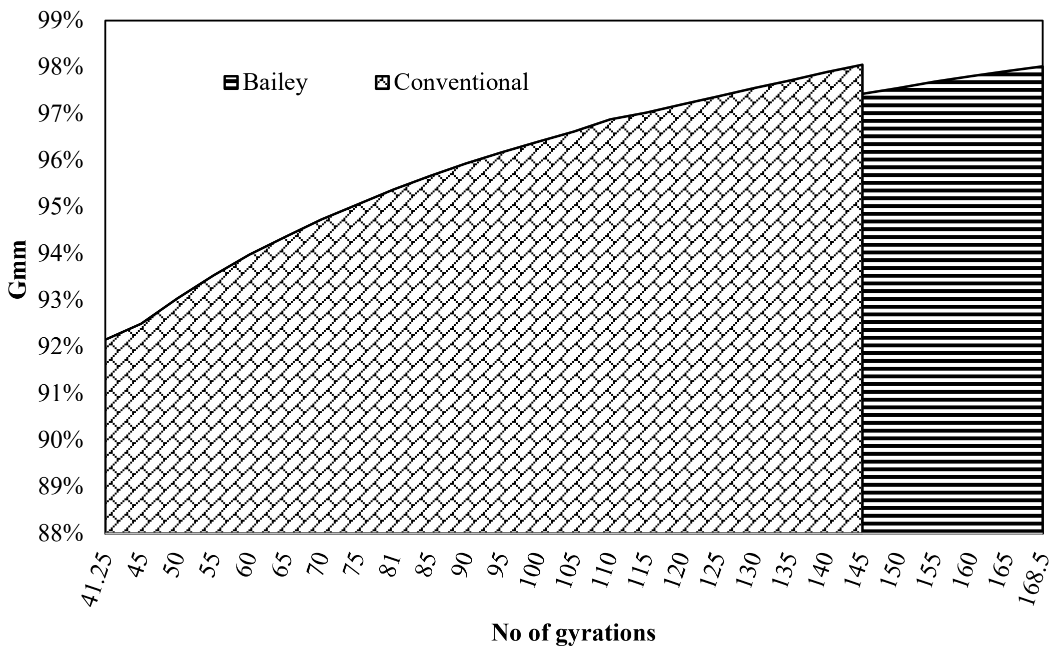


Figure 4.5: Traffic Densification Index of Conventional and Bailey Mix

Mathematically, it is the difference of gyrations at 98% and 92%. When the compaction is final and traffic is open (approx. 92 % of Gmm), pavement comes under densification due to traffic loading. Most road agencies have mixture design requirements of compaction of mixture to be up to 96 % to 97 % of Gmm (3% to 4 % air voids) this simulate the density that attains at early life of pavement due to traffic loading. The 98% of Gmm is considered the critical density, at which the mixture is approaching the plastic failure zone. Higher value of TDI is the indication that mixture has better resistance to bear traffic loading and pavement can serve for long term. The obtained values of TDI for Bailey and Conventional mixtures are 127 and 105.5, respectively.

It may be noted from Figure 4.4 & 4.5 that asphalt mixture with same NMAS has different compactibility indices, which are mainly due to the method of development of aggregate gradations. Asphalt mixture prepared using the Bailey method yield higher index. The above result is in favor of Bailey mix, it is economical as well as durable as compare to conventional mixtures. SGC data is helpful to determine the stability of mixtures and screen out the mix at early stages.

4.8.4 Wheel Tracker Test

Cooper Wheel Tracker (WT) as shown in Figure 4.6 was utilized to determine the rut resistance of asphalt mixtures. This device measures the effects of traffic loading and environment loading by quantifying relative percentage reduction in thickness of the specimen in the wheel path. Wheel tracking device applies wheel load of range 720 N on a slab, temperature range 25 to 70 °C. A loaded wheel (700 ± 20 N) tracked with simple harmonic motion through a distance of 305 mm on specimens under specified conditions of speed (53 passes per minute) at specified temperatures. Development of the rut was monitored with LVDT and the rut depth was quantified as rut resistance of mixtures at the end of the test.

Figure 4.6: Cooper Wheel Tracker

Slab specimens from both the asphalt mixtures were tested under the WT machine at 40 and 50°C, at an air void of 5.5±0.5 and for 10,000 number of passes. Two replicates tested at each temperature. Wheel tracker test results at these temperatures are shown in Figures 4.7 and 4.8. In Wheel Tracker test, asphalt mixture with conventional aggregate gradation showed almost 78% more rut value, rut rate (slope coefficient), and intercept coefficient at 50°C.

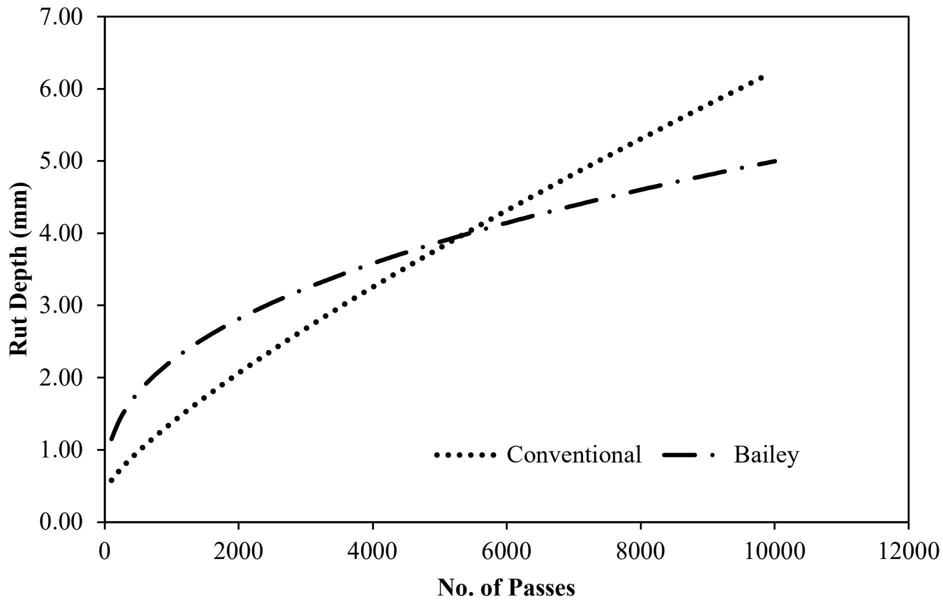


Figure 4.7: Comparison of Conventional and Bailey at 40ºC

A fair comparison has been present in Figure 4.7 and 4.8 at 40 and 50 °C. It may be noted from Figures 4.7 and 4.8 that asphalt mixture prepared with Bailey aggregate gradation showed better resistance to rut as compared to conventional gradation mixture.

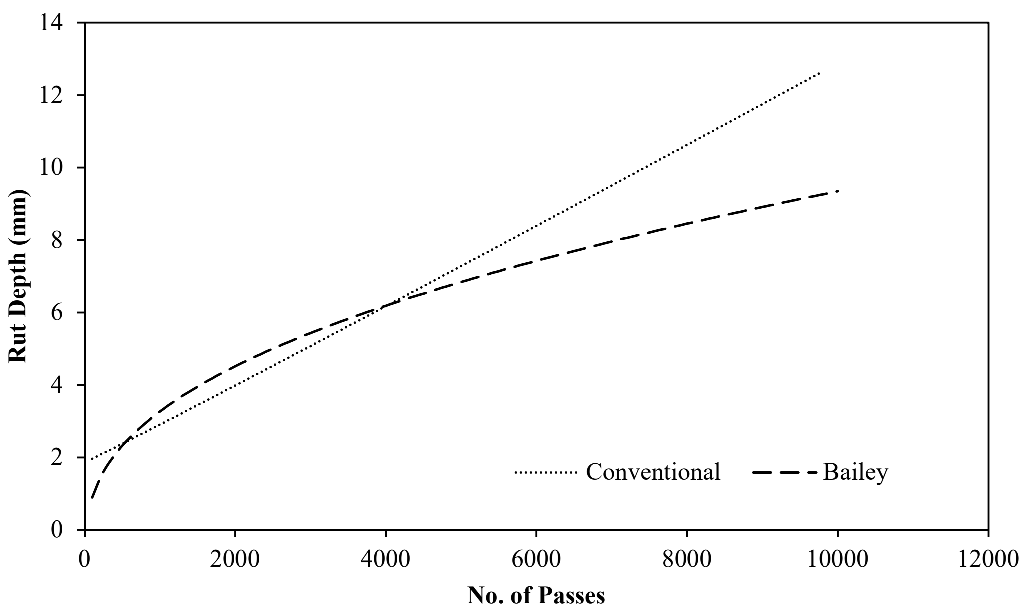


Figure 4.8: Comparison of Conventional and Bailey at 50ºC.

Figure 4.9 shows the overall summary of WT test results. It may be noted from Figure 4.9 that asphalt mixture prepared with NHA conventional aggregate gradation for wearing course yield 13.2 mm rut at 50 °C, which is above the threshold value of 12.5mm.

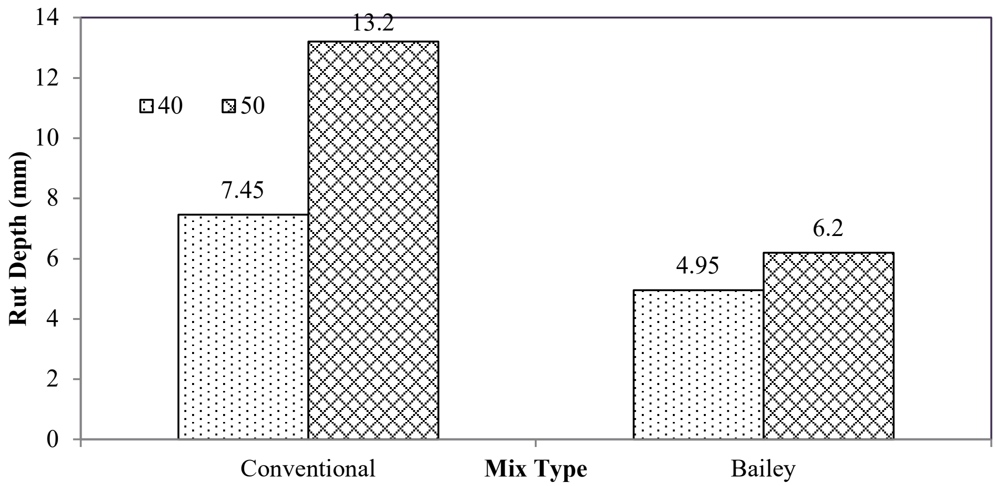


Figure 4.9: Rut depth Comparison of Conventional and Bailey

Thus, we can conclude that Bailey mix is far better as compared to conventional mixtures. The WT results are in line with the asphalt mixture compactibility trends.

4.8.5 Dynamic Modulus Test

Dynamic modulus (E*) is a complex number used to relate the stress to strain relationship. This number can be obtained by applying a sinusoidal loading in a frequency domain that the material exhibits on a linear viscoelastic behavior. Amplitude of the sinusoidal stress when divided by the sinusoidal strain at any given time and given load frequency, a steady state response results is termed as complex modulus. Phase Angle is the angle subtended by sinusoidal applied stress and the resulting strain in a controlled Stress-Test. The phase angle values range from zero (for pure elastic materials) to 90° (for pure viscous materials). A frequency sweep test was run to ascertain the dynamic modulus value in a range of frequency domain in accordance with AASHTO TP 62. Master curves were developed using the dynamic modulus data. Time and temperature superposition principle is the basis for the development of master curves and the shift factors to predict the temperature sensitivity of asphalt mixtures. The Arrhenius and log-linear equations have been utilized for asphalt mixtures. A number of models have been suggested by different research studies to obtain the best fit out of the master curves.

Polynomial fitting functions were utilized for the development of master curve at high temperature. However; a single polynomial model is not suitable for fitting the complete master curve because it swings at low and high temperatures, resulting in irrational modulus prediction when utilized to extrapolate outside the range of data. To overcome this problem presented by the polynomial model, research put forward at the University of Maryland represents that the modulus master curve for HMA mixtures can be expressed by a sigmoidal function defined by the following relationship. Master curves fitting with sigmoidal functions were adopted in this research study to the measured compressive dynamic (complex) modulus test data. The shifting could be done by solving the shift factors simultaneously with the coefficients of the sigmoidal function. Following relationship has been used to develop master curves.

$$Log(|E^*|) = \delta + \frac{\alpha}{1 + e^{\beta} + \gamma \log{(t_r)}} \quad \text{4-10}$$

Where

$\lvert E^*\rvert$	= dynamic modulus
t_r	= time of loading at the reference temperature (reduced time)
δ	= minimum modulus value
$\delta + \alpha$	= maximum modulus value
β, γ	= parameters describing the shape of the sigmoidal function

Parameter γ affects the steepness of the function (rate of change between minimum and maximum) and β, the horizontal position of the turning point. Parameters δ and α depends on aggregate gradation, binder amount and air void content. Parameters β and γ, on the other hand, rely on the characteristics of the asphalt binder and the magnitude of δ and α. Test data at different temperature was shifted to a reference temperature in order to develop a single smooth master curve representing the effects of all temperatures in a frequency domain. Shift factor is the shift desired at each temperature a (T), which has same value for a given temperature.

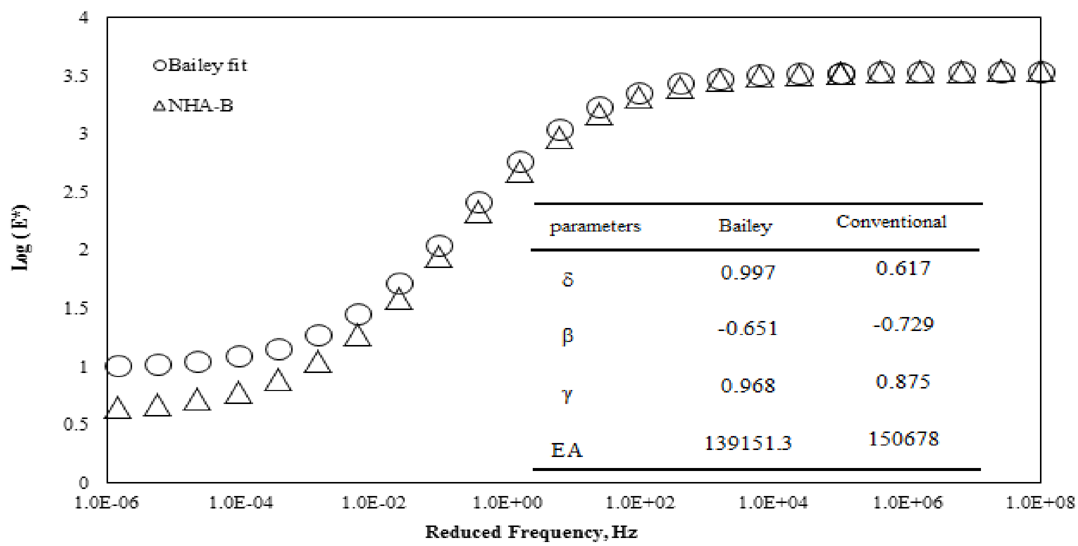


Figure 4.10: Comparison of Master Curve Developed Using the Dynamic Modulus Test Data

Asphalt mixture's specimens of 150mm diameter and 170mm height were prepared using a gyratory compactor. The specimens were extracted using a core cutter machine and trimmed to a final dimension of 100mm diameter and 150mm height. The recorded air voids in the fabricated specimens were in a range of 5.5±0.5%. The fabricated specimens in replicates were tested at 4.4, 21.1, 37.7, and 54.4 °C and at a frequency of 25, 10, 5, 1, 0.5, and 0.1 Hz. A Sinusoidal loading wave with a specific stress level was applied under a controlled temperature environment and the values of dynamic modulus were obtained at the termination of the test. When a material is under sinusoidal loading, the ratios of absolute value of maximum peak stresses to the maximum strains for a given materials is called dynamic modulus. A master curve using the dynamic modulus data was constructed at a reference temperature of 21.1°C as shown in Figure 4.10:

It may be noted from Figure 4.10 that asphalt mixture with Bailey gradation showed higher values of E* in relatively low frequency region. Mixture with Bailey gradation offer more resistance to permanent deformation as compared to conventional mixtures. This confirms the trends obtained in other tests performed in this study.

A comparison of E* results between both the asphalt mixtures has also been presented in Figure 4.11 to ascertain the effect of temperature and frequency levels on complex modulus. Average values of E* have been used at all frequency levels and temperatures to predict their influence.

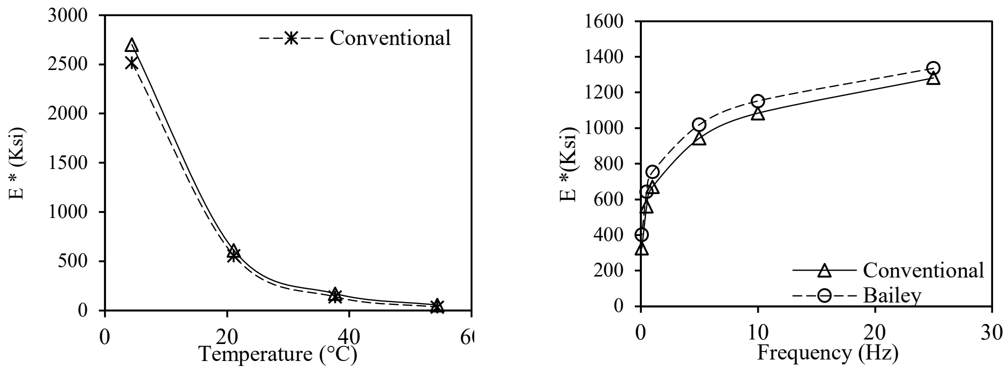


Figure 4.11: Comparison of Dynamic Modulus Values at Different Temperatures and Frequency Levels.

It may be noted from Figure 4.11 that the mean value of E * in case of asphalt mixture with Bailey gradation showed more stiffness than conventional mixture, irrespective of test temperature or the frequency level. The results shown in Figure 4.11 are in line with other performance test results in the present study. Two way ANOVA analyses were also performed on the dynamic modulus test data of Bailey and conventional asphalt mixture considering the entire variable range and reported in Table 4.10 and 4.11, respectively.

Table 4.10: ANOVA Analysis of Bailey Mixture

Source	Df	SS	MS	F	P
Temp	3	27452372	9150791	83.1	0
Freq	5	2408720	481744	4.38	0.012
Error	15	1651570	110105		
Total	23	31512662			
S=331.8	R-Square=94.76 %		R-Square (Adj)=91.96 %		

Table 4.11: ANOVA Analysis of Conventional Mixture

Source	Df	SS	MS	F	P
Temp	3	24254934	8084978	56.44	0
Freq	5	2532825	506565	3.54	0.026
Error	15	2418821	143255		
Total	23	28936579			
S=378.5	R-Square=92.57 %		R-Square (Adj)=88.61 %		

Statistical analysis revealed that p value is less than 0.05 for both the mixtures. A significant relationship is therefore existing among E *, temperature and frequency. The p value is defined as the probability, under the assumption of no effect or no difference (the null hypothesis), of obtaining a result equal to or more extreme than what was actually observed. Whereas p value for frequency is higher than in temperature case (probability that Frequency will not affect E * is high as compared to temperature), which means frequency is relatively less significant than temperature effect.

The degree of determinacy R^2 among the variable were observed to be 94.76% and 92.57 % indicating strong relationship. F-value is essentially the ratio of the variation among groups to the variation within groups. F-value of 83.1 and 56.44 as shown in Table 4.10 & 4.11were recorded for Bailey and conventional mixture, respectively. A relatively large F-value suggests that the variation among groups is largely caused by a given variable or experimental manipulation (in our example, the temperature and Frequency), rather than chances of variation.

4.8.6 Plastic Deformation (Creep Behavior)

The resistance to permanent deformation or the creep behavior of asphalt mixtures is one of the important parameters that determine its ability to resist traffic loading especially in hot climatic areas. Uniaxial repeated load permanent deformation (URLPD) or the creep test has commonly been used in various research studies to ascertain laboratory based rutting resistance of asphalt mixtures at relatively higher temperatures. Different testing procedures have been used to link with the mechanistic-empirical approaches for this purpose. Most of the research studies use linear viscoelastic theory for the prediction of asphalt mixtures permanent deformation characteristics under different loading and temperature conditions.

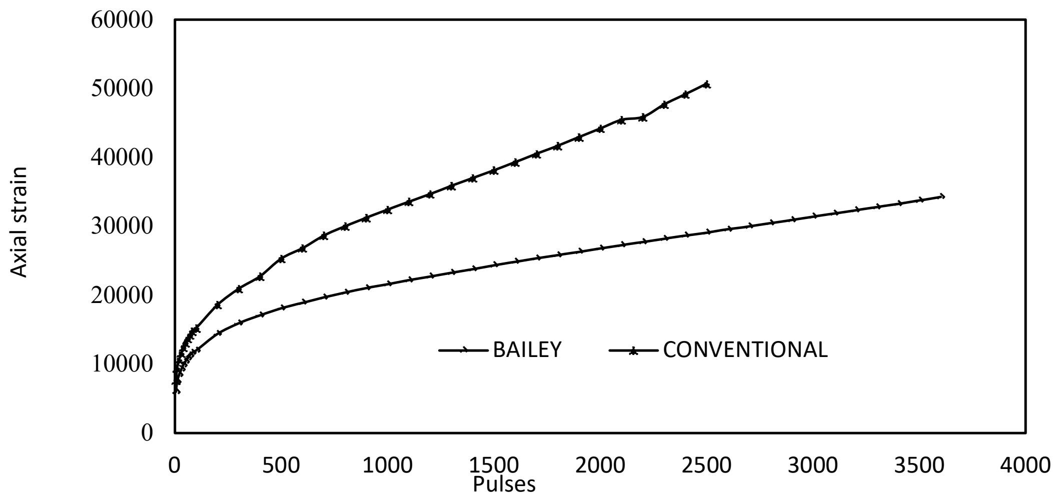


Figure 4.12: Comparison of Axial Strain Versus Pulses for Bailey and Conventional Gradation

Specimens of 100mm diameter and 74mm thickness were fabricated at an air void of 5.5±0.5 % and conditioned for at least two hours before conducting the test. Specimens of both the asphalt mixtures were tested for 3600 load pulses in replicates at 25, 40, and 55°C and at stress level of 500 kPa. A square loading pulse with 0.1 second loading and 0.9 second unloading period at a stress level of 500 kPa was applied in URLPD test on asphalt samples. Strains development in the samples as a result of loading pulse was recorded using two linear variable differential transformers. A progression of permanent deformation curve against load cycle can be developed at the termination of each test. A typical plot showing a comparison of development of axial strain of both the asphalt mixtures under the application of load pulse has been presented in Figure 4.12

It may be noted from Figure 4.12 that asphalt mixture with Bailey method showed less accumulation of axial strain as compared to conventional mix. This observation is again in line with previous result as presented in this study.

4.8.7 Fatigue Behavior

Asphalt pavements exhibits fatigue associated distress under repetition of loading especially at intermediate and low temperatures. Several approaches have been used to determine the fatigue behavior of asphaltic mixture. Adequately designed pavement has very low chance of fatigue cracking because of low strains development. Fatigue depends on pavement structure, age of the pavement and construction materials used in pavement. Characterization of asphalt pavement material using the dynamic modulus is a common approach. Model of dynamic modulus of mix design can be used to predict fatigue cracking. Dynamic modulus contains storage modulus and loss modulus. Loss modulus $[E^{*}]\sin\emptyset$ represents the energy that is dissipated / load. Loss modulus manifests the fatigue characteristics of asphalt mixture.

Asphalt mixture's specimens were conditioned and tested at 5°C in a temperature controlled chamber and tested in replicates at a strain level of 800 microstrains, frequency of 5 Hz in Four Point Beam Fatigue Test (FPBFT) to generate fatigue curves. A reduction of 50% in the initial stiffness was defined as the failure criterion. The corresponding number of repetitions was defined as fatigue life (Nf), which is believed to simulate the field fatigue behavior of asphalt mixtures under vehicular loading. Figure 4.13 presents typical fatigue curves drawn at 5°C and 5Hz.

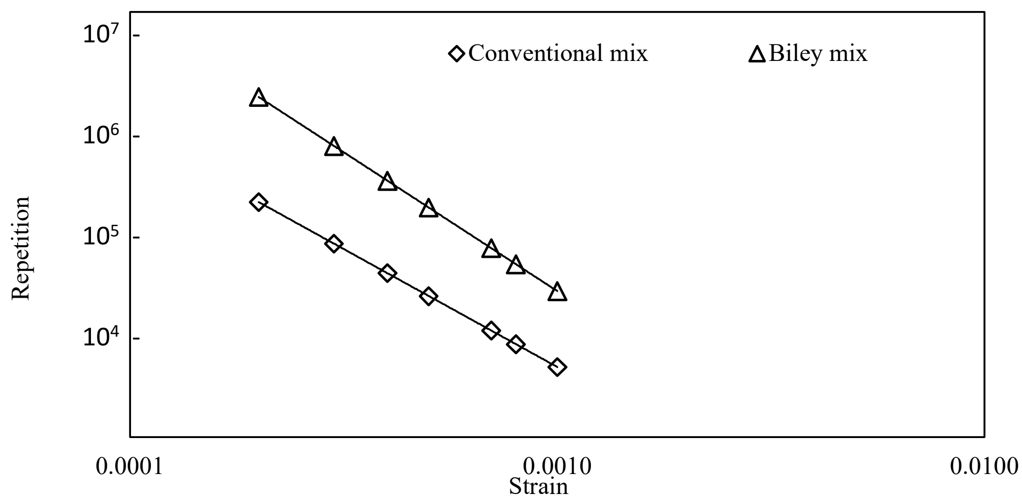


Figure 4.13: Typical Fatigue Curves Drawn at 5°C and 5Hz.

It may be noted from Figure 4.13 that the fatigue life of Bailey mixture is more than the conventional mixture under the test conditions of this study. The authors believe that the improvement in the fatigue behavior of asphalt mixture along with stiffness is due to the improvement in the aggregate gradation.

The study thus revealed that improvement in the aggregate skeleton improves the quality of asphalt mixture. Aggregate gradation development based on Bailey method, not only controls the volumetric parameters, but also improves the quality of mixtures. A fair comparison has been presented in the form of asphalt mixture's performance testing. Both the stiffness as well as fatigue performance of asphalt mixtures was ascertained using typical laboratory based testing procedures. It was observed during the testing that asphalt mixture prepared using the Bailey method improved its fatigue and rutting resistance.

4.9 Conclusions

Following conclusions have been drawn from this study:

- This study found reasonable relationship between the voids in coarse aggregate (VCA) and maximum achievable packing density (Gsb) using the local aggregates. The method of controlling the aggregate gradation volumetric parameters using voids in coarse aggregate and bulk specific gravity as developed by Bailey works fairly well on the local aggregates. The trends of relationships among the volumetric parameters in aggregate gradations with different Nominal Maximum Aggregate Sizes (NMAS) are in line with previous studies as reported in this paper. The criteria of optimizing the VCA and Gsb along with their boundary limits works fairly well and it minimizes the segregation chances during the mass production. The defined Bailey limits of different ratios can be adopted for developing the target aggregate gradation in Pakistan and the asphalt mixtures prepared within the defined limits perform better than the conventional asphalt mixtures.
- Conventional and modified aggregate gradation in the light of Bailey's method with same NMAS was utilized in the asphalt mixtures preparation. Significant variations in both the asphalt mixture design parameters were observed, which is an indication of the effect of aggregate gradation. Three basic parameters i.e., like asphalt mixture's compactibility, stiffness and fatigue resistance were investigated through a number of relevant tests.
- Asphalt mixture compactibility test aims to measure the amount of energy stored during the compaction process. Compaction densification and traffic densification are the two common indices that have been utilized to ascertain asphalt mixture's compactibility. Density at 8th gyration was recorded and compared with Gyration at 92%, 98%, which represents the compaction tendency of asphalt mixture during construction. The results of both the indices show that asphalt mixture with improved gradation by Bailey methods are stable and revealed relatively better compaction and resistance to further compaction once a target density is achieved.
- Asphalt mixture prepared with improved aggregate gradation using Bailey method showed better results both in fatigue as well as in rut resistance based performance testing. In Wheel Tracker test, asphalt mixture with conventional aggregate gradation showed almost 78% more rut value at 50°C with higher the rut rate (slope coefficient), and intercept coefficient. Master curve developed at 21.1 °C based on the dynamic modulus test data better characterized the mixture with lower sensitivity in case Bailey mixture. The p-value, F-value and the degree of determinacy by two way ANOVA analysis confirm the accuracy of the data and the tolerable degree of variations among the test results. The creep response of Bailey asphalt mixtures at higher temperatures was better than conventional mixture. Similarly, the fatigue life of Bailey asphalt mixtures was more than the conventional mixture.
- In general, Bailey method improved the aggregate gradation marginally and the performance of asphalt mixture prepared with the improved version of gradation performed better than conventional mixture. Authors feel confident in recommending the improved gradation by Bailey Method as a replacement of conventional NHA Class-B gradations.

5 Dynamic Modulus and Creep Stiffness

5.1 Introduction

In recent years, pavement performance has gained a lot of attention as it serves the basis for mechanistic-empirical (M-E) design approach. Dynamic modulus (|E*|) is one of the features of the M-E design approach used as design input in the structural design of pavement in Mechanistic-Empirical Pavement Design Guide (M-EPDG). The dynamic modulus test correlates with the field performance of hot mix asphalt (HMA) and complements the mix design criteria.

M-EPDG approach is relatively new for Pakistan and yet to be fully implemented. However, prior to implementation, it is very important to carry out laboratory evaluation of different HMA mixtures to predict their field response and correlate laboratory results with field results. To this end, dynamic modulus is lead candidate performance test that completely characterizes the asphalt mixtures for a range of temperatures (-10 to 60 °C) and loading frequencies in (0.1 to 25 Hz).

Usually, laboratory testing of dynamic modulus is carried out using asphalt mixture performance tester (AMPT) or Simple performance test protocols (SPT). In this research study the dynamic modulus (|E*|) test was conducted at a range of temperatures 4.4 to 54.4°C and frequencies 0.1 to 25Hz using asphalt mixture performance tester (AMPT). This research study also presents the catalogue for default dynamic modulus values for mixtures tested at temperatures ranges and frequencies by generating the master curves which in turn provide the basis for the implementation of mechanistic-empirical analysis and design - an approach which may be more appropriate for heavy axle loads/ tyre pressures and climatic conditions prevalent in Pakistan.

Further, the Flow tests (Flow Number and Flow Time) are also conducted which are explicatory indices for analysis and evaluation of rutting potential of various asphalt mixtures, subjected to static and repeated loading. The Flow tests at an effective temperature of 54.4°C and stress level of 300 kPa was conducted using Asphalt Mixture Performance Tester. The test results indicated that all mixtures experienced tertiary flow state in Flow Number (FN) test whereas no mixtures reached the tertiary flow state in Flow Time (FT) test. Data smoothening technique (five point moving average method) was employed for removing the resonance in raw data obtained from SPT software. Performance of mixtures were compared using FN values, cycle number at which the 50,000 micro strains occurred in specimen and intercept obtained from regression analysis of total permanent strain. The observed accumulated axial strains at the time of termination were used for comparison of mixtures as tertiary phase is not exhibited in FT tests. Two statistical models of different functional formulation (Power and First order multiple regression) were developed using FN and mix volumetric data.

5.2 Experimental Design

A methodology of the research study was planned in order to conduct the simple performance test for investigating the factors affecting dynamic modulus in order to compare different asphalt concrete mixtures. This research study included four (04) asphalt wearing course and four (04) asphalt base course gradations using aggregate from Margalla quarry. Penetration grade binder i.e., 60/70 from Attock Refinery Limited (ARL) was used. The optimum bitumen content and volumetric properties were determined using standard Marshall mix design and based on the results performance testing was carried out on specimens fabricated through Superpave gyratory compactor.

Table 5.1: Test Matrix for Dynamic Modulus Evaluation of Asphalt Mixtures

Test Temperature (°C)	Loading Frequency (Hz)	Layer & Gradation							
		Wearing Course				Base Course			
		NHA-A	NHA-B	SP-1	MS-2	NHA-A	NHA-B	SP-2	DBM
4.4	0.1	✓	✓	✓	✓	✓	✓	✓	✓
	0.5	✓	✓	✓	✓	✓	✓	✓	✓
	1	✓	✓	✓	✓	✓	✓	✓	✓
	5	✓	✓	✓	✓	✓	✓	✓	✓
	10	✓	✓	✓	✓	✓	✓	✓	✓
	25	✓	✓	✓	✓	✓	✓	✓	✓
21.1	0.1	✓	✓	✓	✓	✓	✓	✓	✓
	0.5	✓	✓	✓	✓	✓	✓	✓	✓
	1	✓	✓	✓	✓	✓	✓	✓	✓
	5	✓	✓	✓	✓	✓	✓	✓	✓
	10	✓	✓	✓	✓	✓	✓	✓	✓
	25	✓	✓	✓	✓	✓	✓	✓	✓
37.8	0.1	✓	✓	✓	✓	✓	✓	✓	✓
	0.5	✓	✓	✓	✓	✓	✓	✓	✓
	1	✓	✓	✓	✓	✓	✓	✓	✓
	5	✓	✓	✓	✓	✓	✓	✓	✓
	10	✓	✓	✓	✓	✓	✓	✓	✓
	25	✓	✓	✓	✓	✓	✓	✓	✓
54.4	0.1	✓	✓	✓	✓	✓	✓	✓	✓
	0.5	✓	✓	✓	✓	✓	✓	✓	✓
	1	✓	✓	✓	✓	✓	✓	✓	✓
	5	✓	✓	✓	✓	✓	✓	✓	✓
	10	✓	✓	✓	✓	✓	✓	✓	✓
	25	✓	✓	✓	✓	✓	✓	✓	✓

Simple Performance Tester (SPT) was used to conduct the Flow Number (FN) and Flow Time (FT) tests. The FN test was performed in accordance with AASHTO TP 79 (AASHTO, 2010). Testing was conducted on four asphalt wearing course mixtures and four base course mixtures. Each mix was tested for flow time and flow number at static and repeated unconfined stress of 300 kPa respectively. Tests were conducted at single effective temperature of 54.4°C and each specimen was pre-conditioned for two (02) hours. Table 5.2 shows the test matrix for the flow number and flow time test. Trimmed and polished specimens were subjected to a repeated axial haversine compressive load pulse of 0.1 second followed by rest period of 0.9 second. The test was set to be terminated at 10,000 cycles or until maximum accumulated permanent strain in specimen reached 5% or whichever came first (NCHRP, 2004). The FT test was performed in accordance with NCHRP reports where preconditioned specimen is subjected to a constant axial load until it fails and permanent deformation is measured with respect to the load time.

Table 5.2: Test Matrix for Flow Number and Flow Time Test

Test Conditions		Flow Number	Flow Time
Layer & Gradation		54°C, 300kPa	54°C, 300kPa
Wearing Course	NHA – A NHA - B SP - 1 MS – 2	12 specimens (Triplicate of each gradation)	12 specimens (Triplicate of each gradation)
Base Course	NHA – A NHA - B SP - 2 DBM	12 specimens (Triplicate of each gradation)	12 specimens (Triplicate of each gradation)

5.2.1 Sample Preparation

Optimum bitumen content evaluated from Marshall method was used for preparation of the specimens using Superpave gyratory compactor (SGC). The specimens were prepared in accordance with ASTM D3496-99, "Standard Practice for preparation of bituminous mixture specimens for Dynamic Modulus Testing". Triplicate specimens were prepared for each mix of both wearing and base course mixtures respectively. Figure 5.1 shows the specimens prepared from gyratory compactor with approximate height of 170mm and diameter of 150mm. Similar protocol was followed for the preparation of Tests specimens for FT and FN tests.

Specimens were labelled as per their gradation and with suffix of "W" for wearing and "B" for wearing while triplicate specimens were named by numeric value for e.g., (NHA W-1). After preparation and compaction in gyratory compactor, the specimens were further cored using a coring machine for extraction of 100mm core from the centre and trimmed as prescribed in AASHTO TP 62-07. The height of the specimens was trimmed to 150mm using saw cutter. As per specification the height to diameter ratio was kept as 1.5. Figure 5.2 shows the cored specimens, trimmed to desired ratio along with the waster rings.

It was ensured that specimens were compacted to desired air voids with allowable limit not exceeding ± 0.5%. After preparation of specimens to desired dimensions and air voids, gauge points (studs) were fixed using 5 minutes epoxy glue. These studs on specimens help to measure the axial deformation/strain during the test using linear variable differential transformer (LVDT). Studs were fixed to specimens using gauge point fixing jig which is illustrated in Figure 5.3 below. The gauge point fixing jig machine applies pressure for forty five (45) minutes on the studs so that the glue is hardened completely around the studs. Once the studs are in place, specimens were removed from the jig and fixed with the clamps. These clamps are designed in such a way to accommodate the LVDTs for measuring axial deformation as shown in the Figure 5.4. The LVDT's are checked through computer software for proper functioning and measurement of deformation. If the bars of LVDT's are in motion and within range (i.e.0.49 mm), it can measure the axial deformation.

The specimen is then kept in an environmental chamber to equilibrate with testing temperatures as stated in AASHTO TP 62-07. At higher temperatures, the studs start to loosen up as glue softens. Hence, greater care needs to be taken while performing test at higher temperatures.

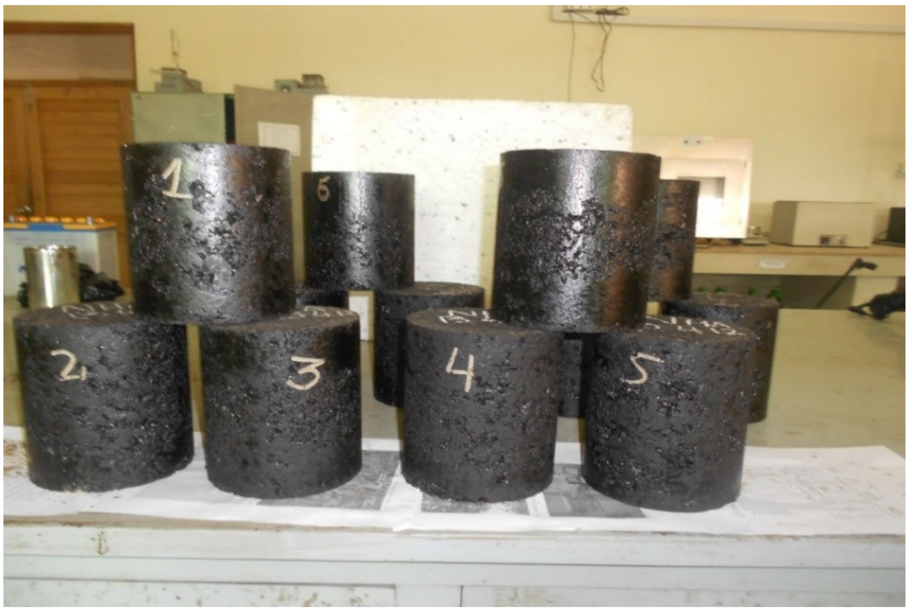


Figure 5.1: Specimen Prepared Using Gyratory Compactor

Figure 5.2: Cored and Trimmed Specimen with Waster Rings

Figure 5.3: Gauge Point Fixing Jig

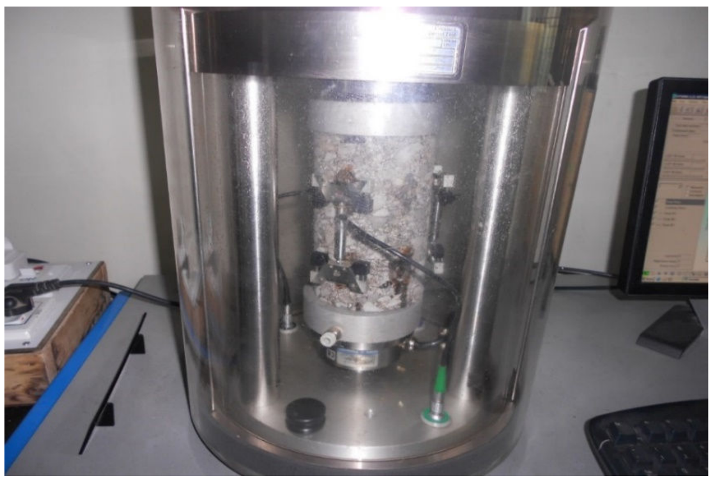


Figure 5.4: LVDT Mounted on the Specimen with Transducers

5.2.2 Laboratory Testing

The AASHTO TP 62-07 standard was used for performing dynamic modulus test which requires cylindrical specimen. Sinusoidal loading is applied and dynamic modulus along with phase angle is calculated at required frequency and temperature. With the application of sinusoidal compressive stress, the ratio of stress to axial recoverable strain gives the dynamic modulus along with the phase angle. Same setup was used to conduct the FN and FT test in the Simple Performance Tester (SPT). The FN test was performed in accordance with AASHTO TP 79. The FT test was performed in accordance with NCHRP reports where preconditioned specimen is subjected to a constant axial load until it fails and permanent deformation is measured with respect to the load time.

5.2.2.1 Dynamic Modulus Testing

Simple Performance Tester (SPT) apparatus was used for evaluation of Dynamic Modulus. In this apparatus, specimen is placed in an environmental chamber for maintaining temperature with an inbuilt, hydraulic actuator and pump. The refrigeration and heating units are attached to the computer system along with data collection system.

Figure 5.5 shows the hardware configuration for measuring dynamic modulus through simple performance tester (SPT) machine, using UTS 6 software.

Figure 5.5: Simple Performance Tester (SPT)

Once the studs are fixed to the specimen, it is placed in an environment chamber with transducers (LVDT) mounted at three points which are 120 degrees apart. The environmental chamber is closed and left to equilibrate with given test temperature. Tests were conducted as per standard testing procedure outlined in AASHTO TP 62-07. Minimum equilibrium time for temperature as defined in AASHTO 2007 is given below in Table 5.3:

Table 5.3: Recommended Temperature Time

Specimen Temperature (°C)	Time from Room Temperature (Hrs)	Time from Previous Test Temperature (Hrs)
-10	Overnight	Overnight
4	Overnight	4 Hrs or overnight
21	1	3
37	2	2
54	3	1

Once the required temperature is attained, the desired frequencies are selected through the computer menu option and in this case 25, 10, 5, 1, 0.5 and 0.1 Hz frequencies were selected. It may be noted that prior to start the test, the software requires the initial modulus value which is obtained by tuning option of software where sinusoidal load is applied for 9 cycles. The test procedure is such that it starts from highest frequency and proceed to lowest frequency i.e., from 25 Hz to 0.1 Hz. After completion of test, the software automatically generates the output which yields dynamic modulus value and phase angle at given test temperature and selected frequencies. The test results have been utilized in development of master curves for both asphalt wearing and base course mixtures.

5.2.2.2 Flow Number (FN) & Flow Time (FT) Testing

Simple performance tester (SPT) machine is capable of applying cyclic loading to compacted asphalt mixture specimen over a wide range of temperatures as well as frequencies as explained earlier. A salient feature of equipment is its own software and built-in test modules for each test type (i.e., flow number, flow time, etc.) which are installed in computer system attached to it. In FN and FT tests after completion of pre-conditioning time, specimen is subjected to selected stress level. All tests were performed at single temperature and stress level of 54.4°C and 300 kPa, respectively as recommended in earlier research work (Witczak et al. 2002).

The SPT equipment was limited to apply maximum stress level of 460 kPa for one hour time duration therefore confined testing was not possible. The desired stress level for testing was 300 kPa (43.5 Psi). Trial tests were conducted on various samples before selection of this stress level to ensure that in a suitable time most of these specimens would reach tertiary flow.

Termination of test is selected as maximum 5% of accumulated axial strain. The target test termination strain value is selected since the finite element studies show that the 0.5inch or 12.5mm rutting is expected at 5% strain. The maximum value of load repetition is set at 10,000 cycles. Test termination shall occur upon maximum limit of strain or cycles whichever comes first.

Flow number test differs in stress application from flow time test. Static loading is applied in flow time test while dynamic or repeated load is applied in flow number test in haversine wavelength form. A haversine type of loading is applied with pattern of 0.1 seconds loading followed by 0.9 seconds rest period. Equivalent pulse time conversion for haversine loading at rate of 10Hz frequency closely represents speed of 110km/h, posted speed for heavy traffic on national motorways. Upper and lower friction reducer is also applied in between the sample and plates when the specimen is placed inside the testing chamber to avoid eccentric loading. Specimen is centred visually in-line with the load actuator. Axial deformation is recorded against every load cycle.

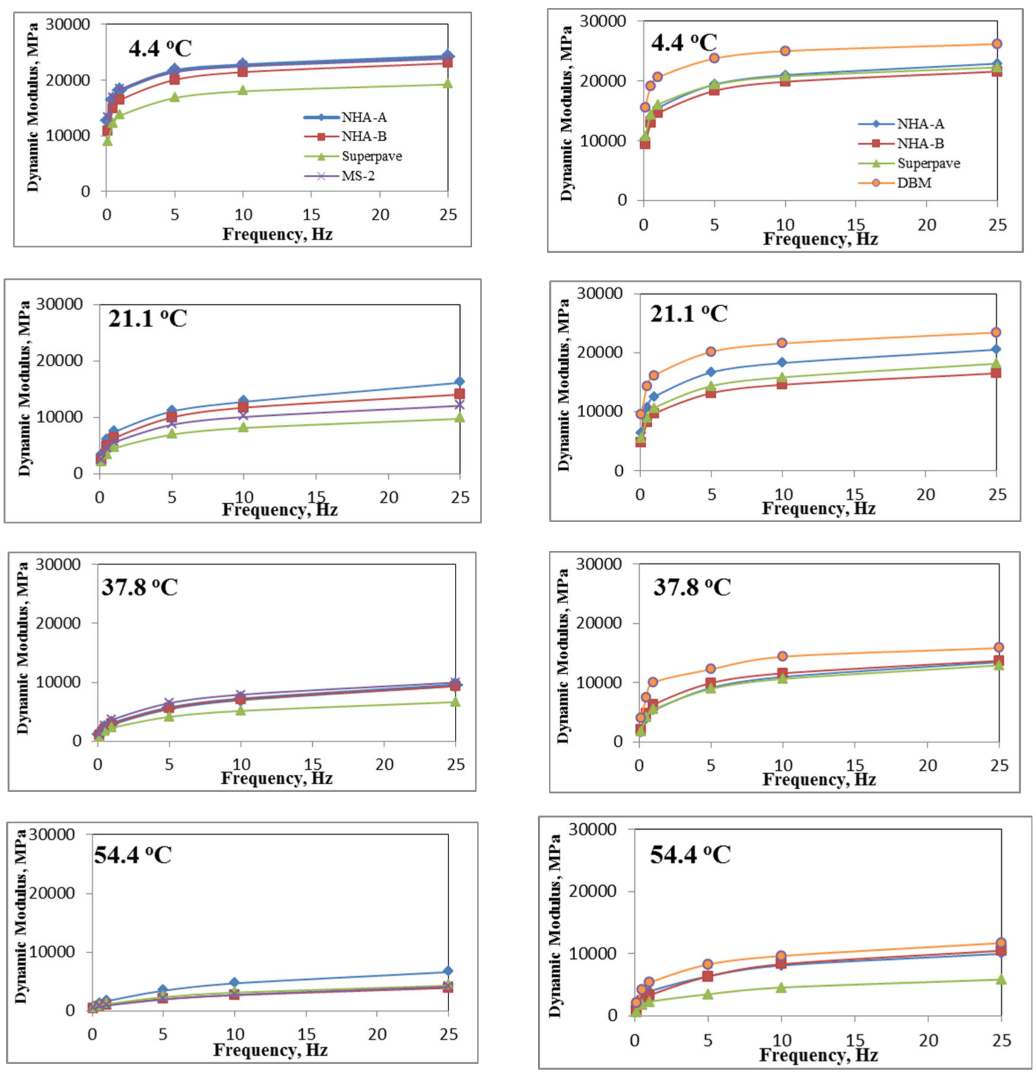
5.3 Experimental Results

Statistical analysis was carried out on experimental test results of dynamic modulus, flow number and flow time. Factors affecting the dynamic modulus i.e., temperature, frequency and mixture properties were evaluated. Master curves were developed for each mix and presented collectively for asphalt wearing and base course mixtures. Statistical analysis mainly included the design of experiment i.e., full factorial design; analysis of variance (ANOVA), interaction plot and sensitivity analysis. Additionally, resistance to fatigue is also calculated for both asphalt wearing and base course mixtures.

Flow number and flow time results obtained from SPT is firstly exported to excel data sheets from AMPT software file using SPT software named as SPT Flow. The data in excel sheets is further used for analysis. Data smoothening technique is employed for removing any error in order to obtain correct values of flow time and flow number. Five point moving average method is used for removing the resonance in raw data obtained from SPT. The software gives the measured strain (in microstrains) against each cycle which is used to calculate the strain rate. The strain rate is plotted for calculating the corrected flow number value.

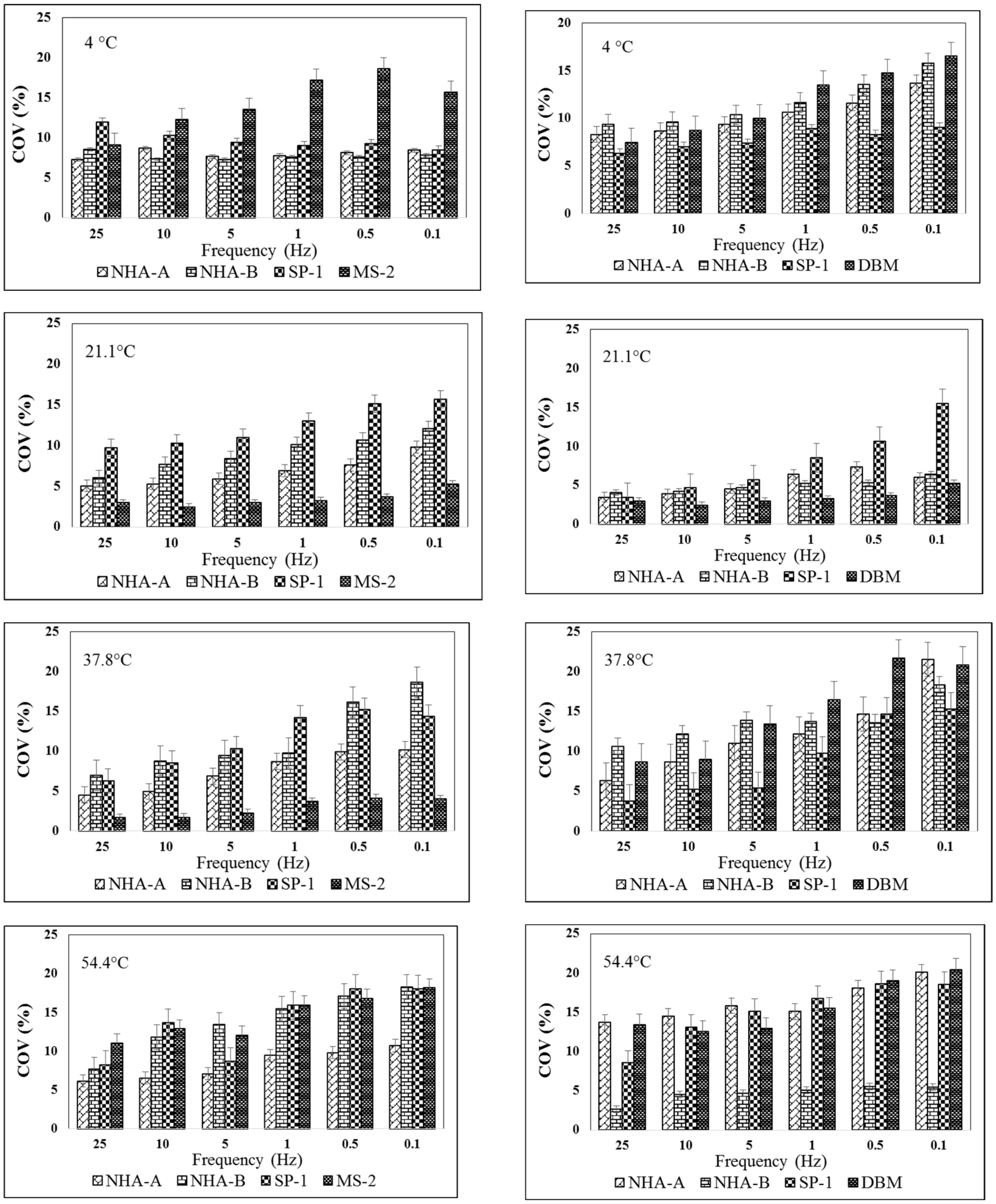
5.3.1 Dynamic Modulus Test Results

Compressive sinusoidal load was applied to simulate the dynamic movement of vehicles. Triplicate specimens were tested for each mix for the range of temperatures and loading frequencies to obtain a desired accuracy limit of ±12%. The recorded data acquired from software were employed for development of master curves, estimation of resistance to fatigue and sensitivity analysis. Coefficient of variance (COV) was calculated as a ratio of standard deviation to the mean of observed dynamic modulus values for comparing the degree of variation from one specimen to another for a given gradation. The average |E*| values for triplicate specimens and COV is presented in Figure 5.6 and Figure 5.7, respectively. The minimum and maximum calculated COV were 1.17% and 20.71% for asphalt wearing course (Figure 5.7a), and 2.67 % and 21.99% (Figure 5.7b), for base course mixtures. In general, the highest COV were obtained at the high temperatures and low frequencies, where the deformation measured was relatively high.



Asphalt Wearing Course Mixtures　　Asphalt Base Course Mixtures

Figure 5.6: Dynamic Modulus of Asphalt Mixtures Tested



a) Asphalt Wearing Course Mixtures

b) Asphalt Base Course Mixtures

Figure 5.7: COV of Asphalt Mixtures Tested

5.3.1.1 Statistical Analysis for Dynamic Modulus

A two-level factorial design of experiment was conducted in order to determine the significant factors affecting the dynamic modulus. In this experiment, each parameter is specified by two levels; high and low. The factors considered for both types of mixtures are temperature, frequency and nominal maximum aggregate size (NMAS). Table 5.4 illustrates the levels for factorial design along with abbreviation of each factor and measured unit. The temperature and frequency remain same for all mixtures while NMAS changes with respect to mixtures. The main effect is the difference in mean response between low and high level of a factor, while interaction effect is mean difference between effects of one factor at extreme values i.e., high and low level values of other.

Table 5.4: Factors for Two-Level Factorial Analysis

Factors	Abb.	Units	Low Level		High Level	
			WC	BC	WC	BC
Temperature	A	°C	4.4	4.4	54.4	54.4
Frequency	B	Hz	0.1	0.1	25	25
NMAS	C	Mm	12.5	25	19	37.5

The confidence level for experiment was taken as 95% (significance level, $\alpha = 0.05$). The negative and positive sign with effect shows that the factor is inversely and directly related with dynamic modulus (response), respectively whereas the numerical value displays the strength of effect (Table. 5.5). The significance of effects or otherwise can be assessed by p-value less than or greater than $\alpha = 0.05$, respectively. In case of asphalt base course mixtures, the temperature and frequency are analogous to wearing course mixtures in significance while NMAS is insignificant in case of base course mixtures. Significance/insignificance of factors affecting the dynamic modulus has also been presented in the normal plots of standardized effect (Figure 5.8).

The main effect is plotted versus high and low levels of factors (see Figure. 5.9). The steep slope of line signifies strength of relationship between factor and response. Both temperature and frequency have steep slope line which notifies their significance for both types of mixtures whereas the impact of NMAS is marginally significant on dynamic modulus as line does not have notable slope for wearing course mixtures and is insignificant for base course mixtures as shown by a straight line.

Table 5.5: Main and Interaction Effect Estimates of |E*|

Factors	Wearing mixtures		Base mixtures	
	Effects	p-value	Effects	p-value
Main effect				
A	-16398	0	-14921	0
B	7591	0	9443	0
C	1791	0.033	492	0.546
Two factors interaction				
A*B	-2211	0.098	-960	0.463
A*C	-573	0.606	1206	0.271
B*C	752	0.448	692	0.478
Three Factors interaction				
A*B*C	256	0.847	271	0.836

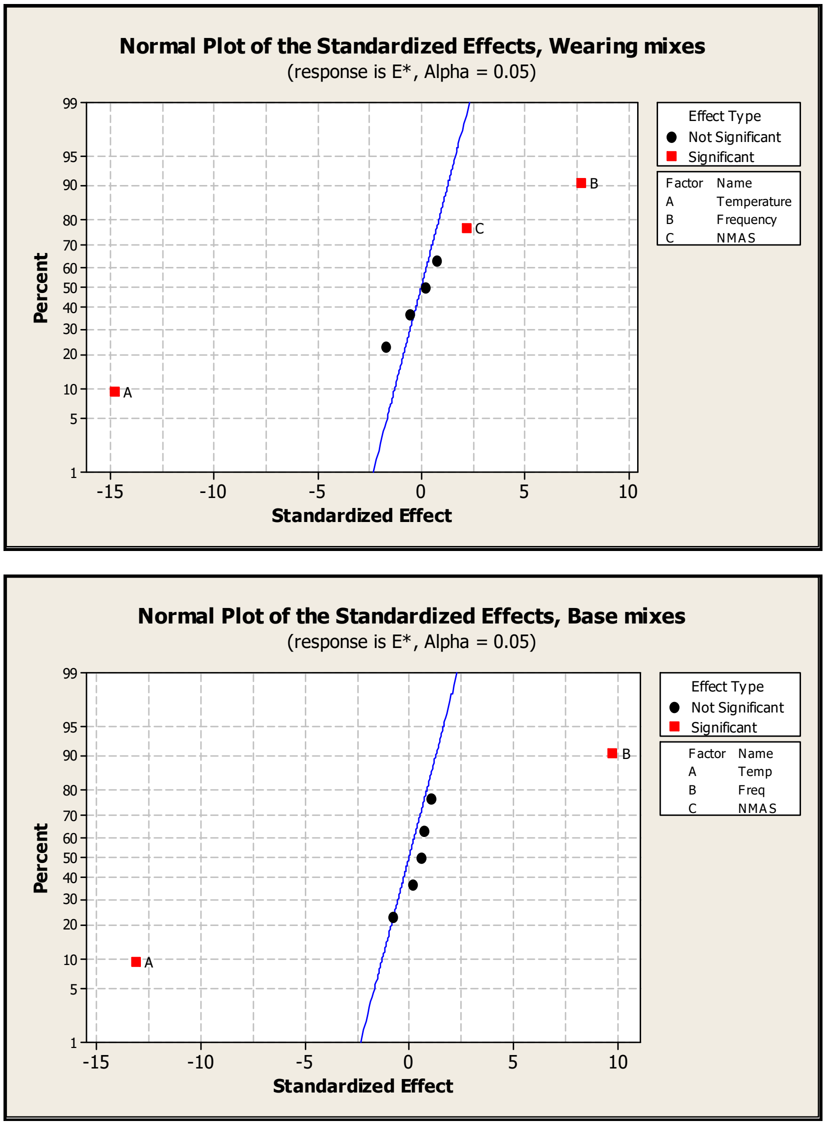
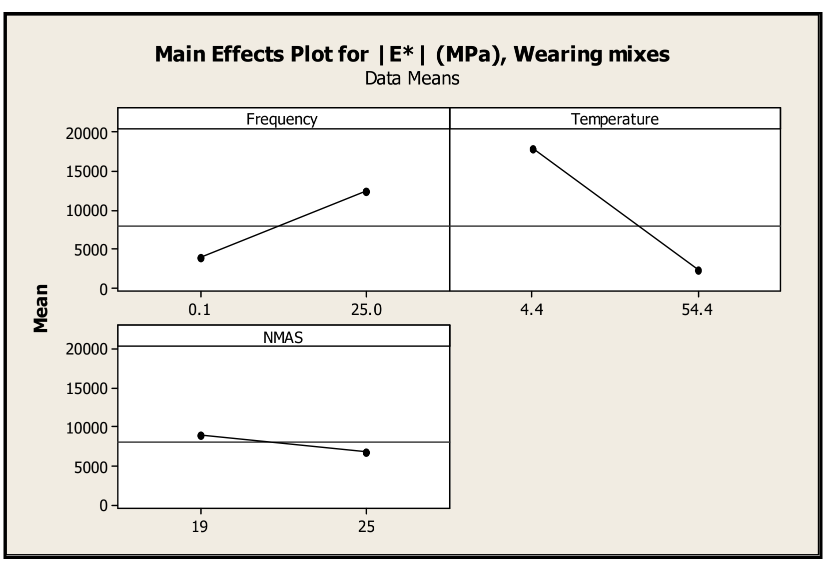


Figure 5.8: Cumulative Normal Probability of Effect Estimates of |E*|



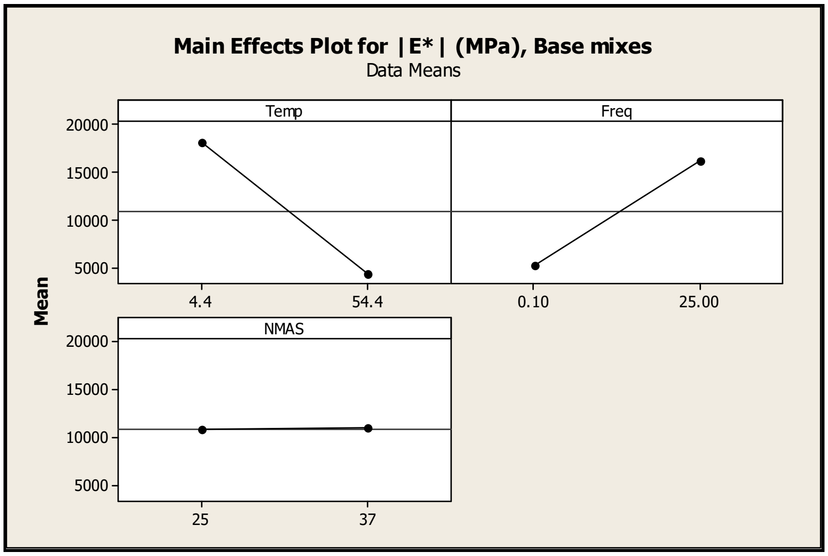


Figure 5.9: Main Effect Plots of Factors of |E*|

5.3.1.2 Development of Master Curves

A master curve of an asphalt mixture permits comparison of linear viscoelastic properties of material, when testing has been conducted using different stress rates (loading frequencies) and test temperatures. The results obtained from laboratory tests using simple performance tester were used to develop master curves, which ultimately helps in determining the pavement response. The average result is taken for each temperature and used while developing master curve for average |E*| at reference temperature of 21°C using the time-temperature superposition principle in which each temperature |E*| value is shifted to reference temperature to get a smooth uniform curve.

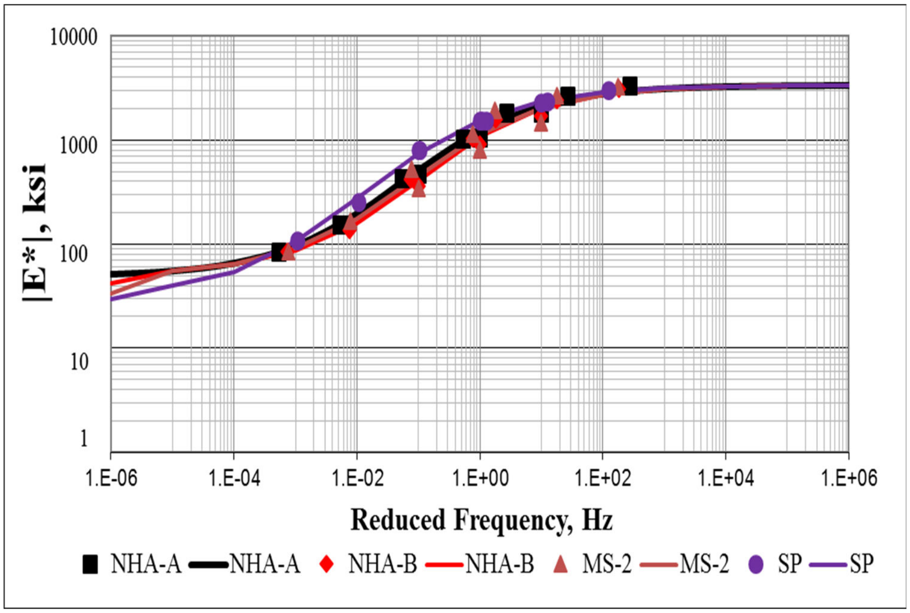
Master curves were generated using Microsoft Excel® sheet that was established under NCHRP project 9-29 which regress/minimize the error sum of square (SSE) to fit the curve. The general form of sigmoidal function used to develop a master curve is presented in Equation 5-1 below:

$$\boldsymbol{Log}\ |\boldsymbol{E}^*| = \boldsymbol{\delta} + \frac{\boldsymbol{\alpha}}{\mathbf{1} + \boldsymbol{e}^{\boldsymbol{\beta}+\boldsymbol{\gamma}(\boldsymbol{log f_r})}} \qquad \text{5-1}$$

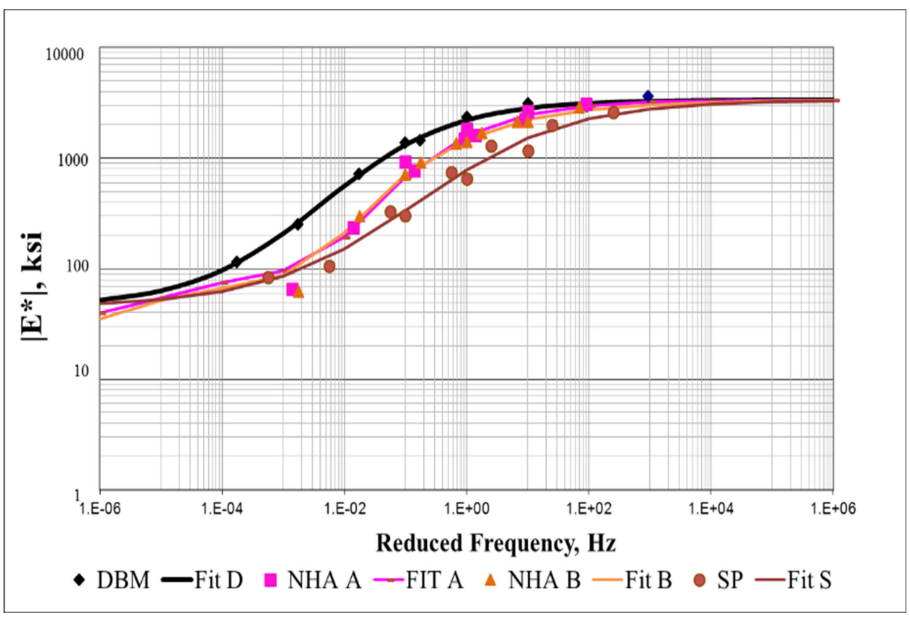
Where;

Log (|E*|)	=	log of dynamic modulus
δ	=	minimum modulus value
f_r	=	reduced frequency
α	=	span of modulus value
β, γ	=	shape parameters

The parameters used for development of master curves are tabulated in Table 5.6. The maximum stiffness of mix is function of binder at lower temperature, while at higher temperatures aggregate interlock dominates the binder effect. The master curve for all wearing course mixtures is presented in Figure 5.10 (a), which indicates that all curves merge to one and no variation is observed at higher and lower frequency. Even though it is hard to distinguish between the lines, the graph shows that the NHA–A mix has the highest dynamic modulus values at all frequencies while Superpave mix has the lowest dynamic modulus values at all frequencies. This indicates that the dynamic modulus test is sensitive to variation in the mix volumetric properties, gradation type and optimum bitumen content. Figure 5.10 (b) illustrates the dynamic modulus master curves for asphalt base course mixtures and variation due to decrease in |E*| values from 21.1 to 37.8°C can be observed at intermediate frequencies. This variation is attributable to the difference in aggregate interlock/mechanical behaviour that causes difference in stiffness of various base course mixtures.



(a) Wearing Course Mixtures



(b) Base Course Mixtures

Figure 5.10: Dynamic Modulus Master Curves for Asphalt Base Course Mixtures

Table 5.6: Master Curve Parameter

Material	Mix type	Parameters				
		δ	α	β	γ	R^2
Asphalt Wearing Course	NHA-A	49.20	3311.10	-1.16	-0.94	0.98
	NHA-B	54.14	3302.90	-0.97	-1.01	0.97
	SP–1	27.94	3332.40	-1.64	-0.87	0.99
	MS-2	50.00	3310.30	-1.05	-0.95	0.87
Asphalt Base Course	NHA-A	4.15	3356.15	-2.13	-0.83	0.98
	NHA-B	0.11	3360.20	-2.45	-0.72	0.99
	SP–2	46.10	3314.21	-0.67	-0.82	0.94
	DBM	45.72	3314.59	-2.22	-0.94	0.98

5.3.1.3 Estimation of Fatigue Resistance

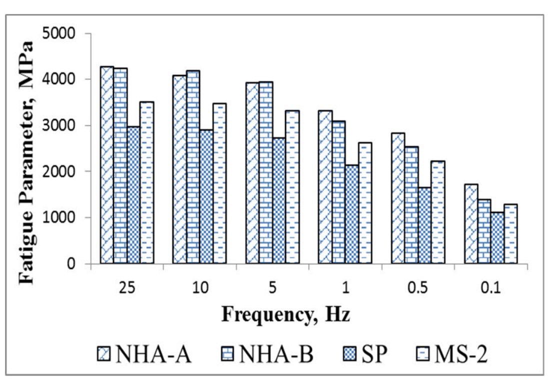
Fatigue parameter is derived from dynamic modulus of asphalt mixtures to evaluate the resistance to fatigue (Ye et al 2009). The fatigue parameter is the product of dynamic response (|E*|) and viscoelastic behaviour of mix; i.e., phase angle (φ) and can be estimated using Equation 5-2. Higher values of fatigue parameter represent poor resistance to fatigue cracking and vice versa. Also, fatigue behaviour is inversely proportional to stiffness of mix.

$$\text{Fatigue parameter } = |E^*| \times sin\varphi \qquad 5\text{-}2$$

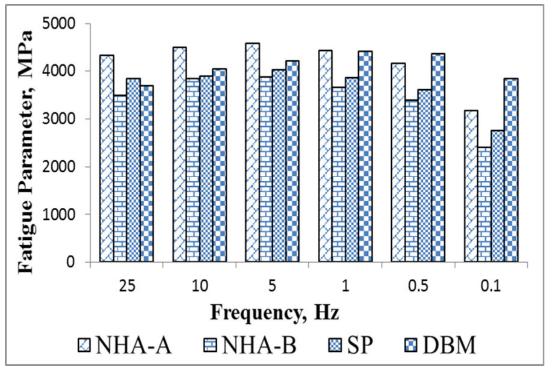
Where;

|E*| = Dynamic modulus (MPa)
φ = Phase angle (Degree)

Fatigue parameter evaluates endurance to fatigue of asphalt mixtures analogous to |G*|×sin δ for asphalt binders. The fatigue parameters (Equation 5-2) of all mixtures were compared at 21°C and varying frequencies. Superpave gradation for wearing mix and NHA-B base mix has relatively higher resistance to fatigue as indicated by lower fatigue parameter value as compared to the other mixtures (see Figure 5.11). The higher resistance to fatigue cracking in relatively finer asphalt wearing and base course mixtures is attributable to distract in the stress fabrication of asphalt mixtures that ultimately led to arrest the crack initiation.



(a) Asphalt Wearing Course Mixtures



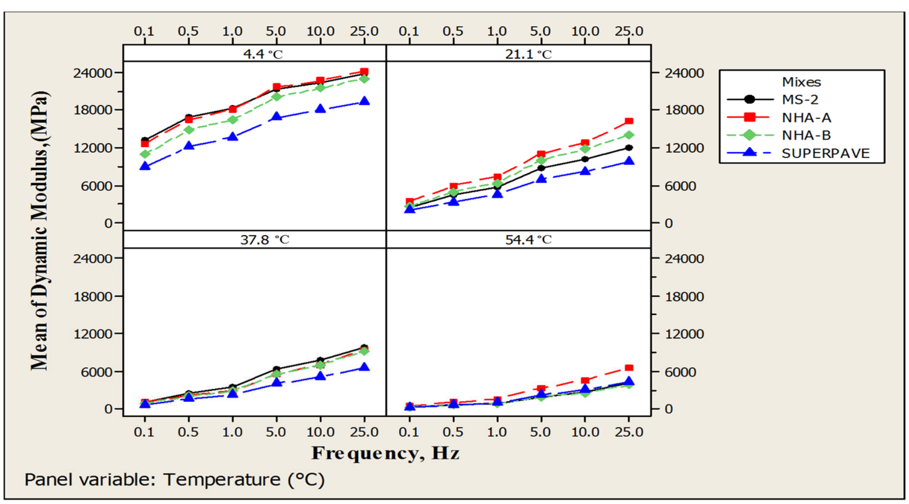
(b) Asphalt Base Course Mixtures

Figure 5.11: Fatigue Parameter of Asphalt Mixtures at 21°C

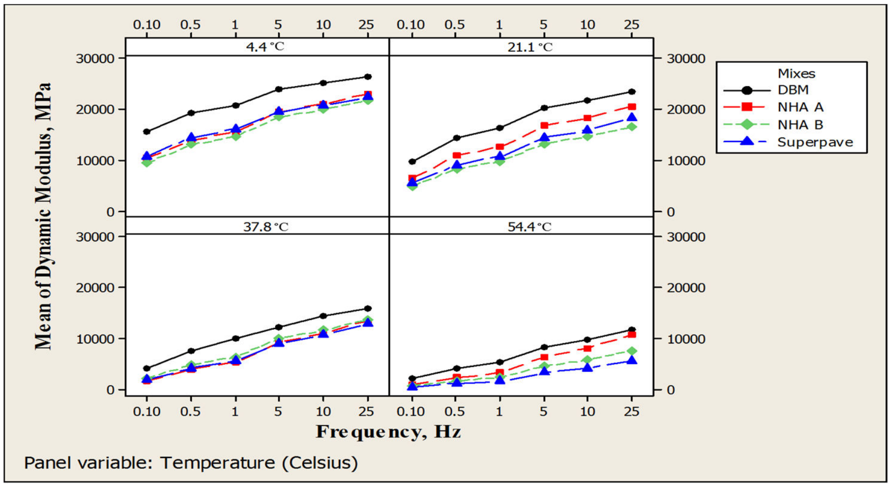
5.3.1.4 Sensitivity Analysis of Dynamic Modulus

Sensitivity analysis is a technique used to determine effect of different input variables to dependent variable subjected to certain set of conditions. It can be very useful while testing the robustness of the output which is based on certain input parameters. For this study, the selected input parameters included the loading frequency and test temperature while output is dynamic modulus.

From the test results, it is clear that dynamic modulus has inverse relation with test temperature and direct relation with loading frequency. Graphically the average test results are expressed in form of isothermal and isochronal curves. It has been observed that dynamic modulus values are significantly higher at lower temperatures while increased flexibility/viscoelasticity at higher warmer temperatures resulted in lower dynamic modulus. Among the mixtures tested, NHA-A wearing mix and DBM base mix showed higher dynamic modulus values (see Figure 5.12). The reason for NHA-A wearing mix exhibiting relatively high dynamic modulus could be attributed to coarser nature of mix which become stiffer at lower and higher temperature and ultimately higher |E*| is achieved. Similarly, DBM mix has dense aggregate – binder matrix which makes it stiffer and viable to produce relatively higher |E*| values at lower and higher temperatures.

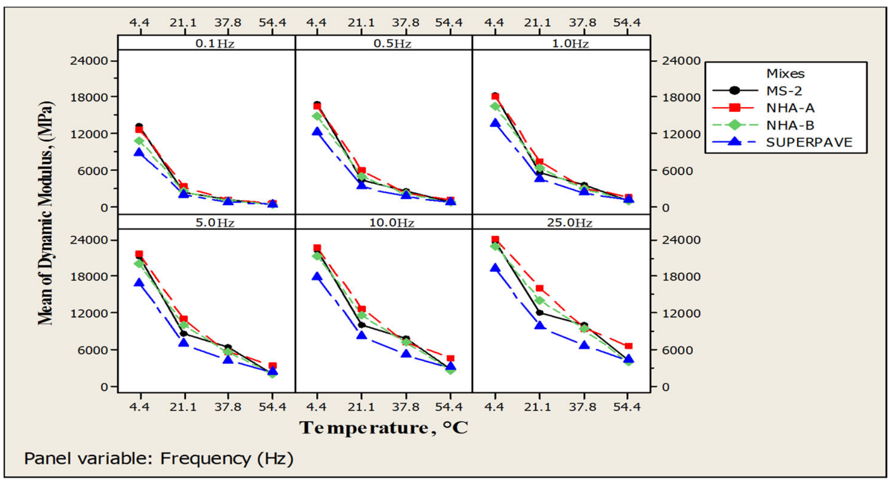


(a) Asphalt Wearing Course Mixtures

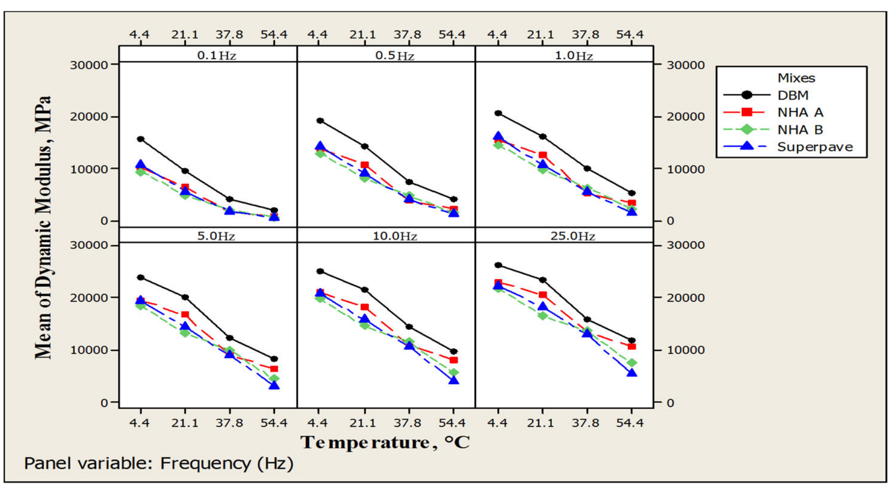


(b) Asphalt Base Course Mixtures

Figure 5.12: Isothermal Curves for Dynamic Modulus of Asphalt Mixtures



(a) Asphalt Wearing Course Mixtures



(b) Asphalt Base Course Mixtures

Figure 5.13: Isochronal Curves for Dynamic Modulus of Asphalt Mixtures

Figure 5.13 indicates variation of dynamic modulus of all mixtures at single frequency and range of temperatures in individual panel. It is observed that NHA-A wearing mix had relatively higher dynamic modulus than other mixtures while in case of base mixtures, DBM mix showed higher dynamic modulus values at all frequencies.

5.3.2 Flow Number (FN) Test Results

Repeated load (FN) test employs a repeated dynamic load test to determine the permanent deformation characteristics of paving materials for several thousand repetitions and record the cumulative permanent deformation as a function of the number of cycles (repetitions) over the testing period. An important element in the design of rut-resistant pavements is the screening of asphalt mixtures for rut susceptibility during mix design. The time to tertiary flow failure is considered a good indicator of the rutting resistance of a given mixture. This can be quantified through the FN as measured in a repeated load permanent deformation test. The repeated load test is one of the best methods for assessing the permanent deformation potential of asphalt mixtures. Witczak et al. (2002) defined the FN as the loading cycle number where tertiary deformation starts. The FN is more analogous to field conditions since the loading to pavement is not continuous. The FN allows the asphalt pavement a period to recover some of the strain induced by the loading. The FN attempts to identify the resistance of a mixture to permanent deformation by measuring the shear deformation that occurs because of haversine loading. The most important output of the FN test is the curve of accumulated strain against the number of load cycles, which describes the rutting resistance of mixtures (Figure 5.16). The relationship between the accumulated strain and loading cycles is based on the rutting mechanism, densification, and shear flow.

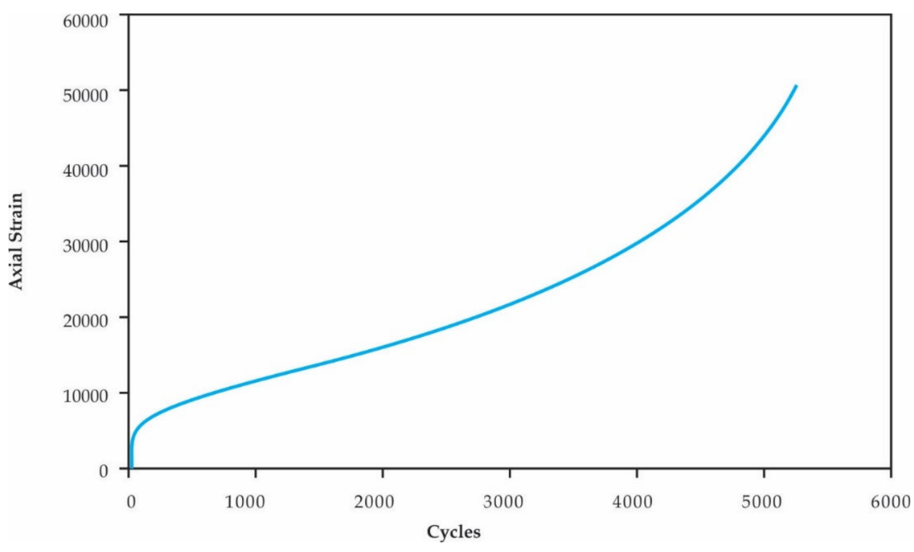
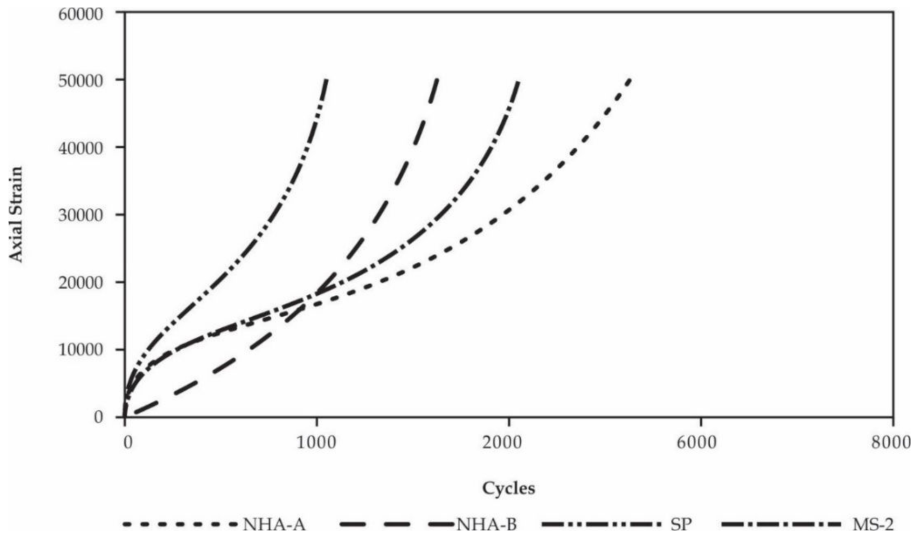
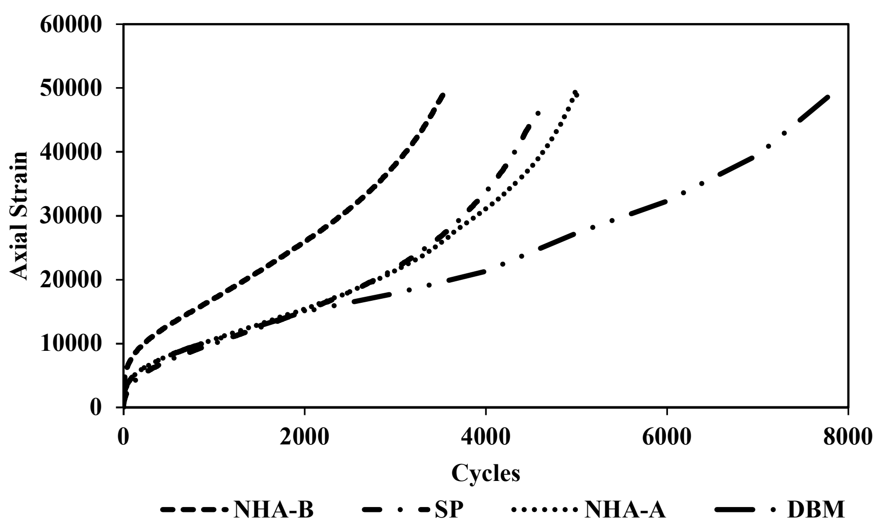


Figure 5.14: Typical Data from Flow Number Test

In this study, the test specimens were conditioned for two (02) hours at 54.4°C temperature. Triplicate specimen for each mixture were tested for FN. Tertiary phase of deformation was observed for all mixtures tested where tests had to be terminated before 10,000 cycles at their maximum allowed strain limit i.e., 50,000 macrostrains. The results obtained from FN test are presented as accumulated strain versus time for mixtures (see Figure 5.17). It can be seen that NHA-A wearing course mix comparatively has the highest number of load cycles at all phases among the tested gradations, it sustains higher number of load repetition to achieve three phases (Primary, Secondary, and Tertiary) (see Figure 5.17). However, DBM in case of asphalt base course mixtures has maximum number of load cycles.



(a) Asphalt Wearing Course Mixtures



(b) Asphalt Base Course Mixtures

Figure 5.15: Accumulated Strain versus Time Plot

Various researchers have reported that whenever specimen attains tertiary state, data obtained from software represent resonance which leads to false FN instead of true FN. As stated previously, all wearing and base course mixtures have attained tertiary state, hence, it was considered necessary to remove resonance/ noise in data so that true FN could be obtained. Data smoothening technique was employed for removing data resonance in order to obtain corrected values of FN. Five point moving average method was used for removing the resonance in raw data obtained from simple performance tester.

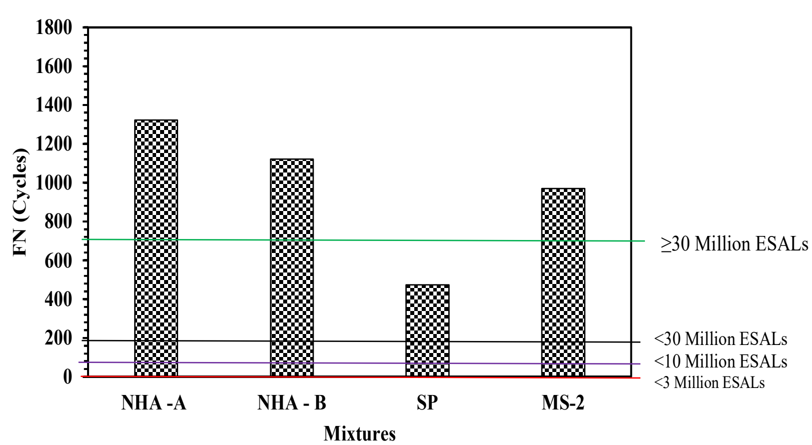
The strain rate against a designated cycle is obtained by half of difference of adjacent cycles. Then smoothening of strain rate moving average at five intervals is used. Figure 5.17 shows smoothened data after applying smoothening technique of moving average at five points. The coefficient of determination (R^2) of best fit line of smoothened data is also shown along with a fourth order polynomial equation as typical example. As the FN is the start point of tertiary deformation zone and can be reported as the lowest point in relationship of rate of change of compliance to loading time. In order to confirm FN value visible as lowest on curve, the equation is solved by putting y equals to zero as stated by Witczak (2002) that theoretically FN is cycle number corresponding to a rate of change of permanent strain equal to zero.

In order to rank the asphalt mixtures, the precursor is to check whether obtained FN complies with recommended FN values as stated in NCHRP 9-33 for the minimum FN criteria against traffic level (see Table 5.9). It can be seen in Figure 5.18 that all mixtures can be used for heavy traffic (≥30 Million ESALs) except Superpave mixture (asphalt wearing as well as base course). However, Superpave mixture can be used as an alternative for medium level traffic (10 to <30 ESALs).

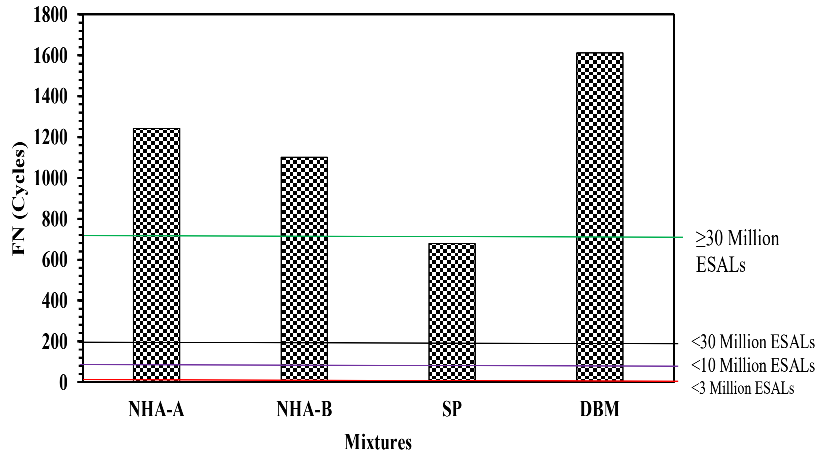
Table 5.7: Recommended Minimum Flow Number Values in NCHRP 9–33

Traffic Level (Million ESALs)	Minimum Flow Number (Cycle)
<3	-
3 to <10	50
10 to <30	190
>30	740

Apart from two conventional procedures opted for determination of FN i-e termination of test at achieving 10,000 load cycles or 5% accumulated strain. The regression coefficients βo (Intercept) and β1 (slope) can be employed to determine rutting susceptibility of asphalt concrete mixtures. Regression coefficients can be obtained from secondary zone of the log of compliance to log of time plot. It is reported that tertiary zone of asphalt mixture should be ignored as regression analysis is dependent on material-test conditions. To obtain the regression constants, the total accumulated permanent strains are modelled with respect to time (load cycles) as shown in Figure 5.19.



(a) Asphalt Wearing Course Mixtures



(b) Asphalt Base Course Mixtures

Figure 5.16: Flow Number Values with Traffic Level for Asphalt Mixtures

It may be noted that the plot does not contain strain values after FN (Tertiary zone) and these two parameters ignores the tertiary zone, so that corrected value can be determined, hence, one should restrict the strain values till the secondary phase. In general, inelastic behavior of asphaltic material can be predicted using regression coefficients.

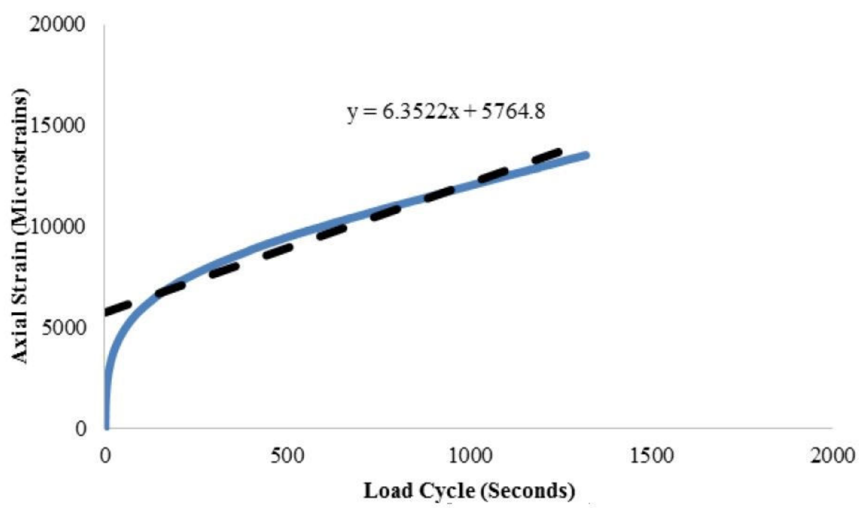


Figure 5.17: Accumulative Axial Strain versus Cycle Plots

5.3.3 Flow Time (FT) Test Results

The FT or static creep test was conducted on triplicate specimens of each mixture using SPT. The stress level and temperature are same as used in FN test. The results obtained from FT tests showed that none of the mixtures attained the tertiary phase of deformation. Hence, data smoothening technique was not required and data obtained from AMPT software was used for further analysis and restricted to the comparison of accumulated strain only. The FT results indicated that all mixtures exceeded the maximum cycles of 10,000 and accumulated strains were used for purpose of comparison. It can be inferred from Table 5.11 that NHA-A wearing course and DBM base course mixtures had relatively smaller number of accumulated strains at time of termination which suggests that both of these mixtures have better resistance to permanent deformation over the regimes tested for selected mixtures. However, these results can be correlated with FN results where same mixtures performed reasonably well among the other mixtures.

Table 5.8: Accumulated Strain Values at Termination for Asphalt Mixtures

Wearing Course			**Base Course**		
Mixtures	Cycles	Acc. Strain	Mixtures	Cycles	Acc. Strain
NHA –A	> 10000	7894	NHA-A	> 10000	9337
NHA – B	> 10000	9658	NHA-B	> 10000	10286
SP–1	> 10000	11319	SP–2	> 10000	7831
MS-2	> 10000	10530	DBM	> 10000	5769

Flow time value for all mixtures is greater than or equal to ten thousand cycles as tertiary flow is not exhibited. Figure 5.20 is the plot showing a comparison of accumulated strain at termination.

Base course mixtures are tested for flow time test with same parameter and procedure as of wearing course mixtures. The results obtained are presented in table 5.11 and in Figure 5.21. At intermediate cycles, the same performance trend is observed but with minimum differences among the mixtures accumulated strain. As we moved towards the maximum number of cycles, strain rate increases but the ranking trend remained same. So, the accumulated axial strain is tabulated as well as plotted against maximum number of cycles just to show some significant difference of strain values of mixtures with each other.

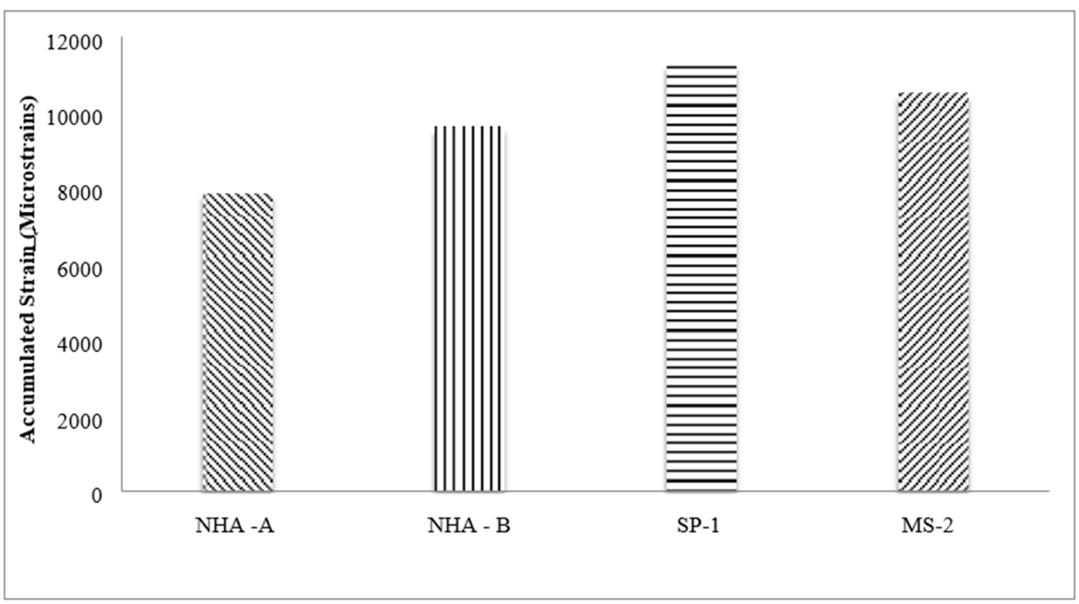


Figure 5.18: Accumulated strain values at same cycle number 10,000 – Asphalt Wearing Course

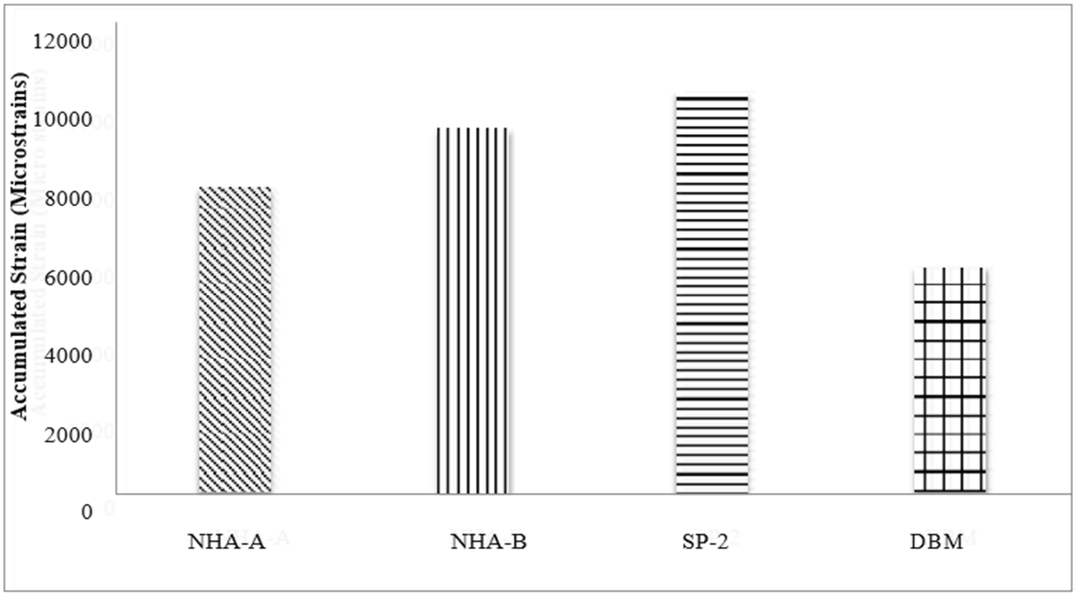


Figure 5.19: Accumulated strain values at termination – Asphalt Base Course

5.4 Conclusions

Dynamic Modulus Testing of AC Mixtures

- Dynamic modulus testing of different asphalt concrete mixtures at various temperatures (4.4 to 54.4°C) and frequencies (0.1 to 25 Hz) was carried out. Laboratory obtained results were employed to two-level factorial design analysis to determine the factors affecting the dynamic modulus and phase angle.
- Results show that test temperature and loading frequency have significant impact on |E*| and phase angle, whereas; NMAS is significant in wearing course and insignificant in base course mixtures. Other factors such as binder content, gradation and aggregate source have no influence on the measured dynamic modulus.
- Sensitivity of dynamic modulus values to the input parameters revealed that for a given loading frequency, an increase in temperature (from 21.1 to 37.8 °C), translated into 45% and 43% drop in |E*| values on average for wearing and base course mixtures, respectively. Similarly, for a given temperature, an increase in loading frequency (from 0.1 to 25Hz), 80% and 67% of variation in |E*| values on average was attributed for wearing and base course mixtures, respectively.
- The tested gradations/mixtures using Margalla aggregate and 60/70 penetration grade bitumen, NHA-A wearing course mix and DBM base course mix are relatively more stiff exhibiting higher values of dynamic modulus. However, on evaluating the resistance to fatigue cracking, Superpave wearing course mix and NHA-B base course mix have shown relatively better resistance to fatigue cracking.
- Isothermal and Isochronal plots revealed that Superpave wearing mix has higher phase angle and less drop is observed hence it shows viscous behaviour of mix. The DBM and Superpave mixtures of base course have higher phase angle and behaves as viscous at higher temperature while DBM mix is more of elastic in nature at lower temperatures.

Flow Number and Flow Time Testing of AC Mixtures

- The wearing course mixtures are ranked on basis of their performance shown by all three analysis as; NHA-A>NHA-B> MS-2 >SP-1. SP-1 is not recommended for traffic level with ≥ 30Million ESALs.
- For base course mixtures, results showed that performance of DBM mix is consistently better in all comparisons. Flow number values at start of third phase (tertiary phase) of deformation shows that DBM mix may be the best performing mix against rutting distress.
- The base course mixtures are ranked on basis of their comparison of performances as; DBM>NHA-A>NHA-B> SP-2.
- Given the tested gradations/mixtures using limestone aggregate and 60/70 penetration grade bitumen, the FT test results indicate that NHA-A mix has 0.65 times lesser accumulated strains on average at the time of termination of the test than other tested mixtures.
- FN and FT test results would facilitate the pavement design engineers and practitioners to select suitable rut-resistant mixtures and helps in performance prediction of AC mixtures.

6 Resilient Modulus

6.1 Introduction

Resilient modulus of hot mix asphalt (HMA) is an important parameter in analysis of pavement structural response under repeated traffic loading. This research study attempts to characterize different asphalt concrete mixtures using resilient modulus test. The resilient modulus is elastic modulus that is used in the layered elastic theory for pavement design. Although asphalt concrete is not truly elastic and is susceptible to permanent deformation with each load application, if the magnitude of repeated loading is small compared with the strength of the material, the deformation under each load repetition is almost recoverable that can be considered elastic. Permanent deformation in asphalt concrete mix may occur at the early stage of load applications due to densification, which is characterized as a plastic strain. As the number of load repetitions increase, the plastic strain due to each load repetition becomes minimal.

In laboratory the resilient modulus of an asphalt mix can be determined by various forms of repeated load tests. In the repeated load indirect diametral tension test, a compressive load with a haversine waveform is applied through a loading strip to the vertical diametral plane of a cylindrical specimen and the successive horizontal recoverable deformation is measured. If an asphalt layer of typical thickness is exposed to a bending action, then the radial instead of the vertical stiffness of the asphalt layer will resist the applied stress. Thus, the diametral test provides a more appropriate assessment of the stiffness of the asphalt layer than tests performed in the vertical direction. Therefore, diametral test results are mostly attractive for estimating radial tensile strain for fatigue analysis. The diametral test has additional benefits since thin cores can also be tested which allows more measurements over the depth of thick asphalt layers.

6.2 Factors Affecting Resilient Modulus

There are numerous factors that affect the resilient modulus of asphalt mixtures using indirect diametral tension test arrangement. These include temperature, loading waveform, pulse duration applied to the specimens, thickness and diameter of specimen and nominal maximum size of aggregate of a particular gradation used in a mixture. Numerous research studies have been carried out to evaluate parameters that influence the resilient modulus of asphalt mixtures. A brief description of factors affecting the resilient modulus of asphalt concrete mixtures is as follows:

6.2.1 Temperature

Temperature is the most important factor for performance of pavement structure since it largely affects the resilient modulus, fatigue properties and the plastic strains within the asphalt concrete mixtures. For temperature higher than 20°C, resilient modulus of HMA decreases quickly and reaches to much lower values at 40°C. Stroup et al. (1997) conducted a wide investigation on the effect of temperature and load duration on resilient modulus of asphalt mixtures. The load duration of 0.1 and 1.0 sec at various temperatures including -18°C, 1°C, 25°C and 40°C were investigated. It was found that as the load duration increases, the resilient modulus reduces at all temperatures except at -18°C. At -18°C, a marginal increase in the resilient modulus value was observed. Ziari et al. (2005) concluded in their investigation that the resilient modulus rapidly reduces with increasing temperature largely due to softening of the bitumen. Kamal et al. (2005) investigated the resilient performance of asphalt concrete mixture by varying temperatures and reported a reduction of almost 85% in resilient modulus for rise in temperature from 25°C to 40°C.

6.2.2 Load Duration and Magnitude

The influential effect of the load pulse length and magnitude on the performance of asphalt mix has been widely investigated. Almudaiheem et al. (1991) concluded that the percentage of indirect tensile strength of specific asphalt mixture used as a cyclic load affects the resilient modulus value. Tests were performed on the specimens having cyclic load ranging from 10% to 30% of indirect tensile strength of similar specimen having identical mixture properties. On average 4% difference in resilient modulus values was found for the samples having 4% asphalt content at load degree of 1000 and 2700 N. Loulizi et al. (2002), concluded that being a viscoelastic material the loading time affects the properties of asphalt mix and suggested that in dynamic tests the load cycle time should be 0.03sec to simulate loading duration of moving trucks at an average speed. Saleh et al. (2006) reported that the load pulse length had substantial effect on resilient modulus values. They found that the resilient modulus reduced with the increase in the load pulse period (i.e., longer time duration) due to the development of high strains whereas the load pulse form and strain level had insignificant effect on the resilient modulus of asphalt concrete mixtures.

6.2.3 Specimen Diameter

A 4-inch (100mm) or 6-inch (150mm) diameter specimens having thickness range from 1.5 (38mm) to 2.5-inch (63mm) can be used for determination of resilient modulus of asphalt mixtures through indirect diametral tension test arrangement. The test specimens can be fabricated in the laboratory or obtained from field coring. Lim et al. (1995) carried out resilient modulus (diametral testing) on specimens with varying diameters. The specimen diameter range included 4-inch, 5-inch and 6-inch with identical diameter/height ratio of 1.6. It was found that with similar aggregate gradation and bitumen content, the resilient modulus reduced with the increase in specimen diameter. They concluded that specimen diameter i.e., geometry of the testing specimen may have influence on the resilient modulus values.

6.2.4 Aggregate Gradation

Lim et al. (1995) studied the influential effect of specimen diameter to maximum nominal stone size fraction on resilient modulus. It was observed that resilient modulus reduces as the ratio of specimen diameter to maximum nominal aggregate size improved. It was concluded that greater resilient modulus values may result while using a smaller diameter specimen with large stone size. Pan et al. (2005) performed laboratory tests to study the influential effect of material properties on the resilient modulus of asphalt mixture and found that the coarse aggregate morphology is the dominant factor that affects the resilient modulus. They observed that by using coarse aggregates having uneven morphologies enhance the resilient modulus values at temperature of 25°C with different asphalt combinations. It was also observed that different aggregate gradations did not considerably affect the correlation between the coarse aggregate morphology and the resilient modulus of asphalt mixtures. Saleh et al. (2006) carried out research to relate factors that influence the resilient modulus on 4-inch and 6-inch laboratory compacted specimens. Higher resilient modulus values were observed with coarser gradations and concluded aggregate interlocking to be a dominant factor. Similarly, Jahromi et al. (2009) also found that the maximum nominal size was the most substantial factor influencing the resilient modulus.

6.3 Experimental Design

This research includes dynamic load testing (resilient modulus test) carried out on various asphalt mixtures prepared in the laboratory with different aggregate gradations pertaining to asphaltic wearing course. This study incorporates four (04) aggregate gradations of wearing course and two (02) bitumen penetrations grades from two sources. Asphalt wearing coarse aggregate gradations include NHA-A, NHA-B, Superpave-1 & MS-2 and bitumen penetration grades include NRL 40/50, NRL 60/70 & ARL 60/70.

The detailed experimental design/test matrix was established including different variables before the performance testing. Table 6.1 shows the detailed test matrix for performance testing of selected asphalt wearing course concrete mixtures.

Table 6.1: Test Matrix of Performance Testing

Temperature (°C)	Loading (ms)	Gradation	Margalla				Ubhan Shah		
			4 inch Dia		6 inch Dia		4 inch Dia		
			NRL 40/50	ARL 60/70	NRL 40/50	ARL 60/70	NRL 40/50	NRL 60/70	ARL 60/70
25	100	NHA–A	√	√	√	√	√	√	√
		NHA–B	√	√	√	√	√	√	√
		SP–1	√	√	√	√	√	√	√
		MS–2	√	√	√	√	√	√	√
	300	NHA–A	√	√	√	√	√	√	√
		NHA–B	√	√	√	√	√	√	√
		SP–1	√	√	√	√	√	√	√
		MS–2	√	√	√	√	√	√	√
40	100	NHA–A	√	√	√	√	√	√	√
		NHA–B	√	√	√	√	√	√	√
		SP–1	√	√	√	√	√	√	√
		MS–2	√	√	√	√	√	√	√
	300	NHA–A	√	√	√	√	√	√	√
		NHA–B	√	√	√	√	√	√	√
		SP–1	√	√	√	√	√	√	√
		MS–2	√	√	√	√	√	√	√
Total			**336 Specimens with 3 replicates**						

6.3.1 Sample Preparation

After determination of optimum asphalt content (OAC) for each mix, samples for performance testing were prepared using Gyratory Compactor. The development of Superpave Gyratory Compactor aids to improve mix design to simulate actual field compaction and particle orientation in the laboratory. Sample size of 150mm (6-inch) in diameter and 190.5 mm (7.5-inch) in height were subjected to 125 numbers of gyrations as specified in Superpave mix design for traffic loading ≥ 30 million ESALs. For preparation of samples having the same void content and volumetric properties as in the Marshall Mix design, weight of aggregate was calculated through back calculation. With the help of specimen's volume and maximum specific gravity (Gmm) value of mixture the weight of aggregate was calculated for specimen preparation having the same volumetric properties at target 4% air voids.

Once compacted in the Gyratory compactor, specimens for performance testing were prepared using core and saw cutting machines. For each mix type 5 numbers of specimens were prepared, two specimens for indirect tensile strength testing and three specimens for resilient modulus testing, respectively.

6.3.2 Laboratory Testing

In this research study, repeated load indirect diametral tension test setup was selected to determine the resilient modulus of selected hot mix asphalt mixtures due to its simplicity and availability in the laboratory. Tests were conducted in Universal Testing Machine (UTM-25). Indirect tensile strength of asphalt mixtures was obtained prior to resilient modulus testing.

6.3.2.1 Indirect Tensile Strength Determination

Prior to the resilient modulus test, two specimens were subjected to indirect tension test (ASTM D 6931) and average of two values was taken. The specimen was positioned in the test jig above the bottom loading plate and then top loading plate was placed to grip the specimen. Test was conducted by applying a compressive load across its vertical diametral plane at a controlled deformation rate of 50mm/min at 25°C. The stress at which failure occurred was taken as the indirect tensile strength of the specimen. For resilient modulus test, 20% of that strength value was taken. Indirect tension test was performed for each mix type with the help of two replicates each using two types of sample diameters (4-inch and 6-inch).

6.3.2.2 Resilient Modulus Testing

After performing indirect tension test, the actual resilient modulus tests were performed on the remaining three specimens of each mixture. The metallic fixtures for LVDTs (linear variable differential transducers) were installed in the jig. The LVDTs are used for measuring the linear horizontal displacement. The specimen was then fixed into the jig with the help of clamping screws between the loading plates. Afterwards, the jig was shifted for resilient modulus testing into universal testing machine chamber. The LVDTs were installed and adjusted to operate within their range.

As mentioned in ASTM D 4123, the peak loading force was taken as 20% of the failure load and seating force was kept 10% of the peak loading force. Poisson's ratio was assumed as 0.4. With target test parameters i.e., temperature (25°C or 40°C), load pulse width (100ms or 300ms), pulse repetition period (1000ms), and conditioning pulse count (100), specimen was subjected to haversine loading. The indirect tension modulus software recorded and presented the force and displacement as the conditioning stage continued. At the end of the conditioning stage, i.e., after 100 conditioning pulses, the levels display automatically invoked. The out of range LVDTs were adjusted and by closing the level display window, automatically 5 pulses of nearly constant deformation were applied to conclude the test.

After completing conditioning pulse deflections, load values were recorded by the software for the last 5 pulses. These values were used to determine the resilient modulus with mean value calculated by the software. Three resilient modulus values were determined for each mix type at two different load duration periods, at two different temperatures and for two different diameter specimens, respectively.

6.4 Experimental Results

This section contains the detailed analysis of results obtained from the resilient modulus testing. For analysis Microsoft Excel and the most understandable statistical software recognized as MINITAB-15 was used. The results obtained from data analysis are presented with graphs such as relative performance plots, normal probability plot, half normal probability plot and factorial plots

6.4.1 Indirect Tensile Strength Results

Indirect tension test was performed for each mix type with the help of two replicates each using two types of sample diameters (4-inch and 6-inch). Table 6.2 & 6.3 shows the values for indirect tensile strength test for all the selected wearing course mixtures, respectively:

Table 6.2: Results of Indirect Tensile Strength Test for 4-inch Dia Specimens

Aggregate Source & Gradations		Bitumen Type	Specimen Thickness (Inch)	Test Temperature (°C)	Peak Force (kN)
Margalla	NHA-A	NRL 40/50	2	25	8.103
	NHA-B		2	25	7.811
	SP-1		2	25	7.489
	MS-2		2	25	7.840
	NHA-A	ARL 60/70	2	25	7.725
	NHA-B		2	25	8.105
	SP-1		2	25	6.721
	MS-2		2	25	7.618

Table 6.3: Results of Indirect Tensile Strength Test for 6-inch Dia Specimens

Aggregate Source & Gradations		Bitumen Type	Specimen Thickness (Inch)	Test Temperature (°C)	Peak Force (kN)
Margalla	NHA-A	NRL 40/50	2	25	8.190
	NHA-B		2	25	8.627
	SP-1		2	25	8.771
	MS-2		2	25	9.039
	NHA-A	ARL 60/70	2	25	8.739
	NHA-B		2	25	8.677
	SP-1		2	25	8.441
	MS-2		2	25	9.217

6.4.2 Resilient Modulus Test Results

In this research, five factors were considered where two factors pertain to material and three factors are condition based. Material based factors are the aggregate gradation and the bitumen type while the condition based factors pertain to testing regime such as test temperature, load pulse duration and diameter of specimen. Table 6.4 shows the factors considered in the resilient modulus testing. Each experimental condition was replicated three times to obtain a reasonable estimate of experimental error. Table 6.5 and 6.6 shows the average values of three replicates specimens obtained from resilient modulus testing for each experimental condition.

Table 6.4: Factors Considered in Resilient Modulus Testing

Sr. No.	Factors	Factor Type	Units
1	Nominal Maximum Aggregate Size	Material Based Factors	Mm
2	Bitumen Type/source		0.1mm
3	Temperature	Condition Based Factors	°C
4	Load Pulse Duration		ms
5	Diameter of Specimen		Inch

Table 6.5: Resilient Modulus Test Results for Margalla Aggregate

Gradation	Temperature (°C)	Load Duration (ms)	Resilient Modulus (MPa)			
			NRL 40/50		ARL 60/70	
			4 inch Dia Specimen	6 inch Dia Specimen	4 inch Dia Specimen	6 inch Dia Specimen
NHA–A	25	100	11431	8958	8758	7390
		300	7176	6255	6080	4518
	40	100	4840	3364	2559	1770
		300	2169	1985	1657	1081
NHA–B	25	100	10402	8958	8382	7908
		300	7019	5832	5835	4866
	40	100	4825	3333	2313	2139
		300	2273	1895	1174	976
SP–1	25	100	9585	7251	5541	5105
		300	6262	4582	3338	3182
	40	100	2849	2193	1184	1017
		300	1476	1260	653	562
MS–2	25	100	9521	7426	8103	6802
		300	6481	5446	5564	3473
	40	100	2751	2431	1694	1322
		300	1540	1370	977	844

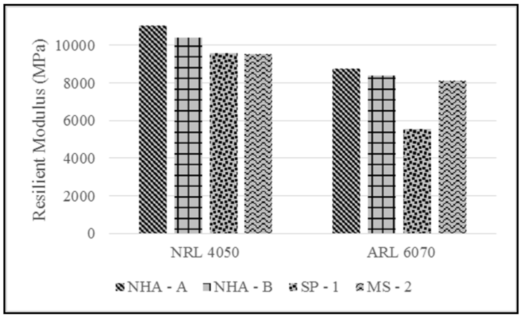
Table 6.6: Resilient Modulus Test Results for Ubhan Shah Aggregate

Gradation	Temperature (°C)	Load Duration (ms)	Resilient Modulus (MPa)		
			NRL 40/50	NRL 60/70	ARL 60/70
			4 inch Dia Specimen	4 inch Dia Specimen	4 inch Dia Specimen
NHA–A	25	100	10236	9276	9276
		300	8508	7398	7398
	40	100	5401	4013	4013
		300	2617	2163	2163
NHA–B	25	100	9331	8453	8453
		300	7038	6388	6388
	40	100	5379	3965	3965
		300	2267	1754	1754
SP–1	25	100	8093	7201	7201
		300	6031	5276	5276
	40	100	3163	2037	2037
		300	1533	997	997
MS–2	25	100	8656	7534	7534
		300	7247	6010	6010
	40	100	3027	2576	2576
		300	1377	1204	1204

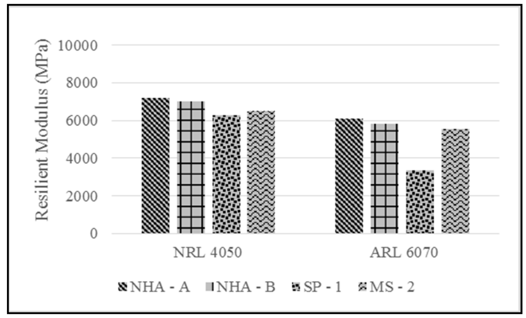
6.4.3 Relative Performance Plots

The main objective of this research was to characterize the various asphalt concrete mixtures and to analyse the relative performance of the mixtures using different wearing course gradations and different binder types/sources. Among the mixtures of Margalla aggregate with NRL 40/50 binder; NHA-A relatively performed consistently well followed by NHA-B, MS-2 and Superpave-1 in all test conditions. Moreover, 39% decrease in the resilient modulus values was observed due to the load pulse duration change from 100ms to 300ms simulating the fast and slow vehicle speeds of 64 km/h and 22 km/h, respectively. Almost 68% decrease in the resilient modulus values was observed due to the temperature change from 25°C to 40°C. Resilient modulus values of specimens having diameter 6 inch (150 mm) were found comparatively lower than that of diameter 4 inch (100 mm) and percentage decrease was 17%.

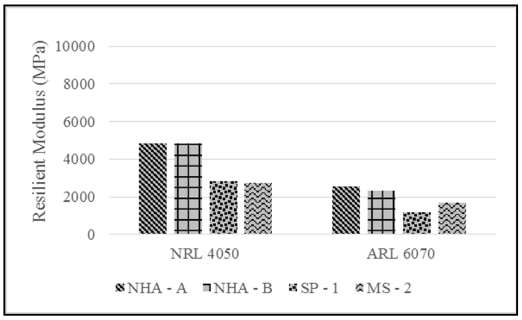
Similar findings were observed in mixtures comprising ARL 60/70 binder; where NHA-A relatively performed well followed by NHA-B, MS-2 and Superpave-1 consistently in all test conditions. Moreover, 40% decrease in the resilient modulus values was observed due to the load pulse duration change from 100ms to 300ms simulating the fast and slow vehicle speeds of 64 km/h and 22 km/h, respectively. Almost 78% decrease in the resilient modulus values was observed due to the temperature change from 25°C to 40°C. Resilient modulus values of specimens having diameter 6 inch (150 mm) were found comparatively lower than that of diameter 4 inch (100 mm) and percentage decrease was 18%. Figure 6.1 and 6.2 shows the charts for relative performance of Margalla aggregate mixtures with NRL 40/50 and ARL 60/70 binder. Figure 6.3 shows the relative performance charts for the Ubhan Shah aggregate mixtures with NRL 40/50, NRL 60/70 and ARL 60/70 binder.



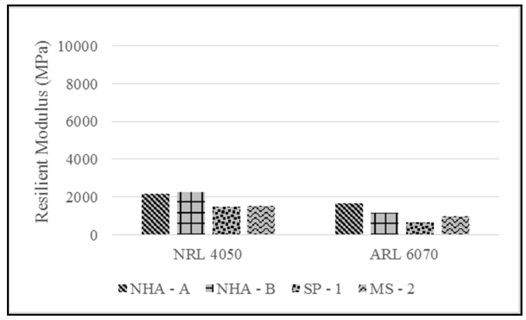
25°C Temperature and 100ms Load Duration



25°C Temperature and 300ms Load Duration

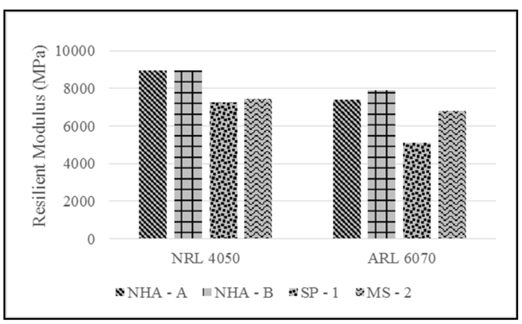


40°C Temperature and 100ms Load Duration

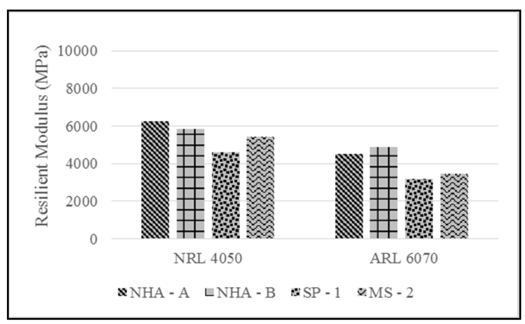


40°C Temperature and 300ms Load Duration

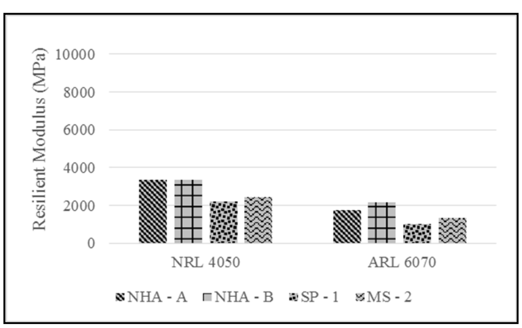
Figure 6.1: Relative Performance of 4 inch Dia. Samples for Margalla Aggregate



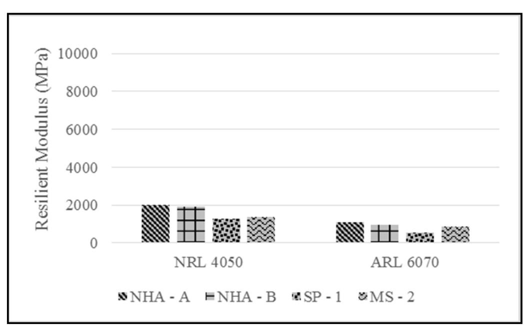
25°C Temperature and 100ms Load Duration



25°C Temperature and 300ms Load Duration

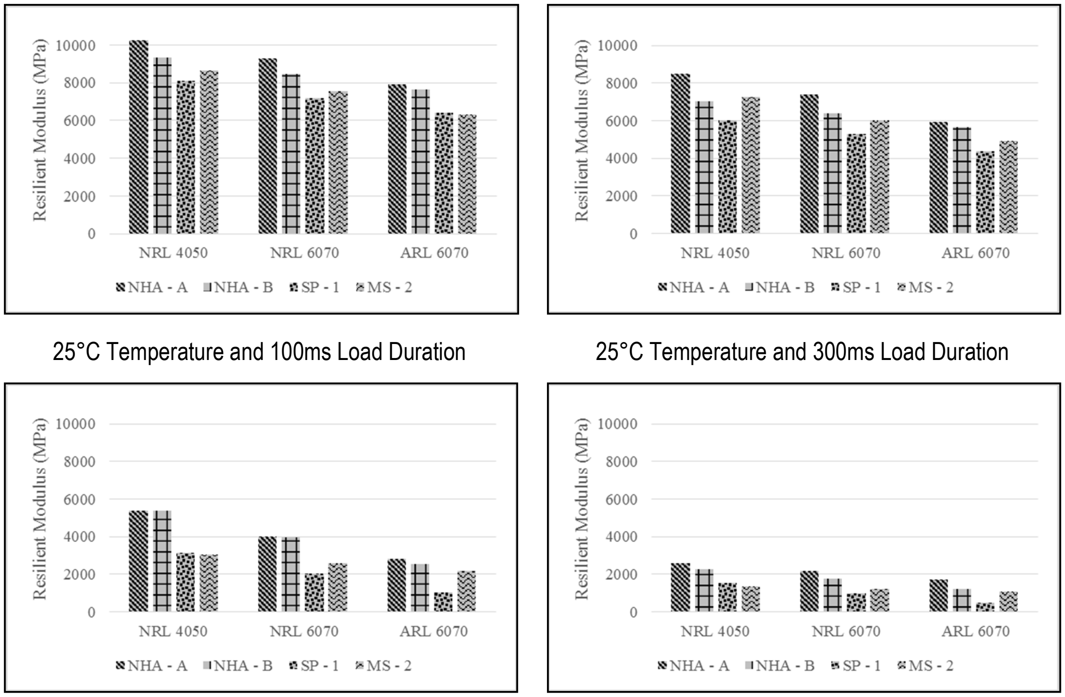


40°C Temperature and 100ms Load Duration



40°C Temperature and 300ms Load Duration

Figure 6.2: Relative Performance of 6 inch Dia. Samples for Margalla Aggregate



25°C Temperature and 100ms Load Duration

25°C Temperature and 300ms Load Duration

40°C Temperature and 100ms Load Duration

40°C Temperature and 300ms Load Duration

Figure 6.3: Relative Performance of 4 inch Dia. Samples for Ubhan Shah Aggregate

Figure 6.4 shows the comparison between low and high temperature on which change in the resilient modulus values due to different specimen's diameter were observed. The change in resilient modulus values from 6-inch specimens to 4-inch specimens was observed relatively similar at 40°C temperature than 25°C as the slopes of both lines are same. So, it is concluded that both the specimen diameters have almost same sensitivity at the temperature change and percentage decrease in the resilient modulus values due to temperature change is similar for both 4-inch and 6-inch diameter specimens.

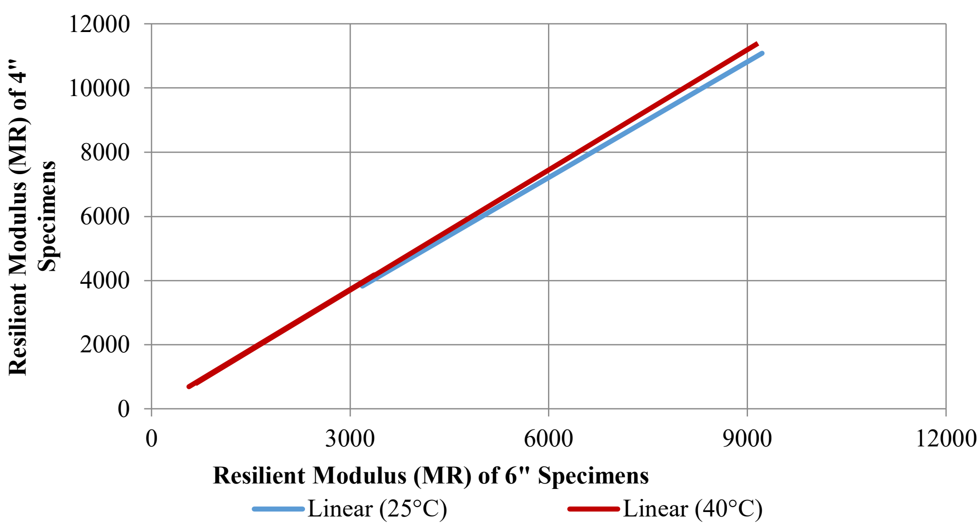


Figure 6.4: Effect of Specimen Diameter Change at Low and High Temperature

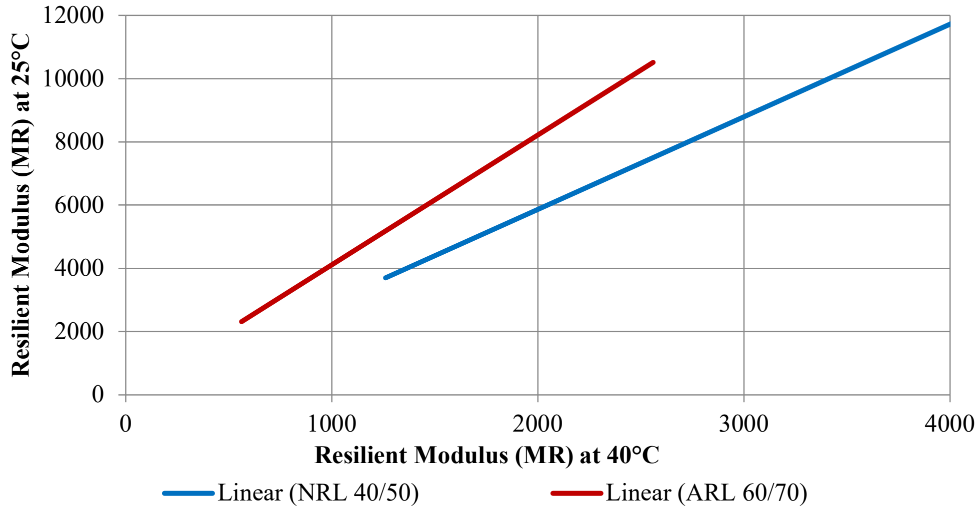


Figure 6.5: Effect of Temperature Change on Bitumen Type/Source

Figure 6.5 shows the comparison between 40/50 and 60/70 binders for which change in the resilient modulus values due to different temperatures was observed. The change in resilient modulus values from 25°C temperature to 40°C temperature was observed relatively higher with ARL 60/70 than NRL 40/50. The percentage decrease in the resilient modulus values due to temperature change was noted to be similar for both 4-inch and 6-inch diameter specimens. Therefore, it can be reasonably concluded that both the specimen diameters have almost similar sensitivity to temperature.

6.4.4 Factorial Design

Factorial design of experiments is a statistical technique to study the effect of factors on the response variable. It is very difficult to study effect of factors when number of factors is more against a single response variable. As such, this technique is very useful to study the effect of individual factors and their combined effect on the response variable. In this research two-level factorial design using statistical software MINITAB. The response variable was the resilient modulus value and five different factors were considered to check their effect on resilient modulus value. Each factor had two levels, low level and higher level. Table 6.7 shows the factors considered in the resilient modulus testing with their respective abbreviations and high and low levels.

Table 6.7: Considered Factors along their Low and High Levels

Abbreviation	Factors	Levels		Units
A	Temperature	25	40	°C
B	Load Pulse Duration	100	300	ms
C	Bitumen Type	45	67	mm-10
D	Nominal Maximum Aggregate Size	19.0	12.5	Mm
E	Diameter of Specimen	4	6	Inch

Table 6.8 shows the estimates of main factor effects as well as their combined effect (2-way interactions). The effect can be defined as it is the difference in response values due to any factor at the two levels (low & high) while the interaction effect can be defined

as the mean difference between effect of one factor at high and low level values of other factor. The design of experiments was conducted at 95% confidence interval with significance level of α=0.05. The significance of any factor can be judged through its *p*-value against the significance level. If *p*-value is less than the 0.05 then the factor is considered as significant. All five individual factors have *p*-value <0.0001 that shows that all individual factors are significant. All 2-way interaction of factors are also significant except Load Duration*Diameter, Bitumen Type*Nominal Maximum Aggregate Size and Nominal Maximum Aggregate Size*Diameter.

Table 6.8: Effects and P-values of Individual Factors and Their Interactions

Factors	Effect	P-value
Temperature	-4850	<0.0001
Load Duration	-1979	<0.0001
Bitumen Type	-1268	<0.0001
Nominal Maximum Aggregate Size	1996	<0.0001
Diameter	-852	<0.0001
Temperature*Load Duration	837	<0.0001
Temperature*Bitumen Type	297	0.001
Temperature*Nominal Maximum Aggregate Size	-226	0.021
Temperature*Diameter	468	0.000
Load Duration*Bitumen Type	202	0.019
Load Duration*Nominal Maximum Aggregate Size	-263	0.007
Load Duration*Diameter	86	0.309
Bitumen Type*Nominal Maximum Aggregate Size	86	0.387
Bitumen Type*Diameter	177	0.039
Nominal Maximum Aggregate Size*Diameter	-35	0.716

Table 6.9 represents the analysis of variance (ANOVA) of the data including main effects and 2-way interaction effects for resilient modulus of HMA mixtures. Degree of freedom (DF) for main effects is 5 and 2-way interacting effects is 10, which means that total 5 main factors and 10 interacting factors explain the variation in the resilient modulus values. Significance of the factors can be judged through the *p*-value which is <0.0001for main factors as well as 2-way interacting factors. F value generally greater than 10 also shows the significance of the factors.

Table 6.9: ANOVA for Resilient Modulus Testing

Source	DF	Sum of Sq	Mean Sum of Sq	F	P
Main Effects	5	3254346781	529877607	654.74	<0.0001
2-Way Interaction	10	128483662	12848366	15.88	<0.0001
Residual Error	149	120585087	809296		
Pure Error	110	48543981	441309		
Total	165	3523087243			

6.4.4.1 Significant Effects and Interaction Plots

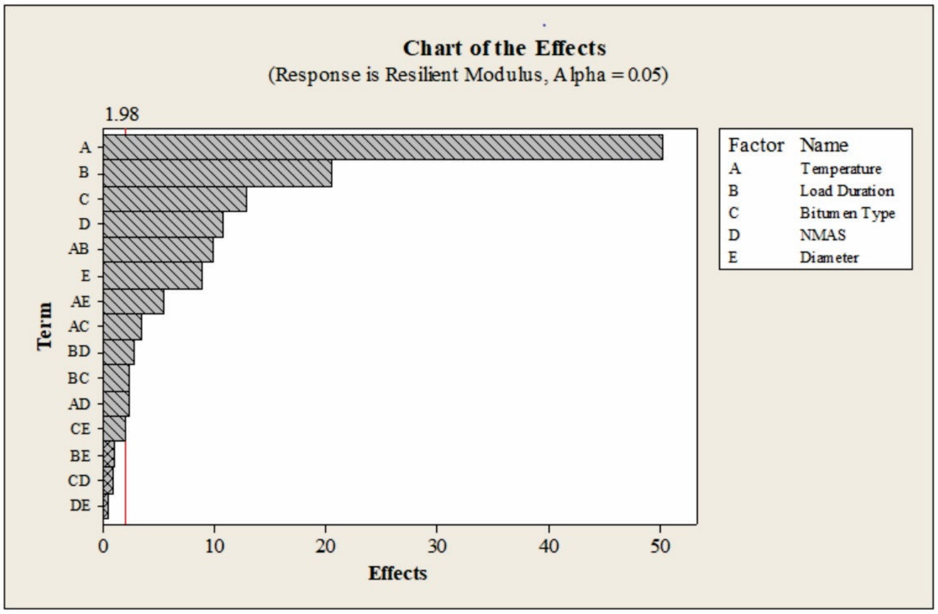


Figure 6.6: Pareto Chart Showing Significant Factors for Resilient Modulus

Figure 6.6 shows the significance of the individual and combined on resilient modulus of HMA mixtures. T-critical reference line is drawn on the chart that indicates the bars crossing the reference line are significant. Bars are showing the significant as well as insignificant factors. Last three bars on the left side of T-critical line are showing insignificant factors. It is very clear that, among all individual factors, temperature (A) was the most significant factor affecting the resilient modulus followed by load duration (B), bitumen type (C), Nominal maximum aggregate size (D) and diameter of specimen (E). Moreover, 2-way interactions of all factors were critical except load duration*Diameter (BE), bitumen type*nominal maximum aggregate size (CD) and nominal maximum aggregate size*diameter (DE).

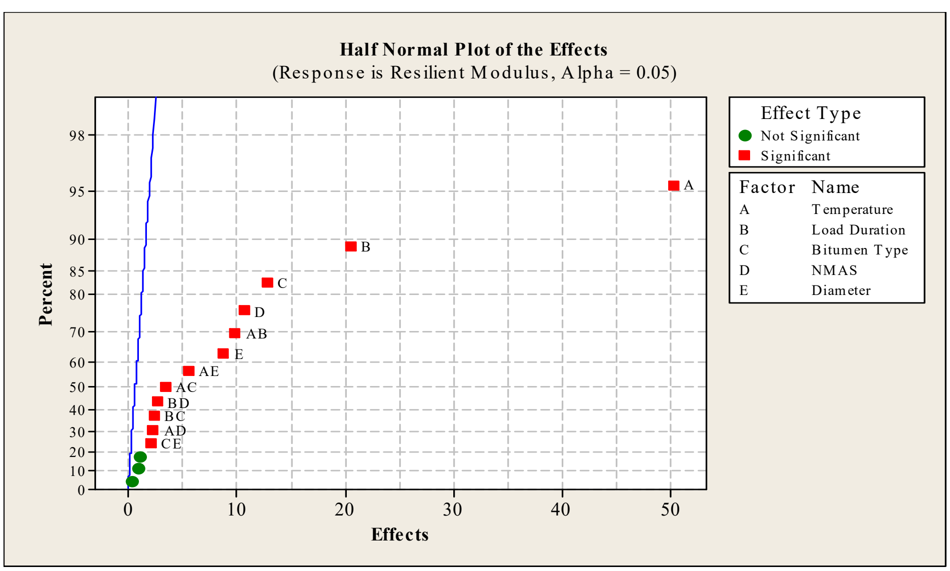


Figure 6.7: Half Normal Plot of Standardized Effect

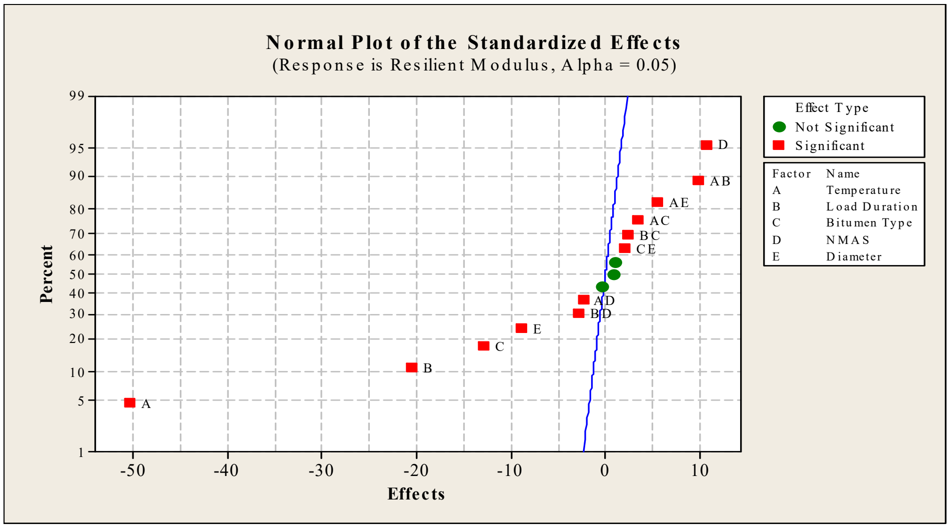


Figure 6.8: Normal Plot of Standardized Effect

Figure 6.7 and 6.8 shows the cumulative half normal and normal probability plots of standardized effects for resilient modulus of HMA mixtures at 95% confidence interval. Plots are also showing significant and insignificant factors with the help of red and black markers respectively:

6.4.4.2 Factorial Plots

Factorial plots tell the effect of main factors and their combined effect on the response variable. Figure 6.9 shows the main factors affecting the resilient modulus value of HMA mixtures. Slope of line shows how much strong the effect is or how much the factor is significant. All five main factors have inverse relationship with the resilient modulus value except nominal maximum aggregate size which has direct relationship with resilient modulus value:

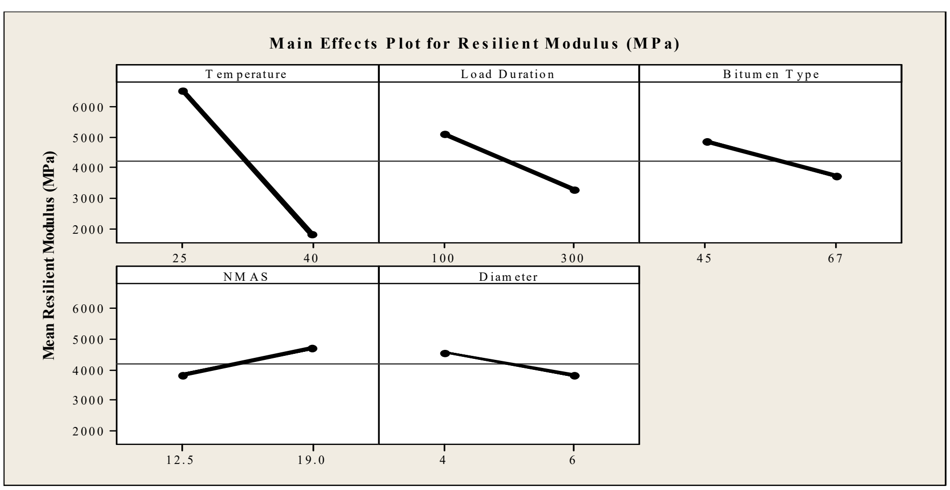


Figure 6.9: Main Effect Plot for Resilient Modulus

Figure 6.10 illustrates the interaction plots for all five factors in which 2-way interaction of factors is plotted. Interaction of temperature with all other four factors is significant as the distance between the lines is more while in the case of nominal maximum aggregate size with specimen diameter lines are parallel and are closed to each other showing insignificance:

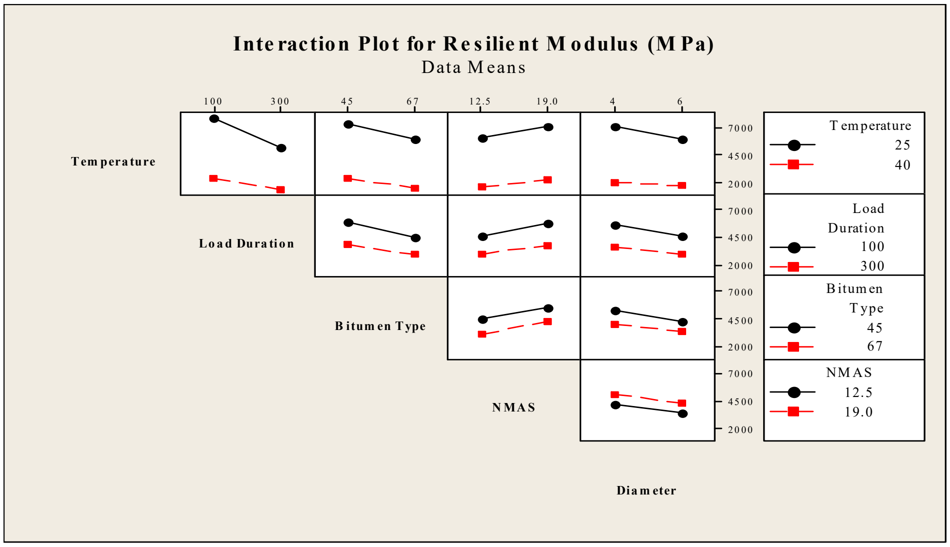
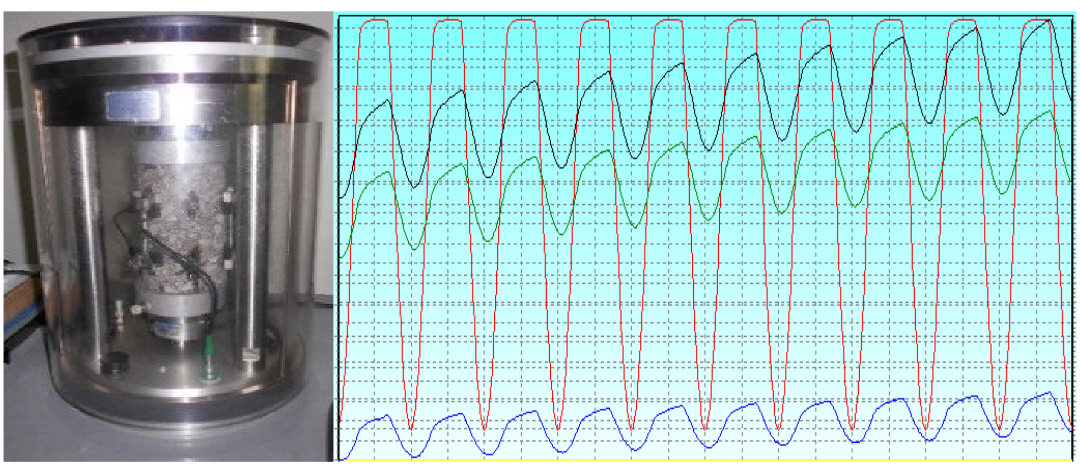


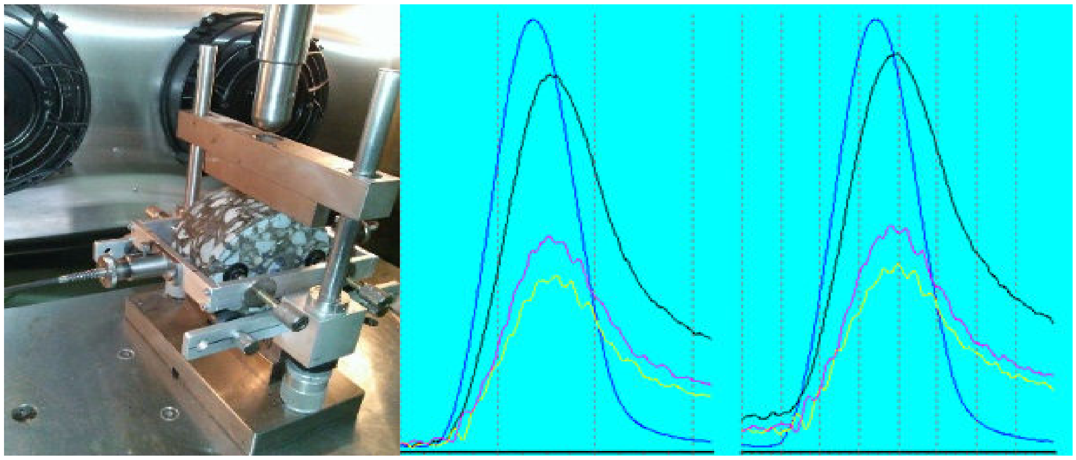
Figure 6.10: Interaction Effects Plot for Resilient Modulus

6.4.5 Discussion and Comparison of Resilient Modulus (MR) and Dynamic Modulus (|E*|) Values

Witczak (1999) carried out a study, which was summarised in NCHRP 1-37A project, that differentiated between |E*| and MR test for AC mixtures. The key differences like loading pattern, loading period, and specimen geometry as shown in Figure 6.11 were reported. Several other research studies also pinpointed various differences between |E*| and MR (Witczak, 1999; Drescher et al. 1997; Zhang et al. 1997; Kim et al. 2004), which are summarized in Table 6.10.



(a) Dynamic modulus



(b) Resilient modulus

Figure 6.11: Test setups and loading patterns

Table 6.10: Comparison between |E*| and MR Test Methods

S. No	Parameter	Dynamic Modulus	Resilient Modulus
1	Loading Pattern	Continuous compressive sinusoidal loading with no rest period	Waveform haversine loading with given rest/loading period
2	Effect of Geometry	Restricted to height to diameter ratio of 1.5, which nullify the effect of geometry	Can be performed on 100 and150mm diameter specimen which significantly influence stiffness parameter
3	Test temperature	Generally performed at four/five test temperature	Usually performed at room temperature and/or 40 °C
4	Loading Frequency	Performed at varying frequencies ranging from 0.01 to 25 Hz	Restricted to perform at single loading frequency of 1 Hz.
5	Testing time	Requires 6-8 hours	Requires 1-2 hours
6	Preparation of Specimens	Laborious and long	Laborious but short

Apart from this, several researchers attempted to develop the correlation between these two test methods (Birgisson et al. 2004; Loulizi et al. 2006; Ping and Xiao (2006)). Birgisson et al. (2004) determined the tensile dynamic complex modulus from indirect tensile strength (IDT) tests based on their developed testing and analysis protocols. This study found dynamic complex modulus to be in agreement with MR and testing frequency for a given range of testing temperatures and loading frequencies. Loulizi et al. (2006) studied the relationship of |E*| and MR tests and confirm that there exists a correlation between these two tests methods. Similarly, Ping and Xiao (2006) carried out a comparative study on |E*| and MR and report that |E*| performed at 4 and 5 Hz was strongly correlated with MR performed at a frequency of 1 Hz (0.1 sec). These researches either used a single testing temperature and/or loading duration to determine the relationship between MR and |E*|. Building further upon this, the developed correlations in these studies were not rigorously tested e.g., using their own dataset as well as data acquired from already published literature. The rigorous testing of any developed correlations is an essentially important consideration; and low predictive power of these correlations poses concerns for the pavement analysts and transport agencies to solely rely on the correlations for prediction of |E*| for design purposes. However, the literature is devoid of any evidence of rigorous testing of developed correlations and our understanding on this important aspect of |E*| and MR relationship remains elusive. Given the differences between these two test methods and earlier work carried out by various researchers, this study aimed to derive a correlation between |E*| and MR in order to save the agencies' inventory and ensure smooth transition from empirical to M-E design.

During sample preparation and testing, it was ensured that air voids are within limits of 4% ± 0.5 for both performance tests i.e., |E*| and MR, so that effect of air voids on performance of mixtures could be nullified. A comparison between MR and |E*| values was carried out to ascertain the frequency at which |E*| complement MR values at 25 °C and/or 40 °C. To determine the appropriate loading frequency, two approaches are used: (a) by using loading time (0.03 sec) to calculate the equivalent angular frequency ($\omega = 1/t$), which would be approximately 33 rad/s, and dividing the obtained angular frequency by 2π to obtain the loading frequency (which in our study is found to be 5 Hz) (Loulizi et al. 2006); and (b) by using simple linear regression approach at various loading frequencies. From simple linear regression analysis (Table 6.11), it can be inferred that |E*| is increasing with increase in MR for a particular loading frequency. Further, the comparison of various loading frequencies (25 Hz to 0.1 Hz) is presented in Table 6.11 (note that comparisons at 40 °C are found to be insignificant and meaningless trends; see Table 6.11). A strong correlation is observed for |E*| values at 5 Hz loading frequency and 25°C temperature (|E*| at this temperature is obtained using interpolation) with the MR at same temperature and 300ms loading duration (R^2 = 0.85).

Table 6.11: Relationship of |E*| and MR

Relationship	R^2	Temperature (ºC)	Frequency (Hz)
$\lvert E^*\rvert = 1.6071\text{MR}$	0.58	25	25
$\lvert E^*\rvert = 1.3084\text{MR}$	0.67		10
$\lvert E^*\rvert = 1.0764\text{MR}$	0.85		5
$\lvert E^*\rvert = 0.74941\text{MR}$	0.58		1
$\lvert E^*\rvert = 0.5638\text{MR}$	0.57		0.5
$\lvert E^*\rvert = 0.0823\text{MR}$	0.08		0.1
$\lvert E^*\rvert = 0.6623\text{MR}$	0.02	40	25
$\lvert E^*\rvert = 0.7331\text{MR}$	0.08		10
$\lvert E^*\rvert = 0.2789\text{MR}$	0.30		5
$\lvert E^*\rvert = 0.4456\text{MR}$	0.44		1
$\lvert E^*\rvert = 0.5388\text{MR}$	0.10		0.5
$\lvert E^*\rvert = 0.0073\text{MR}$	0.003		0.1

To further confirm the relationship expressed by the developed correlation, the closest interpreted loading frequency which yields an equal value of |E*| from MR would be approximately 5 Hz (Figure 6.12). Hence, this complements our previous findings and it can be concluded that |E*| values obtained at 5 Hz loading frequency may be compared to MR values for a given temperature (25 °C) and loading duration (300 ms).

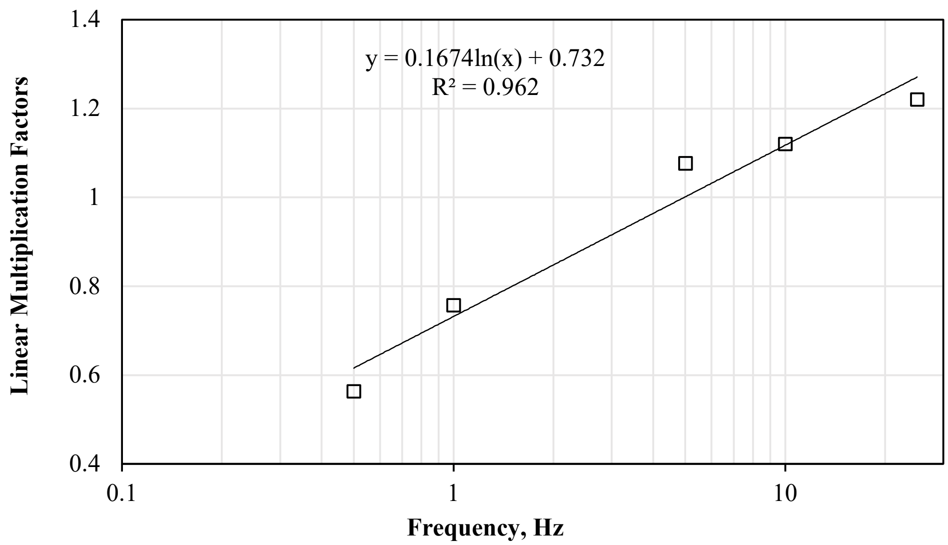


Figure 6.12: Relationship of linear multiplication factors with |E*| loading frequency

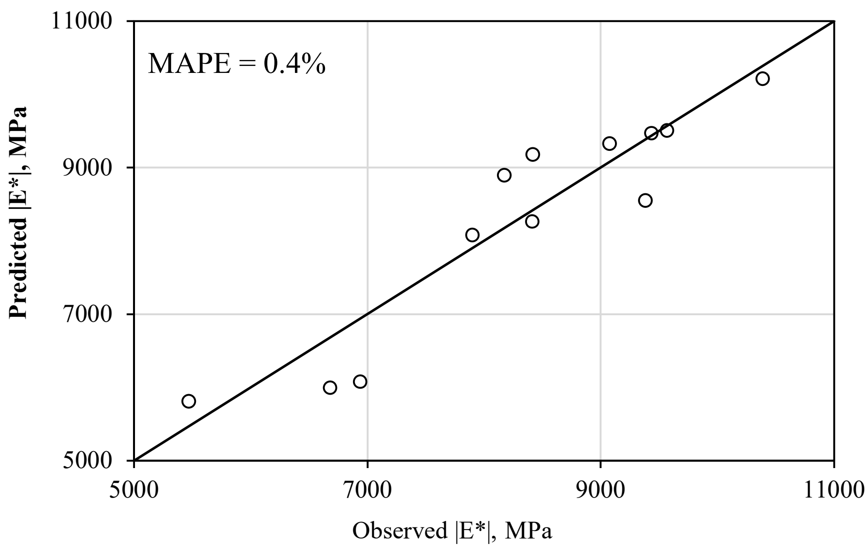


Figure 6.13: Validation plot

One of the main drawbacks of earlier studies is that they failed to provide validation results of the developed correlations. To this end, this study proposed a bi-level testing protocol to assess the predictive capability of the developed correlation. At the first level, the current study's dataset was validated by calculating the Mean Absolute Percent Error (MAPE). The MAPE calculated for the developed correlation is 0.004, which suggests that the developed correlation, on average, under/over predicted the |E*| from true values by 0.4%. To further support this argument, observed |E*| (from the dataset) was plotted against predicted |E*| (from the developed correlation) as shown in Figure 6.13. It is evident from the validation plot that observed and predicted |E*| are very close to Line of Equality (LOE), suggesting that the developed correlation predicts reasonably well.

To further investigate the veracity of the developed correlation, a cross-validation was performed at the second level of testing. For cross-validation, data of seven plant produced AC mixtures were acquired from available literature. |E*| were taken from [28] while MR of same mixtures were reported in Hussain et al. (2016). The testing conditions for both |E*| and MR were similar to the conditions adopted in this study, and thus the results obtained from these studies can serve as good input to verify the veracity of the developed correlation. The testing matrix of AC mixtures used for cross-validation is presented in Table 6.12.

Table 6.12: Details of AC Mixtures Used for Cross-Validation

Variables	Description						
Test Performed	|E*| and MR						
Testing Conditions	|E*| is performed at four test temperatures and six loading frequencies; MR (25 °C and 40 °C) at loading condition of 100 and 300 ms						
Specimen Preparation	Superpave gyratory						
OBC calculation	Marshall mix design procedure						
Mix volumetric, |E*| at 5 Hz and 25 °C and MR at 100ms and 25 °C							
Mix Type/Name	Mix-1	Mix-2	Mix-3	Mix-4	Mix-5	Mix-6	Mix-7
Binder Content (%)	4.0	4.4	4.2	4.2	3.3	3.4	3.5
Air Void (%)	5.6	4.76	5.37	5.76	5.4	6.2	4.9
VMA (%)	14.8	15.3	14.2	13.4	13.7	13.2	12.8
VFA (%)	66.9	63.5	66.5	59.5	58.1	59.2	58.0
Average |E*|, MPa	11811	14720	13803	17037	14734	15141	13348
Average MR, MPa	6441	7107	6746	5940	5477	5172	6233
Metric (U.S) Sieve	Gradation (% Passing)						
37.5 mm ($1\frac{1}{2}$ in)	100	100	100	100	100	100	98.8
25 mm (1 in)	100	100	100	100	82.6	85.4	82.7
19 mm ($\frac{3}{4}$ in)	100	99.9	99.7	100	69.7	71.0	67.4
12.5 mm ($\frac{1}{2}$ in)	82.5	79.5	84.5	80	59.2	59.9	56.2
9.5 mm ($\frac{3}{8}$ in)	68.7	69.4	63.4	67	52.2	53.4	43.5
4.75 mm (No. 4)	49.3	48.7	46	48	41.6	35.5	35.0
2.36 mm (No. 8)	30.8	35.3	30.5	33	24.5	26.4	25.1
0.3 mm (No. 50)	10.5	11.8	10.3	11	9.4	10.8	7.6
0.075 mm (No. 200)	5.2	5.2	5.0	4.7	5.0	4.7	4.2

Analogous to validation performed for own dataset, the MAPE value was also calculated for cross-validation. A MAPE value of 0.56 was obtained, which suggests that the developed correlation either under/over predicts |E*| by about 56% from the true values. Albeit the error is too high for prediction of |E*|, the developed correlation still possesses some predictive power.

The correlation developed in this study was compared with the correlation reported in literature (Ping and Xiao, 2008). Ping and Xiao (2008) found that |E*| can be correlated with MR at a loading frequency of 5 Hz and 25 ºC temperature. In Ping and Xiao' study (2008), air voids (AV), voids in mineral aggregate (VMA), and voids with asphalt (VFA) varied from 3 to 5%, 14 to 16.5%, and 71 to 76%, respectively. For further volumetric details, interested readers are referred to [23].

To compare the predictive power of the developed correlation, similar bi-level protocol was followed and two validations were performed. Firstly, the dataset of this study was used to calculate MAPE value. The MAPE values for this study and Ping's study were 0.4% and 5%, respectively. It is evident that the correlation developed in Ping and Xiao (2008) results higher prediction error compared to our study. To gain further insights into comparison, the data from literature, that is Hussain et al. (2016), (same dataset used for cross-validation) were used. Interestingly, the prediction error of our study and Ping's correlation–measured by MAPE value–was found to be similar (56%). This suggests that both correlations performed equally well in predicting |E*| from MR. However, the prediction error is still too high and can be a concern for the highway agencies and practitioners.

To overcome the issue of high prediction error, there exists a great need to define relationship of |E*| and MR by incorporating additional parameters that can affect both these performance tests. To account for simultaneous effect of various parameters on a single variable, a first-order multiple linear regression approach was used in this study. Prior to development of model, a precursor is to check the assumptions of linear regression which are: linearity, normality, homoscedasticity, and independence of errors. Therefore, the dataset was examined and observed that: variables in dataset showed linear relationship, errors were independent and followed normal distribution with constant variance.

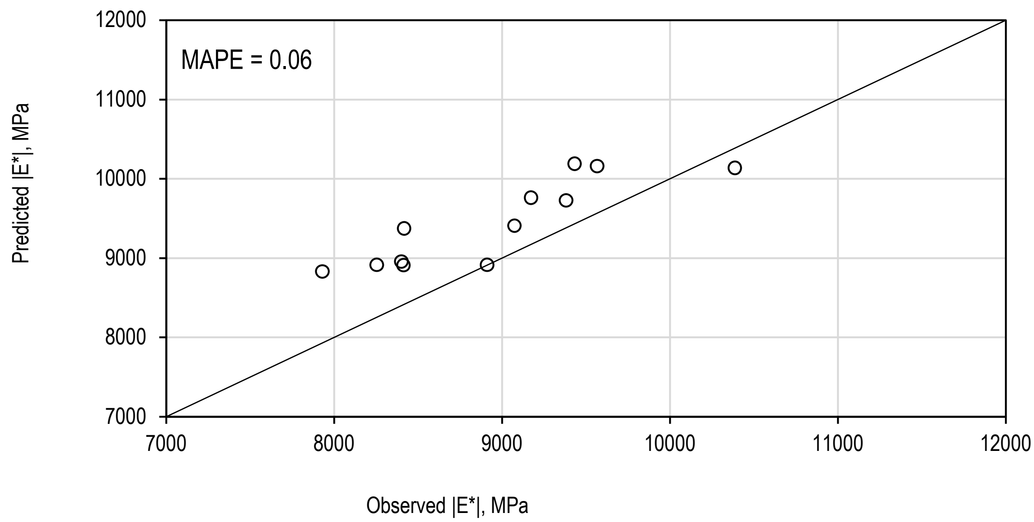
Table 6.13 shows model summary. The model has R^2 of 0.97 which suggests that the model has the capability of capturing 97% of variation in the data. Further, F-value of 82.3 (p-value = <0.0001) indicated that model is better than null model (without any independent variables). All the parameters in the model are statistically significant at 95% confidence level and interpreted below.

Table 6.13: Model Summary

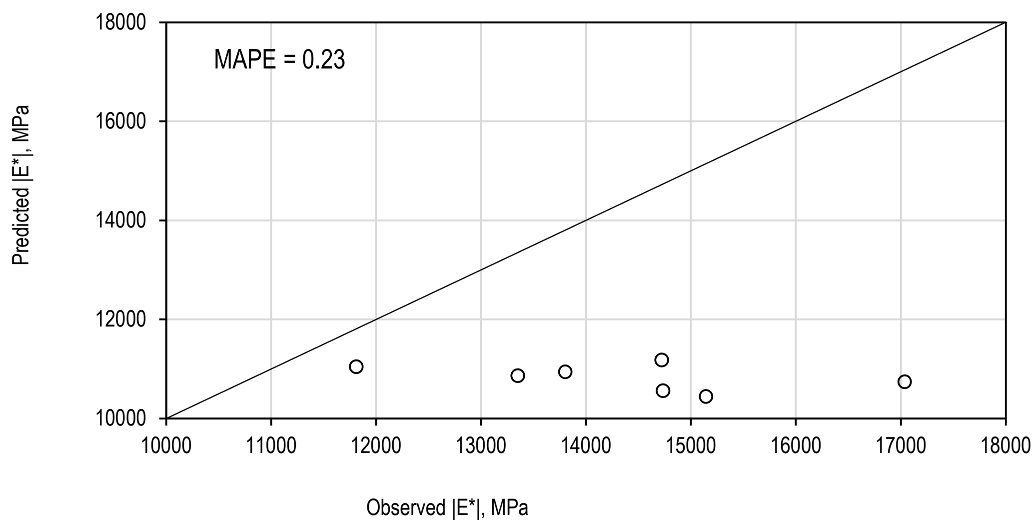
Parameter	Estimate	S.E	t statistics	p-value
Constant	8501	1612	5.27	0.001
MR	0.357	0.135	2.64	0.030
G4.75	-138.46	31.49	-4.40	0.002
VMA	336	131.6	2.55	0.034
R^2	0.97			
Adjusted R^2	0.96			
F3,8	82.85			
N	12			

As expected, MR has positive sign and found to be significant. A positive relation implies that increase in MR tends to increase |E*| and vice versa. This increase is intuitive because increase in MR refers to increase in stiffness of a mix and consequently increase in |E*|. A gradation parameter (percent passing 4.75 mm sieve) has been significant and negative in the model. The negative relation implies that |E*| tends to increase with decrease in gradation parameter because with lower finer particle percentage and consequently higher amount of coarser particles, |E*| would increase and vice versa. Another significant parameter in the model is VMA which is found to be with expected sign. It is worth mentioning here the all variables were tested for multicollinearity using variance inflation factor (VIF) and variables high VIF (>10) were excluded from the final model. VMA and VFA were found to be highly correlated, and thus VFA was removed due to high correlation.

The developed model was tested with proposed bi-level testing protocol. The model showed 6% predictive error (measured by MAPE) by validating using current study dataset i.e., first level of testing (Figure 6.14), while error of 23% was obtained in cross-validating i.e., second level of testing.



(a) Validation Plot at First Level



(b) Validation Plot at Second Level

Figure 6.14: Validation Plots

6.5 Conclusions

Given the tested gradations/mixtures with ARL 60/70 & NRL 40/50 bitumen using Margalla aggregates and based upon Resilient Modulus Test Results following conclusions have been drawn:

- With asphalt binder NRL 40/50, NHA-A relatively performed well followed by NHA-B, MS-2 and Superpsve-1 consistently in all test conditions.
- Similarly, with asphalt binder ARL 60/70, NHA-A relatively performed well followed by NHA-B, MS-2 and Superpave-1 (SP-1) consistently in all test conditions.
- Among the two bitumen sources, NRL 40/50 performed better than ARL 60/70 repetitively in all test conditions.
- Percentage decrease in resilient modulus values due to increase in load pulse duration was observed almost same 39% and 40% in the mixtures prepared with NRL 40/50 and ARL 60/70 respectively.
- Percentage decrease in resilient modulus values due to increase in temperature was relatively more in the mixtures prepared with NRL 40/50 than ARL 60/70. 78% decrease in the resilient modulus values was observed for samples prepared with ARL 60/70 and 68% decrease in the resilient modulus values was observed for samples prepared with NRL 40/50. It can be reasonably concluded that specimen prepared using bitumen penetration grade ARL 60/70 were found more sensitive in temperature change.
- Effect of diameter change on the resilient modulus values of different mixtures was found relatively similar at temperature 25°C and 40°C. Specimens with different diameters have shown almost same percentage decrease in resilient modulus values due to temperature change.
- Among the five main factors temperature was the most significant factor affecting the resilient modulus followed by load duration, bitumen type, nominal maximum size of aggregate and specimen diameter.
- Among the ten (10) interacting factors, interaction of temperature with all other four factors was significant. However, three interactions were insignificant including Load Duration*Diameter, Bitumen Type*Nominal Maximum Aggregate Size and Nominal Maximum Aggregate Size*Diameter.
- The comparison of $|E^*|$ with MR showed that $|E^*|$ at 5 Hz for 25 °C is strongly correlated with MR at same temperature (25 °C) and loading frequency of 300 ms; this correlation showed R^2 of 0.85.
- The developed correlation was rigorously tested using bi-level testing protocol and results suggest that correlation has less predictive power.
- A statistical model was developed and tested; the developed model captured 97% of the variability in the data and predicted reasonably well with an error of 5% and 23% for first and second level of proposed bi-level testing protocol, respectively.

7 Permanent Deformation Evaluation of Asphalt Mixtures for the Development of Rutting Transfer/Shift Functions

Abstract

Rutting or plastic deformation is one of the most frequently observed distress types in the wheel paths of flexible road pavements. This distress may cause early failure of the pavement structure by accelerating the deterioration due to load and environmental detrimental agents. Therefore, it is necessary to simulate the rutting performance of different asphalt mixtures in controlled laboratory conditions, before exposing it to aggressive field conditions. A variety of rutting simulation computer-controlled equipment is available to predict the mixture performance in laboratory at nearly similar conditions to actual in-service pavements. In this study, three types of simulative tests i.e., Cooper Wheel Tracking Test (CWTT), Asphalt Pavement Analyzer test (APA), and Repeated Load Axial Test (RLAT) are performed in accordance with standard procedures on the selected wearing and base course mixtures, and are evaluated according to their performance. Thirty representatives wearing and eight base course mixtures were selected for this study. Three limestone composed aggregate sources, Ubhan Shah, Margalla, Sargodha and three binder types, NRL 40/50, NRL 60/70, ARL 60/70 pen. grade were taken into consideration in the research. Five wearing course gradations, NHA-A, NHA-B, SP-1, SP-2, MS-2; and four base course gradations NHA-A (BC), NHA-B (BC), SP-2 (BC), DBM (BC) were included in the study. The evaluation based on terminal results and permanent deformation/rutting slopes indicated that the impact of temperature variable is observed to be more dominant in case of softer binder and finer graded mixtures.

The mixture ranking on the basis of terminal performance criteria is almost consistent for all the performance tests as translated by Spearman's rank correlation coefficient of 0.81, 0.80, and 0.73 for ranking obtained by various combinations of laboratory rutting performance tests using Cooper Wheel Tracking Equipment, Asphalt Pavement Analyzer and Repeated Load Axial Apparatus. The wearing course asphalt mixtures included in the testing regime were categorized on the basis of rank product statistic computed from rank order in individual laboratory performance tests. The aggregate source and gradation related properties significantly affecting permanent deformation behavior of asphalt mixtures were combined to formulate novel indices named as 'aggregate source index' and 'aggregate gradation index'. General full factorial design was found as an efficient tool to statistically quantify the significance of different factors on the output variables. The results of general full factorial design revealed that temperature condition is most significant contributing factor affecting the rutting propensity of asphalt mixtures, followed by binder penetration grade, aggregates source index, aggregate gradation index, and stress condition. In order to compare

relative rutting predictability of different performance tests in terms of the time required to achieve cumulative rut depth of one millimeter, it has been observed that Asphalt Pavement Analyzer test takes maximum time to reach one millimeter rut depth, hence, it was concluded that rut depth observed in Cooper Wheel Tracking Test and permanent strain/cycle observed in Repeated Load Axial Test has most significant correlation with R^2 of 0.7263.

Laboratory rutting shift functions also known as laboratory rutting prediction models were developed as a final product of this research study, utilizing couple of statistical techniques. Multiple non-linear regression technique with Cobb-Douglas formulation was used as a first technique to develop laboratory shift functions. The multilayer perceptron algorithm of artificial neural network technique is also used to develop the shift function with same set of variables. Significant independent variables included in the models are temperature in °C, number of passes in case of Cooper Wheel Tracking Test or number of cycles in case of Asphalt Pavement Analyzer test/Repeated Load Axial Test, bitumen penetration value in $1/10^{th}$ of mm, aggregate source index, aggregate gradation index, and stress condition. The comparison of two modeling techniques was done by plotting the predicted versus measured values. The research study revealed that artificial neural networking technique relatively better predicts the rutting output parameters as compared to non-linear regression technique.

7.1 Introduction

Visually, rutting is recognized by a continuous depression parallel to traffic direction along the wheel paths. It is normally observed at the locations where heavy traffic travels at slow speed, such as intersections, bus bays, turning lanes etc.

Due to high temperature, high tire pressures and unregulated heavy truck traffic in Pakistan, the pavements are subjected to very high stresses and strains resulting into premature failure. Around the world low cost and better ride quality forces the use of asphaltic concrete pavements; resulting nearly 80% of the roadways being constructed using asphalt concrete. Rutting, fatigue cracking, and low temperature cracking (shrinkage cracking) are the most common distresses being observed in flexible pavements. It is very important to adopt proper design, construction, and quality control procedures to build better quality roads and avoid premature failures. Though a stiffer pavement layer improves load sustaining ability of the subgrade, but at the same time it is observed that the stiffer the pavement layer the more would be the tensile stresses at the bottom of asphaltic pavement layer, resulting in more susceptibility to cracking. Therefore, the designers must adopt a reasonable compromise between stiffness and flexibility.

Material quality of surface, base, subbase and subgrade layers should be well designed, failing to which, it would be impossible to reduce rutting susceptibility, no matter how much layer thickness is provided or how much construction quality control is to be taken care of (Parker, 1990). Rutting in flexible asphaltic pavements is generally dependent on various factors such as asphalt mixture properties, which further include aggregate gradation, types of aggregates, binder type and its properties and extent of applied compaction efforts. Factors related to loading pattern include types of vehicles, tire material, tire pressure, speed of vehicles & axle load. Similarly environmental factors, such as climatic conditions of the area and pavement temperature also affect the type and extent of rutting. Gu et al., (2015) have reported that low vehicle speed and overloading significantly increases the rutting potential of a pavement. Sel et al., (2014) conducted statistical analyses & concluded that there is a significant effect of test temperature and binder type on the Hamburg wheel tracker test results. A study by Xiao et al. (2007) showed that as air voids in the modified mixtures decrease, the rut depth from the APA test also decreases.

Zulkati et al., (2012) reported that a repetitive uniaxial compressive load on cylindrical specimens of asphalt-concrete mixtures provides a reasonable simulation of asphalt pavement subjected to repetitive axle loads. For determining simulated rutting resistance of recycled asphalt mixtures, Hamburg Wheel Tracking Test (HWTT) was used by Hafeez et al., (2014). Zhang et al. (2015) have reported that the response of asphalt mixtures to various test conditions by Partial Triaxial Compression Test (PTCT) is in accordance with that of typical triaxial compression tests. Doyle and Howard (2013) have reported that PURWheel (Purdue University Wheel Tracker)-dry and APA (Asphalt Pavement Analyzer) were observed to be equally good indicators of rutting potential. Al-Hadidy and Yi-Qiu (2011) & Chandra and Choudhary (2013) used wheel tracking test to study the effect of modifiers in asphalt mixtures. The Asphalt Pavement Analyzer (APA) and repeated simple shear test at constant height (RSST-CH) were validated as useful Accelerated Pavement Testing (APT) tools by Martin and Park, (2003). Choubane et al., (2000), Rushing et al., (2012) & Kim et al. (2009) inferred that the APA may be used as an effective tool to rank asphalt mixtures in terms of their respective rut performance. Repeated load axial tests or dynamic creep tests were used by Shafiei and Namin (2014), Al-Khateeb et al., (2013) and Chen et al., (2011) to simulate the effect of various fillers and modifiers on rutting susceptibility of asphalt mixtures.

Rushing and Little (2014) presented results from a laboratory study to identify a performance test for accepting hot asphalt mixtures for constructing airport pavements. Statistical analyses performed on the results indicated that the rate of increase in permanent strain and the flow time value determined from triaxial static creep testing provides the strongest correlation to APA rutting. Chen et al. (2007) concluded that a strong correlation exists between the French Loaded Wheel Tracker (FLWT), Dynamic Stability (DS) and APA rut depth for the mixtures studied.

A rutting transfer function or prediction model is an equation that is used to predict pavement life in terms of a number of repetitions (or loading cycles) to failure. Apeagyei (2011) developed multiple regression models to describe the relationship among flow number (FN), dynamic modulus, and gradation. The prediction model between laboratory rut depth and the dynamic stability of each asphaltic layer, and the longitudinal grade was developed by Wang et al., (2009) using multiple, nonlinear regression analysis. Ahmed & Erlingsson (2014) carried out two different types of tests: an extra-large wheel-tracking (ELWT) test and a full-scale accelerated pavement test using a heavy vehicle simulator (HVS) to compute model parameters for Mechanistic Empirical Pavement Design Guide (MEPDG) model. Tapkin et al., (2009) presented an application of Artificial Neural Networking (ANN) technique to predict permanent strain of polypropylene modified asphalt mixtures in a repeated load axial test. Mirzahosseini et al., (2013) developed a ANN based rutting propensity model for asphalt mixtures; percentages of coarse aggregate, filler, bitumen, air voids, voids in mineral aggregate, and Marshall Quotient were employed as the predictor variables using flow number test. Archilla and Madanat (2000) developed a non-linear rutting regression model based on observations of AASHO Road test between number of ESALs along with thawing index as an environmental parameter. Rutting was modeled as a function of traffic, temperature, and mix characteristics, including voids filled with asphalt, asphalt content, in-place air voids, surface area, and the densification slope by Archilla (2006). Hu et al., (2011) presented a new mechanistic-empirical M-E rutting model developed for hot-mixed asphalt HMA overlay thickness design and analysis. A research described by Suh and Cho (2014) explained the development of a model for estimating the rutting performance of dense graded asphalt concrete pavement under various temperatures and loads.

7.2 Objectives

The research was conducted to achieve the following objectives:-

- To evaluate and rank the rutting susceptibility of different representative asphalt mixtures used in road pavements.
- To compare and correlate the rutting predictability of different performance tests.
- To statistically identify and analyze the factors influencing the rutting susceptibility of asphalt mixtures.
- To develop and validate rutting shift functions based on laboratory performance data.

7.3 Research Methodology

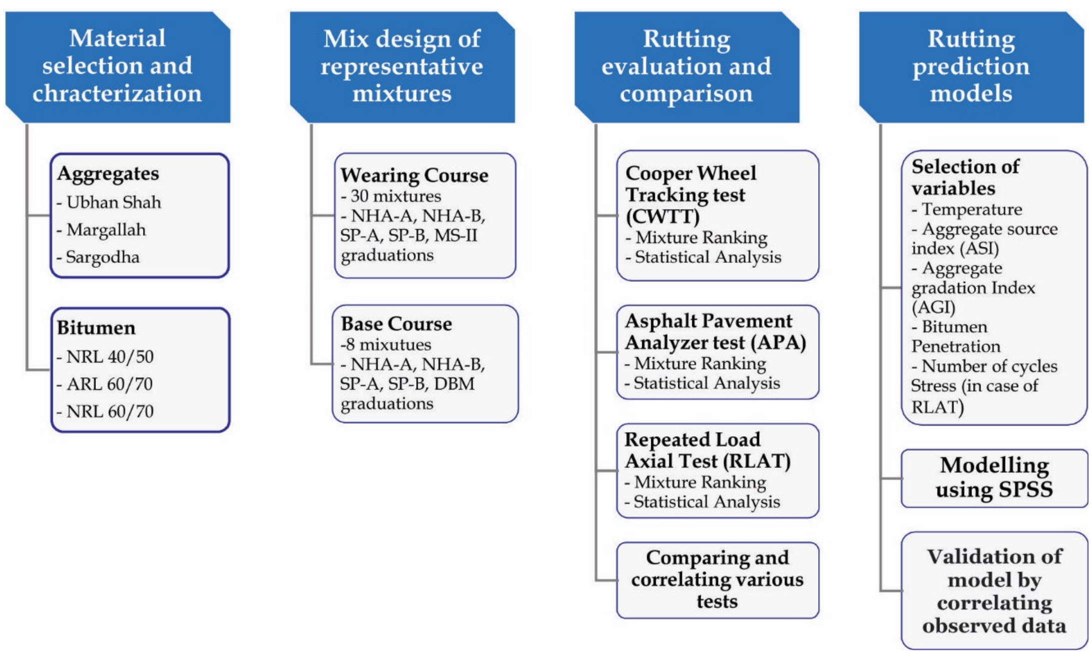


Figure 7.1: Flowchart for Experimental Program

The methodology used to achieve the research objectives has been summarized in Figure 7.1. Three aggregate types (i.e., Ubhan Shah, Margalla, and Sargodha) and three bitumen types (NRL 60/70, ARL 60/70, and NRL 40/50) has been selected in the research. Cumulatively thirty eight representative mixtures designed by Marshall mixture design method have been selected in present research study for both the base course and wearing course layers. Laboratory rutting performance of these mixtures has been evaluated using three globally accepted simulation tests. Based on the results of these tests, rutting shift functions/prediction models have been developed using different statistical modeling techniques in SPSS statistical analysis software. As can be seen from Figure 7.1, the methodology was planned in four steps i.e., constituent materials selection and characterization; selection of representative asphalt mixtures and their mix design; rutting performance evaluation, comparison and correlation between various test procedures; and development of rutting shift functions/prediction models using different statistical techniques.

7.3.1 Materials and Methods:

Aggregates

Three aggregate sources namely Ubhan Shah Quarry in Sindh Province, Sargodha and Margalla Quarry in Punjab province were selected. Ubhan Shah aggregate quarry represents the southern part of country while Margalla and Sargodha quarry represents the northern part. Results of the physical and mechanical tests carried out on aggregates have been tabulated in Table 7.1. The centerline gradations selected for the study has been shown in Figure 7.2 and Figure 7.3.

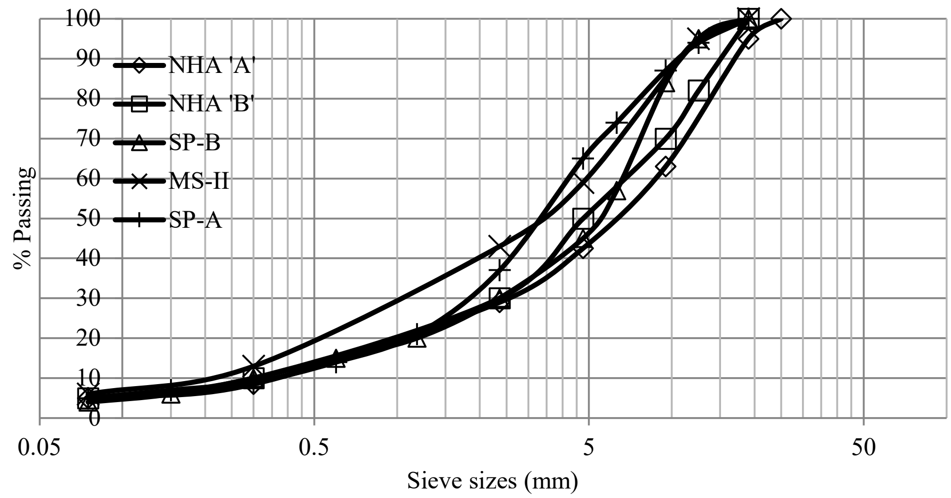


Figure 7.2: Centerline Gradations Curves of Asphaltic Wearing Course

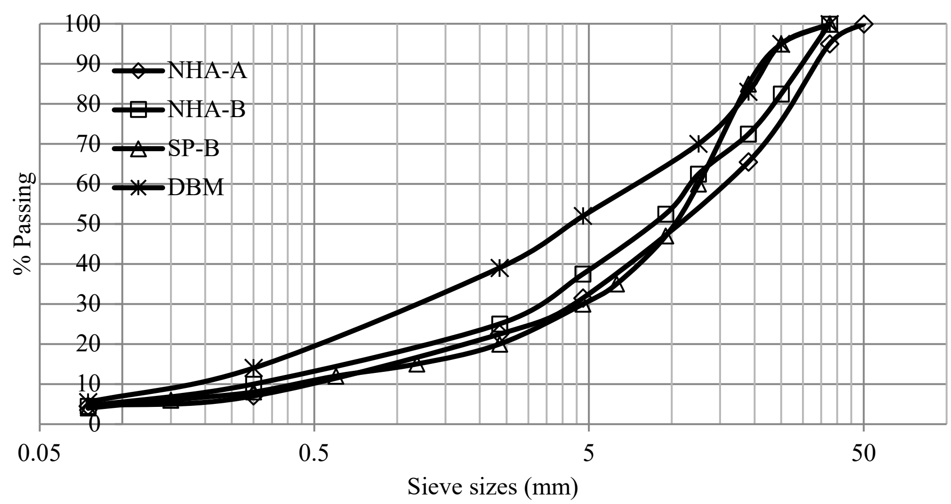


Figure 7.3: Centerline Gradations Curves of Asphaltic Base Course

Figure 7.2 indicates that, among wearing course gradations, NHA-A (WC) centerline gradation has a minimum while SP-1 (WC) has a maximum percentage of material passing # 4 sieve while, among base course gradations NHA-A (BC) is coarser while DBM (BC) is finer, as shown in Figure 7.3.

Table 7.1:Tests of Aggregates

Tests	Standard	Aggregate Source			Specification Limits
		Ubhan Shah	Margalla	Sargodha	
Fractured Particles (Two faces)	ASTM D 5821	100	100	100	90% (min)
Flakiness Index	BS 812-108	12	4.75	8	10% (max)
Elongation Index	BS 812-109	16.6	2.2	6	10% (max)
Sand Equivalent value	ASTM D 2419	82	75	70	50% (min)
Los Angeles Abrasion value	ASTM C 131	21	23	22.5	30% (max)
Water Absorption	ASTM C 127	0.88	1.02	0.96	2% (max)
Soundness (Coarse)	ASTM C 88	0.17	7.1	4.3	8% (max)
Soundness (Fine)	ASTM C 88	2.44	4.7	3.6	8% (max)
Uncompacted Voids (Fine)	ASTM C 1252	41	37.5	40	45% (min)

It has been inferred from Table 7.1 that the aggregates procured from Ubhan Shah Quarry are flakier and more elongated as compared to that of Margalla and Sargodha Quarry. Sand equivalent value, Los Angeles Abrasion value, water absorption, soundness and uncompacted voids of aggregate samples of all three sources are well within the NHA specified limits.

Bitumen

Three different binders were selected from two sources i.e., NRL 40/50 & NRL 60/70 pen. grade from National Oil Refinery Karachi (NRL) and ARL 60/70 pen. grade from Attock oil refinery Attock (ARL), which are commonly used for flexible pavements in Pakistan. Qualitative analysis of bitumen has been tabulated in Table 7.2.

Table 7.2: Qualitative Tests of Binders

Binder Source	Title	Standard	Units	Tests Results
NRL 60/70	Penetration at 25°C	ASTM D 5	(1/10th of mm)	70
	Softening point	ASTM D 36	°C	46
	Ductility	ASTM C 88	cm	100
	Flash & Fire point	ASTM C 142	°C	291
NRL 40/50	Penetration at 25°C	ASTM D 5	(1/10th of mm)	49
	Softening point	ASTM D 36	°C	49
	Ductility	ASTM C 88	cm	100
	Flash & Fire point	ASTM C 142	°C	296
ARL 60/70	Penetration at 25°C	ASTM D 5	(1/10th of mm)	64
	Softening point	ASTM D 36	°C	48
	Ductility	ASTM C 88	cm	100
	Flash & Fire point	ASTM C 142	°C	262

Asphalt Mixtures

Marshall mixture design procedure had been adopted for different asphalt mixtures. A typical average value of 4% air voids had been used in the design procedure, as air voids range is 3-5% according to Superpave specification criteria. The complete test matrix including mixture designation, optimum bitumen contents (OBC) along with test conditions has been summarized in Table 7.3 below;

Table 7.3: Test Matrix for CWTT, APA and RLAT

Sr No.	Mixture ID	OBC (%)	CWTT		APA		RLAT					
							Stress: 300 kPa			Stress: 500 kPa		
			40°C	50°C	40°C	50°C	25°C	40°C	50°C	25°C	40°C	50°C
1	NHA-A WC US NRL 60/70	3.9	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓
2	NHA-B WC US NRL 60/70	4.7	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓
3	SP-2 WC US NRL 60/70	4.6	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓
4	SP-1 WC US NRL 60/70	5.4	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓
5	MS-2 WC US NRL 60/70	4.7	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓
6	NHA-A WC US NRL 40/50	3.8	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓
7	NHA-B WC US NRL 40/50	4.6	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓
8	SP-2 WC US NRL 40/50	4.4	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓
9	SP-1 WC US NRL 40/50	5.5	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓
10	MS-2 WC US NRL 40/50	4.8	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓
11	NHA-A WC M ARL 60/70	4.0	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓
12	NHA-B WC M ARL 60/70	4.1	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓
13	SP-2 WC M ARL 60/70	4.4	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓
14	SP-1 WC M ARL 60/70	5.0	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓
15	MS-2 WC M ARL 60/70	4.8	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓
16	NHA-A WC M NRL 40/50	3.9	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓
17	NHA-B WC M NRL 40/50	4.4	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓
18	SP-2 WC M NRL 40/50	4.5	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓
19	SP-1 WC M NRL 40/50	5.3	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓
20	MS-2 WC M NRL 40/50	4.7	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓
21	NHA-A WC S ARL 60/70	4.0	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓
22	NHA-B WC S ARL 60/70	4.6	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓
23	SP-2 WC S ARL 60/70	4.4	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓
24	SP-1 WC S ARL 60/70	5.6	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓
25	MS-2 WC S ARL 60/70	4.9	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓
26	NHA-A WC S NRL 40/50	3.9	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓
27	NHA-B WC S NRL 40/50	4.5	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓
28	SP-2 WC S NRL 40/50	4.6	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓
29	SP-1 WC S NRL 40/50	5.5	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓
30	MS-2 WC S NRL 40/50	4.8	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓
31	NHA-A BC US NRL 60/70	3.1	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓

Sr No.	Mixture ID	OBC (%)	CWTT		APA		RLAT					
							Stress: 300 kPa			Stress: 500 kPa		
			40°C	50°C	40°C	50°C	25°C	40°C	50°C	25°C	40°C	50°C
32	NHA-B BC US NRL 60/70	3.2	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓
33	SP-2 BC US NRL 60/70	3.9	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓
34	DBM BC US NRL 60/70	4.0	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓
35	NHA-A BC M ARL 60/70	3.3	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓
36	NHA-B BC M ARL 60/70	3.7	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓
37	SP-2 BC M ARL 60/70	3.6	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓
38	DBM BC M ARL 60/70	3.9	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓

In Table 7.3 above, NHA-A, NHA-B, SP-1, SP-2, MS-2, DBM indicates gradations, WC & BC stands for wearing course and base course, respectively while US, S & M stands for Ubhan shah aggregate, Sargodha aggregate and Margalla aggregate respectively; NRL & ARL indicates National Refinery Limited and Attock Refinery Limited, respectively as binder source, and 40/50 & 60/70 shows binder penetration grade.

Samples were prepared at OBC for three rutting performance tests i.e., Cooper Wheel Tracking Test (CWTT), Asphalt Pavement Analyzer test (APA) & Repeated Load Axial Test (RLAT). Table 7.4 shows the dimensions of compacted specimens and the adopted compaction protocols of various performance tests.

Table 7.4: Sample Preparation Protocols for CWTT, APA And RLAT

	CWTT	APA	RLAT
Sample shape	**Slab**	**Cylindrical**	**Cylindrical**
Sample size i) AWC ii) ABC	 300 x 300 x 50 mm 300 x 300 x 100 mm	150 mm dia. and 75 mm thick	150 mm dia. and 75 mm thick
Mixing Temp.	155 ± 5°C	155 ± 5°C	155 ± 5°C
Compaction Temp.	140 ± 5°C	140 ± 5°C	140 ± 5°C
Compactor	Roller	Gyratory	Gyratory
Compaction conditions	2.5 bar, 3.5 bar, 4.5 bar, 4 bar at stage 1, 2, 3 & 4 respectively	600 kPa pressure, 1.25° Gyration angle	600 kPa pressure, 1.25° Gyration angle
Air voids (%)	6 ± 1	6 ± 1	6 ± 1
Batch weight (kg)	11 (WC), 22 (BC)	7	7

The steps involved in sample preparation have been shown in Figure 7.4. Sieving of aggregates was performed using standard sieve sizes as specified for each gradation. Batch weight was prepared by weighing aggregates retained on each sieve size. The aggregate material was placed in oven for 24 hours prior to mixing at a temperature of 110°C. Each mixture was prepared at mixing temperature of 155±5°C and temperature during mixing was monitored by digital temperature gauge. After mixing, compaction was done by either Cooper roller compactor or Superpave gyratory compactor, depending upon the shape of specimens, to achieve air voids of 6 ± 1 %. Two replicates for each test condition were prepared for three rutting performance tests. Cumulatively more than 760 samples were tested as a whole.

CWTT is used to assess the susceptibility of an asphalt mixture to deform plastically at high temperatures under pressure caused by traffic. Cooper wheel tracking machine enables the test specimen in its cradle to be moved backwards and forwards under the loaded wheel. The center of the contact area of a solid rubber tire of 200 mm diameter and 20 mm thickness undergoes simple harmonic motion with respect to the center of top surface of the test specimen, with a total distance of travel 230 ± 10 mm at a frequency of 26.5 ± 1.0 cycles per minute. The terminal number of passes is limited to 10,000 with loading magnitude of 700 N. The failure rut depth is considered to be 12.5 mm in line with past literature. The test was conducted in accordance with EN 12697-22.

a) Weighing Material

b) Mixing of Materials

c) Roller Compactor

d) Gyratory Compactor

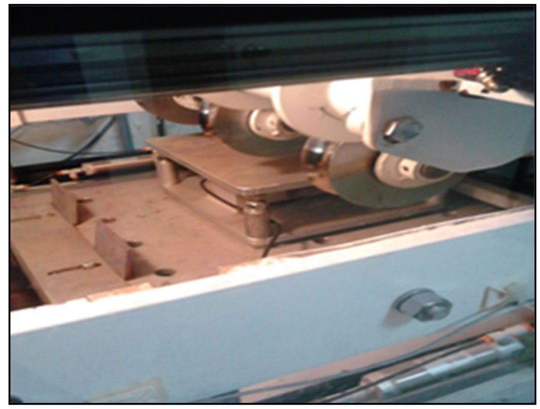
e) CWTT Sample f) RLAT Sample

Figure 7.4: Sample preparation for CWTT, APA & RLAT

In Cooper wheel tracking test (CWTT), specimens were conditioned at the specified test temperature of 40°C and 50°C for a minimum period of 2 hours prior to testing. The specimen temperature was monitored by placing a dummy sample in the temperature controlled cabin of wheel tracking machine. Once required temperature was achieved, the test was started through software installed in the attached PC. The rut depth was automatically monitored by the attached LVDT. The computer screen of attached PC showed the rut development curve in increments of 0.01 mm. Test continued until terminal number of passes or failure rut depth, whichever was earlier.

The Asphalt Pavement Analyzer (APA) test is an empirical laboratory test that is assumed to simulate actual field performance of asphalt concrete trafficked by heavy vehicles. The APA test does not measure mixture material properties; therefore, it can be used only to rank relative rut resistance of different mixtures. For APA tests, samples were conditioned for a period of 12 hours prior to testing. The air voids were kept in the range of 6 ± 1% to simulate actual conditions of road pavements. The terminal number of cycles was limited to 8000 with loading magnitude of 100lbs and hose pressure of 100 psi. The loading frequency was 59 Hz, while the failure rut depth was considered to be 5 mm in line with the previous published work (Choubane et al. 2000). The test was conducted in accordance with AASHTO TP 63. Before Asphalt Pavement Analyzer (APA) test, load and height calibration of the equipment was verified. Preheated molded specimens were inserted in the APA machine as shown in Figure 7.5. The chamber doors were closed for minimum of 10 minutes after placing the specimens in order to stabilize the temperature prior to testing. Twenty five cycles were applied to seat the specimens before recording the initial reading. At the completion of maximum cycles, APA test terminated, and the wheels automatically retracted.

a) Height Calibration



b) Load Calibration

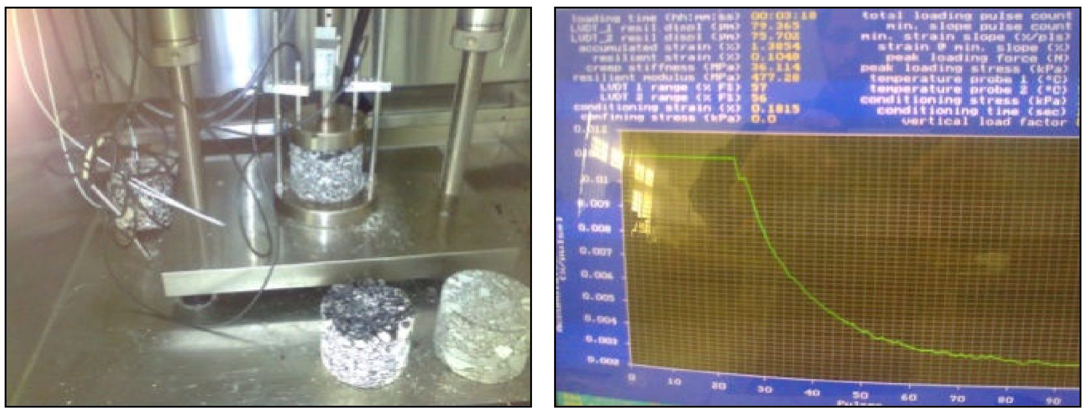
c) Inserting Molded Sample

d) Sample Ready to Be Tested

Figure 7.5: Asphalt Pavement Analyzer Test in Progress

Although the stress distributions within an HMA surface layer on a loaded highway pavement are quite complex, representative axial strains could be selected to serve as a basis for ranking asphalt mixture performance in the laboratory (Rushing and Little, 2014). Repeated Load Axial Test is one among the various tests which can be used to measure these permanent and resilient strains. The test was conducted on 100 mm diameter and 75 mm thick cylindrical core specimens. The air voids were kept in the range of 6 ± 1 % to simulate actual conditions of pavements. The terminal number of cycles was limited to 3600 with a pulse period of 2 sec and loading pulse width of 1 sec. The terminal permanent strain was kept to be 3% and the test was conducted in accordance with EN 12697-25. The test was conducted at two stress levels of 300 and 500 kPa (Tapkin & Keskin, 2013). In Repeated Load Axial Test (RLAT), cylindrical test specimens maintained at particular conditioning temperature, were placed between two plane loading platens. The test specimens were positioned well centered co-axially with the test axis between the two platens. Two displacement transducers were positioned on the loading platen, one opposite to the other. The specimens were subjected to cyclic axial block or square pulsed loading without any confinement pressure. A pre-load of 10 kPa was applied for 100 sec. The accuracy of the preload was ensured to be within ± 10% or less. Immediately after the pre-loading time, the periodic load cycle followed by a rest period of 1±0.05 sec was applied, which corresponds to the loading frequency of 0.5 Hz. During the test the permanent strains and resilient strains were measured after specific number of loading cycles. Cumulative axial permanent strain (εp) was determined as a function of number of cycles. Total duration of a single test was approximately 2 hours. The test terminated at a permanent strain of 3% or 3600 loading cycles, whichever was achieved earlier. The test was conducted in accordance with EN 12697-25. Samples & test assembly has been shown in Figure 7.6.

In CWTT and APA test, samples were tested in the confined state while in RLAT the samples were unconfined. The wheel used in CWTT is composed of solid rubber while in APA test pressurized rubber hose is used to transfer load to the specimen, therefore APA better simulates load transfer mechanism of an actual vehicle wheel. CWTT, APA and RLAT are typically used to rank relative rutting resistance of different mixtures, therefore, the terminal number of passes/cycles and terminal rutting value/ permanent strains were selected from the respective test standard protocols.



a) Fixing Sample in Test Jig b) PC Output During Test

Figure 7.6: Sample Testing for Repeated Load Axial Test

7.3.2 Results and discussion

7.3.2.1 Cooper Wheel Tracking Test

The rutting development curve with rut depth as ordinate and number of passes as abscissa has been observed as the primary computer generated output of this test. The average rut depth at thirteen contact points along the path of loaded wheel has been automatically monitored and reported in the output file. The typical automatically generated profile has been shown in Figure 7.7 below. The top portion of Figure 7.7 shows the rutting development curve along-with number of passes. The test conditions are shown at bottom of the screen.

There are two important observations from the developed rutting curve i.e., first is the terminal value of rut depth in case mixture survived the adopted failure criterion of 12.5 mm at 10,000 wheel passes, and second is the variation of slope of curve along-with increasing loading passes.

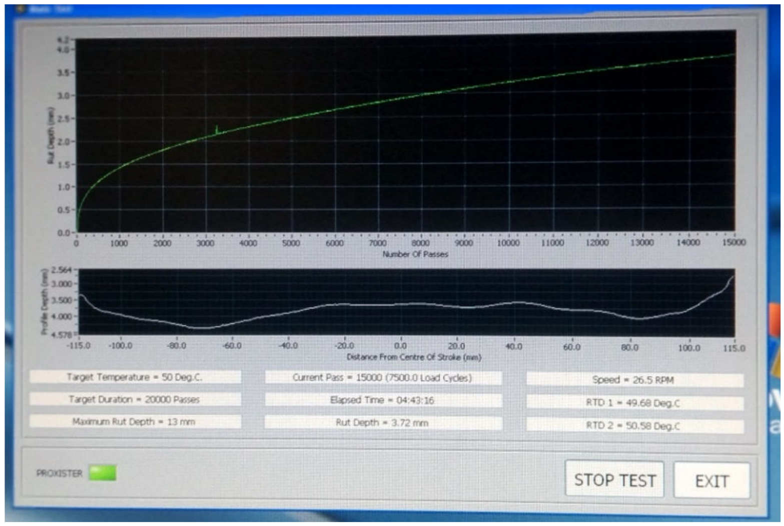
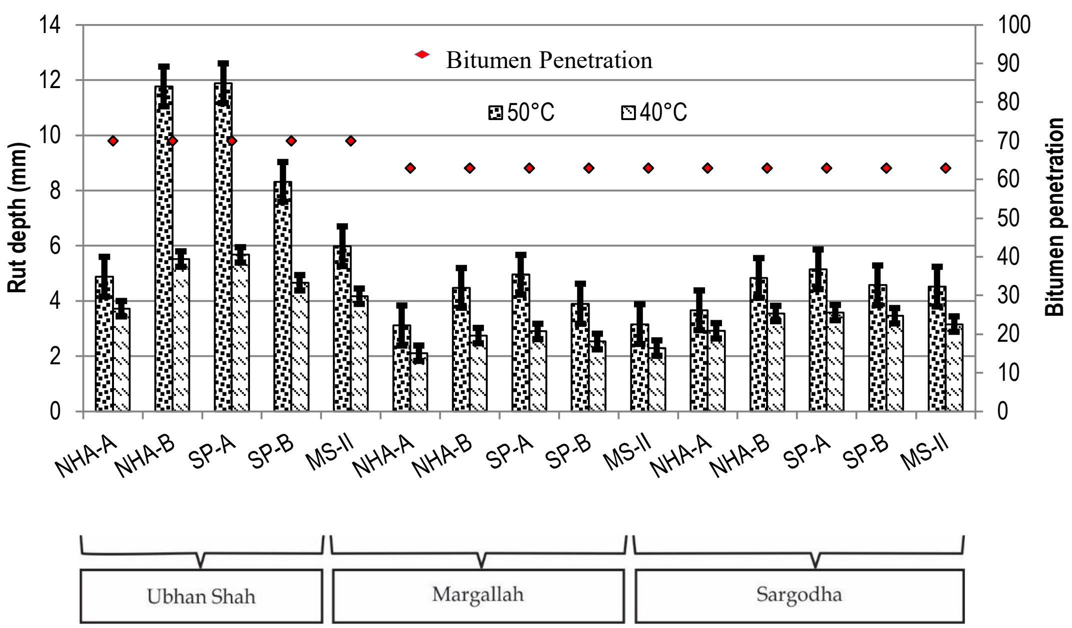


Figure 7.7: Cooper Wheel Tracking Test Software Automatically Generated Display

7.3.2.2 Evaluation Based on Terminal Rut Depth

Ranking or evaluation based on terminal rut depth at 10,000 loading passes of thirty wearing course and eight base course mixtures using Cooper Wheel Tracking Test has been shown in Figure 7.8 and Figure 7.9 respectively. Wearing course and base course mixtures were tested at temperature conditions of 40°C and 50°C. Figure 7.8 shows the relative rutting resistance of selected wearing course mixtures at 10,000 wheel passes. The diamond shaped bullets show the penetration value of bitumen plotted on secondary vertical axis. The bars in top portion of Figure 7.8 shows terminal rut depth values of fifteen wearing course mixtures with 60/70 pen. grade bitumen, while bars in bottom portion show terminal values of fifteen wearing course mixtures with 40/50 pen. grade bitumen at temperature conditions of 50°C and 40°C, respectively. Similarly Figure 7.9 elaborates the relative rutting performance of eight base course mixtures.



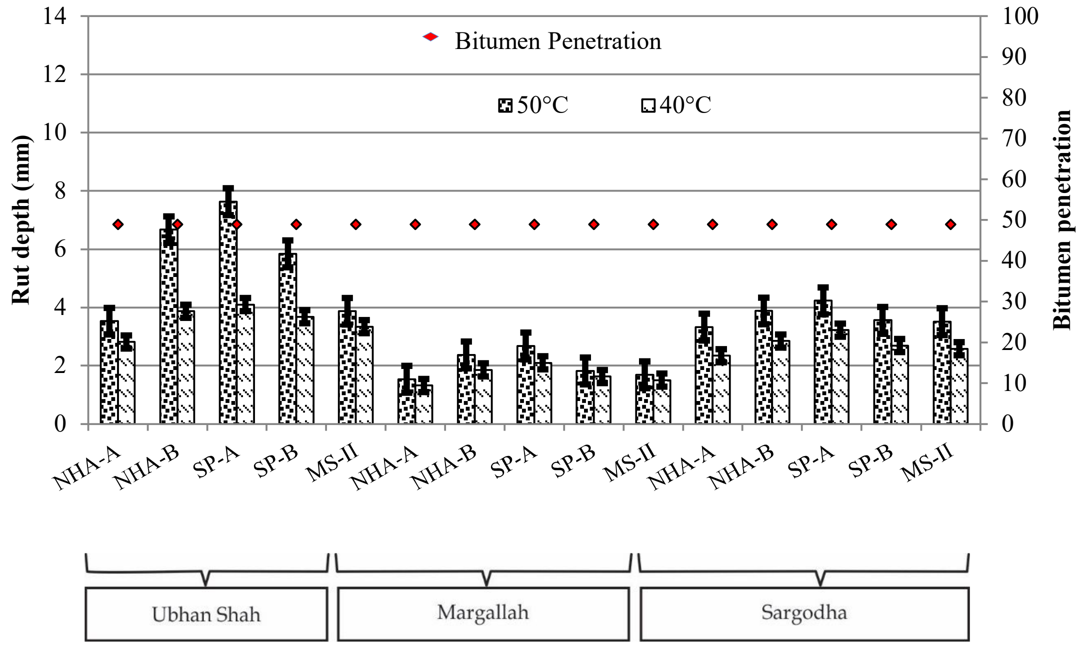


Figure 7.8: Test results for Cooper Wheel Tracking Test (WC)

The results in Figure 7.8 show that all the mixtures completed 10,000 loading passes without reaching the terminal rut depth of 12.5 mm. Almost similar ranking has been observed for all asphalt mixtures at temperature conditions of 40°C and 50°C. The individual ranking showed that mixtures with aggregates of Margalla and bitumen of NRL 40/50 pen. grade are most resistant to rutting susceptibility while Ubhan Shah aggregates and NRL 60/70 bitumen composed mixtures are least resistant to rutting distress. In case of performance comparison on the basis of aggregate type; mixtures with Margalla aggregates performed the best, mixtures with Sargodha aggregates have intermediate performance, while Ubhan Shah aggregates composed mixtures have worst rutting resistance among the selected aggregate types. It is notable that the only aggregate type/source having higher Flakiness Index (FI) than National Highway Authority (NHA) specified limit of 10% is Ubhan Shah aggregate source. The higher rutting propensity of mixtures with Ubhan Shah aggregates is probably due to more percentage of flat and elongated particles as the aggregate particle shape parameters influenced the rutting performance of asphalt mixtures by past researchers (Mahmud et al., 2014, Putrajaya et al., 2014). If we compare on basis of bitumen type, NRL 40/50 pen. grade bitumen composed mixtures performed the best, followed by ARL 60/70 pen. grade and NRL 60/70 pen. grade mixtures, respectively.

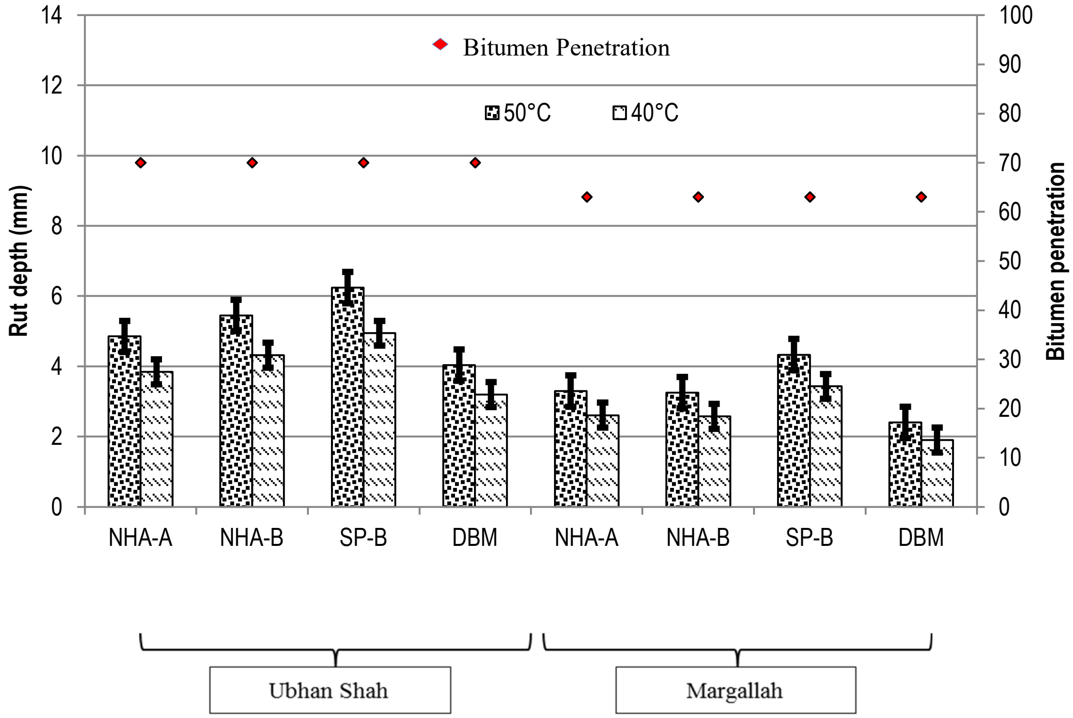


Figure 7.9: Test Results for Cooper Wheel Tracking Test (BC)

Figure 7.9 depicts that SP-2 (BC) graded base course mixture with Ubhan Shah aggregates and NRL 60/70 pen. grade bitumen has maximum vulnerability to be affected by rutting distress, while DBM (BC) graded base course mixture with Margalla aggregates and ARL 60/70 pen. grade bitumen is least prone to rutting failure.

7.3.2.3 Evaluation Based on Slope of Rutting Curve

The rutting slope with loading passes 'N' is another important parameter that can be used to evaluate permanent deformation behavior of asphalt mixtures. The rutting slope in units of 'mm/loading pass' has been computed by Equation 7-1 in which N = Number of passes, p = previously recorded number of passes. In this test data is automatically stored in increments of hundred passes. The rutting slope plots for all the thirty eight asphalt mixtures have been plotted in Figure 7.1 for both temperature conditions.

$$\boldsymbol{Rutting\ Slope\ at\ N = (Rut\ depth_N - Rut\ depth_P)/(N - p)} \qquad \text{7-1}$$

The parameter has been analyzed by plotting rutting slope against number of passes on log-log scale. The graphs shown in Figure 7.10 have been categorized according to aggregate gradation and penetration grade of bitumen used in a mixture.

From Figure 7.10 (a & b) it has been observed that for coarse gradation of NHA-A, the rutting slope near terminal passes approaches close to each other when tested at variable temperature conditions of 40°C and 50°C. This justifies that there is relatively less impact of temperature variation on rutting slope for a coarse graded mixture. This behavior could be due to better interlocking of coarse aggregates as a result of point-to-point contact between coarse particles. Henceforth, the role of bitumen which significantly loses its binding ability with increase in temperature, has been reduced. This trend has been observed irrespective of the aggregate source. It has further been observed that the rutting slope converges as the test had progressed.

Figure 7.10 (c & d) shows the rutting slope plots of relatively fine gradation of NHA-B. It has been observed that for Ubhan Shah aggregate mixtures, the rutting slope diverged with increasing passes in contrast to behavior of coarse graded NHA-A mixtures. The variation among the rutting slope has also increased significantly for mixtures with various aggregate sources near the terminal number of passes. The mixtures with Sargodha aggregates seem more stable as the rutting slope terminates at values significantly close to each other.

Figure 7.10 (e & f) shows the rutting slope of relatively finest gradation of SP-1. It has been observed that for Ubhan Shah and Margalla aggregate mixtures, the rutting slope is initially greater at temperature condition of 40°C with respect to temperature condition of 50°C, while latter on the rutting slope trend became conventional. The variation among the rutting slope has also increased significantly for mixtures with various aggregate sources near the terminal number of passes.

Figure 7.10 (g & h) shows the rutting slope curves of SP-2 graded mixtures. It has been observed that the rutting slope diverged with increasing number of passes. For Ubhan Shah aggregate mixtures, the effect of temperature variation on rutting slope is observed to be more pronounced, as indicated by largely separated rutting slope curves at temperature conditions of 40°C and 50°C. Similar trends have been observed from Figure 7.10 (i & j) for MS-2 graded wearing course mixtures.

For base course mixtures the variability in rutting slope is less in case of coarse graded mixture of NHA-A (BC) gradation as compared to mixtures with relatively finer gradations of NHA-B (BC), SP-2 (BC) and DBM (BC).

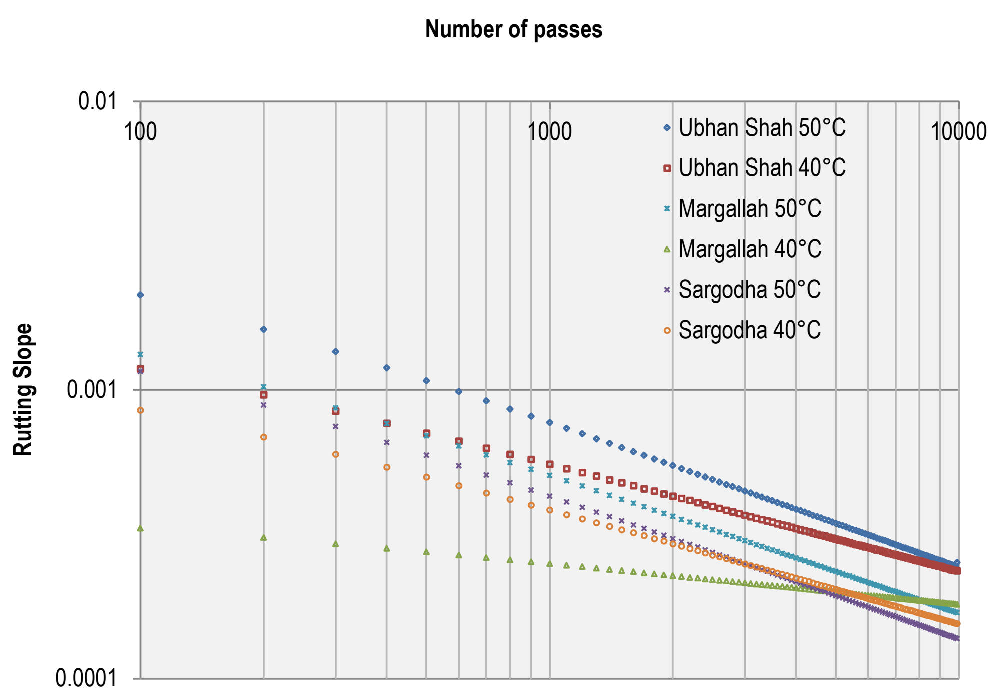


Figure 7-10(a): Rutting Slope Plot for Cooper Wheel Tracking Test For NHA-A Gradation And 60/70 Pen. Grade Bitumen

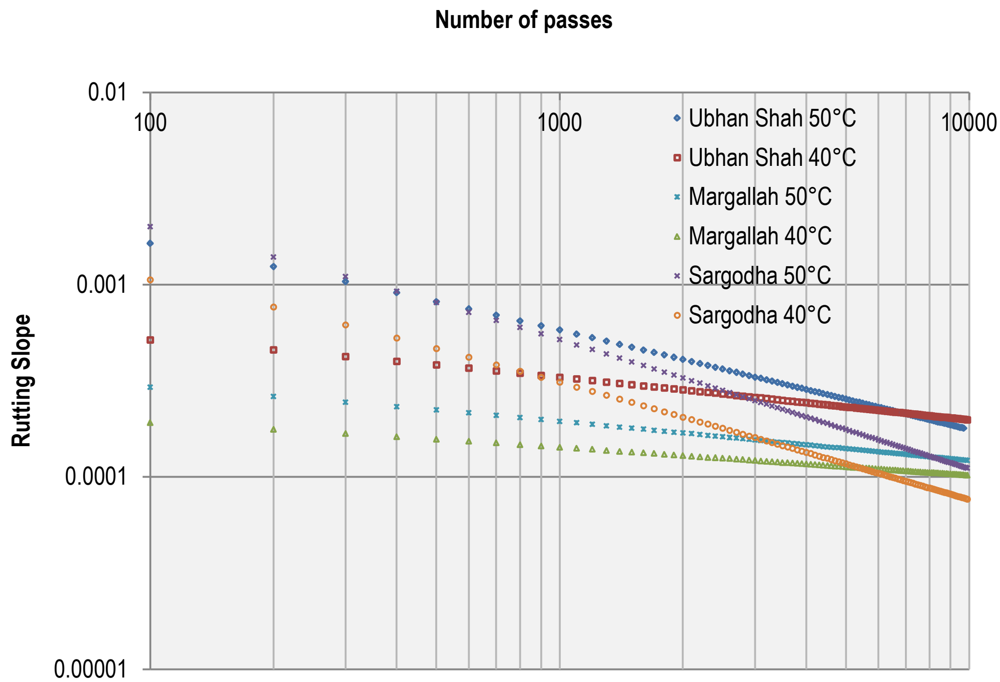


Figure 7-10 (b): Rutting Slope Plot for Cooper Wheel Tracking Test For NHA-A Gradation And 40/50 Pen. Grade Bitumen

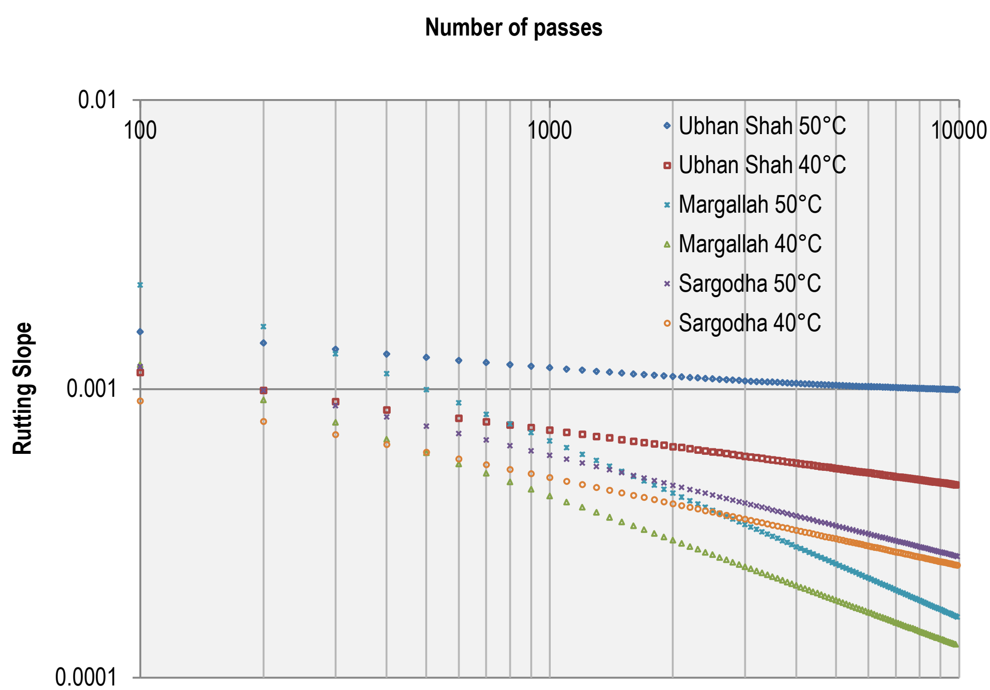


Figure 7-10 (c): Rutting Slope Plot for Cooper Wheel Tracking Test For NHA-B Gradation And 60/70 Pen. Grade Bitumen

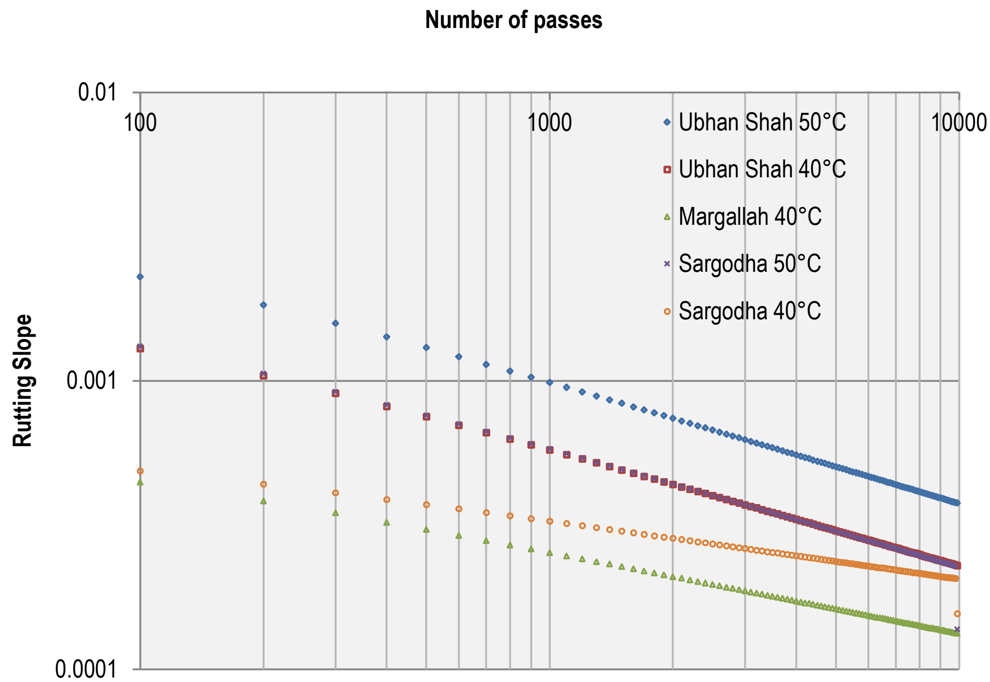


Figure 7.10 (d): Rutting Slope Plot for Cooper Wheel Tracking Test For NHA-B Gradation And 40/50 Pen. Grade Bitumen

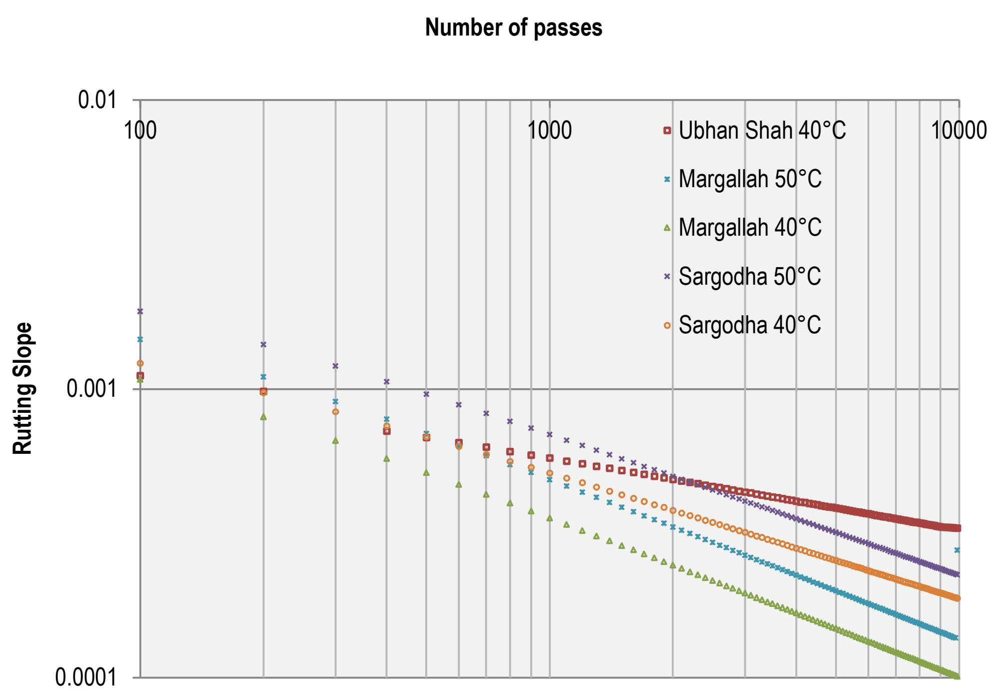


Figure 7.10 (e): Rutting Slope Plot for Cooper Wheel Tracking Test For SP-1 Gradation And 60/70 Pen. Grade Bitumen

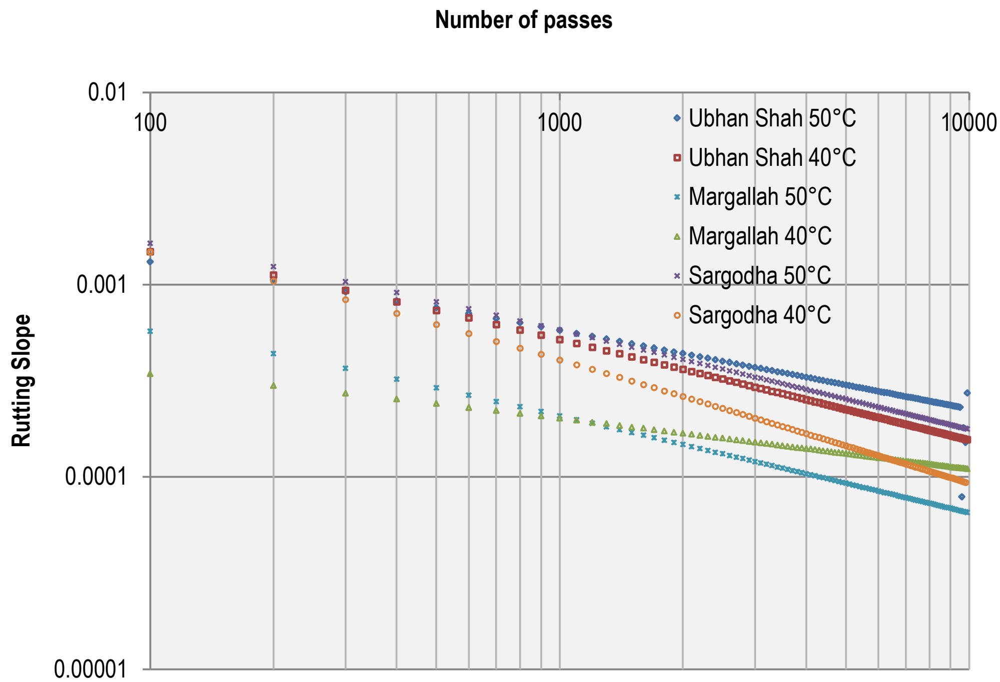


Figure 7.10 (f): Rutting Slope Plot for Cooper Wheel Tracking Test For SP-1 Gradation And 40/50 Pen. Grade Bitumen

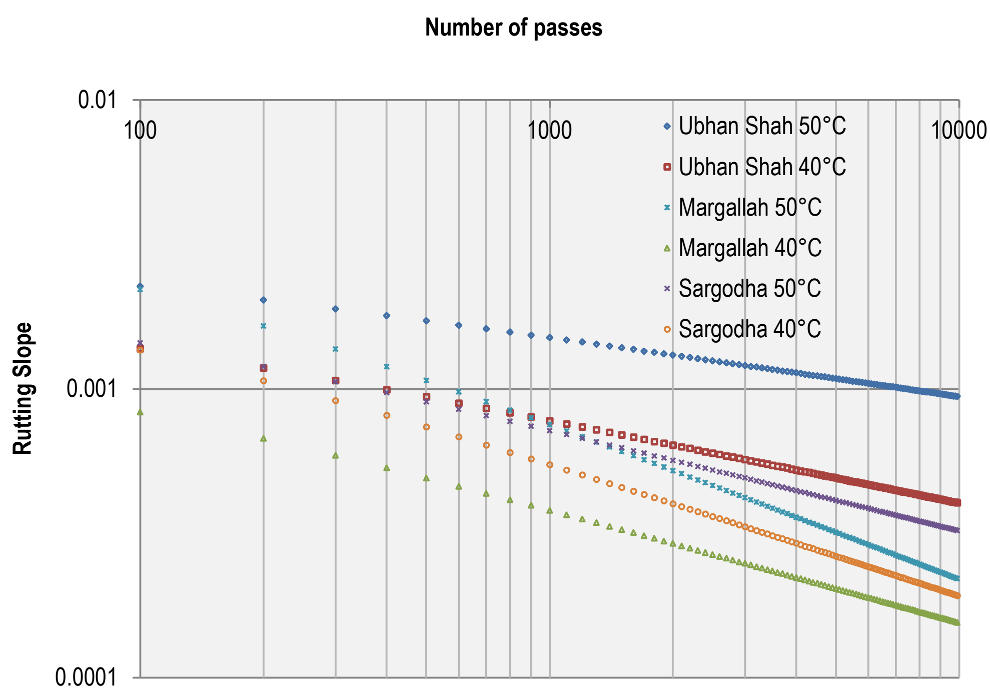


Figure 7.10 (g): Rutting Slope Plot for Cooper Wheel Tracking Test For SP-2 Gradation And 60/70 Pen. Grade Bitumen

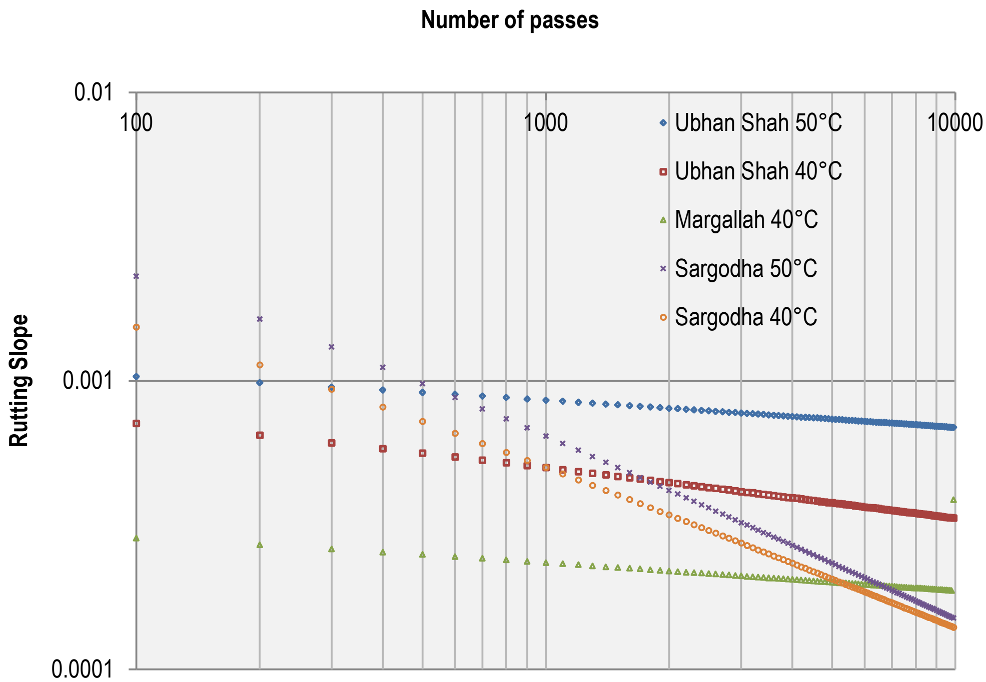


Figure 7.10 (h): Rutting Slope Plot for Cooper Wheel Tracking Test For SP-2 Gradation And 40/50 Pen. Grade Bitumen

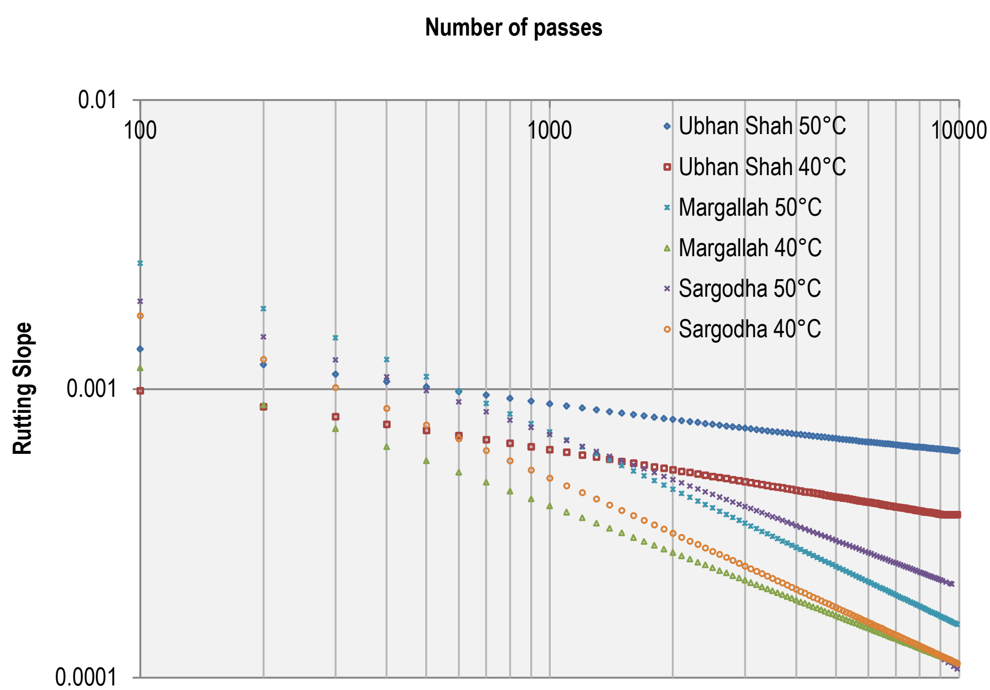


Figure 7.10 (i): Rutting Slope Plot for Cooper Wheel Tracking Test For MS-2 Gradation And 60/70 Pen. Grade Bitumen

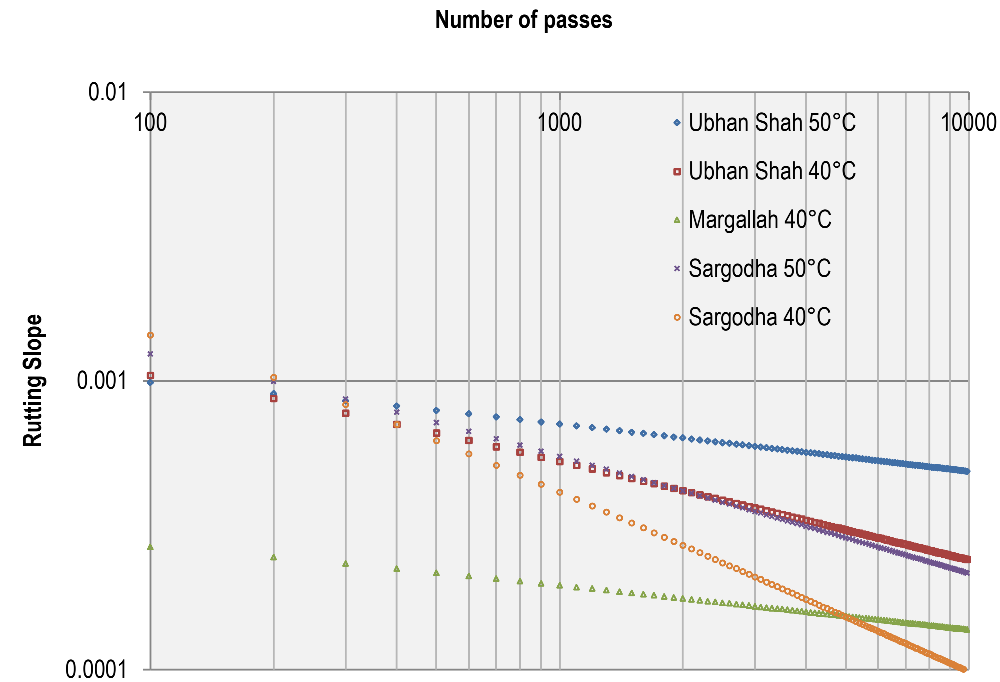


Figure 7.10 (j): Rutting Slope Plot for Cooper Wheel Tracking Test For MS-2 Gradation And 40/50 Pen. Grade Bitumen

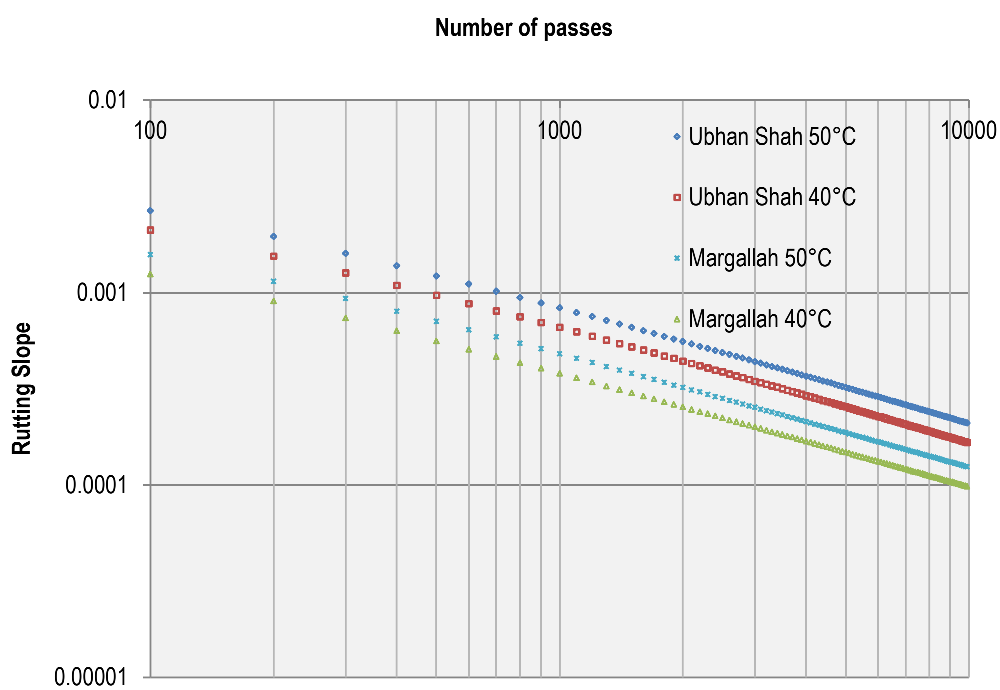


Figure 7.10 (k): Rutting Slope Plot for Cooper Wheel Tracking Test For NHA-A (BC) Gradation And 60/70 Pen. Grade Bitumen

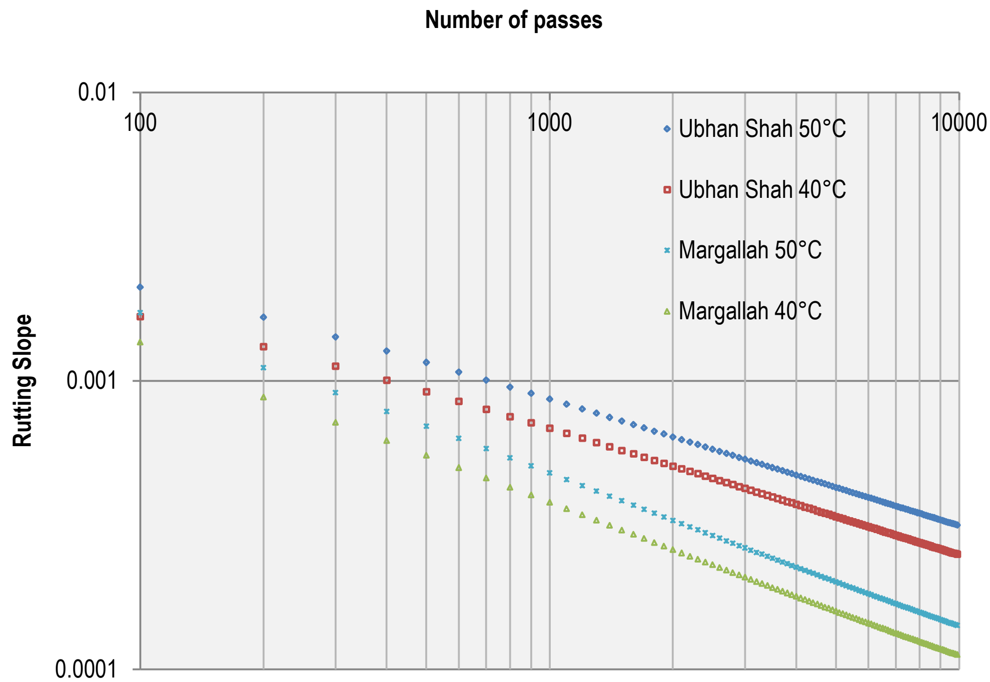


Figure 7.10 (l): Rutting Slope Plot for Cooper Wheel Tracking Test For NHA-B (BC) Gradation And 60/70 Pen. Grade Bitumen

Figure 7.10:Rutting Slope Plots for Cooper Wheel Tracking Test (CWTT)

7.3.2.4 Asphalt Pavement Analyzer Test

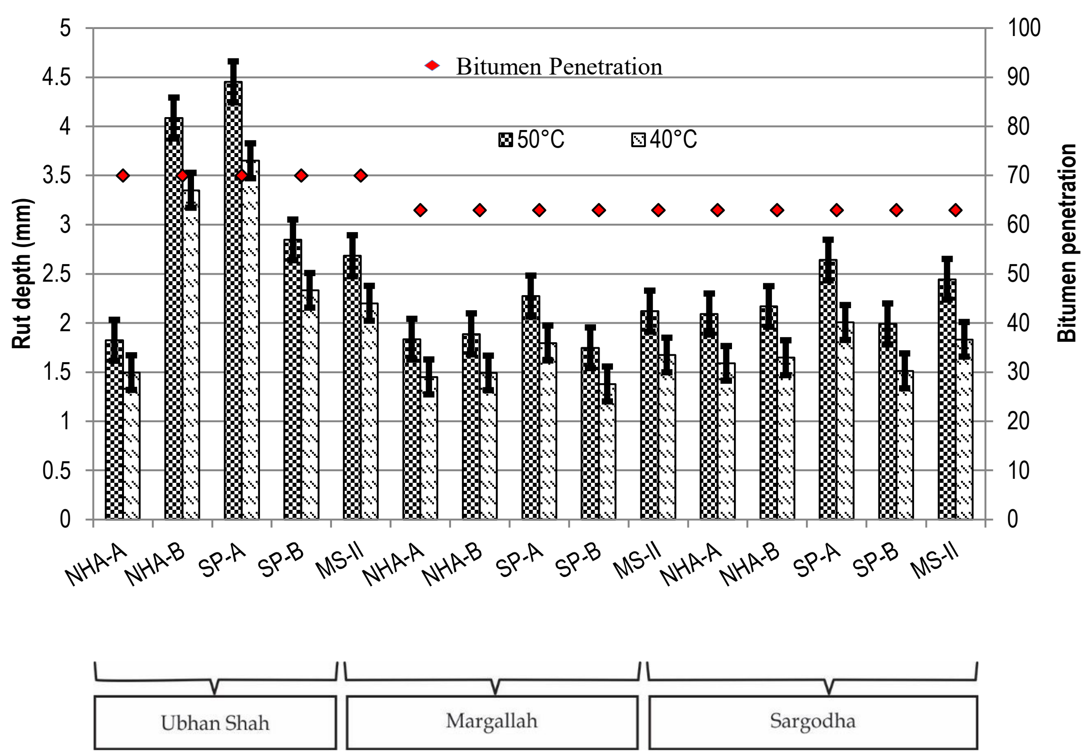
Figure 7.11: Asphalt Pavement Analyzer Test Software Output File

The rutting curve automatically generated during the test has been shown in Figure 7.11. The test conditions including temperature and frequency are shown at top of the screen while bottom portion shows rut development curve of three specimens placed in the temperature controlled enclosed chamber.

7.3.2.5 Evaluation Based on Terminal Rut Depth

Evaluation or ranking based on terminal rut depth at 8,000 loading cycles of thirty wearing course and eight base course mixtures using asphalt pavement analyzer test has been shown in Figure 7.12 and 7.13 respectively. Wearing course and base course mixtures were tested at temperature conditions of 40°C and 50°C. Figure 7.12 shows the relative rutting resistance of selected wearing course mixtures at 8,000 loading cycles. The red diamond shaped bullets shows the penetration value of bitumen plotted on secondary vertical axis. The bars in top portion of Figure 7.12 shows terminal rut depth values of fifteen wearing course mixtures with 60/70 pen. grade bitumen, while bars in bottom portion show terminal values of fifteen wearing course mixtures with 40/50 pen. grade bitumen at temperature conditions of 50°C and 40°C, respectively. Similarly Figure 7.13 elaborates the relative rutting performance of eight base course mixtures.

The individual mixtures evaluation based on terminal APA rut depth indicated identical mixtures ranking as observed in case of CWTT results. The individual ranking showed that NHA-A graded mixtures with aggregates of Margalla and bitumen of NRL 40/50 pen. grade are most resistant to rutting susceptibility while Ubhan Shah aggregates and NRL 60/70 bitumen composed SP-1 graded mixtures are least resistant to rutting distress. Figure 7.8 shows that all the asphalt mixtures reached 8000 cycles prior to achieving failure rut depth of 5 mm. The results plotted in Figure 7.12 further indicate that variability among the terminal rut depth is maximum for Ubhan Shah aggregates composed asphalt mixtures while minimum for mixtures with Margalla aggregates. Among base course mixtures, SP-2 (BC) with Ubhan Shah aggregates and NRL 60/70 pen. grade bitumen showed the maximum rut depth while DBM (BC) with Margalla aggregates and ARL 60/70 pen. grade has minimum value of rut depth in mm at 40°C and 50°C.



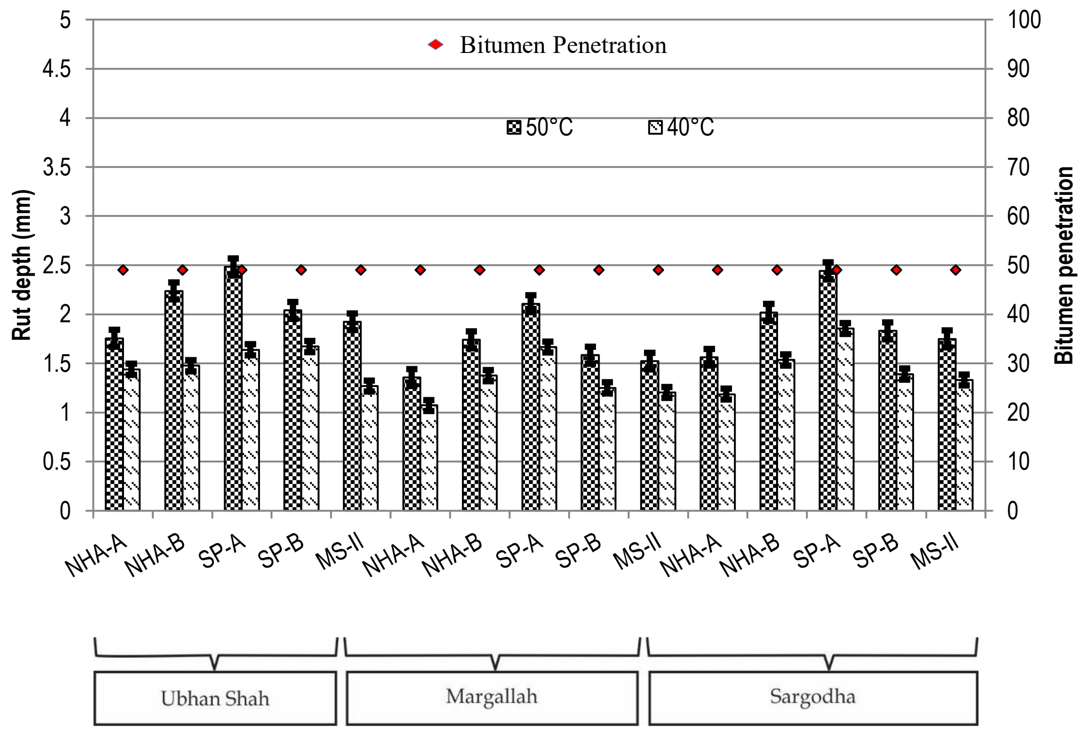


Figure 7.12: Test Results for Asphalt Pavement Analyzer Test (WC)

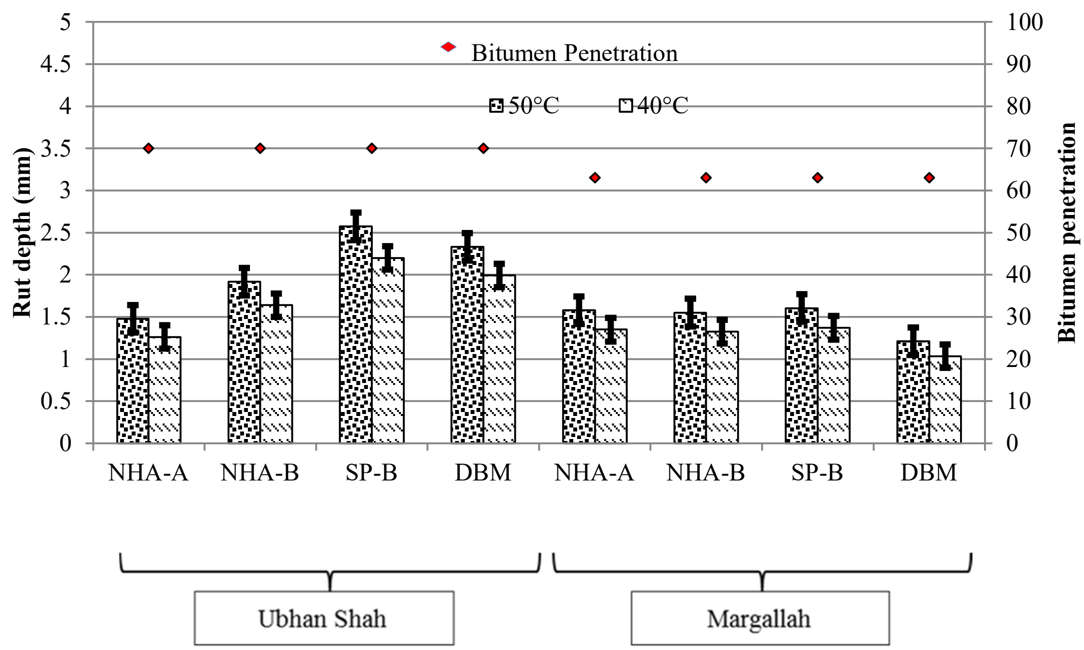


Figure 7.13: Test Results for Asphalt Pavement Analyzer Test (BC)

7.3.2.6 Evaluation Based on Slope of Rutting Curve

The rutting slope, computed by recorded APA rut depth and number of asphalt pavement analyzer cycles/strokes, has been plotted on log-log scale against number of loading cycles in Figure 7.14. It has generally been observed that variability among the rutting slopes of different mixtures is relatively lesser as compared to those of cooper wheel tracking test results. The plotted rutting slope curves are more streamlined and adjacent to each other as compared to those observed in cooper wheel tracking test.

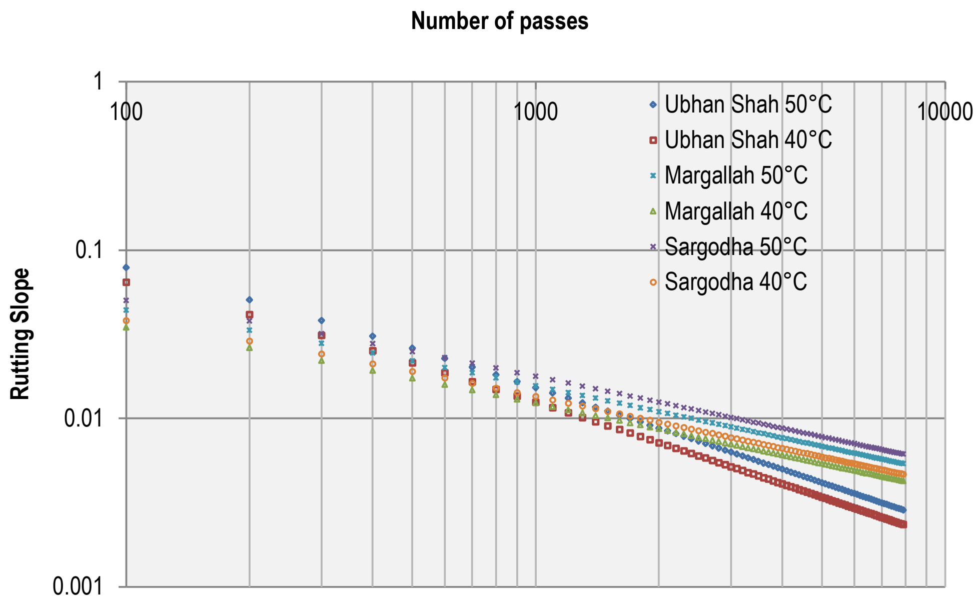


Figure 7.14 (a): Rutting Slope Plot for Asphalt Pavement Analyzer Test For NHA-A Gradation And 60/70 Pen. Grade Bitumen

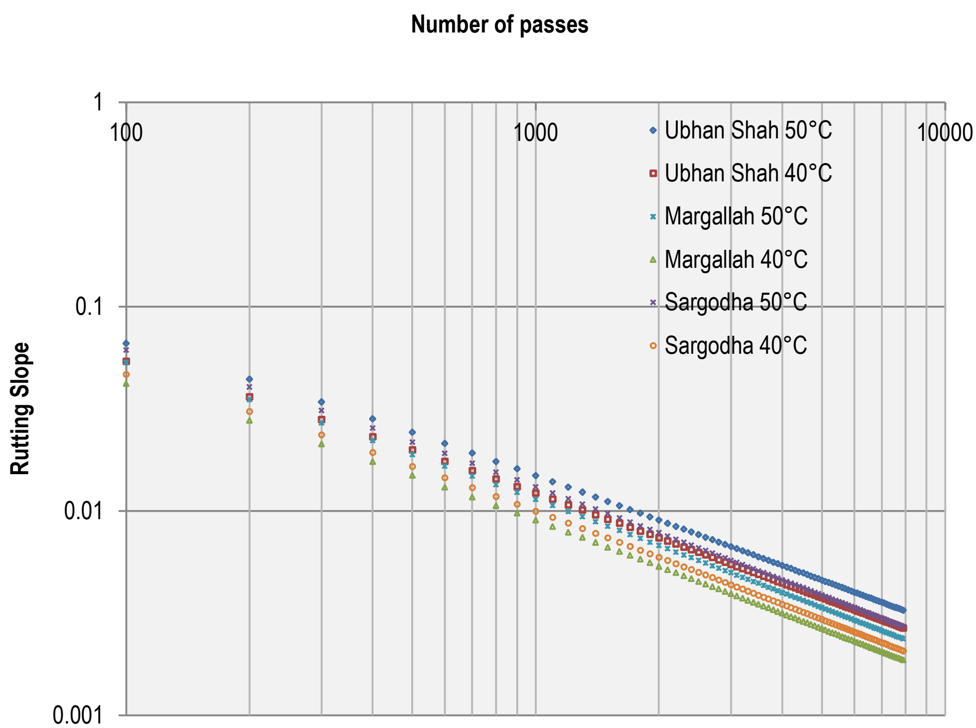


Figure 7.14 (b): Rutting Slope Plot for Asphalt Pavement Analyzer Test For NHA-A Gradation And 40/50 Pen. Grade Bitumen

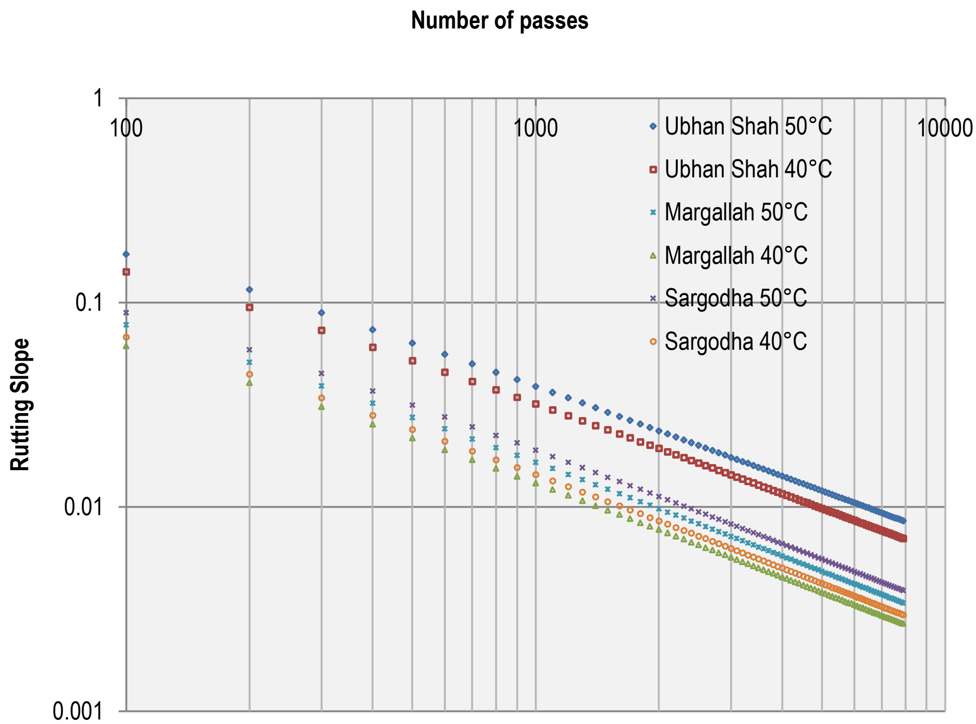


Figure 7.14 (c): Rutting Slope Plot for Asphalt Pavement Analyzer Test For NHA-B Gradation And 60/70 Pen. Grade Bitumen

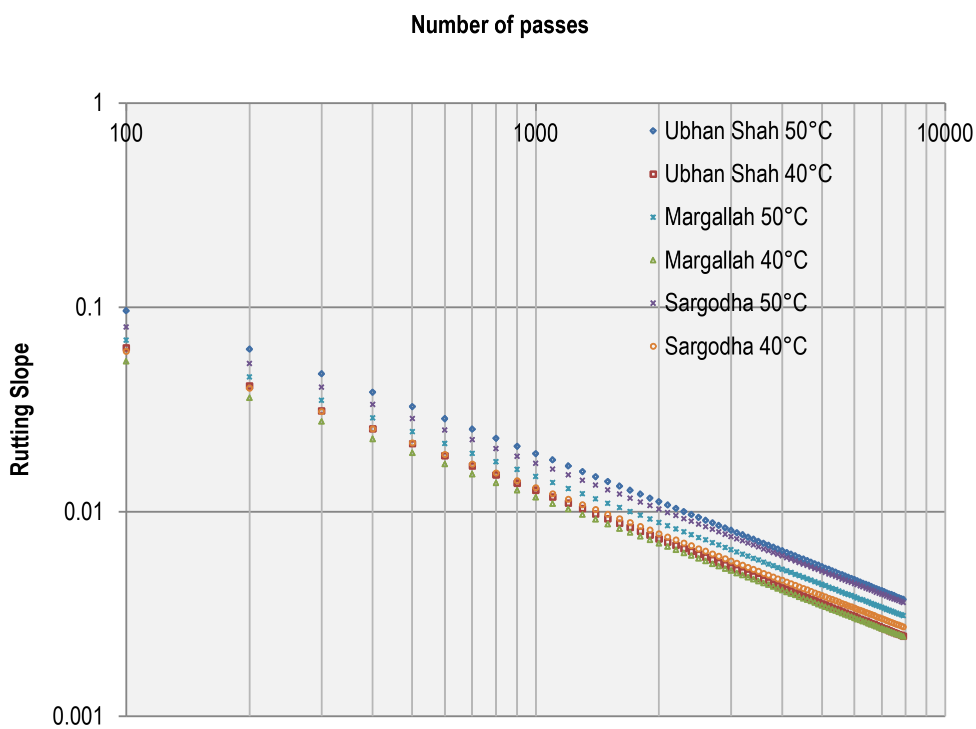


Figure 7.14 (d): Rutting Slope Plot for Asphalt Pavement Analyzer Test For NHA-B Gradation And 40/50 Pen. Grade Bitumen

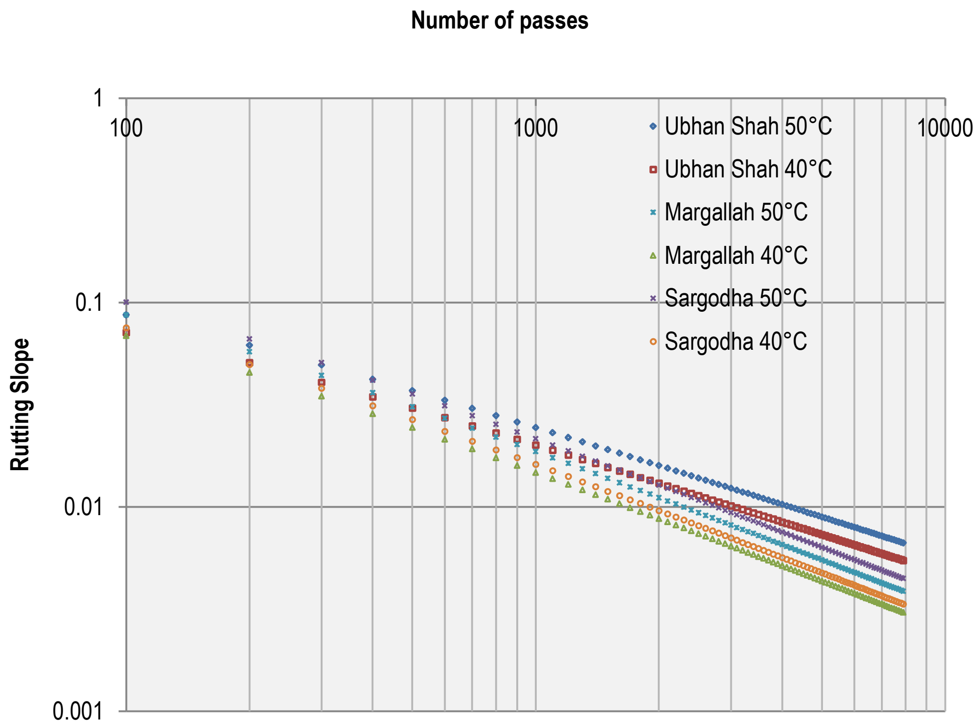


Figure 7.14 (e): Rutting Slope Plot for Asphalt Pavement Analyzer Test For SP-1 Gradation And 60/70 Pen. Grade Bitumen

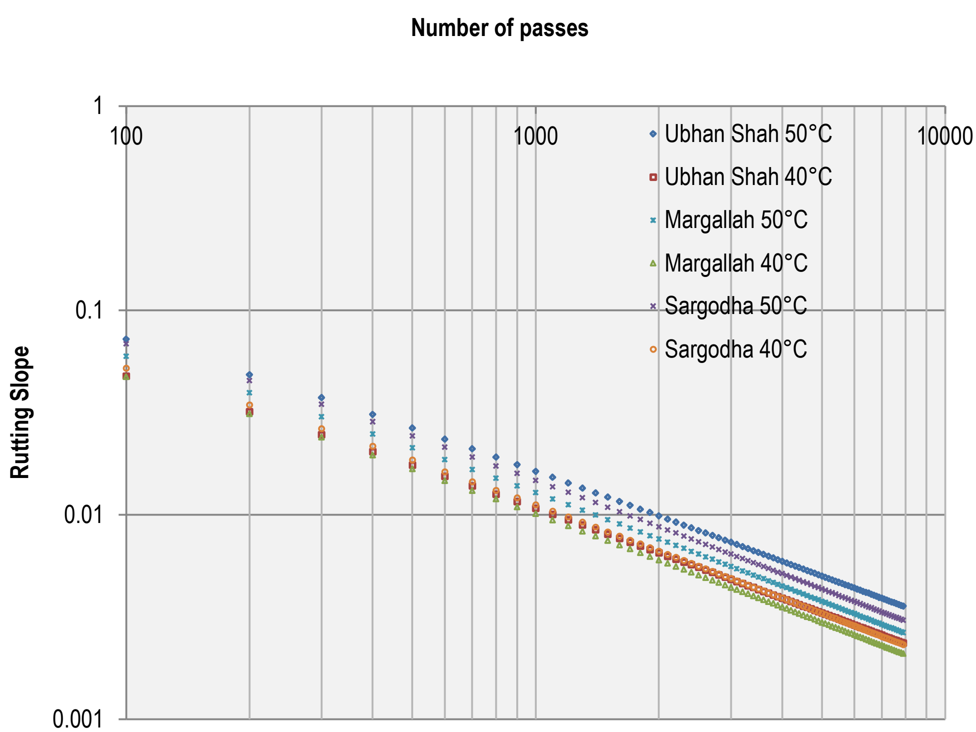


Figure 7.14 (f): Rutting Slope Plot for Asphalt Pavement Analyzer Test For SP-1 Gradation And 40/50 Pen. Grade Bitumen

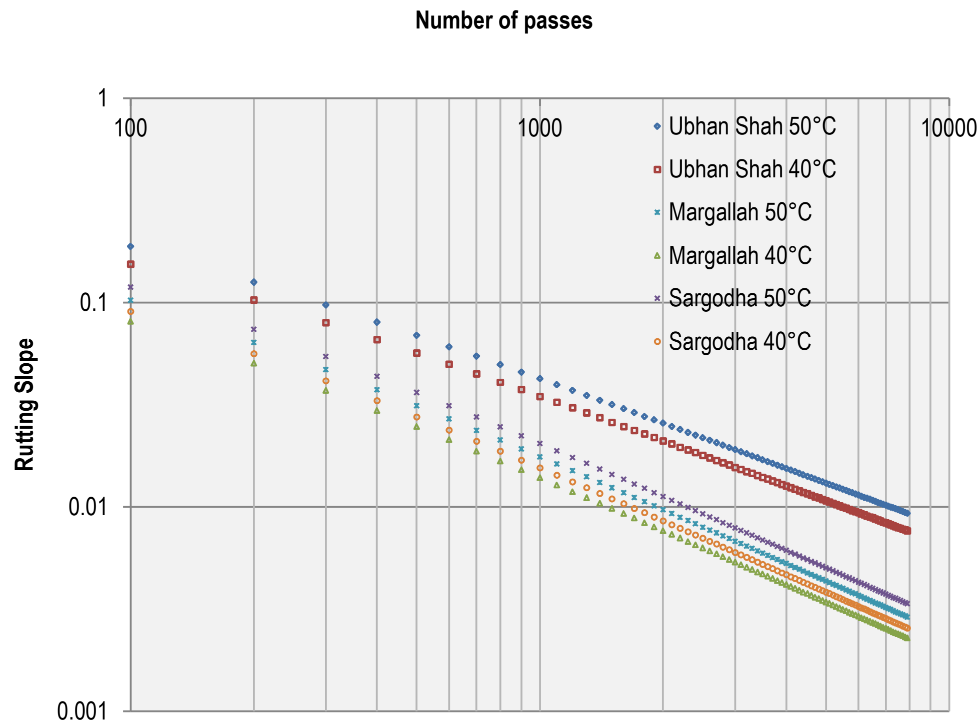


Figure 7.14 (g): Rutting Slope Plot for Asphalt Pavement Analyzer Test For SP-2 Gradation And 60/70 Pen. Grade Bitumen

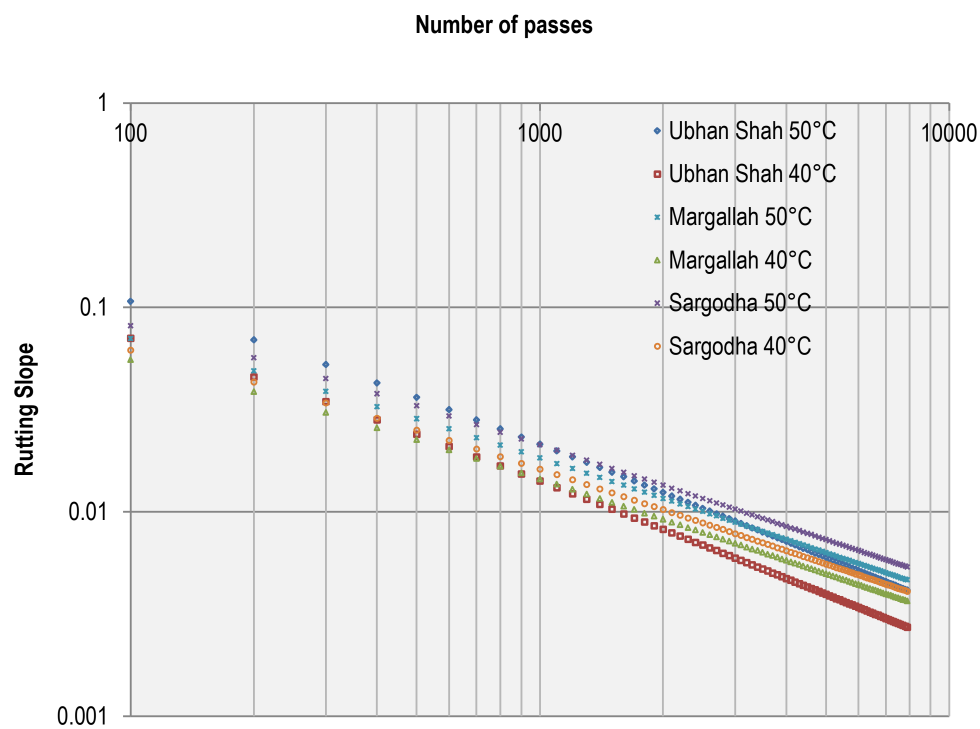


Figure 7.14 (h): Rutting Slope Plot for Asphalt Pavement Analyzer Test For SP-2 Gradation And 40/50 Pen. Grade Bitumen

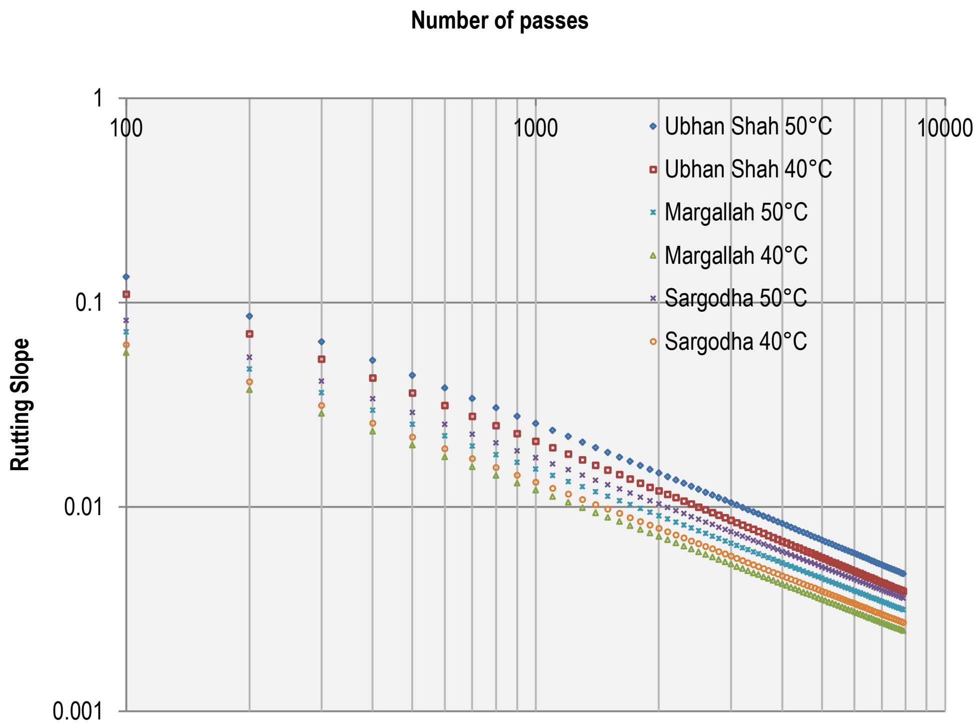


Figure 7.14 (i): Rutting Slope Plot for Asphalt Pavement Analyzer Test For MS-2 Gradation And 60/70 Pen. Grade Bitumen

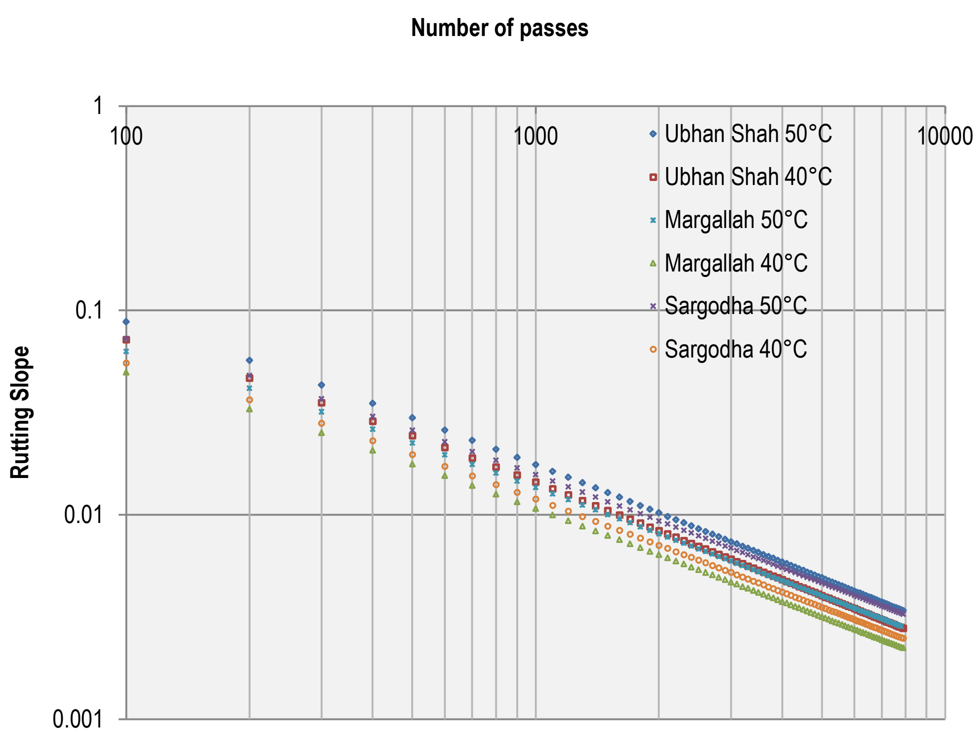


Figure 7.14 (j): Rutting Slope Plot for Asphalt Pavement Analyzer Test For MS-2 Gradation And 40/50 Pen. Grade Bitumen

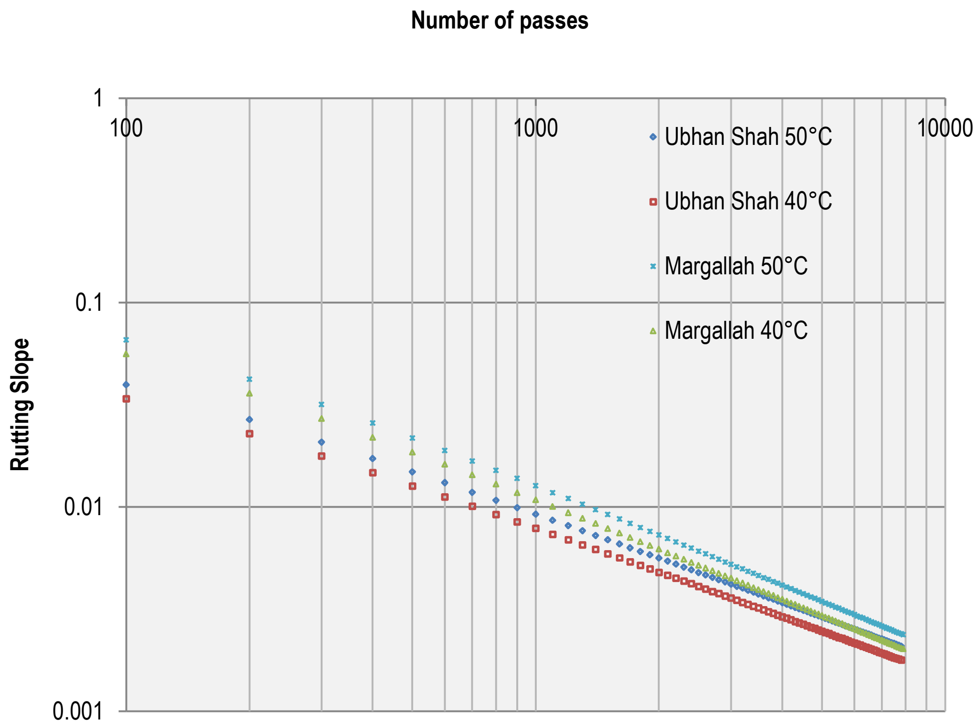


Figure 7.14 (k): Rutting Slope Plot for Asphalt Pavement Analyzer Test For NHA-A (BC) Gradation And 60/70 Pen. Grade Bitumen

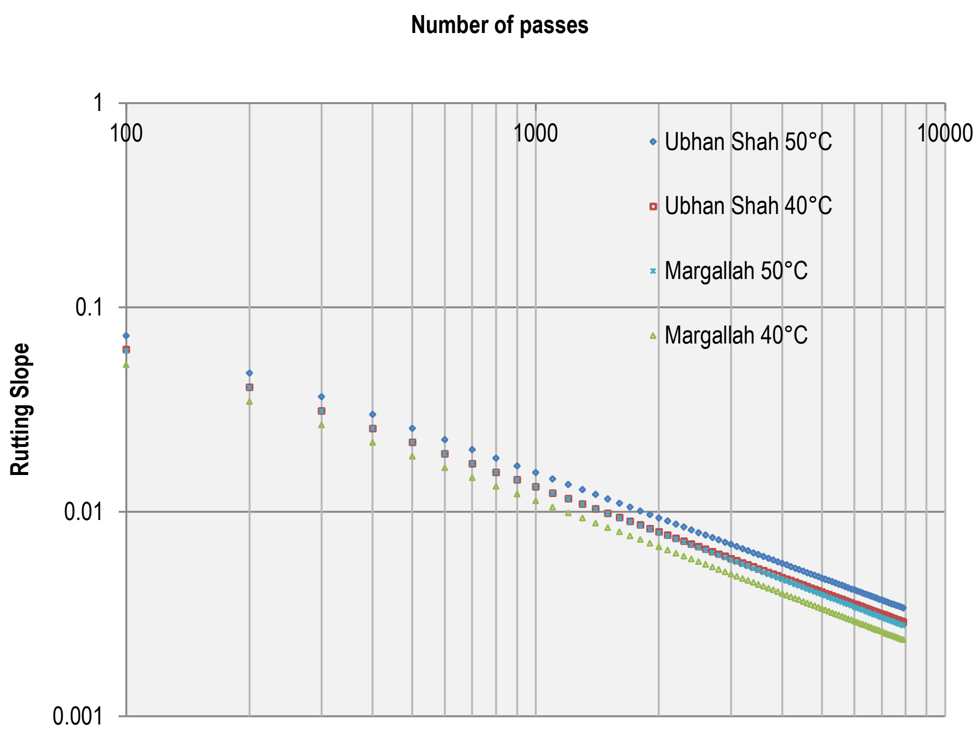


Figure 7.14 (l): Rutting Slope Plot for Asphalt Pavement Analyzer Test For NHA-B (BC) Gradation And 60/70 Pen. Grade Bitumen

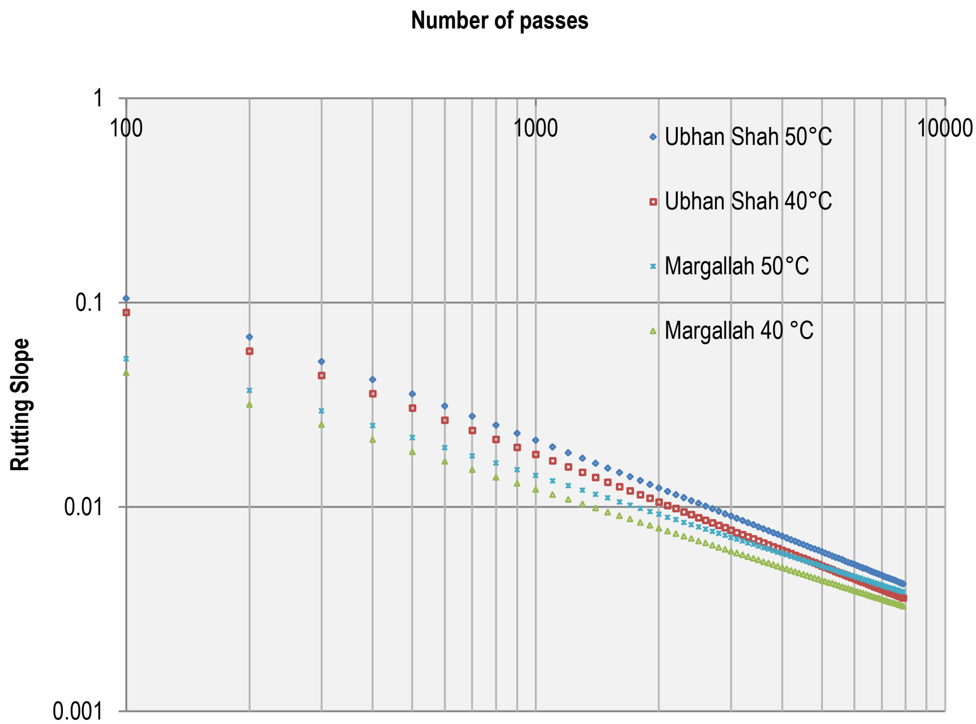


Figure 7.14 (m): Rutting Slope Plot for Asphalt Pavement Analyzer Test For SP-2 (BC) Gradation And 60/70 Pen. Grade Bitumen

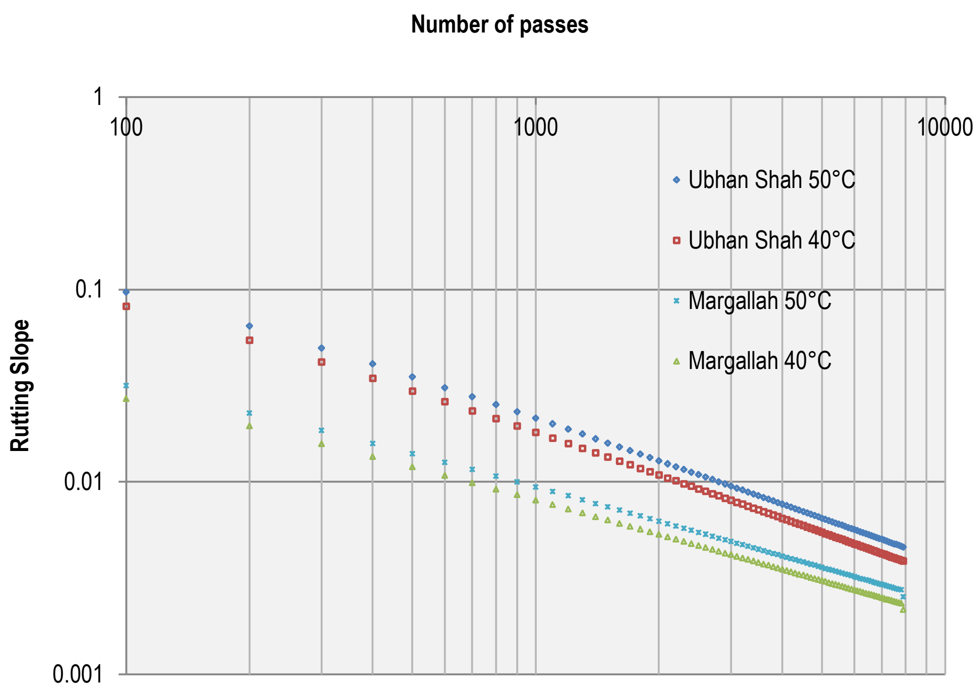


Figure 7.14 (n): Rutting Slope Plot for Asphalt Pavement Analyzer Test for DBM (BC) Gradation And 60/70 Pen. Grade Bitumen

Figure 7.14: Rutting Slope Plots for Asphalt Pavement Analyzer Test

These observations endorsed the improved compaction uniformity of specimens prepared by using Superpave gyratory compactor as compared to those prepared by cooper roller compactor for this specific study. It has been further concluded from Figure 7.14 that mixtures with Ubhan Shah aggregates have large rutting slopes as compared to mixtures with Margalla and Sargodha aggregates. However, for NHA-B and SP-2 graded mixtures with 60/70 pen. grade bitumen the variation is more pronounced as compared to mixtures with other wearing and base course gradations.

7.3.2.7 Repeated Load Axial Test

7.3.2.7.1 Evaluation based on Terminal Permanent Strain

Repeated load axial test (RLAT) was conducted at two stress levels of 300 and 500 kPa and three temperature levels of 25, 40 and 50°C, therefore, cumulative six combinations of testing conditions have been shown for each asphalt mixture in Figure 7.15. As discussed previously, the test was set to terminate at 3% permanent strain or 3600 loading cycles, whichever reached earlier. The ordinate of bar charts in Figure 7.15 comprises of primary and secondary vertical axis. The primary vertical axis measures the terminal number of cycles indicated by grey bars, while secondary vertical axis scales the permanent strain at terminal number of cycles shown by black bars. The abscissa in Figure 7.15 shows each of the tested wearing course asphalt mixture.

It has been observed that the terminal number of cycles consistently reached to 3600 for relatively least aggressive testing condition of 300 kPa stress and 25°C temperature as shown in Figure 7.15a. The corresponding permanent strain values were used to evaluate or rank the relative permanent deformation behavior of various asphalt mixtures. The ranking of mixtures on the basis of mixture gradation is nearly identical for all combinations of bitumen and aggregates at a specific temperature and stress condition. Among the tested samples of asphalt mixtures, SP-1 with Ubhan Shah aggregates and NRL 60/70 pen. grade bitumen has been observed to be most rut susceptible mixture. However, NHA-A with Margalla aggregate and ARL 60/70 pen. grade bitumen showed the maximum rutting resistance among the selected mixtures. Figure 7.15b indicated that when stress was increased to 500 kPa at same temperature condition of 25°C, majority of asphalt mixtures still sustained until terminal condition of 3600 loading cycles was achieved. But the observed permanent strain values were on the higher side as compared to those of initial testing condition. It has also been observed that some asphalt mixtures with Ubhan Shah aggregates reached the failure strain of 3% prior to 3600 loading cycles. Figure 7.15c shows the similar trend that Ubhan Shah mixtures failed prematurely along-with couple of fine graded mixtures with other aggregate sources of Margalla and Sargodha. Figure 7.15d through 15f shows the progressively deteriorating permanent deformation performance of asphalt mixtures with ascending trend in severity of stress and temperature conditions.

The mixture ranking has been observed to be nearly consistent for different temperatures. As temperature increases there is drastic decrease in resistance to permanent accumulated strain. Especially at high stress levels and high temperature conditions, the terminal strain of 3% was achieved at very initial stages. It can also be observed that finer mixtures failed earlier at extreme temperature and loading conditions. The lack of point-to-point contact in case of finely graded mixtures could be probable reason of this behavior (Al-Mosawe et al., 2016).

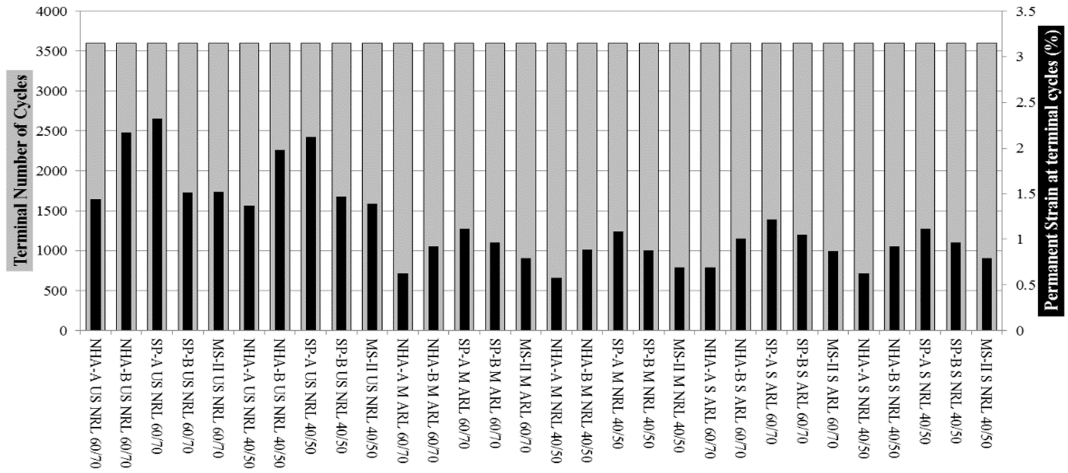


Figure 7.15 (a): Mixture Ranking for Repeated Load Axial Test at 300 kPa Stress And 25°C (WC)

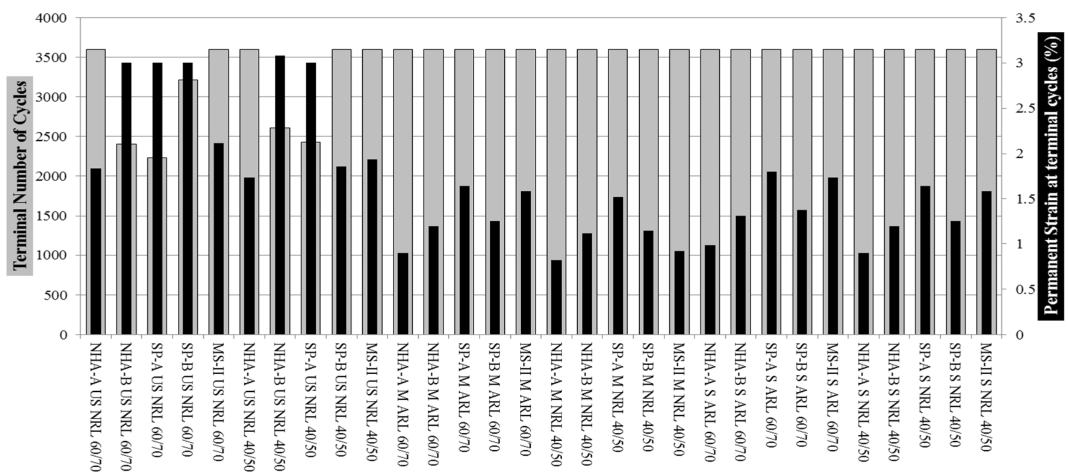


Figure 7.15 (b): Mixture Ranking for Repeated Load Axial Test at 500 kPa Stress And 25°C (WC)

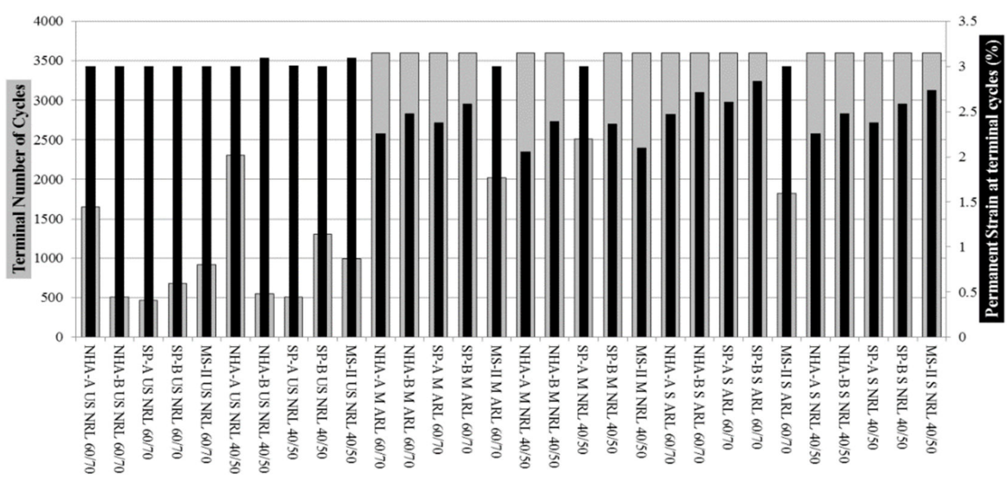


Figure 7.15 (c): Mixture Ranking for Repeated Load Axial Test at 300 kPa Stress And 40°C (WC)

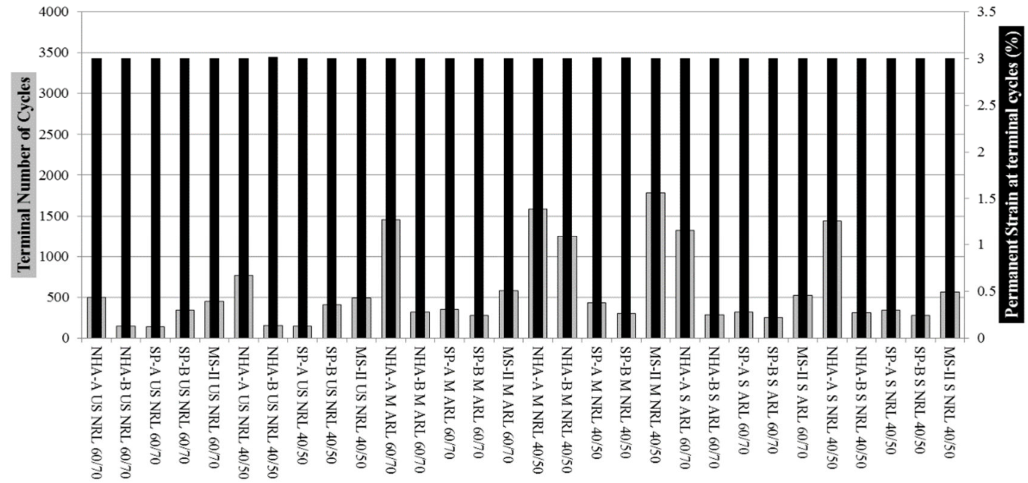


Figure 7.15 (d): Mixture Ranking for Repeated Load Axial Test at 500 kPa Stress And 40°C (WC)

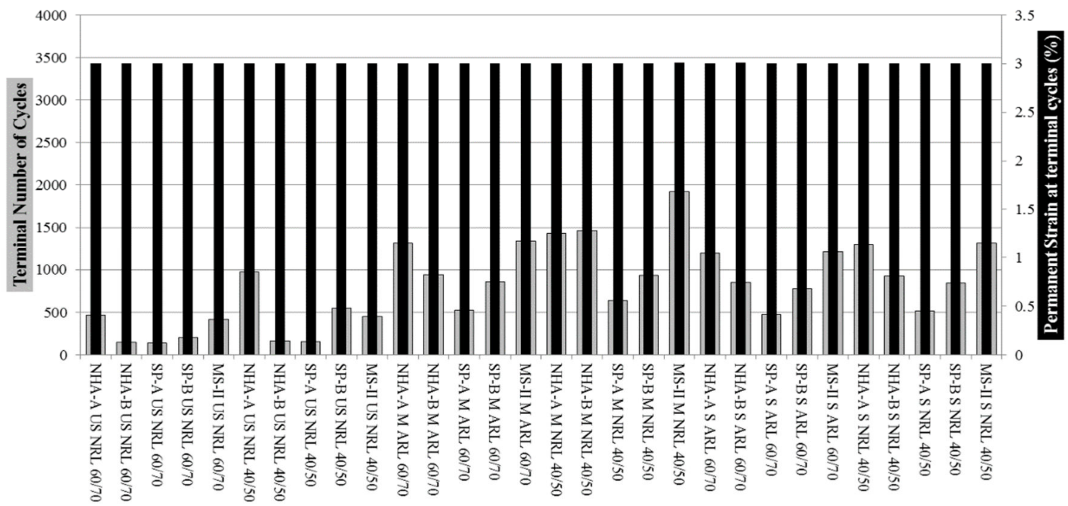


Figure 7.15 (e): Mixture Ranking for Repeated Load Axial Test At 300 kPa Stress And 50°C (WC)

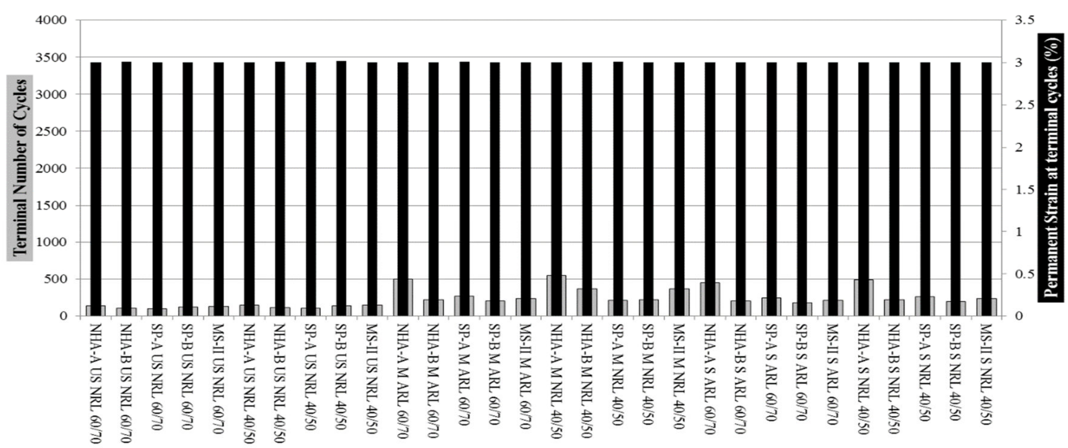


Figure 7.15 (f): Mixture Ranking for Repeated Load Axial Test At 500 kPa Stress And 50°C (WC)

Figure 7.15: Mixture Ranking for Repeated Load Axial Test (WC)

It may be noted from Figure 7.16 that among the tested samples of base course mixtures, SP-2 (BC) with Ubhan Shah aggregates and NRL 60/70 pen. grade bitumen has been observed to be the most rut susceptible mixture among the selected mixtures as terminal value of 3% strain has been achieved at 3160 cycles, while DBM (BC) with Margalla aggregate and ARL 60/70 showed least rut susceptibility i.e., a permanent strain value of 0.90 % at terminal value of 3600 cycles. At high temperatures of 40°C and 50°C the ranking for mixtures with Ubhan Shah aggregates was different from that of Margalla aggregates i.e., DBM (BC) mixture with Ubhan Shah aggregates has greater permanent strain as compared to NHA-A (BC) mixture with Ubhan Shah aggregates, conversely, permanent strain of NHA-A (BC) mixture with Margalla aggregates is greater than that of DBM (BC) mixture with Margalla aggregate. This difference in ranking could be due to the physical and mechanical material properties of both aggregate sources.

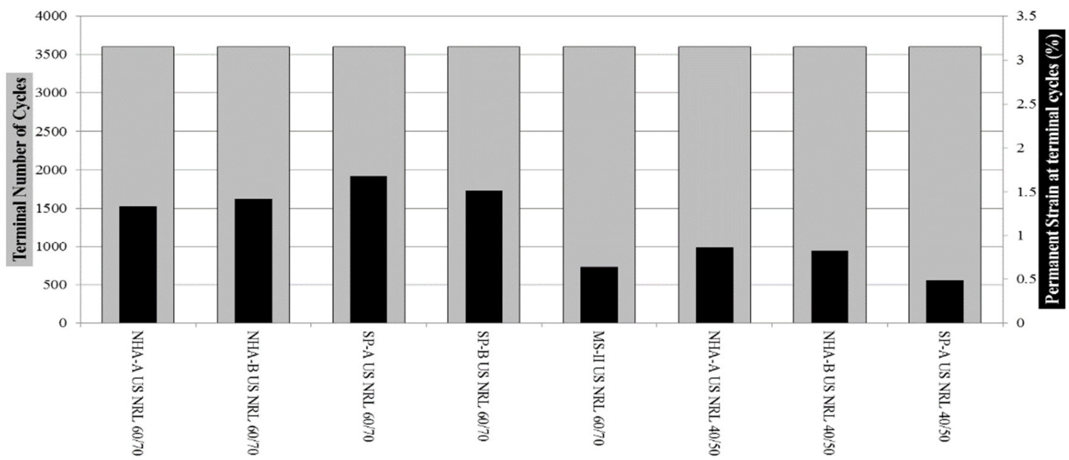


Figure 7.16 (a): Mixture Ranking for Repeated Load Axial Test At 300 kPa Stress And 25°C (BC)

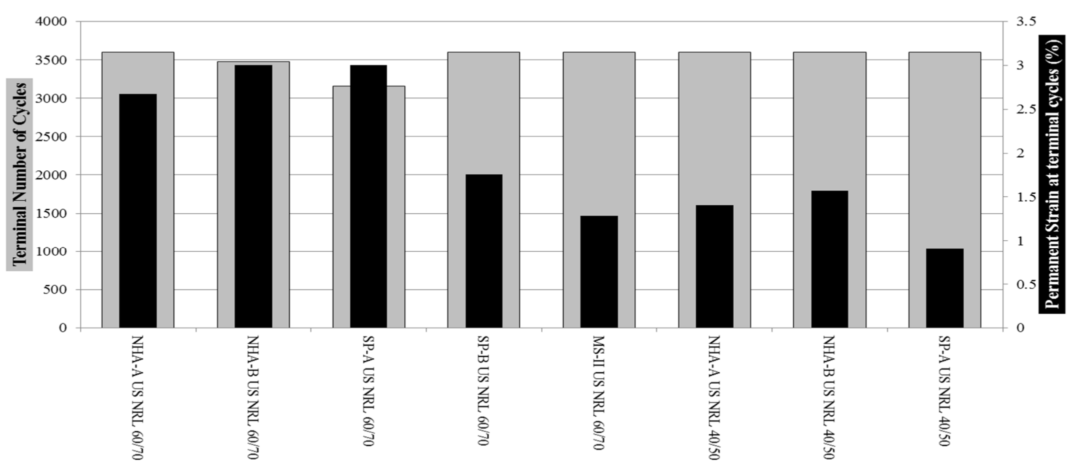


Figure 7.16 (b): Mixture Ranking for Repeated Load Axial Test At 500 kPa Stress And 25°C (BC)

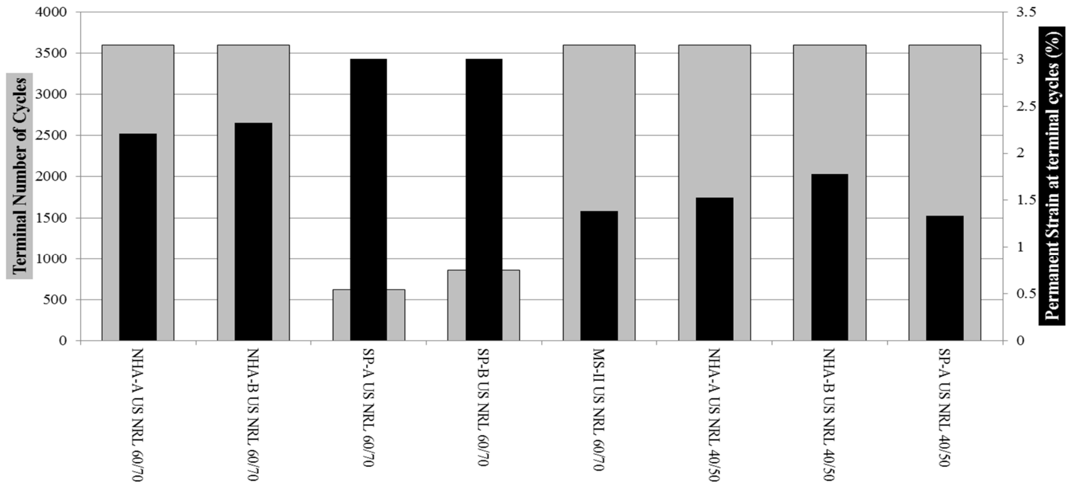


Figure 7.16 (c): Mixture Ranking for Repeated Load Axial Test At 300 kPa Stress And 40°C (BC)

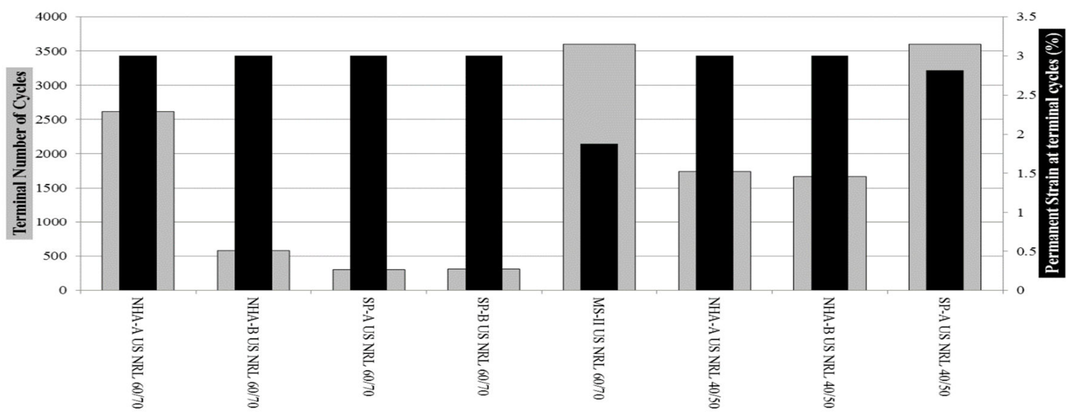


Figure 7.16 (d): Mixture Ranking for Repeated Load Axial Test At 500 kPa Stress And 40°C (BC)

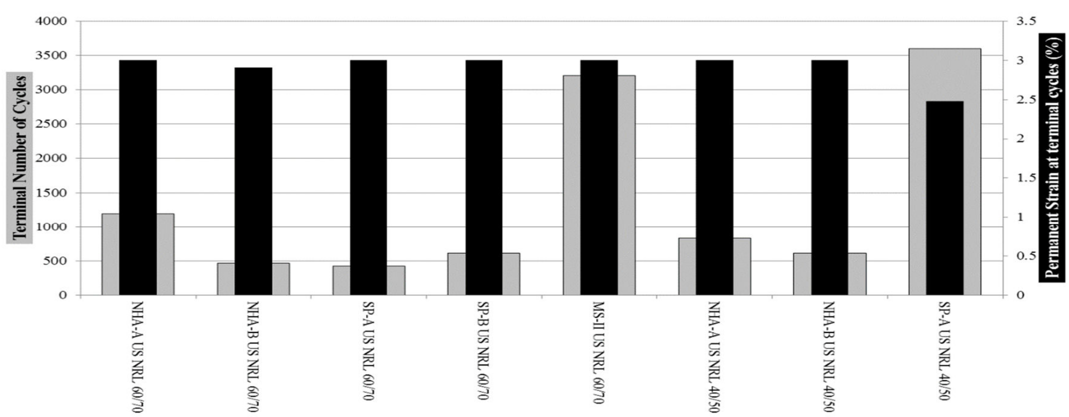


Figure 7.16 (e): Mixture Ranking for Repeated Load Axial Test At 300 kPa Stress And 50°C (BC)

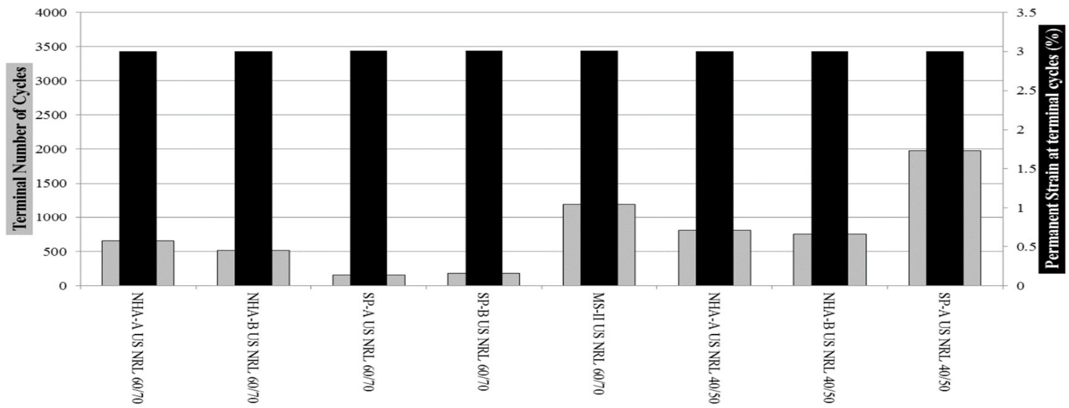


Figure 7.16 (f): Mixture Ranking for Repeated Load Axial Test At 500 kPa Stress And 50°C (BC)

Figure 7.16:Mixture Ranking for Repeated Load Axial Test (BC)

7.3.2.7.2 Evaluation based on permanent strain curves/permanent strain slope curves

The variation of permanent strain slope with loading cycles 'N' is another important parameter which can be used to evaluate permanent deformation behavior of asphalt mixtures. The permanent strain slope has been computed by Equation 7-2 in which N = Number of cycles, p = previous number of cycles. In case of 3% terminal permanent strain achieved at less than 1000 loading cycles, data is automatically stored in increments of single cycle; if test terminates between 1000 and 2000 loading cycles, data is stored in increments of two cycles; and data is stored by default in increments of four cycles if test continued for more than 2000 loading cycles. The permanent strain slope plots for all the thirty asphalt mixtures have been plotted in Figure 7.17 below for two stress and three temperature conditions.

$$Permanent\ Strain\ Slope\ at\ N = \frac{(Permanent\ strain_N - Permanent\ strain_P)}{(N-p)} \qquad 7\text{-}2$$

It has been observed from Figure 7.17a and 17b that permanent strain slope follows nearly similar pattern when stress was increased from 300 kPa to 500 kPa, at the temperature condition of 25°C. Permanent strain slope is divergent initially for various mixtures, but stabilized with the increasing loading cycles. However, the waviness of data points at the stabilized stage is slightly higher in case 500 kPa stress. Figure 7.4c and 7.4d shows the permanent strain slope curves at temperature condition of 40°C. As 40°C is initiation of high temperature range, the effect of stress has been observed to be more pronounced above this temperature. The waviness among permanent strain slope values has been increased by a significant amount with the increase in stress from 300 kPa to 500 kPa for the results at temperature conditions of 40°C and higher. Typical permanent deformation curve of asphalt material can be divided in three stages, primary stage having high initial level of rutting, predominantly associated with volumetric change; secondary stage having small rate of rutting exhibiting a constant rate of change of rutting and tertiary stage with high level of rutting predominantly associated with plastic (shear) deformations under no volume change conditions (Hussan et al., 2017). The tertiary stage of permanent deformation has also been achieved for some mixtures, as indicated by reversal of direction of permanent strain slope. Figure 7.17e and 17f shows the permanent deformation behavior at most severe temperature condition of 50°C. It is clearly visible that asphalt rutting resistance drastically reduces at high temperature; therefore, the permanent strain slope is extremely divergent at temperature of 50°C. It can be concluded from Figure 7.17f that majority of representative wearing course mixtures entered the tertiary zone well before the terminal cycles of 3600. However, due to weak resistance of an unconfined sample, it is hard to isolate the tertiary stage from primary and secondary stage for majority of the asphalt mixtures.

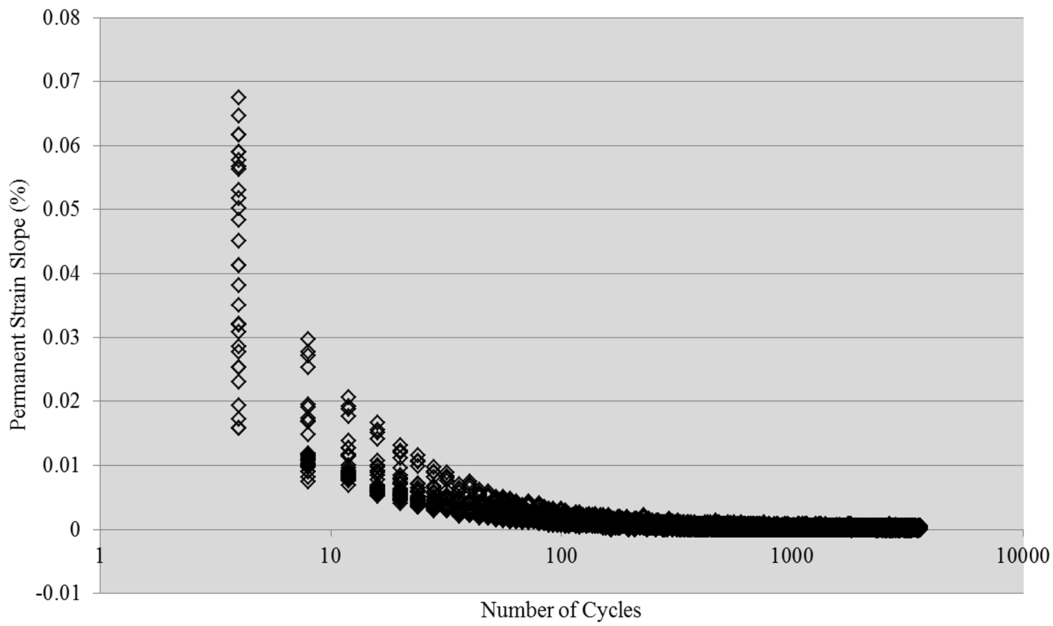


Figure 7.17 (a): Permanent Strain Slope Curves For 300 kPa And 25°C

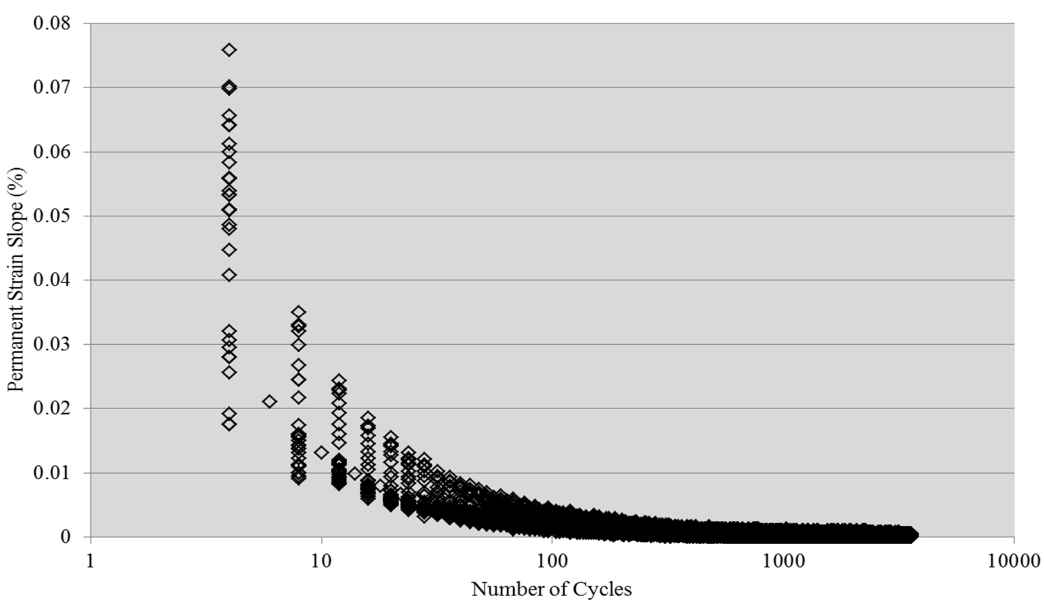


Figure 7.17 (b): Permanent Strain Slope Curves For 500 kPa And 25°C

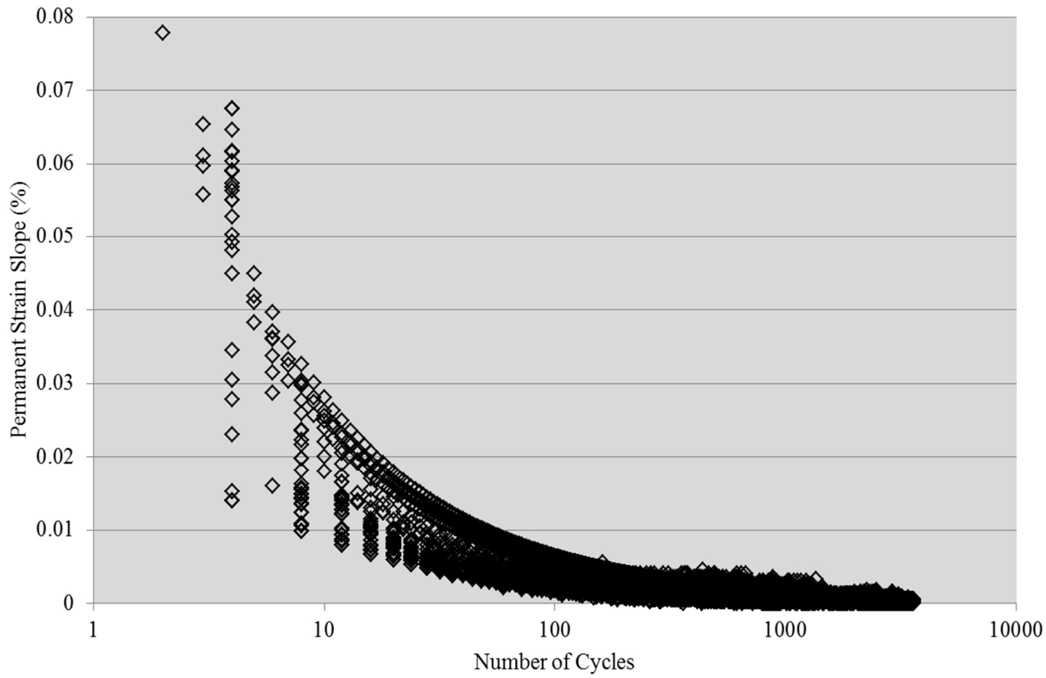


Figure 7.17 (c): Permanent Strain Slope Curves For 300 kPa And 40°C

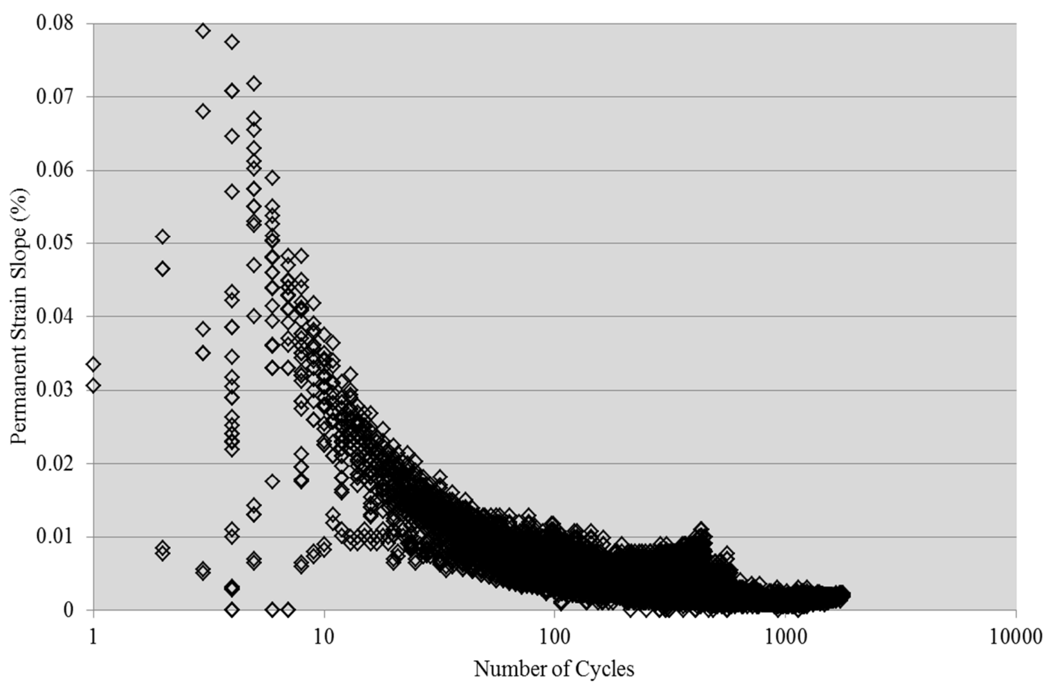


Figure 7.17 (d): Permanent Strain Slope Curves For 500 kPa And 40°C

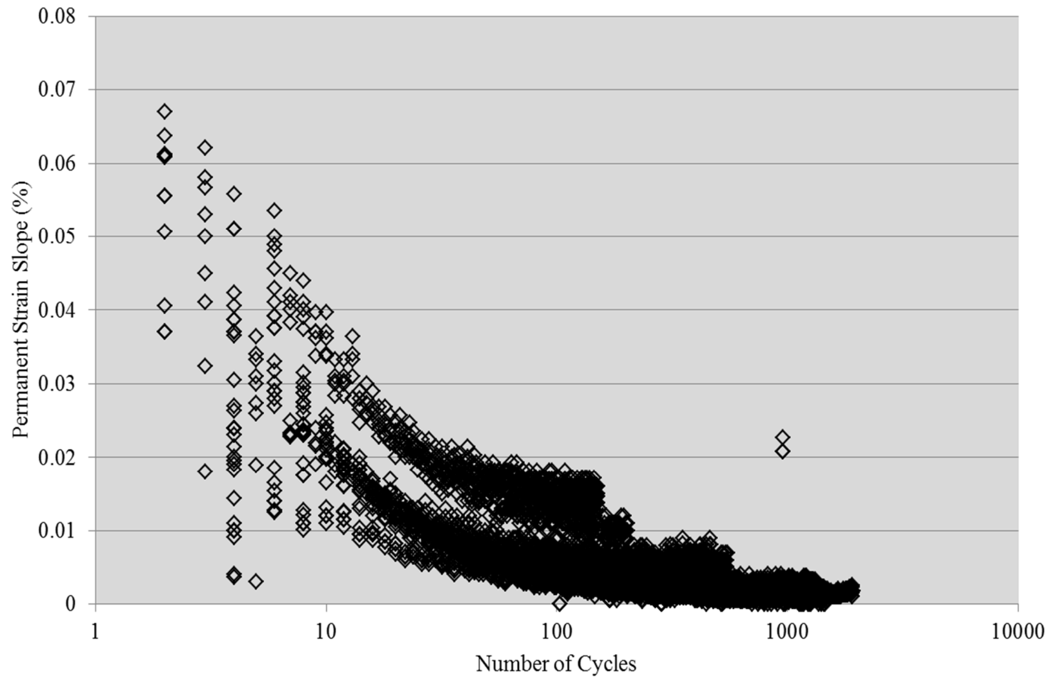


Figure 7.17 (e): Permanent Strain Slope Curves For 300 kPa And 50°C

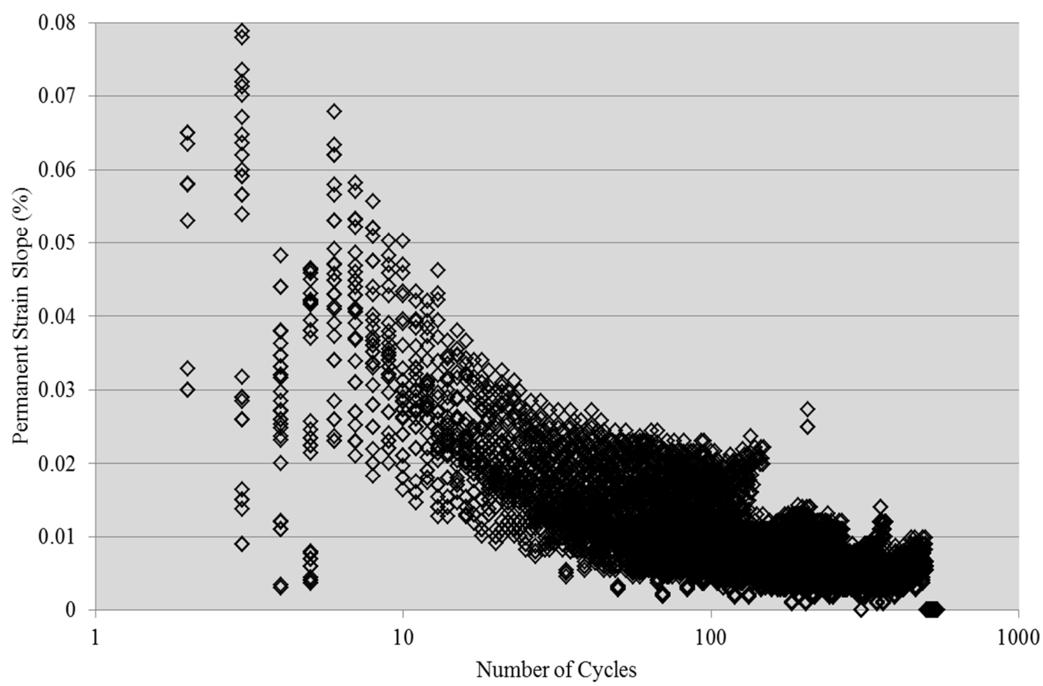


Figure 7.17 (f): Permanent Strain Slope Curves For 500 kPa and 50°C

Figure 7.17: Permanent Strain Slope Curves for Dynamic Creep Test in Uniaxial Mode

7.3.2.8 Comparison/Ranking of Rutting Performance Tests

Different rutting performance tests were conducted according to their own specific standard protocols. Hence, the loading pattern, loading magnitude, loading frequency, specimen shape and compaction procedure vary for each performance test. As diverse facilities exist at different research laboratories, this study compared relative ranking of similar asphalt mixtures influenced by using different rutting performance tests (i.e., asphalt pavement analyzer test, cooper wheel tracking test, and repeated load axial test) as per standard protocols.

Different rutting performance tests could be simultaneously compared by plotting time required to achieve 1 mm absolute rut depth for similar set of asphalt mixtures (Rushing and Little, 2014). The same criterion has been used in this research study as plotted in Figure 7.18. The single loading cycle was completed in 2.26, 1 and 2 sec for CWTT, APA and RLAT, respectively. Since the total height of specimens for RLAT was 75 mm, therefore 1.34 % permanent strain value was equivalent to absolute rut depth of 1 mm.

The study also identified the most suitable rutting performance test. The true performance test could be decided by considering the load transfer mechanism which closely simulate the on-field traffic loading conditions. Cooper wheel tracking test and asphalt pavement analyzer test apply the repeated load by to and fro motion of solid rubber wheel and pressurized rubber hose, respectively; while repeated load apply repeated cyclic square pulse loading on stationery cylindrical specimen. The load transfer methodology of Asphalt pavement analyzer test most closely simulates the real traffic loading as pressurized rubber tires transfer the vehicle load to the underlying pavement layers material in confined conditions.

Figure 7.18 showed that the asphalt mixtures tested in APA test required maximum time to reach 1 mm absolute rut depth as compared to rutting performance results in CWTT and RLAT. This behavior could be due to the fact that APA specimens were more uniformly compacted by Gyratory compactor as compared to CWTT slab specimens compacted by Cooper roller compactor. Moreover, APA samples were tested in the confined state. The better rutting performance of APA test specimens as compared to CWTT and RLAT specimens is probably due to combined effect of compaction homogeneity and confinement. It can be further observed that RLAT specimens achieved the rut depth of 1 mm, earliest as compared to APA and CWTT, which might be due to non-confinement.

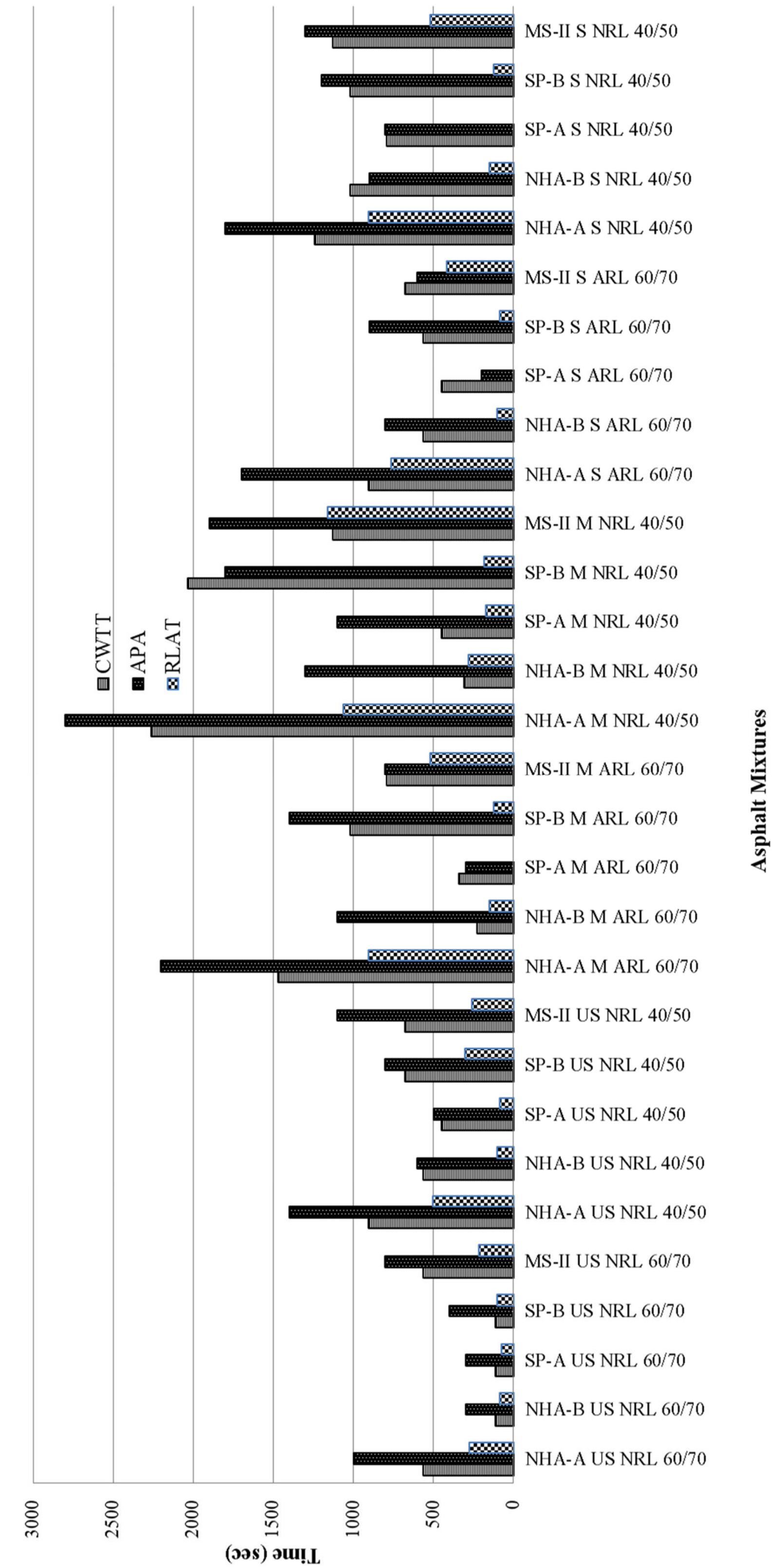


Figure 7.18: Plot of Time (In Seconds) to Achieve Absolute Rut Depth of 1 mm For CWTT, APA & RLAT At 50°C

7.3.2.9 Correlation Between Output Parameters of Different Rutting Performance Tests

This study identified the most suitable combination of output parameters by correlating the output parameters obtained from the three selected rutting performance test procedures. The identification could be helpful to propose suitable quality assessment alternative in case of non-availability of a particular test and convert results obtained from one test to that of other. This research study developed a correlation among the output parameters observed in case of CWTT, APA test and RLAT. Rut depth is output parameter in case of CWTT and APA, while permanent strain or simply strain is output parameter in RLAT. As RLAT terminates at lesser number of cycles at extreme conditions, strain/cycle has been plotted as an output parameter in case of RLAT results. It is notable that the primary purpose of development of these correlations is to compare the relationships between a specific combination of output parameters obtained from three laboratory performance tests. The most suitable combination is needed to be identified. Hence, the coefficient of determination (R^2) should be interpreted in relative terms i.e., R^2 value of 0.73 obtained by correlation developed between CWTT and RLAT results is relatively more significant as compared to the respective correlations developed between CWTT and APA, APA and RLAT with R^2 values of 0.29 and 0.20, respectively. The similar justification of lower absolute R^2 values has been observed in past research by Hussan et al. (2017b). A quadratic function correlates best in various plots. The most significant correlation in terms of coefficient of determination (R^2) exists between CWTT rut depth versus RLAT strain/cycle as shown in Figure 7.19 (a) below.

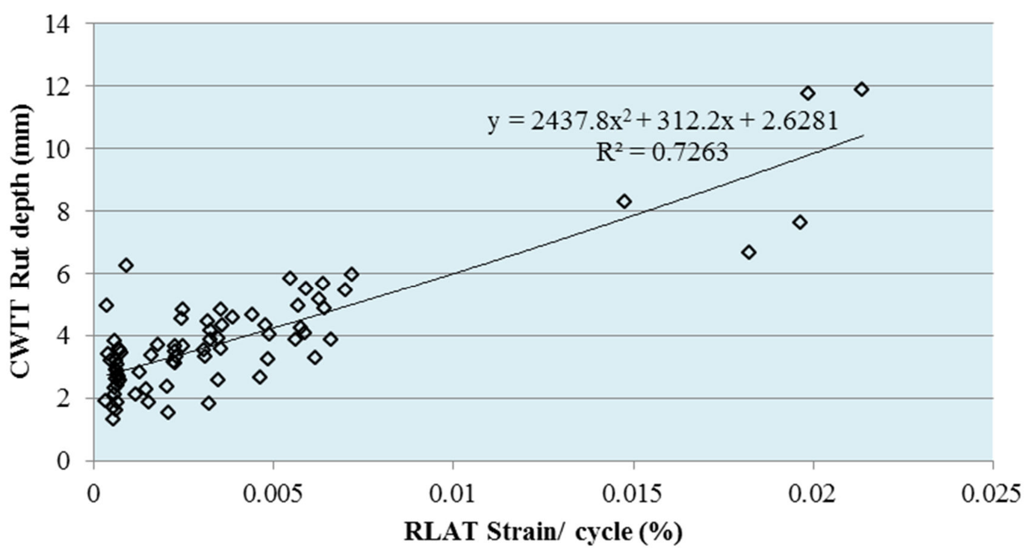


Figure 7.19 (a): Statistical Correlation Between CWTT Rut Depth And RLAT Strain/Cycle

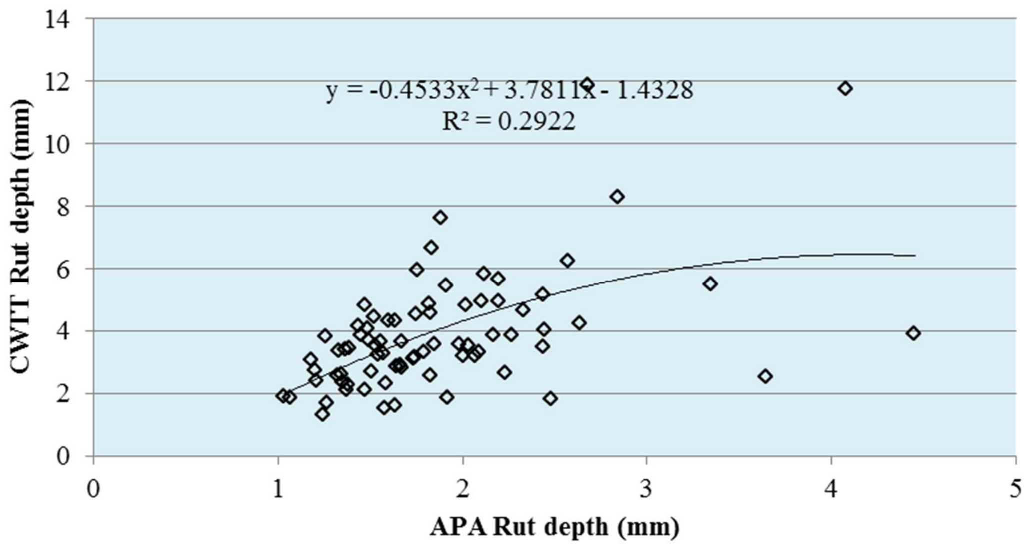


Figure 7.19 (b): Statistical Correlation Between CWTT Rut Depth and APA Rut Depth

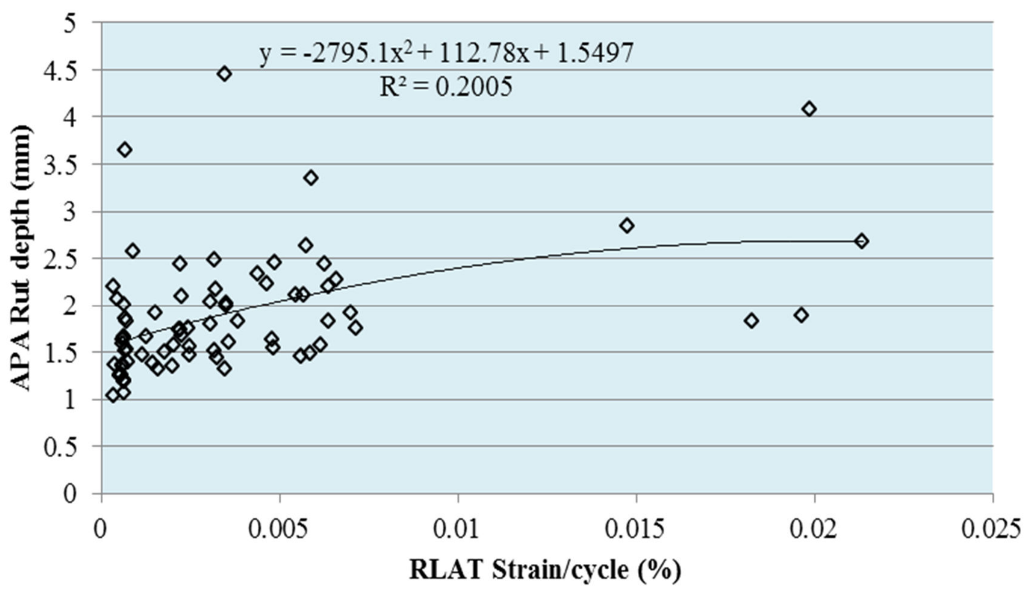


Figure 7.19 (c): Statistical Correlation Between APA Rut Depth and RLAT Strain/Cycle

Figure 7.19: Quadratic Function Correlation Between Output Parameters for CWTT, APA and RLAT at 50°C

7.3.2.10 Comparison/ranking of rutting performance of selected asphalt mixtures

The thirty wearing course mixtures could be ranked on the basis of their laboratory rutting performance in CWTT, APA test and RLAT. The terminal rut depth or terminal permanent strain/terminal number of cycles for various asphalt mixtures could be compared at consistent temperature and stress conditions. Hence, the comparison has been shown in Table 7.5 at constant temperature condition of 50°C with all other test conditions as per respective standard protocols.

Table 7.5: Asphalt Mixtures Performance Comparison by CWTT, APA and RLAT at Temperature Condition of 50°C

Sr No.	Mixture ID	CWTT	APA	RLAT	
		Rut depth (mm)	Rut depth (mm)	Strain (%)	Terminal Cycles
1	NHA-A WC US NRL 60/70	4.880	1.824	3.003	467
2	NHA-B WC US NRL 60/70	11.774	4.085	3.004	151
3	SP-1 WC US NRL 60/70	11.891	4.452	3.001	140
4	SP-2 WC US NRL 60/70	8.310	2.845	3.005	203
5	MS-2 WC US NRL 60/70	5.979	2.685	3.001	417
6	NHA-A WC US NRL 40/50	3.533	1.757	3.002	970
7	NHA-B WC US NRL 40/50	6.677	2.237	3.000	164
8	SP-1 WC US NRL 40/50	7.637	2.483	3.000	152
9	SP-2 WC US NRL 40/50	5.841	2.039	3.005	548
10	MS-2 WC US NRL 40/50	3.866	1.923	3.000	453
11	NHA-A WC M ARL 60/70	3.131	1.837	3.001	1320
12	NHA-B WC M ARL 60/70	4.472	1.889	3.000	938
13	SP-1 WC M ARL 60/70	4.953	2.274	3.001	526
14	SP-2 WC M ARL 60/70	3.902	1.748	3.003	856
15	MS-2 WC M ARL 60/70	3.161	2.121	3.000	1340
16	NHA-A WC M NRL 40/50	1.540	1.356	3.001	1435
17	NHA-B WC M NRL 40/50	2.371	1.740	3.003	1464
18	SP-1 WC M NRL 40/50	2.678	2.109	3.001	642
19	SP-2 WC M NRL 40/50	1.820	1.583	3.005	930
20	MS-2 WC M NRL 40/50	1.694	1.523	3.006	1920
21	NHA-A WC S ARL 60/70	3.664	2.092	3.001	1195
22	NHA-B WC S ARL 60/70	4.839	2.169	3.012	849
23	SP-1 WC S ARL 60/70	5.158	2.640	3.000	476
24	SP-2 WC S ARL 60/70	4.572	1.991	3.001	775
25	MS-2 WC S ARL 60/70	4.525	2.446	3.003	1213
26	NHA-A WC S NRL 40/50	3.335	1.562	3.001	1299
27	NHA-B WC S NRL 40/50	3.880	2.020	3.002	923
28	SP-1 WC S NRL 40/50	4.236	2.442	3.004	517
29	SP-2 WC S NRL 40/50	3.566	1.832	3.000	842
30	MS-2 WC S NRL 40/50	3.513	1.750	3.003	1318

Table 7.5 shows the comparison between rutting resistance of tested wearing course mixtures. It can be revealed from that NHA-A graded mixture with Margalla aggregate and NRL 40/50 pen. grade bitumen is most resistant to rutting distress among the wearing course mixtures in case of CWTT and APA tests, while MS-2 graded mixtures with

Margalla aggregate and NRL 40/50 pen. grade bitumen has maximum rutting resistance in RLAT. SP-1 graded mixture with Ubhan Shah aggregates and NRL 60/70 pen. grade bitumen is found to be the most rutting susceptible asphalt mixture irrespective of the performance test. Table 7.5 further shows that mixtures with Ubhan Shah aggregate source are more susceptible to rutting distress as compared to mixtures with Margalla and Sargodha aggregate source.

In order to rank the selected asphalt mixtures on the basis of laboratory performance, terminal rut depth has been considered for CWTT and APA test results, while terminal cycles to achieve failure strain of 3% have been considered for RLAT results. A rank order based on performance is designated to each mixture. Rank order of 1 designates the highest resistance to permanent deformation, while 30 designates least resistance to permanent deformation of the asphalt mixture as per terminal results. The second through fifth column of Table 7.6 shows the rank order based on performance of each mixture using CWTT, APA and RLAT respectively. It is required to rank the asphalt mixtures on the basis of combined effect of ranking obtained by each laboratory performance tests.

Several non-parametric statistics have been available to combine the ranking obtained by various dependent groups of data i.e., weighted average, Friedman test, Nemenyi test etc. But in this case, the three groups of laboratory performance tests are independent of each other; in other words, rank order obtained by three performance tests of CWTT, APA and RLAT is does not depend on each other. Hence, a suitable statistical test is required to be identified.

Rank product statistic (RP) is a biologically motivated non-parametric statistical parameter that is primarily used to combine the ranking of genes in microarray system obtained by mutually independent replicates at constant condition (Breitling et al. 2004). The mathematical formulation of this parameter has been expressed as geometric mean of rank orders as shown in Equation 7-3 where RP(g) is rank product, ri,g is individual rank order of a specific mixture by a single performance test, and k is number of performance tests.

$$\boldsymbol{RP(g)} = \left(\sum_{\boldsymbol{i=1}}^{\boldsymbol{k}} \boldsymbol{r_{g,i}}\right)^{\frac{1}{k}} \qquad \text{7-3}$$

The rank product statistic can be affectively utilized to fulfill the desired objective of combining the rankings obtained by CWTT, APA, and RLAT. This combination will indicate the overall ranking a specific mixture, which would be helpful for categorization of indigenous asphalt mixtures on the basis of laboratory performance. The more elaborative form of rank product statistic can be expressed as:

$$\boldsymbol{RP} = (\boldsymbol{Rank_{CWTT}} \times \boldsymbol{Rank_{APA}} \times \boldsymbol{Rank_{RLAT}})^{\frac{1}{No.of laboratory tests}} \qquad \text{7-4}$$

The sixth column of Table 7.6 show the rank product statistic values for each selected wearing course mixture. The mixtures are then re-ranked according to rank product statistical parameter and the combined rank order is tabulated in column 7 of Table 7.6.

Table 7.6: Asphalt Mixtures Rank Order at Temperature Condition Of 50°C

Sr No.	Mixture ID	Rank order by CWTT	Rank order by APA	Rank order by RLAT	Rank Product Statistic	Rank order by Rank Product Statistic
1	NHA-A WC US NRL 60/70	21	9	23	16.32	18
2	NHA-B WC US NRL 60/70	29	29	29	29.00	29
3	SP-1 WC US NRL 60/70	30	30	30	30.00	30
4	SP-2 WC US NRL 60/70	28	27	26	26.99	28
5	MS-2 WC US NRL 60/70	25	26	25	25.33	26
6	NHA-A WC US NRL 40/50	10	8	10	9.28	9
7	NHA-B WC US NRL 40/50	26	21	27	24.52	25
8	SP-1 WC US NRL 40/50	27	25	28	26.64	27
9	SP-2 WC US NRL 40/50	24	16	19	19.40	21
10	MS-2 WC US NRL 40/50	13	13	24	15.95	17
11	NHA-A WC M ARL 60/70	6	11	5	6.91	6
12	NHA-B WC M ARL 60/70	17	12	11	13.09	14
13	SP-1 WC M ARL 60/70	22	22	20	21.31	23
14	SP-2 WC M ARL 60/70	15	6	14	10.80	10
15	MS-2 WC M ARL 60/70	7	19	4	8.10	8
16	NHA-A WC M NRL 40/50	1	1	3	1.44	1
17	NHA-B WC M NRL 40/50	4	5	2	3.42	3
18	SP-1 WC M NRL 40/50	5	18	18	11.74	11
19	SP-2 WC M NRL 40/50	3	4	12	5.24	4
20	MS-2 WC M NRL 40/50	2	2	1	1.59	2
21	NHA-A WC S ARL 60/70	12	17	9	12.24	13
22	NHA-B WC S ARL 60/70	20	20	15	18.17	20
23	SP-1 WC S ARL 60/70	23	26	22	23.60	24
24	SP-2 WC S ARL 60/70	19	14	17	16.54	19
25	MS-2 WC S ARL 60/70	18	24	8	15.12	16
26	NHA-A WC S NRL 40/50	8	3	7	5.52	5
27	NHA-B WC S NRL 40/50	14	15	13	13.98	15
28	SP-1 WC S NRL 40/50	16	23	21	19.77	22
29	SP-2 WC S NRL 40/50	11	10	16	12.07	12
30	MS-2 WC S NRL 40/50	9	7	6	7.23	7

Spearman rank order correlation coefficient indicates the non-parametric relationship between rankings of selected asphalt mixtures obtained by different laboratory test methods. It is commonly denoted by ρ and mathematically computed by following formula in which 'di' is difference in ranking obtained by both performance tests for asphalt mixture 'i' and 'n' is total number of asphalt mixtures which is 30 in this study (Cleff, 2014).

$$\rho = 1 - \frac{6 \sum d_i^2}{n(n^2 - 1)} \qquad \text{7-5}$$

The spearman rank order correlation coefficient (ρ) equal to absolute value of 1 represents an ideal condition. The value of 'ρ' appears to be 0.807, 0.728, and 0.852 for the ranking obtained by CWTT & APA test, APA test & RLAT and CWTT & RLAT, respectively. These mathematically computed values of 'ρ' indicate that nearly consistent ranking has been obtained by three laboratory rutting performance with most suitable combination of CWTT and RLAT.

Table 7.7: Sequential Ranking for Asphalt Mixtures in Declining Order Of Permanent Deformation Resistance

CWTT	APA	RLAT	Overall
NHA-A WC M NRL 40/50	NHA-A WC M NRL 40/50	MS-2 WC M NRL 40/50	NHA-A WC M NRL 40/50
MS-2 WC M NRL 40/50	MS-2 WC M NRL 40/50	NHA-B WC M NRL 40/50	MS-2 WC M NRL 40/50
SP-2 WC M NRL 40/50	NHA-A WC S NRL 40/50	NHA-A WC M NRL 40/50	NHA-B WC M NRL 40/50
NHA-B WC M NRL 40/50	SP-2 WC M NRL 40/50	MS-2 WC M ARL 60/70	SP-2 WC M NRL 40/50
SP-1 WC M NRL 40/50	NHA-B WC M NRL 40/50	NHA-A WC M ARL 60/70	NHA-A WC S NRL 40/50
NHA-A WC M ARL 60/70	SP-2 WC M ARL 60/70	MS-2 WC S NRL 40/50	NHA-A WC M ARL 60/70
MS-2 WC M ARL 60/70	MS-2 WC S NRL 40/50	NHA-A WC S NRL 40/50	MS-2 WC S NRL 40/50
NHA-A WC S NRL 40/50	NHA-A WC US NRL 40/50	MS-2 WC S ARL 60/70	MS-2 WC M ARL 60/70
MS-2 WC S NRL 40/50	NHA-A WC US NRL 60/70	NHA-A WC S ARL 60/70	NHA-A WC US NRL 40/50
NHA-A WC US NRL 40/50	SP-2 WC S NRL 40/50	NHA-A WC US NRL 40/50	SP-2 WC M ARL 60/70
SP-2 WC S NRL 40/50	NHA-A WC M ARL 60/70	NHA-B WC M ARL 60/70	SP-1 WC M NRL 40/50
NHA-A WC S ARL 60/70	NHA-B WC M ARL 60/70	SP-2 WC M NRL 40/50	SP-2 WC S NRL 40/50
MS-2 WC US NRL 40/50	MS-2 WC US NRL 40/50	NHA-B WC S NRL 40/50	NHA-A WC S ARL 60/70
NHA-B WC S NRL 40/50	SP-2 WC S ARL 60/70	SP-2 WC M ARL 60/70	NHA-B WC M ARL 60/70
SP-2 WC M ARL 60/70	NHA-B WC S NRL 40/50	NHA-B WC S ARL 60/70	NHA-B WC S NRL 40/50
SP-1 WC S NRL 40/50	SP-2 WC US NRL 40/50	SP-2 WC S NRL 40/50	MS-2 WC S ARL 60/70
NHA-B WC M ARL 60/70	NHA-A WC S ARL 60/70	SP-2 WC S ARL 60/70	MS-2 WC US NRL 40/50
MS-2 WC S ARL 60/70	SP-1 WC M NRL 40/50	SP-1 WC M NRL 40/50	NHA-A WC US NRL 60/70
SP-2 WC S ARL 60/70	MS-2 WC M ARL 60/70	SP-2 WC US NRL 40/50	SP-2 WC S ARL 60/70
NHA-B WC S ARL 60/70	NHA-B WC S ARL 60/70	SP-1 WC M ARL 60/70	NHA-B WC S ARL 60/70
NHA-A WC US NRL 60/70	NHA-B WC US NRL 40/50	SP-1 WC S NRL 40/50	SP-2 WC US NRL 40/50
SP-1 WC M ARL 60/70	SP-1 WC M ARL 60/70	SP-1 WC S ARL 60/70	SP-1 WC S NRL 40/50
SP-1 WC S ARL 60/70	SP-1 WC S NRL 40/50	NHA-A WC US NRL 60/70	SP-1 WC M ARL 60/70
SP-2 WC US NRL 40/50	MS-2 WC S ARL 60/70	MS-2 WC US NRL 40/50	SP-1 WC S ARL 60/70
MS-2 WC US NRL 60/70	SP-1 WC US NRL 40/50	MS-2 WC US NRL 60/70	NHA-B WC US NRL 40/50
NHA-B WC US NRL 40/50	SP-1 WC S ARL 60/70	SP-2 WC US NRL 60/70	MS-2 WC US NRL 60/70
SP-1 WC US NRL 40/50	MS-2 WC US NRL 60/70	NHA-B WC US NRL 40/50	SP-1 WC US NRL 40/50
SP-2 WC US NRL 60/70	SP-2 WC US NRL 60/70	SP-1 WC US NRL 40/50	SP-2 WC US NRL 60/70
NHA-B WC US NRL 60/70	NHA-B WC US NRL 60/70	NHA-B WC US NRL 60/70	NHA-B WC US NRL 60/70
SP-1 WC US NRL 60/70	SP-1 WC US NRL 60/70	SP-1 WC US NRL 60/70	SP-1 WC US NRL 60/70

The ranking based on successively deteriorating permanent deformation resistance has been tabulated in Table 7.7. The ranking based on individual performance tests has been presented in first three columns while fourth column indicates the overall ranking based on combined rank product statistic. The overall ranking can be used to classify the asphalt mixtures in testing regime in three broad categories. The categories of good, average and bad performing asphalt mixtures can be differentiated by color scheme of green, yellow and red respectively. This classification could be helpful in determining the suitability of asphalt mixtures included in testing regime, which are to be laid in the field at different regions with prevailing environmental and loading conditions. As this classification is based on high temperature performance of tested asphalt mixtures, further investigation with respect to low temperature performance is required to be conducted in future studies.

7.4 Statistical Analysis

7.4.1 Statistical Summary

The distribution of data and descriptive statistics derived from output results of three laboratory rutting performance tests can be effectively presented by individual statistical summary plots as shown in Figure 7.20a through Figure 7.20c. These plots have a significance to identify mean value of rut depth/permanent strain rate obtained from the combined results for all selected mixtures tested by each rutting performance test. The plots further specify the deviation of data points from normal distribution about the mean value in terms of statistical parameters of standard deviation, variance, skewness and kurtosis for a specific performance test. Hence, the plots will contribute in quantifying the variability among output data points for a particular performance test.

Several measures of central tendency have been used to explain the central properties of a set of observations by a single representative value. Mean and median are among the most significant measures of central tendency. The mean is computed by dividing the sum of all observations by the number of observations. Figure 7.20 shows mean rut depth of 3.91 and 1.94 mm for CWTT and APA test, respectively; and strain/cycle of 0.00557 percent for RLAT. Median is the central point above and below which fifty percent of data exist. It has been computed by arranging the whole set of data in ascending order, central value or average of two central values is required median in case of odd and even number of data points, respectively. Standard deviation and variance are commonly computed measures of variation/dispersion in a particular set of observations. Variance is defined as the arithmetic mean of squared distances of individual data points with respect to mean. Standard deviation is simply the square root of variance. The standard deviation of 2.07, 0.67 and 0.0071 has been reported for CWTT, APA test, and RLAT respectively. Skewness is the extent to which the data is not symmetrical. Whether the skewness value is 0, positive, or negative reveals information about the shape of the data. Kurtosis indicates how the peak and tails of a distribution differ from the normal distribution. N is number of observations for a particular data set.

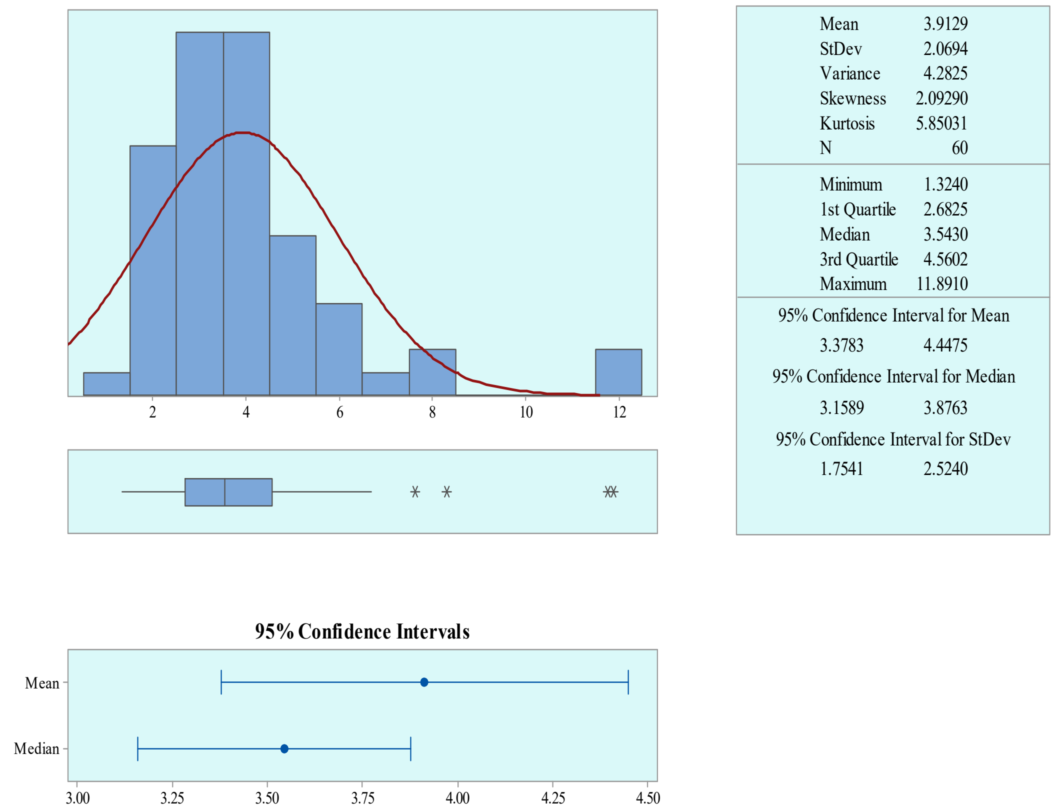


Figure 7.20 (a): Summary Plot for Cooper Wheel Tracking Test

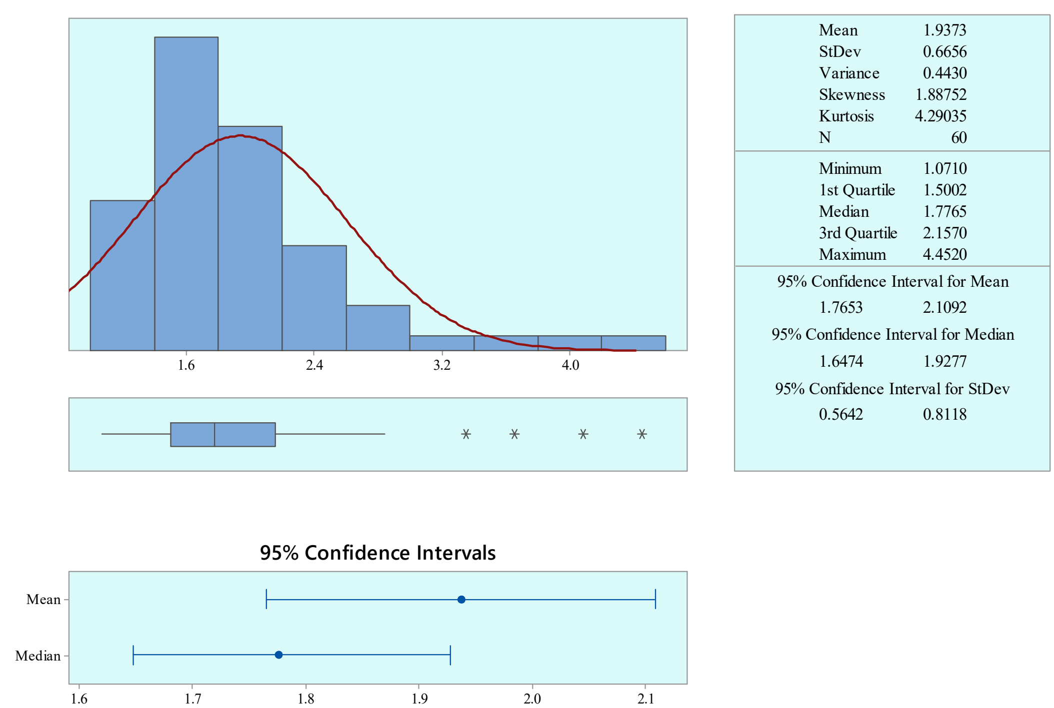


Figure 7.20 (b): Summary Plot for Asphalt Pavement Analyzer Test

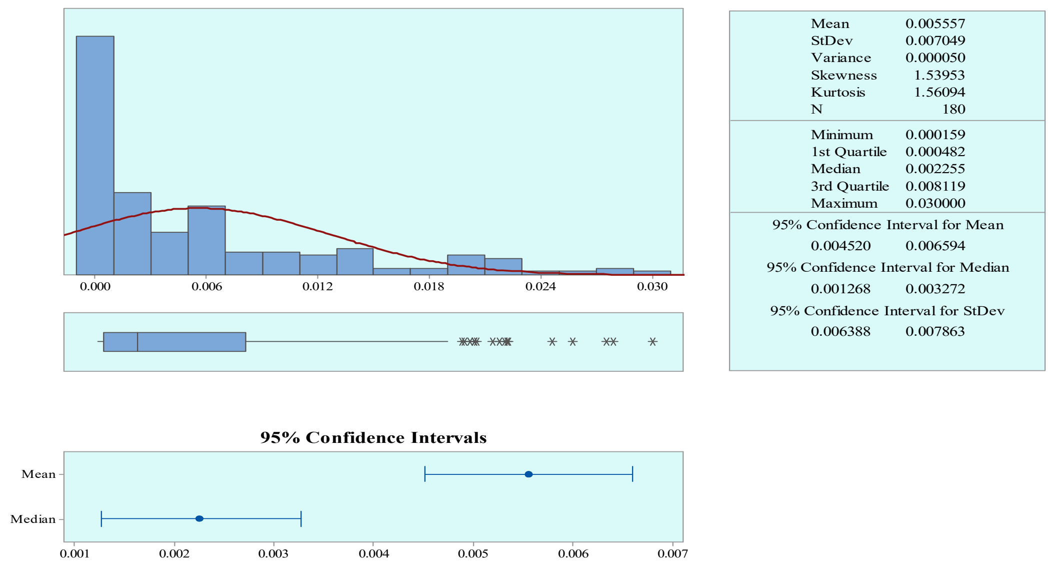


Figure 7.20 (c): Summary Plot for Repeated Load Axial Test

Figure 7.20:Summary Plots For CWTT, APA And RLAT

7.4.2 Boxplots

Boxplot is a very useful visualization graph indicating the distribution of particular data set. It is a summary plot which graphically indicates the median, quartiles, and extreme outliers. The dark line in the middle of the boxes is the median. The bottom of the box indicates the 1st quartile. Twenty-five percent of data points have values below the 1st quartile. The top of the box represents the 3rd quartile. Twenty-five percent of data points have values above the 3rd quartile. This means that 50% of the observations data lie within the box known as inter quartile range (IQR). The perpendicular lines that extend from the boxes are called inner fences or whiskers. These extend to 1.5 times the IQR; if no observation has a value in that range, extends to the minimum or maximum values. Outliers are data points which exist between 1.5 and 3 times the IQR, beyond the whiskers, and are indicated by circular bullets. Extremes shown by stars or asterisks are cases with values more than 3 times the IQR. The terminologies have been demonstrated in Figure 7.21 below.

The boxplots have been plotted for the results of all the three rutting performance tests of CWTT, APA test and RLAT as shown in Figure 7.22. The boxplots have been categorized by independent variables of bitumen penetration, types of aggregates and temperature in case of CWTT and APA test, while stress has been included in case of RLAT.

For CWTT, it has been observed that variability among data points is generally enhanced with increase in penetration grade of bitumen. The anomalous trend exists for rutting performance of mixtures with aggregate source of Sargodha and bitumen of ARL 60/70 (Pen=63) at temperature condition of 40°C, in which spread of resultant rut depth is reduced by increasing bitumen penetration, as indicated by smaller IQR. It has been further observed that the median value increases with increase in bitumen penetration and temperature for different types of aggregates.

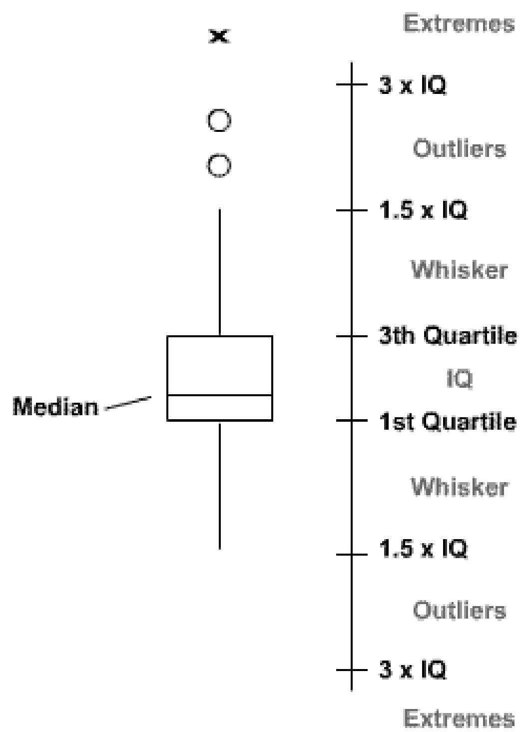


Figure 7.21: Important Terminologies of Typical Boxplot

In case of APA test, similar trends have been observed, particularly variability in rutting results has been predominantly increased when Ubhan Shah aggregates and NRL 60/70 bitumen (Pen=72) has been used.

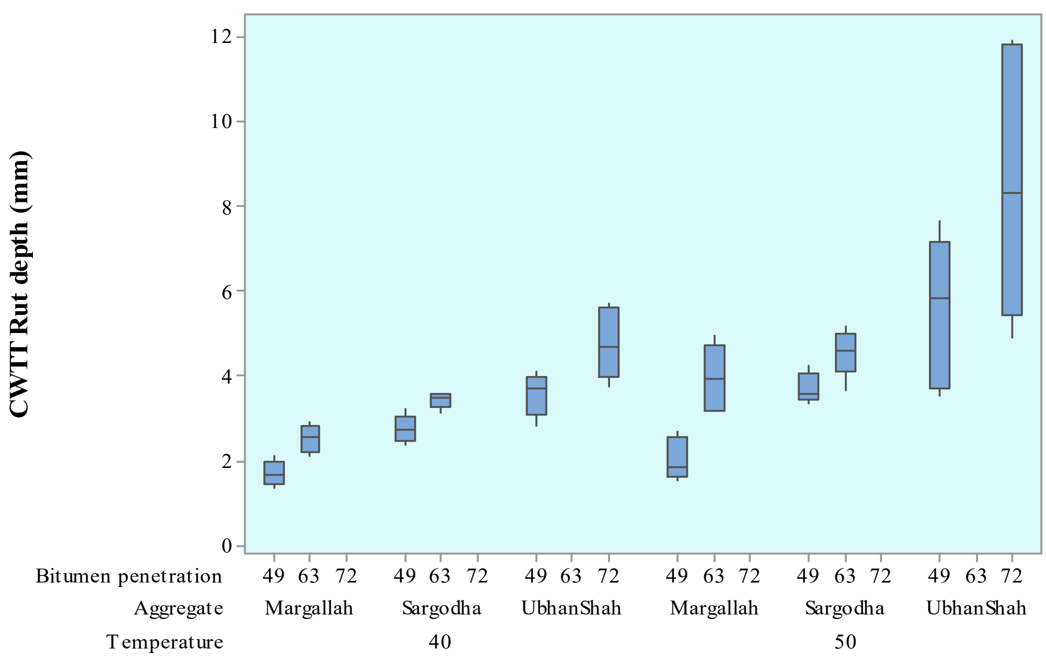


Figure 7.22 (a): Boxplot for Cooper Wheel Tracking Test

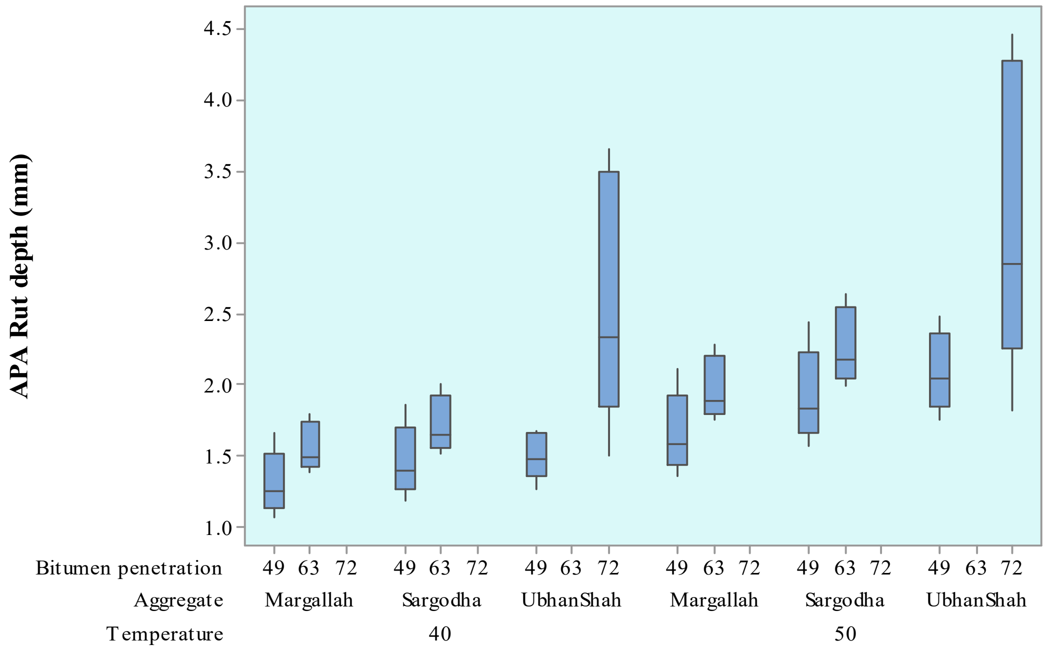


Figure 7.22(b): Boxplot for Asphalt Pavement Analyzer Test

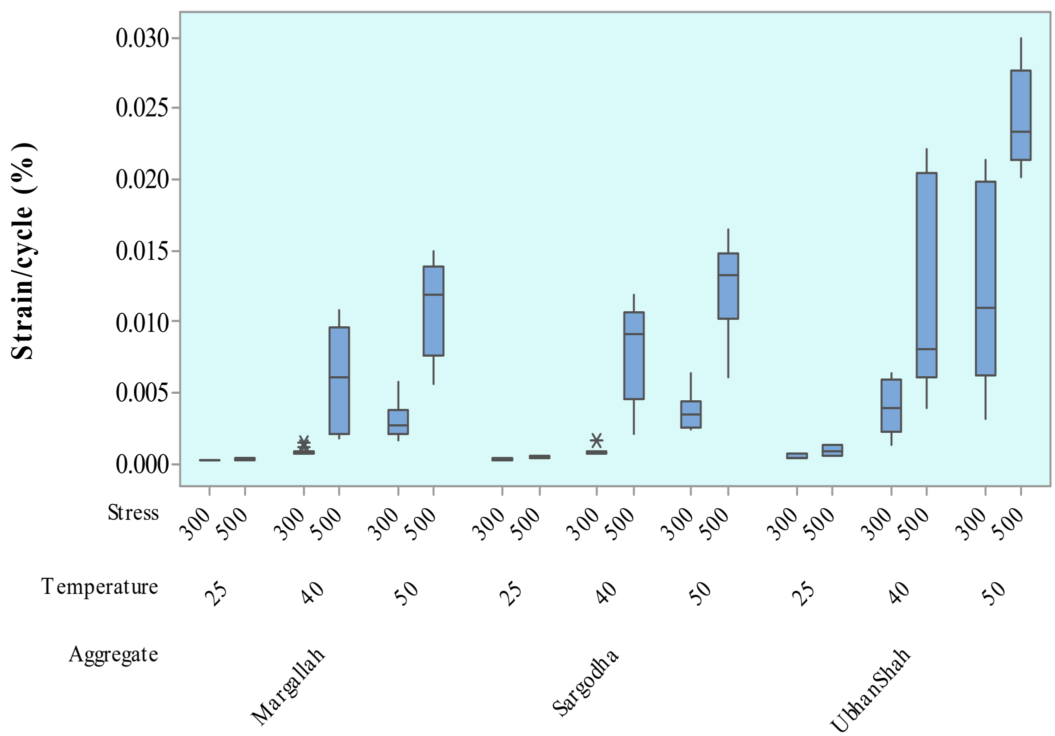


Figure 7.22 (c): Boxplot for Repeated Load Axial Test

Figure 7.22: Boxplots for CWTT, APA And RLAT

Strain/cycle variable has been plotted as dependent variable while stress, temperature and types of aggregates are graphed as independent variables in boxplots of rutting performance results of RLAT. It has been observed that variability of data points is very less in case of lower temperature condition of 25°C, no matter which combination of aggregate source, temperature or stress condition has been considered. A couple of extreme outliers at this least temperature condition are also visible as indicated by asterisks. If the temperature is increased to 40°C, the data spread becomes much more sensitive to the stress condition. Similar trends have been observed when temperature is further increased to 50°C. An anomalous trend has been monitored in case of Ubhan Shah aggregate and NRL 60/70 bitumen (Pen=72) composed mixtures at temperature and stress condition of 50°C and 500 kPa, respectively i.e., variability indicated by boxplot has been reduced with increasing stress. This behavior could be due to the fact that unconfined test specimen crumbles prematurely at these severe conditions, hence reached the failure strain at lesser and consistent number of loading cycles.

7.5 Design of Experiment using General Full Factorial Design

An experiment can be defined as a series of activities in which input variables are varied according to a specific criterion in order to quantify the significance of variation of these input variables on the response/output variable. The main purpose of any experiment is optimization of the response variable. The linking of this optimization of experiment with statistical science is termed as design of experiment (DOE). The initial step of DOE is to define the problem and identify the probable significantly affecting factors on the basis of experience or past research. After that the region of interest should be defined. The region of interest clarifies the range of variation in each of the selected factors. Lastly the factors are varied between selected levels in order to optimize the response/output variable. Different techniques are used for conducting DOE. Full factorial design is a commonly used technique for optimizing and statistically quantifying the influence of different input variables on the response variable in a number of science and engineering problems (Cavazzuti, 2013).

Full factorial design is widely used in the experiments involving several factors where it is necessary to study the joint effect of these factors on a response. The full factorial design of experiment consists of all the possible combinations of levels for all the input factors. General full factorial design is a factorial design methodology used to conduct controlled experiments with variable levels of input factors. The general full factorial design process is composed of selecting vital inputs at all possible levels, determining the interaction between these inputs, and estimating the output on the basis of these interactions.

The selection of independent variables is the most critical phase of the adopted technique of general full factorial design. The selection is based upon the practically relevant critical overview of significance of various variables for laboratory and field rutting performance of asphalt mixtures. The subsequent discussion elaborates the importance of various material, environment, and loading related variables on the basis of technical review of past studies.

Aggregate source plays a vital role in characterizing the rutting propensity of asphalt mixtures. Aggregate source can be classified by petrographic analysis of the parent rock or physical/ mechanical properties of selected aggregate material. As the permanent deformation performance of asphalt mixtures' is required to be analyzed in this study, it is better to classify the aggregates by their engineering properties. The aggregate shape properties greatly influence asphalt mixtures' permanent deformation behavior (Cheung and Dawson, 2003; Chen et al., 2005a; Bessa at al., 2014). The coarse aggregate shape properties are represented by flakiness and elongation indices in present national specifications. The significant effect of flat/flaky particles on laboratory performance of asphalt mixtures has been observed by Mahmud et al., (2014) and Putrajaya et al., (2014). Presence of large amount of flaky/flat aggregate particles reduces the stability of asphalt mixture on application of dynamic traffic load (Chen et al., 2005a). Flat/flaky particles decrease the permanent deformation resistance and creep stiffness of asphalt mixtures (Darabadi and Taherkhani, 2015). Buchanan (2000) observed that the flakiness index (FI) to elongation index (EI) ratios less than 3, might negatively affect the rutting susceptibility of asphalt mixtures. In this research study, FI/EI ratios are less than 3 for all aggregate sources. Hence, the FI/EI ratio parameter can be effectively used to classify the coarse fraction of aggregate materials. The shape properties of finer fraction of aggregates are designated by fine aggregate angularity (FAA). FAA significantly affects the rutting propensity of asphalt mixtures (Ramli et al. 2013; Kim et al. 2009; Topal and Sangoz, 2005). FAA is experimentally computed in the laboratory using un-compacted voids test. Therefore, un-compacted voids test results should also be used to classify the fine aggregate fraction. Along-with the shape parameters, degradation resistance of aggregates also influences the permanent deformation response. Loss Angeles abrasion value (LAAV) test is one of the economical and readily available test procedures listed in national material specifications that can simultaneously quantify the aggregate resistance to impact and abrasion. Aggregate materials with lower LAAV are observed to exhibit higher strengths for laboratory compacted asphalt mixture specimens (Amirkhanian et al. 1991). Aho et al. (2001) concluded that the aggregate materials with higher LAAV along-with higher percentage of flaky & elongated particles degrade at a faster rate during field compaction. Therefore, three engineering properties of flakiness and elongation indices ratios of coarse aggregates, fine aggregate angularity and degradation resistance by Los Angeles abrasion test should be combined to quantitatively designate an aggregate source. This proposed innovative combination can be termed as aggregate source index (ASI) and has been mathematically expressed as:

$$\boldsymbol{ASI = LAAV \times \frac{FI}{EI} \times V_{f,un}} \tag{7-6}$$

Where 'ASI' is aggregate source index, 'LAAV' is Los Angeles abrasion value, 'FI' is flakiness index, 'EI' is elongation index and '$V_{f,un}$' is un-compacted voids for fine aggregate fraction.

Bitumen source also influences the rutting distress performance of asphalt mixtures. The physical properties of bitumen varied by relative proportions of asphaltenes and aromatics in the crude oil from which bitumen is extracted. Penetration grade of bitumen is one of frequently computed physical property. It indicates the relative hardness of bitumen at the intermediate temperature of 25°C. Although the penetration value does not compute the exact response of bitumen at extremely high temperatures, it shows relative stiffness at standard room temperature. Bitumen acts as the visco-elastic binding material in asphalt mixtures at different frequencies and temperature conditions. Its properties greatly influence the rutting performance of asphalt mixtures. Penetration grade is one of the simplified and commonly used bitumen properties which are included in quality control specifications (Williams et al., 2004). As binders are commercially classified by penetration grading system in the country instead of viscosity grading or performance grading system.

Aggregate gradation is defined as particle size distribution within the aggregate bitumen matrix of asphalt mixture and expressed in terms percentage passing of total material through each standard sieve size included in aggregate sieve analysis procedure. The permanent deformation resistance of asphalt mixtures is significantly influenced by the aggregate gradation of asphalt mixtures (Al-Mosawe et al. 2015; Liu et al. 2017; Golalipour et al. 2012; Cao et al. 2016). The coarseness and fineness of aggregate gradation has maximum impact on rutting performance of asphalt mixtures (Khedr and Breakah, 2011). The greater proportion of coarse aggregates increases the particle-to-particle contact, henceforth, reduces the influence of binder stiffness, on the performance of asphalt mixtures (Hussan et al. 2017a). The fineness can be identified by percentage of total material passing 4.75 mm (#4) standard sieve, while coarseness is obtained by percentage of total material retained on 4.75 mm (#4) standard sieve. Additionally, the filler content, identified by percentage of material passing 0.075 mm (#200) standard sieve, has also been observed to remarkably effect the rutting susceptibility of asphalt mixtures [Remisova, (2015); Qasim et al. (2017)]. In order to accommodate the combined effect of all these gradation parameters, a new index termed as 'aggregate gradation index' (AGI) has been introduced. It can be mathematically expressed as:

$$\boldsymbol{AGI} = \frac{\boldsymbol{P_{coarse}}}{\boldsymbol{P_{fine}}} \times \boldsymbol{P_{passing\ \#200\ sieve}} \qquad 7\text{-}7$$

Where 'P_{coarse}' is percentage of material retained on 4.75 mm sieve, 'P_{fine}' is percentage of material passing 4.75 mm sieve, and '$P_{passing\ \#200\ sieve}$' is percentage of material passing 0.075 mm (#200) sieve.

The effect of test temperature is directly attributable to the properties of bitumen as bitumen being visco-elastic material, reduces its binding ability with the increasing temperature. Temperature variable is observed to have most significant effect on rutting performance of asphalt mixtures among various environmental factors (Feyissa 2009; Qiao et al., 2013). Therefore, temperature has been considered as significantly affecting environmental factor.

Variables related to loading magnitude must be adopted in the analysis finally. Loading stress and number of loading cycles have been considered as load related factors (Baoteng and Maina, 2012).

It is notable that certain variables related to asphalt mixture i.e., optimum bitumen content (OBC), stability, voids in mineral aggregates (VMA), voids filled with asphalt (VFA), and maximum theoretical specific gravity of the mixture (Gmm) are not incorporated because these properties were already optimized during Marshall mixture design for each combination of aggregate, bitumen and gradation. Furthermore, air voids are consistently maintained at in-situ representative range of 6±0.5 percent, henceforth Gmb could not be considered as an independent variable.

Conclusively, five factors of temperature, aggregate source index, bitumen penetration value, aggregate gradation index are considered for CWTT and APA results, while additional factor of stress is also adopted in case of RLAT. The response variable is rut depth for CWTT and APA test, while strain/cycle is considered as response variable in case of RLAT. In case of general full factorial design of experiment, each factor is to be considered at any number of levels. Table 7.8 shows the factors that have been considered in the general full factorial design with their respective abbreviations and respective levels as considered in test matrix. Entering these four factors in software resulted in ninety combinations (2 levels of temperature x 3 levels of ASI x 3 levels of Bitumen penetration x 5 levels of AGI).

Table 7.8: Variables/Factors Considered for General Full Factorial Design

Factors	Levels					Units
Temperature	25*	40	50	---	---	°C
Stress*	300	500	---	---	---	kPa
Aggregate source index (ASI)	Ubhan Shah	Margalla	Sargodha			
	645.74	1198.30	906.67	---	---	
Bitumen Penetration	NRL 40/50	ARL 60/70	NRL 60/70			$1/10^{th}$ of mm
	49	63	72	---	---	
Aggregate gradation index (AGI)	NHA-A	NHA-B	SP-1	SP-2	MS-2	
	6.76	5.00	2.69	4.89	4.17	

* In case of Repeated load axial test only

7.5.1 Analytical Tools of General Full Factorial Design

For analysis of factors and their interaction by general full factorial design of experiment, various tools are used for significance check, variation of response with levels and residual analysis. These tools are explained and interpreted in the following section.

7.5.1.1 Main Effect Plots

The Main Effect is defined as difference in the mean response between various levels of a dependent factor or variable. The main effect plot is a plot of the average response values at each level of a factor. The sign of a main effect tells about direction of the effect, i.e., whether the mean response value increases or decreases. The magnitude tells about the strength of the effect. The effects of temperature, aggregate source index, bitumen penetration and aggregate gradation index are shown in Figure 7.23. It is clear from the temperature plot that rut depth is much lower for 40°C as compared to 50°C temperatures. The plot for aggregate source index shows that lower the aggregate source index of aggregates, greater is the rut depth. The bitumen penetration plot shows the softer bitumen is more rut susceptible. The plot for aggregate gradation index shows that lower the aggregate gradation index of aggregates, greater is the rut depth. It has been observed that mean rut depth for SP-2 graded mixtures is less than that of NHA-B graded mixtures in spite of decrease in percentage of aggregates finer than 4.75 mm sieve. This anomalous behavior could be due to the fact that adopted gradation of SP-2 has 84% material coarser than 9.5 mm sieve while NHA-B has only 70% material retained on 9.5 mm sieve; henceforth, the cumulative proportion of large sized coarse aggregate particles is more.

7.5.1.2 Interaction Plots

Figure 7.24 shows the interaction plot between different factors. It is clear from the plot that all the single and 2-way interactions are significant and are represented by non-parallel lines. From Figure 7.24, it can be reported that mean rut depth increases with temperature and bitumen penetration value while decreases with aggregate source index and aggregate gradation index. However, impact of temperature is more pronounced in case of mixtures with softer binder and lower aggregate source and aggregate gradation indices.

7.5.1.3 Diagnostic Testing by Residual Plots

The residuals obtained after general full factorial design should be considered for certain diagnostic checks. It is mandatory requirement that certain assumptions i.e., normal distribution, constant variance and, independence must be verified for residuals after performing general full factorial design. If any of the above-mentioned assumptions is not satisfied, the result interpreted from the design of experiments becomes unreliable.

7.5.1.4 Normality Assumption Plots

Normal distribution assumption was checked by plotting histograms of residuals obtained from general full factorial design as shown in Figure 7.25. It has been shown that the residuals are nearly normally distributed about mean of zero. A slight departure in case of histogram plotted in case of CWTT and RLAT as shown in Figure 7.25a and Figure 7.25c did not fall into the category of serious violation of the assumption. Normal probability plot is another procedure to check the normal distribution of errors about the mean. The plots shown in bottom portion of Figure 7.25 shows that residuals lie near to normal distribution line, hence the normality assumption is broadly satisfied.

7.5.1.5 Residual vs. Fitted values Plots

Residual vs. Fitted values plot is a scatter plot with residuals plotted as ordinate while fitted values (responses) as abscissa. It is required that the data points should be randomly scattered around zero residual line for validation of assumption of constant variance. In case of violation of assumption of constant variance, the scatterness in residual values increases with increasing fitted values. Hence, the cloud of residual points adopts the shape of horizontal funnel opening towards the end with larger fitted values. The residual versus fitted plots shown in Figure 7.26 indicate that the residual points are randomly scattered around zero residual line. No megaphone shaped pattern or horizontal opening funnel like structure has been observed in residual versus fitted plots obtained in this study. Figure 7.26c shows an unorthodox pattern of distribution of residuals for RLAT permanent strain/cycle as response variable. But this pattern does not violate the constant variance assumption, as no horizontally opening funnel like pattern was observed.

7.5.1.6 Residual vs. Order Values Plots

Residual versus order value plots is used to check the third assumption of independence of residuals. The correlation between residuals can be detected by residual versus order plots. A pattern of continuously positive or negative residuals shows positive correlation between residuals and hence the independence of residuals assumption has been violated. This violation creates doubts about the interpreted results, and the correction is difficult to apply at the later stage, so it is important to prevent the problem, if possible, at the time of data collection. The proper randomization of experiment with respect to observation order is an important step in obtaining independence. The residual versus order plots shown in Figure 7.27 indicate that no correlation exists between residual points, as the points are randomly distributed and no definite pattern has been observed.

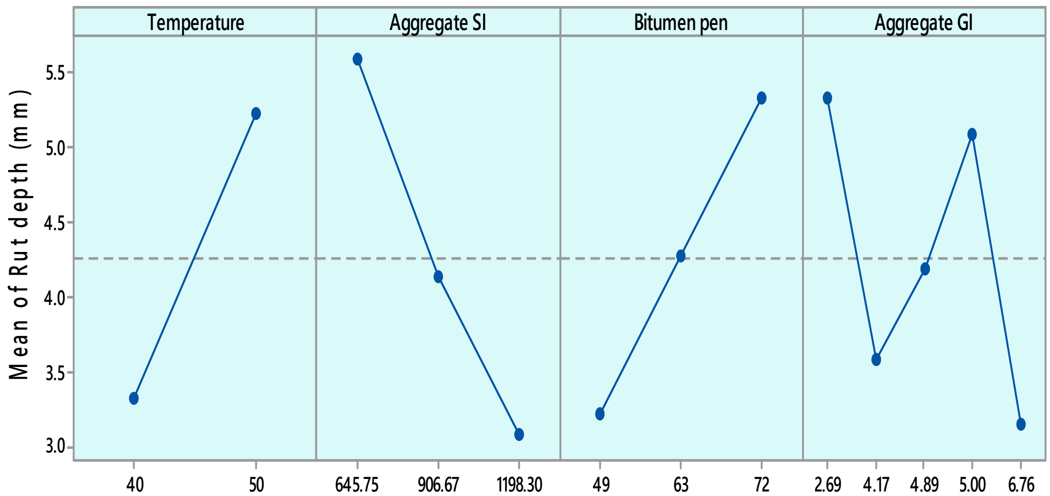


Figure 7.23 (a): Main Effect Plots for Cooper Wheel Tracking Test

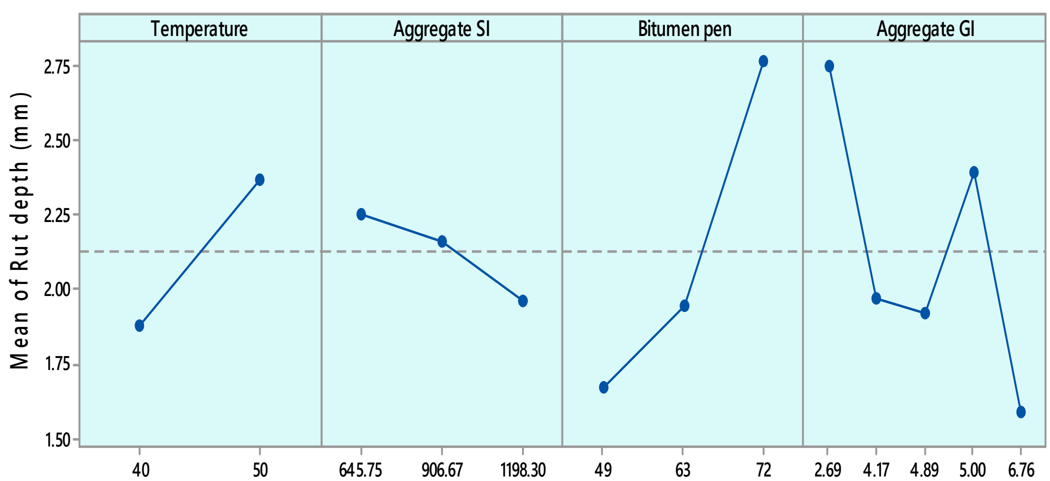


Figure 7.23(b): Main Effect Plots for Asphalt Pavement Analyzer Test

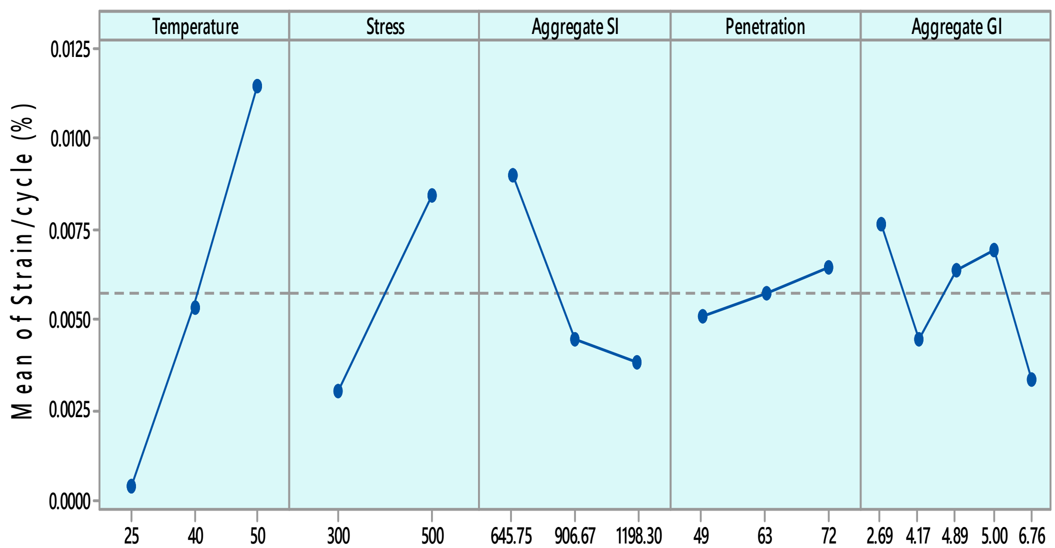


Figure 7.23 (c): Main Effect Plots for Repeated Load Axial Test

Figure 7.23: Main Effect Plots For CWTT, APA And RLAT

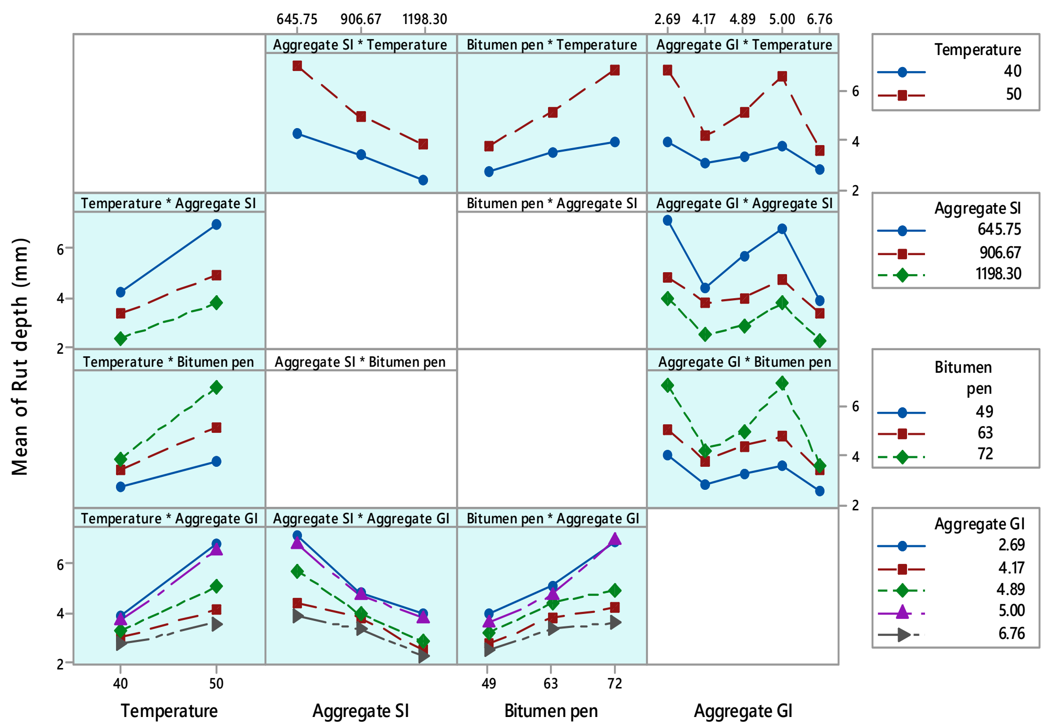


Figure 7.24 (a): Interaction Plots for Cooper Wheel Tracking Test

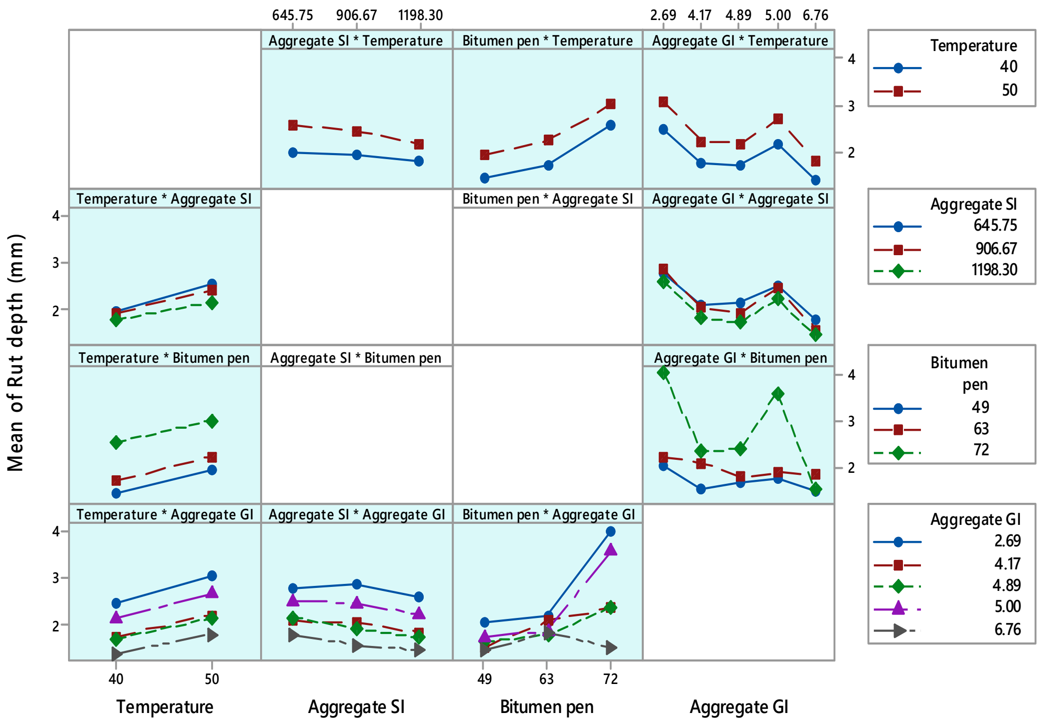


Figure 7.24 (b): Interaction Plots for Asphalt Pavement Analyzer Test

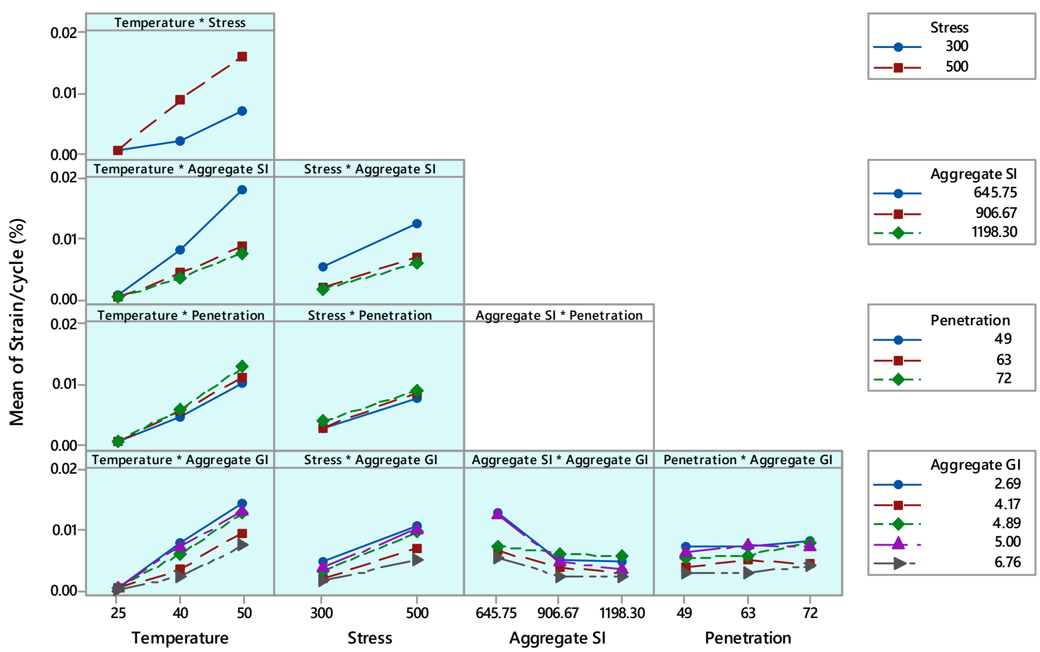


Figure 7.24 (c): Interaction Plots for Repeated Load Axial Test

Figure 7.24: Interaction Plots For CWTT, APA And RLAT

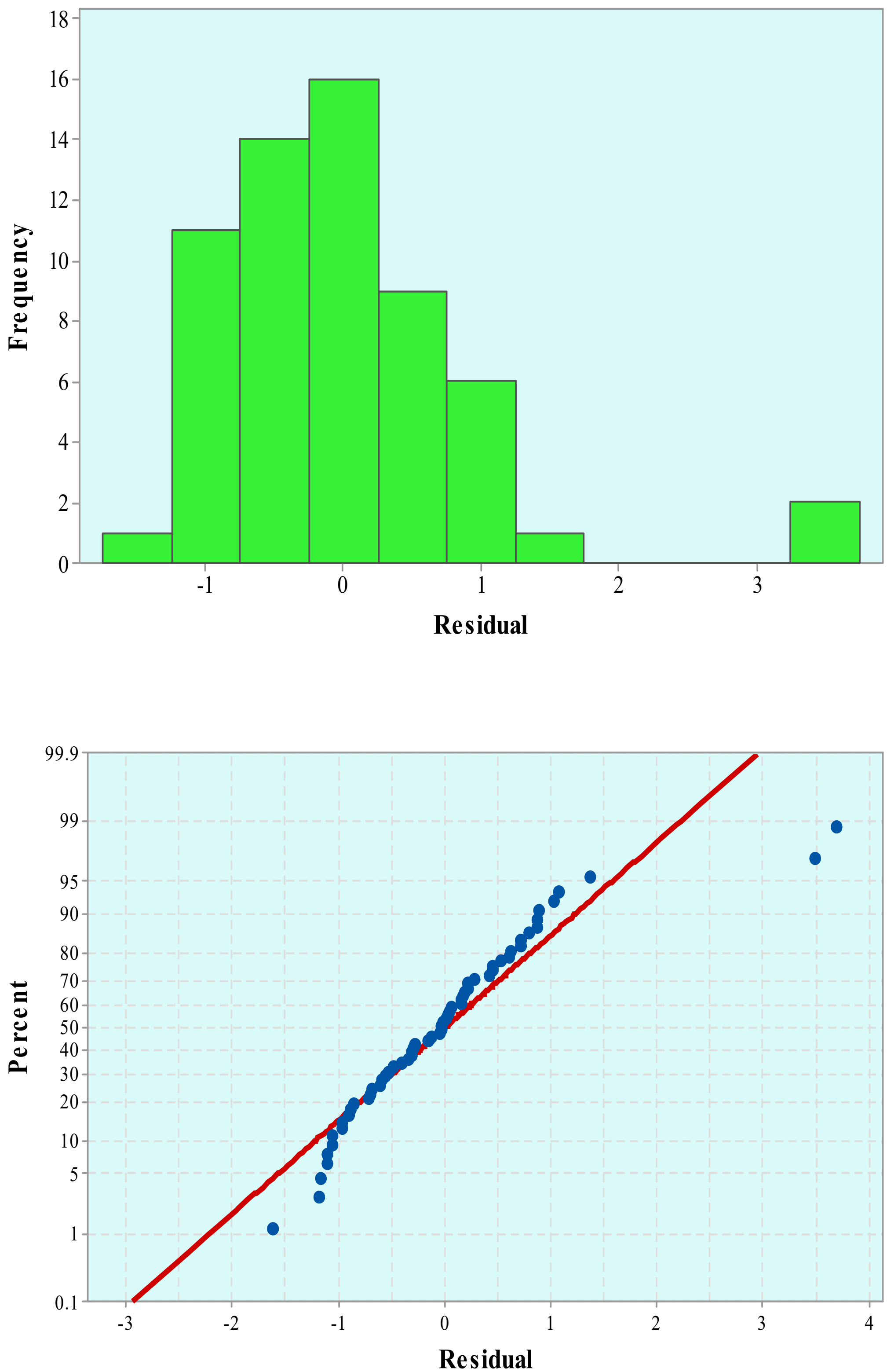
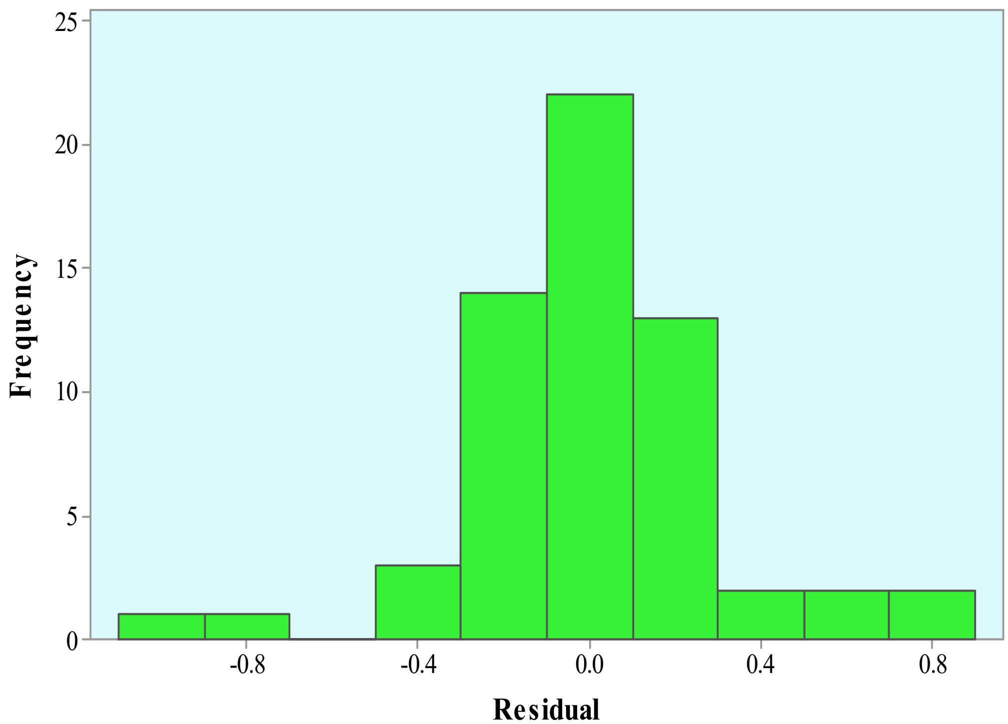


Figure 7.25 (a): Normality Check Plots for Cooper Wheel Tracking Test



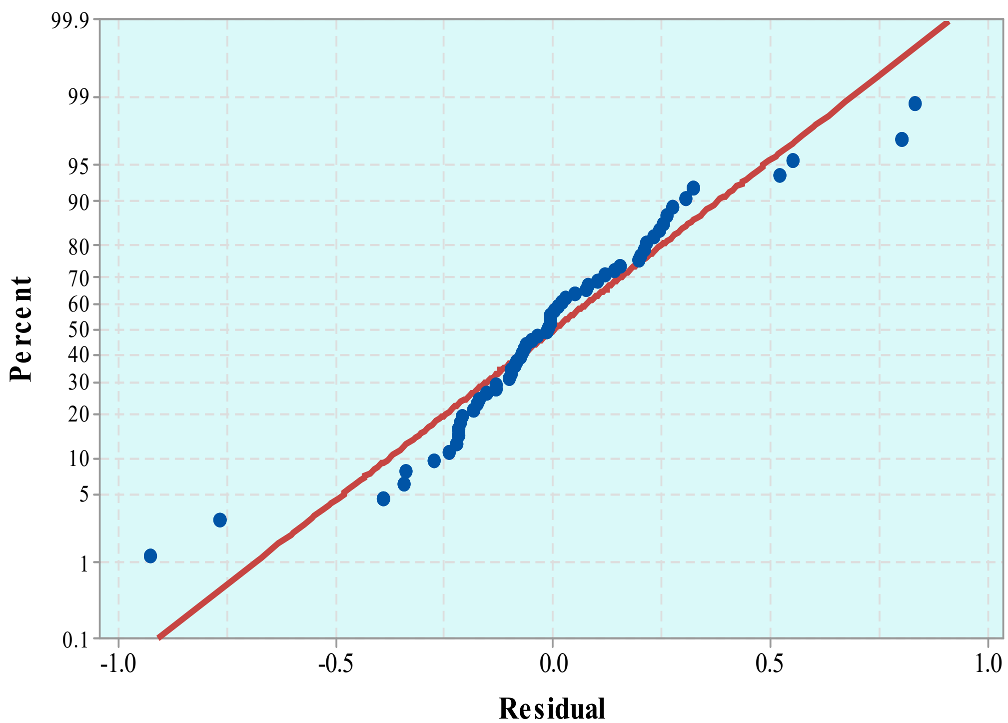
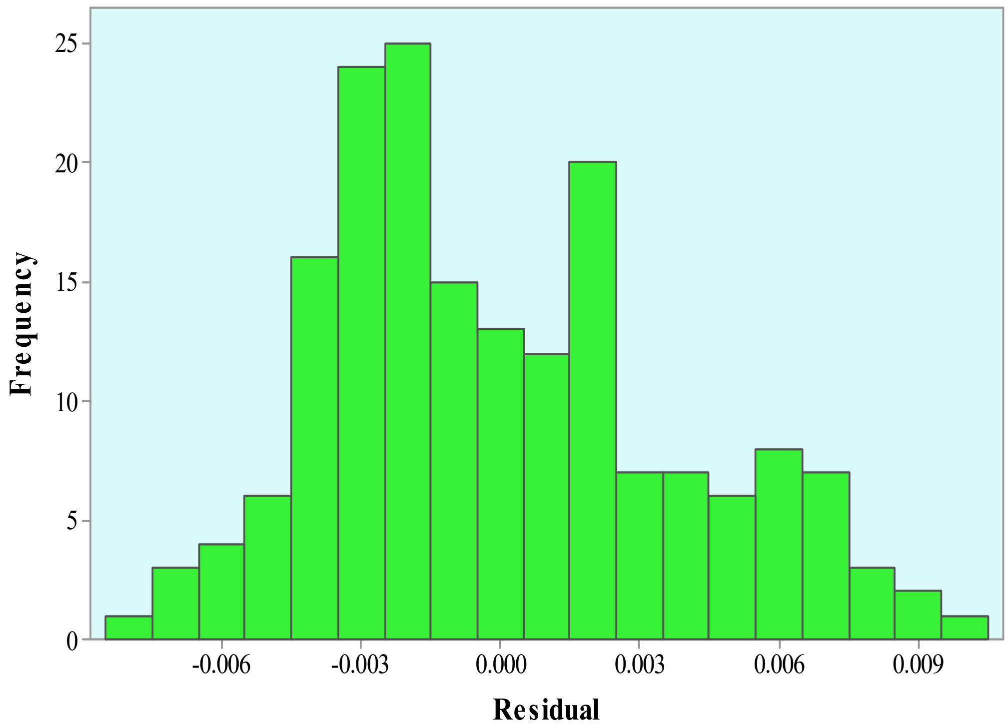


Figure 7.25 (b): Normality Check Plots for Asphalt Pavement Analyzer Test



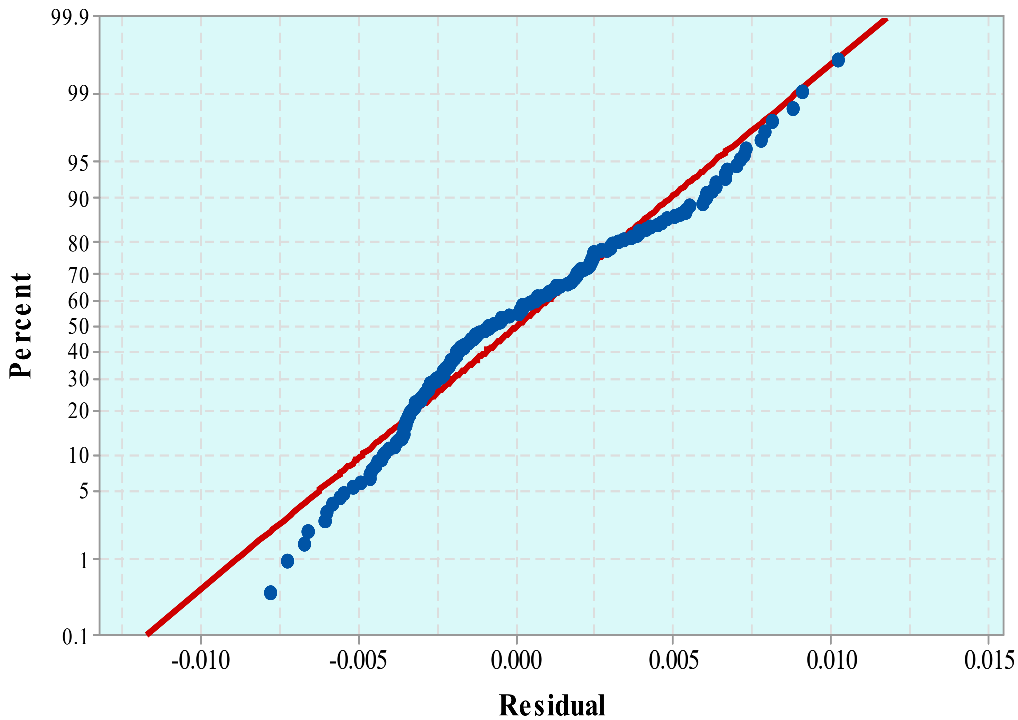


Figure 7.25 (c): Normality Check Plots for Repeated Load Axial Test

Figure 7.25: Residual Normality Check Plots for CWTT, APA And RLAT

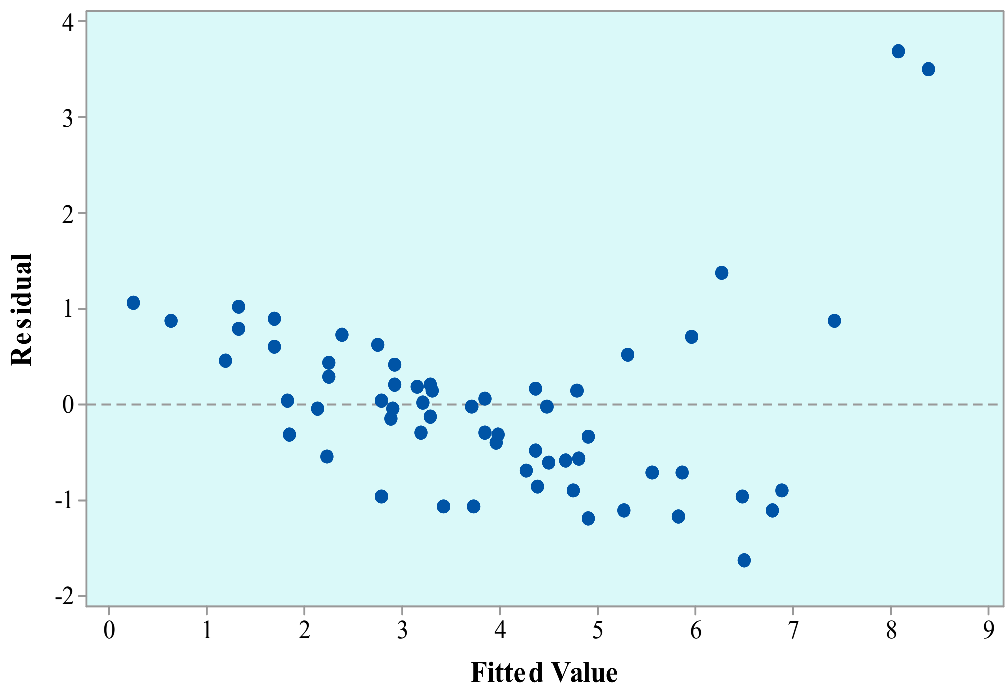


Figure 7.26 (a): Fitted Versus Residual Plot for Cooper Wheel Tracking Test

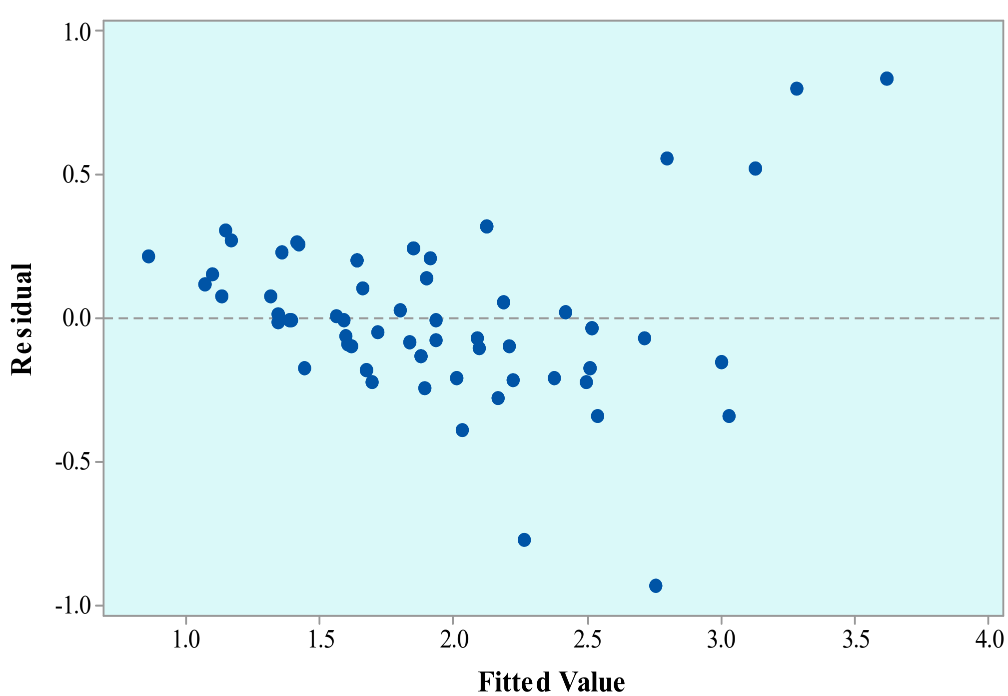


Figure 7.26 (b): Fitted Versus Residual Plot for Asphalt Pavement Analyzer Test

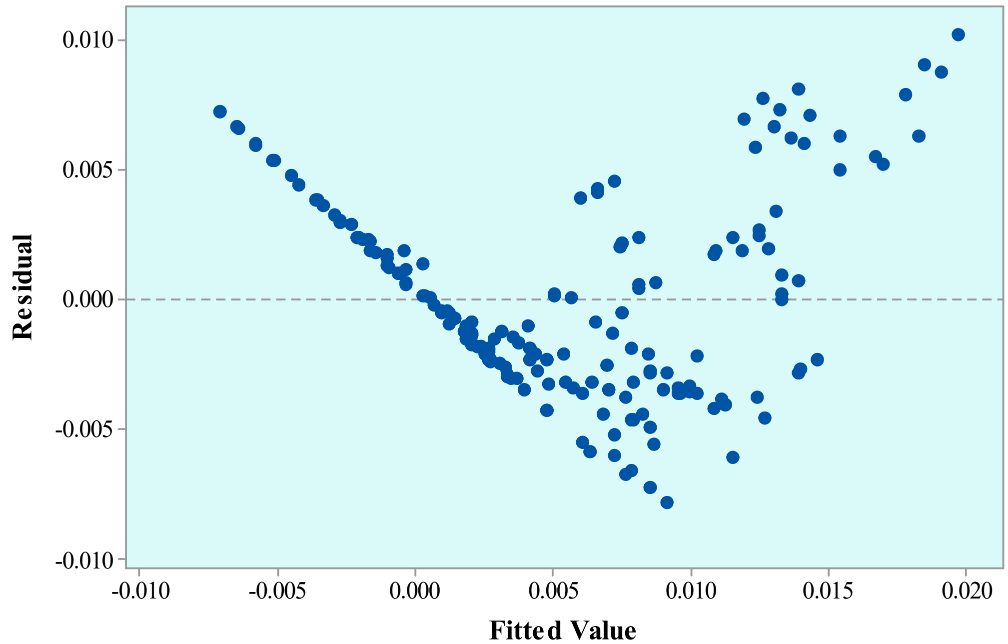


Figure 7.26 (c): Fitted Versus Residual Plot for Repeated Load Axial Test

Figure 7.26: Fitted Versus Residual Plots For CWTT, APA And RLAT

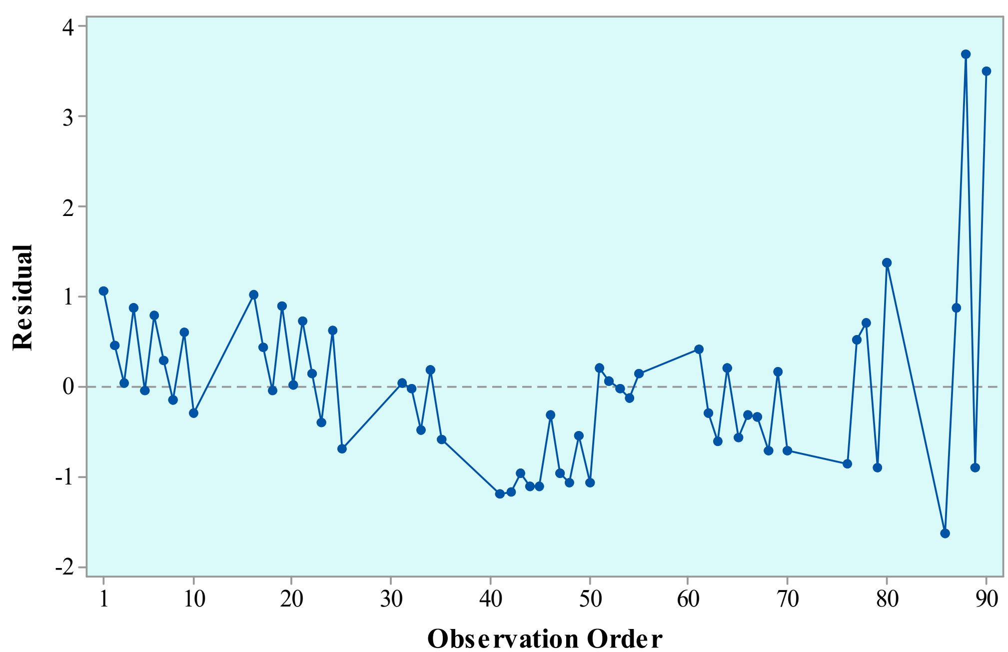


Figure 7.27 (a): Residual Versus Order Values Plot for Cooper Wheel Tracking Test

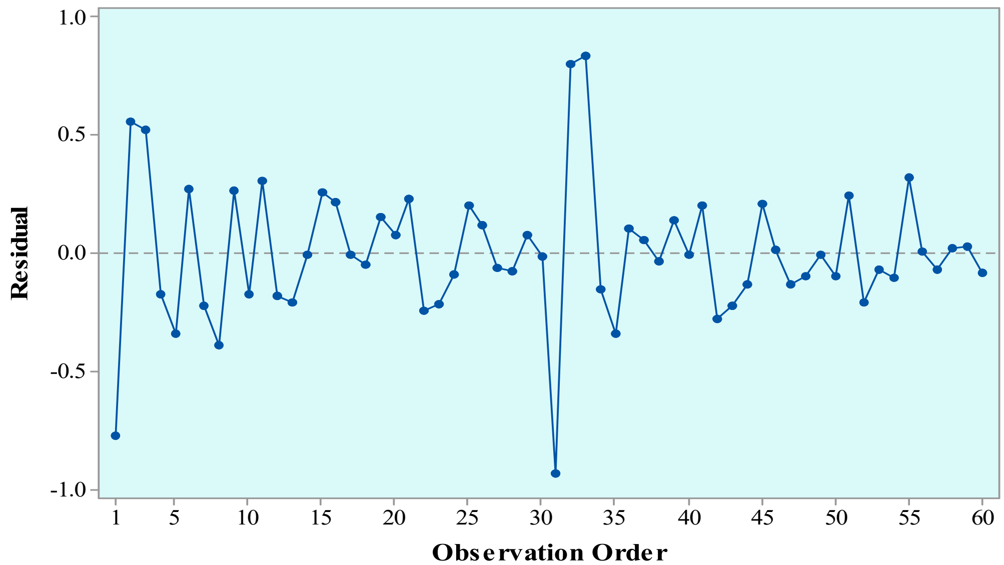


Figure 7.27 (b): Residual Versus Order Values Plot for Asphalt Pavement Analyzer Test

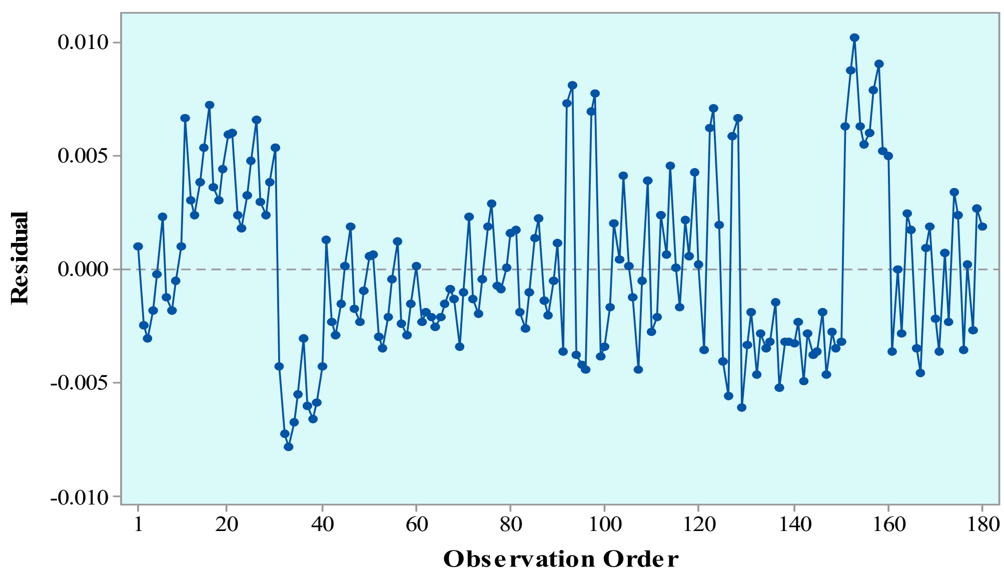


Figure 7.27 (c): Residual Versus Order Values Plot for Asphalt Pavement Analyzer Test

Figure 7.27: Residual Versus Order Values Plots For CWTT, APA and RLAT

7.6 ANOVA Table

Analysis of variance (ANOVA) is a statistical technique to quantify the amount of variation within groups or between groups existing in an experiment. A large amount of variance in experiments generally interprets significant finding from the research question under consideration. ANOVA can be one way or two-way, depending on the expected outcomes of research. The primary purpose of a two-way ANOVA is to understand if there is an interaction between the two or more independent variables and their effect on a dependent variable. In ANOVA the effect of different independent variables i.e., temperature, aggregate source index, bitumen penetration and aggregates gradation index on dependent variable of rut depth can be predicted effectively by using statistical terms such as sum of square, degree of freedom, mean squares, F-value and P-value. Table 7.9 represents the statistical terms in ANOVA output. The sum of squares term in ANOVA output explains the variation in dependent variable due to one or more independent variables. It can be divided into couple of components i.e., adjusted sum of squares and error sum of squares. The adjusted sum of squares for each independent variable quantifies the amount of variation in the response data that is explained by that particular variable while; error sum of squares of independent variables quantifies the proportion of variation in dependent variable that has not been explained by respective independent variables. In Table 7.9, adjusted sum of squares for all four independent variables i.e., aggregate source index, bitumen penetration, aggregate gradation index and temperature has been shown along-with error sum of squares. It has been observed that majority of total sum of squares, 198.75 out of 252.67; 21.015 out of 26.138; 0.0063 out of 0.0089, has been explained by four selected variables for CWTT, APA and RLAT, respectively. In the analysis of variance table, the total degrees of freedom (DF) indicate the amount of useful information available in the data set. The useful information about any population data could be increased by increasing the sample size, which results in greater DF values. DF is calculated by total number of values in an independent variable source minus one e.g., in this study temperature variable can have two possible values 40°C or 50°C, so its degrees of freedom (DF) is calculated by two minus one i.e., one as shown in third column of Table 7.9. Mean squares error term measures the amount of variation an independent variable explains, assuming that all other independent variables are included in the ANOVA model, regardless of their order. Unlike adjusted sums of squares, mean squares consider the degrees of freedom. The values of mean squares are shown in column 4 of Table 7.9. The F-value is the test statistic used to determine whether the particular independent variable is significantly affecting the response/dependent variable or not. Statistical software uses the F-value to calculate the significance-value, which is used to make a decision about the significance of the variables in an ANOVA model. If the obtained F-values are greater than F-critical, then it is understood that p-value<α-value and the variable is significant. So, greater is the F-value from F-critical, lower is p-value from 0.05 and greater is significance of that variable. The procedure of obtaining F-critical from F tables required three parameters. First is numerator degree of freedom DF1, which is degree of freedom of a particular independent variable e.g., DF1 (temperature) =N-1=2-1=1 considered in this study. Second is denominator degree of freedom, DF2, which is error degree of freedom minus number of levels of a particular variable e.g., DF2 (temperature) =50-2= 48 utilized in this research study. The third parameter to be used to obtain F-critical is α-value, which is selected as 0.05 in this research. With the help of these three parameters the F-critical (temperature) is read as 3.94. On similar basis, F-critical values for all independent variables and their interactions have been computed. If the exact value of degrees of freedom is not available in F-table, the nearest value will be selected. Temperature variable has highest F-value of 35.64, 35.02, and 110.0, in case of CWTT, APA and RLAT; among all the significant independent variables. Therefore, it has highest relative significance in the ANOVA model for rut depth or strain/cycle prediction. The p-value is a probability that measures the evidence against the null hypothesis. Lower p-values provide stronger evidence against the null hypothesis. The statistical significance of the effect depends on the P-value, i.e., if the P-value is larger than the selected significance level (α), the effect is not statistically significant but if the P-value is less than or equal to α, then the effect for that particular term is statistically significant. P-value of temperature, aggregate source index, bitumen penetration and aggregate gradation index is less than the selected significance level of 0.05 so these source variables are found to be significant. R-squared (R^2) is the percentage of variation in the response variable that is explained by the model. It is calculated as one minus the ratio of the error sum of squares to the total sum of squares. The higher the R^2 value, the better the model fits data. Adjusted R^2 is the percentage of the variation in the response variable that is explained by the model, adjusted for the number of predictors in the model relative to the number of observations. Table 7.9 shows a significant amount of variation in rut depth or strain/cycle (i.e., 74.82, 76.87, and 68.92) has been explained in case of CWTT, APA and RLAT.

Table 7.9: Analysis of Variance (ANOVA) for CWTT, APA and RLAT

Cooper wheel tracking test						
Term	**DF**	**Adj. SS**	**Adj. MS**	**F-value**	**F-critical**	**P -Value**
Temperature	1	38.44	38.44	35.64	4.03	0.000
Aggregate source index	2	37.97	18.99	17.6	3.18	0.000
Bitumen penetration	2	33.86	16.93	15.7	3.18	0.000
Aggregate gradation index	4	29.94	7.49	6.94	2.56	0.000
Error	50	53.92	1.07			
Total	59	252.67				
R square (adjusted)		74.82%				
Asphalt pavement analyzer test						
Term	DF	Adj. SS	Adj. MS	F-value	F-critical	P -Value
Temperature	1	3.588	3.588	35.02	4.03	0.000
Aggregate source index	2	0.739	0.37	3.61	3.18	0.034
Bitumen penetration	2	6.88	3.44	33.57	3.18	0.000
Aggregate gradation index	4	5.152	1.288	12.57	2.56	0.000
Error	50	5.123	0.103			
Total	59	26.138				
R square (adjusted)		76.87%				
Repeated load axial test						
Term	DF	Adj. SS	Adj. MS	F-value	F-critical	P -Value
Temperature	2	0.0034	0.0017	110	3.06	0.000
Stress	1	0.0013	0.0013	86.33	3.91	0.000
Aggregate source index	2	0.0005	0.0002	15.45	3.06	0.000
Bitumen penetration	2	0.0000	0.0000	1.18	3.06	0.311
Aggregate gradation index	4	0.0005	0.0001	7.4	2.43	0.000
Error	168	0.0026	1.5E-05			
Total	179	0.0089				
R square (adjusted)		68.92%				

7.7 Slope and Intercept Coefficients (a and b)

The power model proposed by Garba (2002) was fitted to the accumulated permanent deformation curves of the selected asphalt mixtures. It is the most commonly used permanent deformation equation. The power models plots as straight line on log-log scale. It has been reported by Rutting and Little (2014) that the slope 'b' and intercept 'a' of this model when plotted on log scale may be used as indicators of rutting resistance. The parameters of the power model 'a' and 'b' were calculated using the method of linear regression, i.e., by fitting straight line to the logεp-logN plot as shown in Figure 7.28.

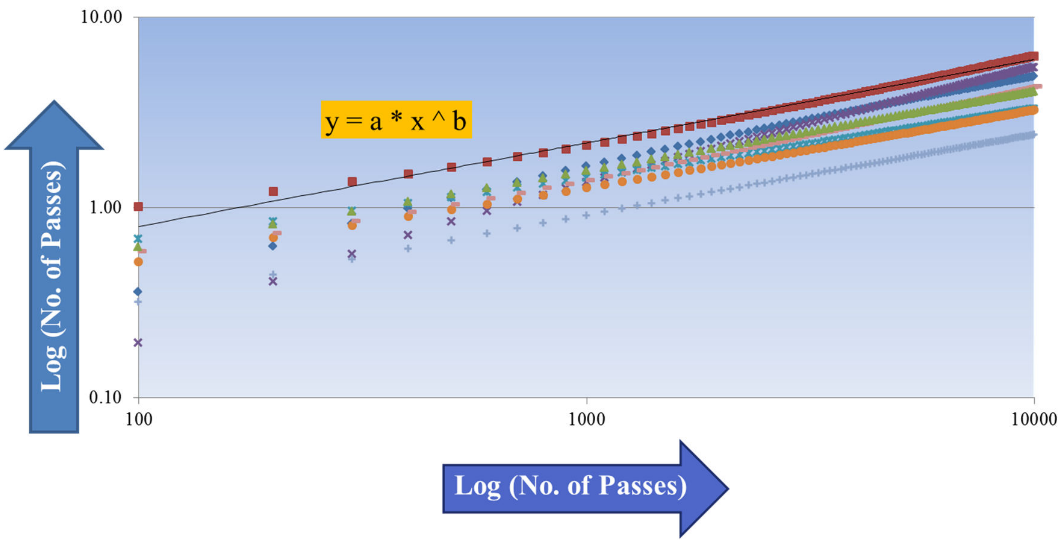


Figure 7.28: Computation of Slope and Intercept Coefficients

Figure 7.29 shows the plot of rut depth versus slope coefficient for both CWTT and APA test. It has been inferred that slope coefficient is directly proportional to the ultimate rut depth for both the rutting simulative tests. Dispersion is greater for CWTT data as compared to APA data. However, R^2 of linear trend line between rut depth and slope coefficient 'b' shows strong correlation in case of CWTT as compared to APA test. The shift of rut depth from APA test to CWTT can also be observed from Figure 7.29. Figure 7.30 shows the scatter plot of permanent strain versus slope coefficient in RLAT at a single stress level of 500 kPa at 25°C and 50°C. For 25°C, slope coefficient increases with an increase in permanent strain. Whereas at 50°C, data points exist in straight vertical line, since samples prematurely achieved terminal strain of 3% before completion of 3600 loading cycles. R^2 of linear trend line between permanent strain and slope coefficient 'b' indicates a more strong correlation in case of low temperature of 25°C. Shift in RLAT results with varying temperature can also be observed from Figure 7.30.

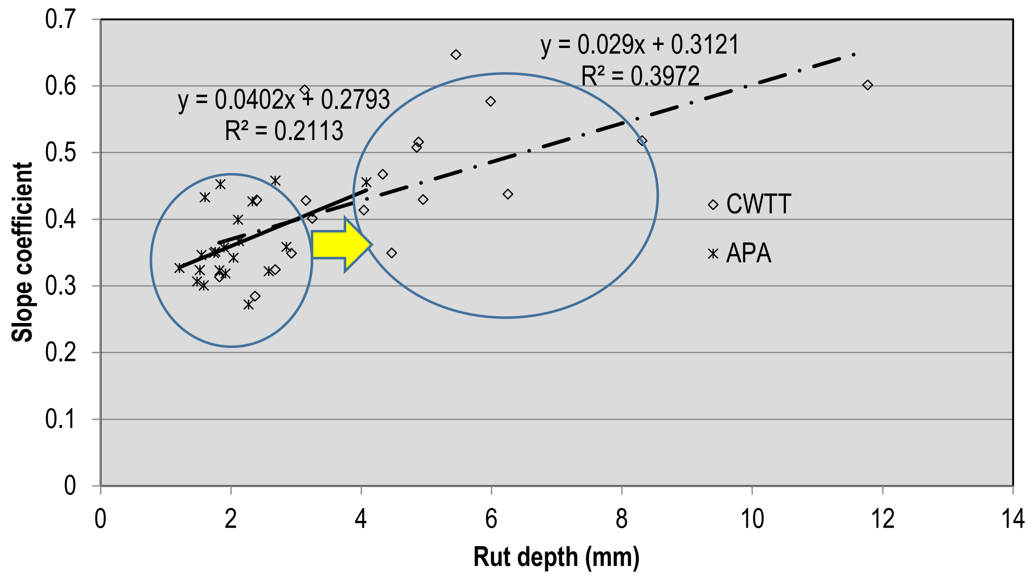


Figure 7.29: Plot of Rut Depth Vs. Slope Coefficient for CWTT and APA Test

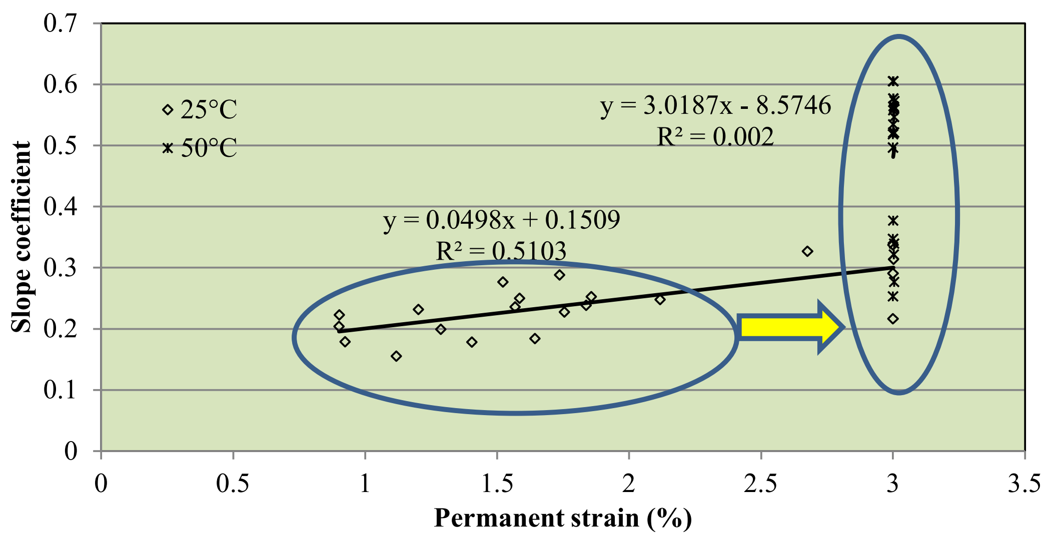


Figure 7.30: Plot of Permanent Strain Vs. Slope Coefficient for RLAT Test at Stress Level Of 500 kPa

7.8 Prediction Modeling

A deterioration shift function/prediction model is a vital element for an effective and successful pavement management system (Mahmood et al., 2016). They are important prerequisites for the empirical portion of mechanistic-empirical pavement design which will be developed by statistical based correlations between in-service pavement performance under actual wheel loads and laboratory based performance. Laboratory based shift functions/prediction models are empirical performance equations developed with a set of significant independent variables affecting the critical material responses e.g., permanent deformation behavior. It is well established fact that rutting of asphalt mixtures is affected by individual material properties, environmental temperature, cycles of wheel load and mixture gradations. However, the effects of all these parameters are not commonly evaluated for prevailing environmental conditions and local materials being used in road pavements in Pakistan. Therefore, one of the objectives of this study was to develop a shift function/prediction model to predict rutting with these variables.

7.8.1 Non-linear Multiple Regression Modeling

The regression technique is generally purposed to predict the values of dependent variable 'y' on the basis of statistically significant combination of one of more independent variables. In many situations, the relationship between 'y' and any single predictor variable is not strong, but knowing the values of several independent variables may considerably reduce uncertainty about the associated 'y' value. In this situation, multiple regression analysis satisfies the purpose. Nonlinear multiple regression is a type of regression in which one of more independent variables are combined by nonlinear functions to predict a dependent variable. Advanced modelers utilized various nonlinear functions for this purpose, such as exponential functions, logarithmic functions, trigonometric functions, power functions, Gaussian function, and Lorenz curves.

SPSS statistical analysis tool is used to develop non-linear multiple regression prediction model for rutting propensity of asphalt mixtures. Non-linear regression model with Cobb-Douglas formulation has been widely used in various empirical studies in the past (Hossain et al., 2012). The same functional formulation better describe the data set in present study. This functional form is characterized by the isoquants that are convex to the origin and thus capture the nonlinear response (rut depth/ permanent strain) to the variations in input variables (bitumen penetration, aggregate source index, aggregate gradation index, loading cycles, temperature and stress). The generic form of Cobb-Douglas function is given in Equation 7-8:

$$\boldsymbol{y = \alpha \times {x_i}^{\beta_i}} \tag{7-8}$$

Where i= number of variables= 1, 2, 3......n

It is generally accepted fact that traffic loading conditions, environmental conditions and constituent material properties have a significant impact on permanent deformation of asphalt mixtures. However, it is required to model the permanent deformation behavior of commonly used asphalt mixtures in the country, on the basis of quantified effect of local environmental conditions and indigenous constituent material properties. The significant independent variables identified by general full factorial design are combined with number of loading passes/cycles to develop a rutting shift function/prediction model for estimating the rutting performance of asphalt mixtures at each loading pass/cycle.

Rut depth and permanent strain are dependent variables for CWTT/APA test and RLAT results respectively. Based on general full factorial design; temperature, number of passes/cycles, bitumen penetration, aggregate source index, aggregate gradation index are included as independent variables in the model. Temperature variable is measured in units of degree centigrade, bitumen penetration in 1/10th of millimeter, aggregate source index in percent and aggregate gradation index in percent. The general equation can be written as

$$Rut\ depth = a * T^b * N^c * ASI^d * AGI^e * P^f \tag{7-9}$$

In case of RLAT, dependent variable of permanent strain has been predicted by additional variable of stress (σ, in units of kPa) along-with previous variables using non-linear multiple regression model. The equation becomes as:

$$\text{Permanent Strain} = a * T^{b} * N^{c} * ASI^{d} * AGI^{e} * P^{f} * \sigma^{g} \quad (7\text{-}10)$$

Where a, b, c, d, e, f, g are non-linear regression parameters

It is a general practice that above form of non-linear model has been adopted, depending on the data and proposed application. The model also contains unknown parameters known as non-linear regression parameters. A popular method to compute values of these parameters is least squares method. According to this method, the estimated values of these parameters are obtained by minimizing the error sum of squares. After several iterations performed to minimize total error sum of squares, the output of regression analysis for rutting data of CWTT, APA, and RLAT has been shown below in Table 7.10.

Table 7.10: Multiple Non-Linear Regression Model Parameters for CWTT, APA and RLAT

Parameter	CWTT		APA		RLAT	
	Estimate	Standard Error	Estimate	Standard Error	Estimate	Standard Error
a	4.80E-05	0.000	0.005	0.001	2.98E-04	0.000
b	2.066	0.029	0.981	0.024	1.718	0.004
c	0.596	0.006	0.333	0.004	0.271	0.001
d	- 0.914	0.013	- 0.549	0.011	- 0.665	0.003
e	- 0.412	0.009	- 0.452	0.008	- 0.318	0.002
f	1.163	0.018	0.899	0.017	0.386	0.004
g	---	---	---	---	0.702	0.003

The 'Standard Error' column of Table 7.10 contains terms equivalent to significance value in linear regression method, so value of standard error below 0.05 indicates that particular parameter and its associated variable is significant for 95% confidence level. All the considered parameters are found to significantly affect the dependent variables of rut depth/permanent strain in case of all three laboratory rutting performance tests. Table 7.10 shows the estimated values of associated regression parameters along with their significance values for three performance tests of CWTT, APA and RLAT.

It is notable that the relative sensitivity of various independent variables included in the model, can be determined by estimated values of regression parameters associated with each independent variable. The rutting shift function/prediction model developed in this research study concluded that temperature variable designated by 'T' has maximum sensitivity among the selected independent variables. The similar highest significance of temperature variable on pavement performance has been observed by Qiao et al., (2013). Bitumen penetration designated by 'P' is second most sensitive independent variable for CWTT and APA results while, stress designated by 'σ' is second most sensitive variable in case of RLAT results.

7.8.1.1 Model Validation

The developed model is validated by x-y scatterplot between the predicted and observed/measured data points. The significantly high value (i.e., ≤ 0.7) of coefficient of determination R^2 for CWTT, APA and RLAT results shown in Figure 7.31 validates the model`. The higher 'R^2' value lowers the error and residuals (difference between actual and predicted values).

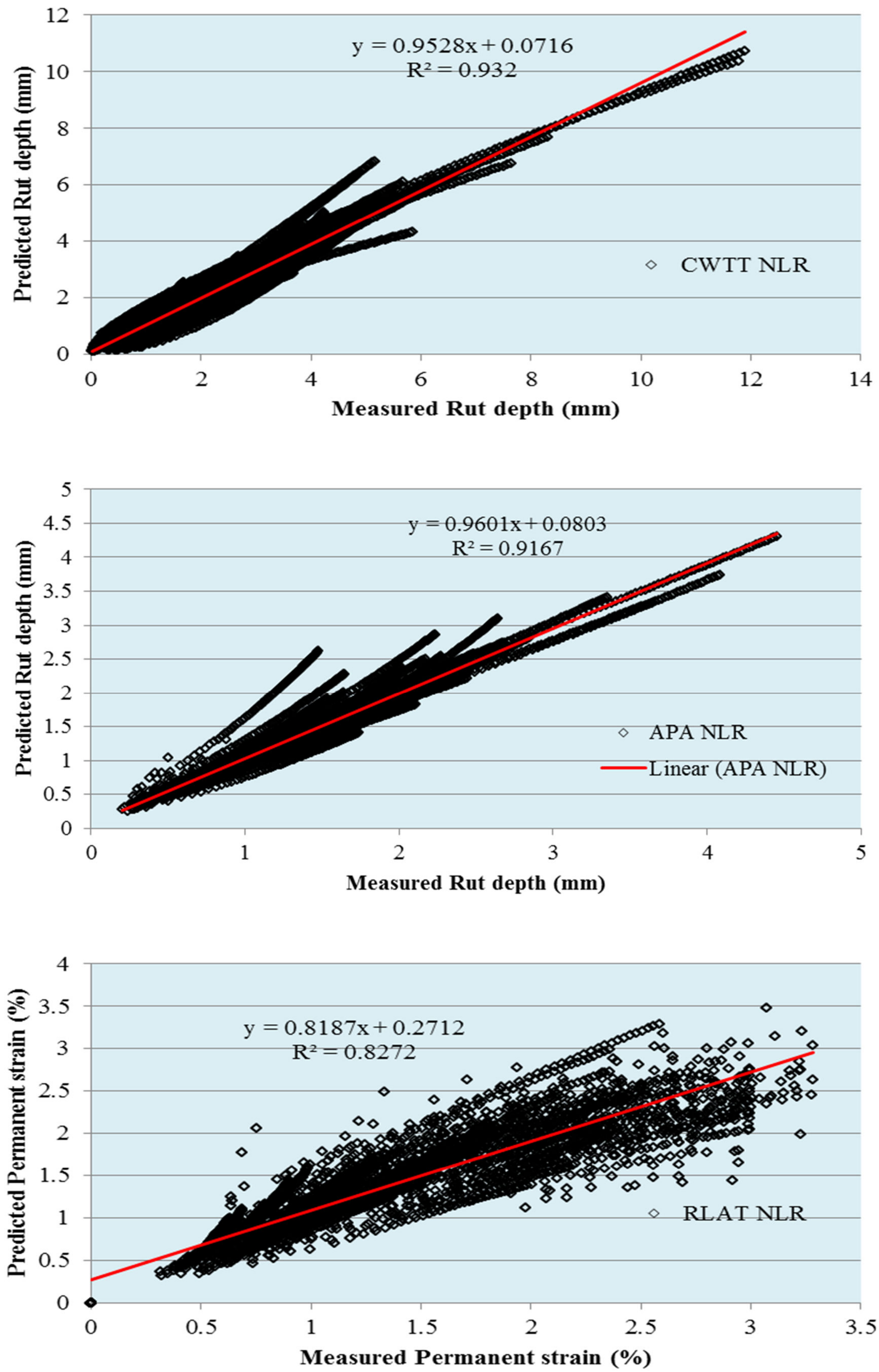


Figure 7.31: Predicted Versus Measured Plot by Non-Linear Regression Model for CWTT, APA, and RLAT

7.8.2 Artificial Neural Network Modeling

The Artificial Neural Network (ANN) approach is an efficient modeling tool which simulates some important features of the human nervous system. Analogous to a human brain, an ANN uses many simple computational elements, named artificial neurons, connected by variable weights. It resembles the human brain in two aspects; knowledge is acquired by the network through a learning process, interneuron connection strengths known as synaptic weights are used to store knowledge. The basic element of the ANN is the artificial neuron, which is mathematical formulation describing the biological neuron. The artificial is composed of three main components i.e., weights, bias and an activation/transfer function. A neuron receives inputs x1, x2, x3….xN attached with a weight of wiN showing the connection strength with that specific input. Each input has been multiplied with the corresponding weights for each connection. A bias θi is a type of connection weight added as constant non-zero value to the summation of inputs and their corresponding weights. The net summation neti is transformed by scalar-to-scalar transfer function to obtain the output y. The Figure 7.32 below shows the details of artificial neuron.

In past few decades several studies have been conducted to connect the artificial neurons to develop a network, and different algorithms have been set up to train them. Back-propagation algorithm has been frequently used to develop artificial neural networks. Back-propagation algorithm is a multilayer local gradient descent reduction technique (Agalbjorn et al. 1997).

ANNs are generally classified by their architecture and topology (either feed-forward or feed-backward). Multi-Layer Perceptron (MLP) is one of most frequently used feed forward ANN architectures developed on the basis of back-propagation algorithm for the classification of regression problems.

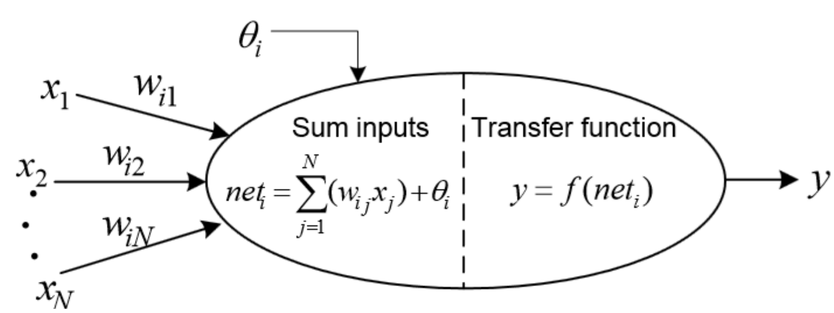


Figure 7.32: Details of A Typical Structure of Artificial Neuron

7.8.3 Multi-Layer Perceptron (MLP)

Multi-Layer Perceptron (MLP) network models are one of the most commonly used neural network architectures in various research areas of engineering, social sciences, strategic studies, medicine etc. MLP is a layered feed forward neural network architecture which optimized the set of inputs along-with bias unit through an activation transfer function to produce desired output. MLP network architecture is generally composed of an input layer comprising of units equivalent to input variables, a hidden layer comprising of several units, and an output layer comprising of output variable. The units in each layer are interconnected to subsequent units of other layers through weighted connections. The MLP transforms n inputs to l outputs through some nonlinear functions. The mostly used activation function is the sigmoid and it is given as follows:

$$x_o = \frac{1}{1 + \exp(-\sum x_h\, w_{ho})} \qquad \text{7-11}$$

Where x_h is activation of h^{th} hidden layer node and W_{ho} is the interconnection between h^{th} hidden layer node and o^{th} output layer node.

As predicting the rutting performance through different material and environmental factors is complex phenomenon, MLP could be efficiently used to accomplish this purpose. The dependent and independent variables included in multiple non-linear regression models were selected and modeled by MLP neural network technique. Hence, rut depth is selected as dependent variable; while temperature, number of passes/cycles, bitumen penetration, ASI, and AGI as independent variables for CWTT and APA test. For RLAT, permanent strain is considered as dependent variable while temperature, number of cycles, bitumen penetration, ASI, AGI and stress are selected as independent variables. SPSS statistical analysis software has been used for MLP ANN modeling. Fifteen different models were developed cumulatively (i.e., 5 mixture gradations x 3 performance tests). Total data input values were 1200 for each of five CWTT models, 960 for each of five APA test models; and 808, 675, 632, 690, 690 for five RLAT models, respectively. Seventy percent data has been used for training and thirty percent for testing as per default criteria in SPSS software. Hyperbolic function has been utilized as an activation function. The MLP ANN architecture for the research study has been shown below in Figure 7.33 and Figure 7.34 for CWTT/APA and RLAT, respectively.

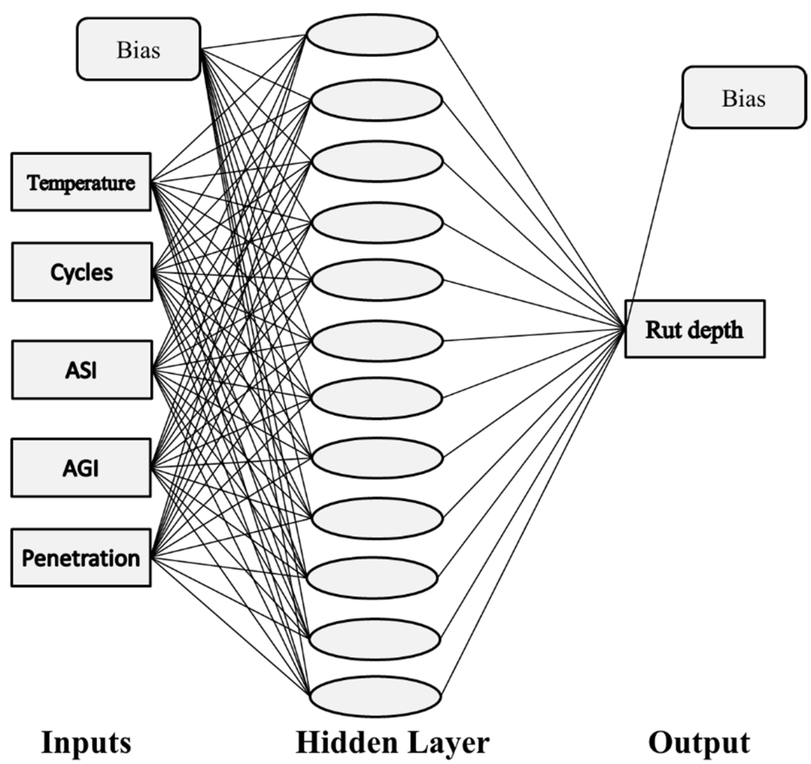


Figure 7.33: ANN Architecture for CWTT and APA

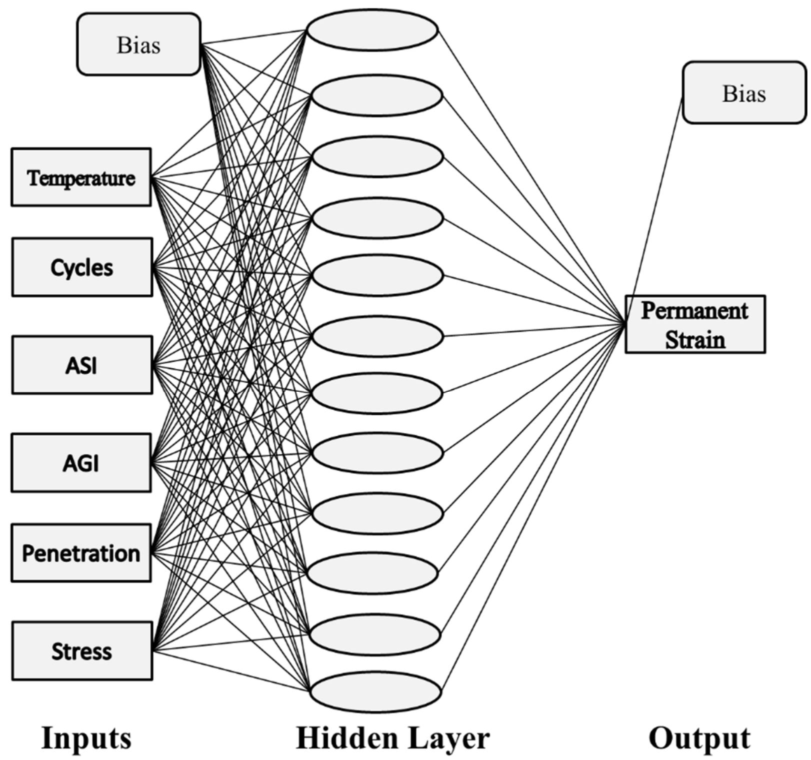


Figure 7.34: ANN Architecture for RLAT

The effectiveness of ANN models for five gradations has been tabulated in Table 7.11. Two parameters of 'sum of squares error' and 'relative error' are shown for both the training and testing data sets. The 'sum of squares error' is displayed when the output layer has scalar variables. This is the error that is minimized by the network during training by performing various iterations. Iterations are performed until single consecutive step occurred without reduction in error. The 'relative error' is the ratio of the sum-of-squares error for the dependent variable to the sum-of-squares error for the "null" model, in which the mean value of the dependent variable is used as the predicted value for each case. These parameters can be computed by Equation 7-12 and Equation 7-13, respectively.

$$E_T(w) = \sum_{r=1}^{s} E_r(w)\ ;\ E_r(w) = \frac{1}{2}\sum_{k=1}^{l}\left(T_k^{(r)} - o_k^{(r)}\right)^2 \tag{7-12}$$

$$RE = \frac{\sum_{r=1}^{s}\sum_{k=1}^{l}\left(T_k^{(r)} - o_k^{(r)}\right)^2}{\sum_{r=1}^{s}\sum_{k=1}^{l}\left(T_k^{(r)} - \overline{o_k}\right)^2} \tag{7-13}$$

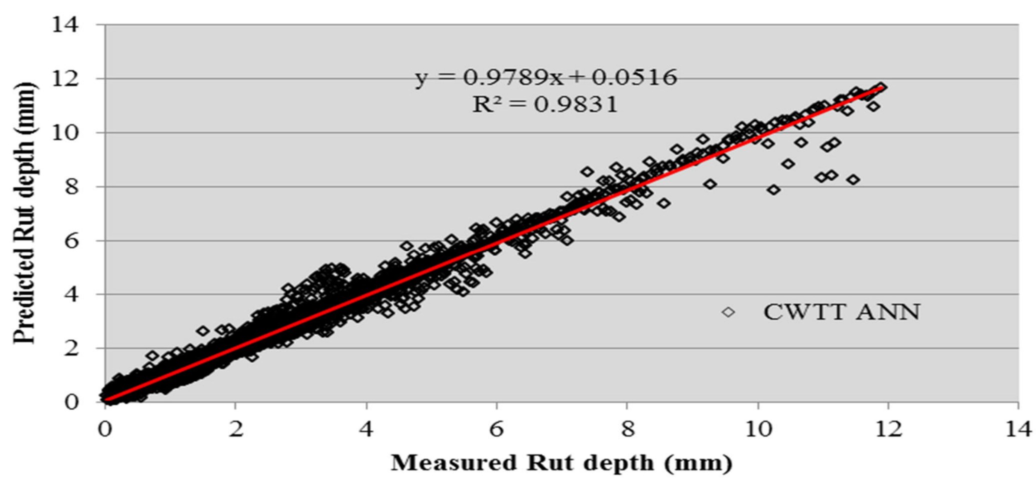
Where, $E_T(w)$= Sum of squares of output error for all training cases, $E_r(w)$= Square of output error for a single input case, s= Number of cases in the data sample, l= Number of neurons in the output layer, $T_k^{(r)}$= Target value of neuron k for case r, $O_K^{(r)}$= Output of neuron k for case r, RE= Residual error, O_k= Mean of $O_K^{(r)}$ over all the cases.

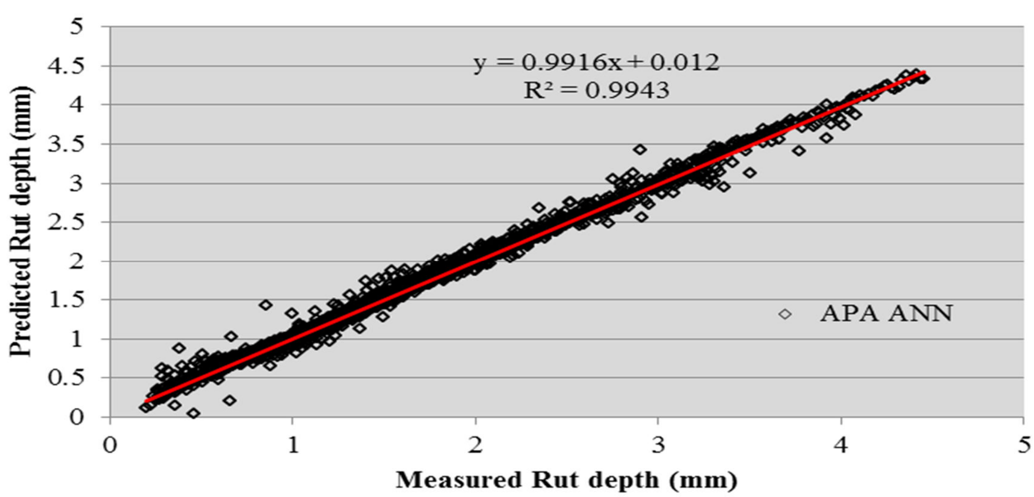
It is desirable to have the minimum values of both the computed parameters. Table 7.11 shows that the developed ANN model most accurately predicted permanent strain for APA test results as compared to rut depth/permanent strain results of CWTT and RLAT results for selected wearing course gradations.

Table 7.11: ANN Model Performance Parameters for Selected Asphalt Mixture Gradations

Parameters	CWTT		APA		RLAT	
	Training	Testing	Training	Testing	Training	Testing
Error sum of squares	44.500	23.882	35.428	19.509	91.769	43.017
Relative Error	0.027	0.038	0.017	0.022	0.075	0.087

The x-y scatter plots between measured versus predicted values from ANN model are shown in Figure 7.35 below for all wearing course gradations and all performance tests. As previously discussed, the quality of any model is judged by the precision with which the predicted values coincide with experimentally observed values. The greater the precision, the greater is the probability of predicted data equivalent to experimental values, and the closer is the coefficient of determination (R^2) between predicted and physically measured/observed values to numerical value of 1. The deviation of individual data points from the linear trend line (with R^2=1) indicates the accuracy of the developed model. The higher R^2 values (i.e., ≥ 0.9) for different gradations and performance tests are indicative of significant dependency of the selected set of independent variables for the prediction of dependent variable selected in the study.





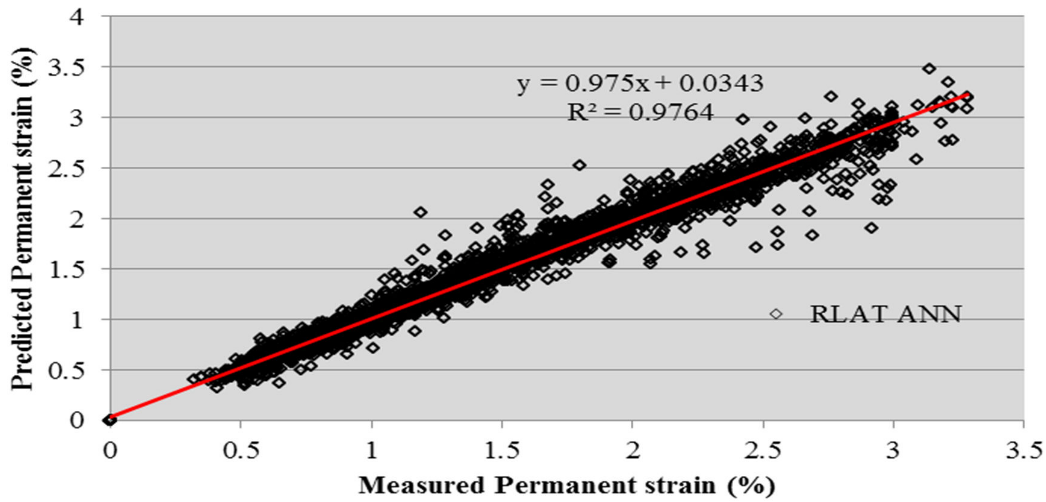


Figure 7.35: Predicted Versus Measured Plot by ANN (MLP) Model for CWTT, APA, And RLAT

7.9 Comparison of Modeling Techniques

The neural network techniques are semi-parametric while regression techniques are parametric modeling techniques. It is notable that the assumptions of normality, constant variance and independence of residuals/errors are mandatory for regression techniques while the output of connectionist techniques such as artificial neural networking do not rely on these assumptions. The neural network techniques use iterative weight adjustments for the selected set of various independent variables to converge the response variable; hence the autocorrelation affect between independent variables does not deteriorate the quality of model, in contrary to regression modeling technique. Hence the performance of both modeling techniques should be compared to specify the more precise statistical modeling technique for permanent deformation of asphalt mixtures.

The coefficient of determination (R^2) between the predicted and measured values reasonably measure the performance of any statistical model. In this study, in addition to R^2, values account for (VAF) and root mean square error (RMSE) indices were also computed to show the performance of developed models as employed by Yilmaz and Kaynar (2011) by Equation 7-14 and Equation 7-15 Where y and y' are the measured and predicted values, respectively while N is total number of observations. The calculated indices are given in Table 7.12. If the VAF is 100 and RMSE is 0, then the model will be excellent.

$$VAF = \left[1 - \frac{var(y - y')}{var\ y}\right] \times 100 \qquad \text{7-14}$$

$$RMSE = \sqrt{\frac{1}{N}\sum_{i=1}^{N}(y - y')^2} \qquad \text{7-15}$$

Mean absolute percentage error (MAPE) which is a measure of accuracy in a fitted series value in statistics was also used for comparison of the prediction performances of the models. MAPE usually expresses accuracy as a percentage as shown in Equation 7-16 below, in which Ai and Pi are actual and predicted values, respectively.

$$MAPE = \frac{1}{N}\sum_{i=1}^{N}\left|\frac{A_i - P_i}{A_i}\right| \times 100 \qquad 7\text{-}16$$

The obtained values of R^2, RMSE, VAF and MAPE, given in Table 7.12, indicated high prediction performances in case of using artificial neural network (ANN) modeling technique as compared with multiple nonlinear regression modeling technique.

Table 7.12: Comparison between Multiple Nonlinear Regression Model and ANN Model

Performance Tests	Dependent Variable	Independent Variables	Multiple Non-linear Regression (MNR)				Artificial Neural Network (ANN)			
			R^2	VAF	RMSE	MAPE	R^2	VAF	RMSE	MAPE
CWTT	Rut depth	T, N, P, ASI, AGI	0.932	93.158	0.420	19.871	0.983	98.309	0.208	7.678
APA	Rut depth	T, N, P, ASI, AGI	0.915	91.464	0.185	8.749	0.994	99.430	0.049	2.210
RLAT	Permanent Strain	T, N, P, ASI, AGI, σ	0.827	82.712	0.298	15.315	0.976	97.635	0.109	4.874

Among the dependent variables shown in column 3 of Table 7.12, T designates temperature (°C), N is number of passes in case of CWTT and number of cycles in case of APA test and RLAT, P is penetration value of bitumen, ASI designates aggregate source index, AGI designates aggregates gradation index, while σ shows stress (kPa).

7.10 Conclusions

The following conclusions have been drawn from this research: -

- Asphalt mixtures are evaluated on the basis of terminal permanent deformation values and rutting/ permanent strain slope plots. The rutting/ permanent strain evaluation plots indicated that certain asphalt mixtures with coarser gradations stabilize with increasing loading cycles at high temperature conditions, hence, the impact of temperature variable is more dominant in case of finer graded mixtures. This behavior is observed due to the fact that there is more point-to-point contact between particles in case of coarser gradations. Therefore, role of temperature susceptible visco-elastic bitumen mastic becomes less dominant in case of coarser mixtures, resulting in relatively lesser variation in rutting slope at increasing temperature. The softness/hardness of binder and the aggregates quality in terms of engineering properties also observed to significantly influence the laboratory permanent deformation evaluation of asphalt mixtures.
- The ranking of different wearing course mixtures was found to be nearly consistent as translated by Spearman's rank correlation coefficient (ρ) of 0.81, 0.73, and 0.85 for ranking obtained by different combinations of laboratory rutting performance tests of CWTT, APA test and RLAT. The wearing course asphalt mixtures included in

the testing regime were categorized on the basis of rank product statistic (RP) computed from rank order in individual laboratory performance tests.

- General full factorial design is found to be an efficient tool to statistically quantify the significance of different factors on the output variables. The results of general full factorial design revealed that temperature is most significant factor affecting the rutting propensity of asphalt mixtures. The other significantly affecting variables include bitumen penetration, aggregate source index (ASI), aggregate gradation index (AGI), and stress condition. The aggregate source and aggregate gradation indices are innovatively developed in this study by combining important properties differentiating aggregate sources and aggregate gradations respectively.
- Each rutting performance test has been conducted according to different specifications. Ranking of relative suitability of each performance test has been done by comparing the time required to achieve one millimeter of rut depth. It has been observed that generally APA test takes maximum time to reach one millimeter rut depth. This behavior is due to more homogeneous and properly compacted specimens, using Superpave gyratory compactor, in case of APA test samples; as compared to Cooper roller compactor compacted slab specimens in case of CWTT. However, un-confinement is the major cause of the minimum time to achieve one millimeter rut depth in case of RLAT, when compared with APA results.
- The dependent variables of each rutting performance test are correlated with each other in order to identify the most suitable combination of laboratory rutting performance tests. The identification might propose suitable quality assessment alternative in case of non-availability of a particular performance test by converting results obtained from one test to that of other. Among different functions, quadratic function is found to have best correlation in terms of coefficient of determination (R^2). It has been concluded that rut depth observed in CWTT and permanent strain/cycle observed in RLAT has most significant correlation with R^2 of 0.7263.
- Rutting shift function / prediction models based on two statistical techniques are developed as a final product of this thesis. Instead of simple linear regression, multiple non-linear regression technique with Cobb-Douglas formulation better predicts the output. The multilayer perceptron (MLP) algorithm of artificial neural network (ANN) technique has also been used to predict the laboratory rutting behavior with same set of variables. The comparison of two modeling techniques reveals that ANN better predicts the rutting parameters when compared to non-linear regression technique.

8 Fatigue Behavior of Asphaltic Concrete Mixtures

8.1 Introduction

A flexible pavement primarily consists of binder and aggregate, and strength of such pavements largely depends on both the ingredients. However, when a flexible pavement is exposed to moving traffic, various distresses manifest on the surface of the flexible pavement such as rutting, fatigue cracking etc. (Y. Ali et al, 2018). These distresses eventually lead to a premature failure of flexible pavements, resulting in a reduced service life, high maintenance, and rehabilitation cost (F. Khodary Moalla Hamed, 2010). Fatigue in AC mixtures is defined as the phenomenon causing cracking (consisting of a crack initiation and propagation phase) due to the tensile strains generated in pavements when subjected to load repetition, temperature variation, and inadequate construction practices collectively (F. Wu, 1992). In general, characterisation of fatigue is carried out using two approaches: the traditional/conventional method using the strain (or stress)-based models (C.L. Monismith et al., 1969), and the dissipated energy method that is defined as a damage indicator of a material (K. Ghuzlan, et al., 2000).

In general, the fatigue behaviour of an AC mix can be expressed as a relationship between the number of cycles to failure and initial strain (AASHTO, 1994), however, the literature review reveals disagreement in the definition of fatigue especially in strain-controlled mode. Some researchers claim that 50% loss in modulus or stiffness value from an initial value should be termed as a fatigue life [(R. Hicks, et al., 1993), (B. Smith, et al., 2000)], while others use phase angle as a parameter to determine the fatigue life in fatigue testing (R. Reese, 1997). R. Reese, (1997) argued that the fatigue failure point could be related to maximum phase angle because the curve of phase angle against time indicates a drastic decline in phase angle when AC mixtures could not attain further distress. In addition, many researchers used 50% loss in pseudo stiffness as a failure criterion [(Y.R. Kim, et al., 1997), (H. J. Lee, et al., 2000)] since pseudo stiffness can closely predict the damage accumulation occurred due to repeated fatigue loading, which did not induce damage because linear viscoelastic time-dependency is eliminated.

Strain and stress controlled modes are respectively used to simulate conditions in thinner and thicker pavements. Tensile strains are accumulated at the bottom of an AC layer in the first mode, while in the latter, the cracking appears on the top of an AC layer due to the localised stresses developed because of the tyre-pavement interaction (J.A. Epps, et al., 1972). The review of the past studies suggests that for a given mix under same conditions, stress and strain mode yields different results (Y. R. Kim, et al., 2003). Moreover, Monismith and Y. Salam (1973), concluded that strain and stress controlled modes of loading are respectively characteristics of asphalt layers less and greater than 50 mm. The difference in these testing modes is explained by S. Brown (1978), using the failure mechanism. The failure is observed at two stages: the first when cracks are manifested at distinct points due to high stresses followed by crack propagation through a mix until the complete failure occurs. The crack propagation in stress controlled mode is very rapid since it mainly depends on the intensity of stress at the tip of cracks. On the other hand, the crack propagation in strain-controlled mode is relatively slower as stress slowly decreases after the initiation stage that refers to a significant decrease in stiffness of the material. The damage caused by cracking and deformation is generally due to change in environmental and traffic loading conditions, which can be related to frequency and magnitude of loading, loading time, and variation in temperature (F. Wu, 1992).

ITFT, due to its simplicity compared to other alternative tests, is very appealing for characterisation of fatigue behaviour of AC mixtures. Literature suggests that the performance of AC mixtures using ITFT showed a shorter fatigue life to failure than the bending beam test (T.W. Kennedy, 1977). ITFT, in the past, is also compared with other two test methods: the trapezoidal cantilever test and the uniaxial tension-compression test. The results reveal that under given set of conditions, ITFT showed equivalent results to the other two tests; and also, ITFT has a tendency to generate fatigue profile in minimum time of four hours, which is relatively quicker than the other two methods. This facilitates the commercial use of ITFT (J.M. Read, 1996). Various studies illustrated the use of other procedures to determine the fatigue life of AC mixtures besides the traditional approaches [(K. Ghuzlan and S. Carpenter, 2000), (K.A. Ghuzlan, 2001), (B.S. Underwood, 2016)]. Most of the studies found in the literature developed a model based on the initial strain and the mix volumetric parameters except (Y. Ali, et al., 2015) where the fatigue life is evaluated using dynamic modulus and phase angle, however, these studies did not develop any model. The effect of different variables, e.g., stiffness, mix volumetric, binder properties, etc. is currently under-represented in the literature and our understanding of this important aspect remains elusive. Table 8.1 illustrates the findings of different past studies on fatigue behaviour of asphalt mixtures.

Table 8.1: Evaluation of Fatigue life in the Literature

Study	Explanatory Variables	Model Functional Form
Lytton et al.	Bitumen content, stiffness, air voids, aggregate type, gradation and angularity	Linear
Harvey and Tsai	Initial stiffness and mix volumetric	Intrinsically linear
Kim et al.	Stress level	Power
Lee and Kim	Pseudo stiffness	Linear
Rodrigues	Traffic speed and the shape of the stress pulse	Quadratic
Hartman	Type of compaction	Linear
Kim et al.	Strain rates and damage growth	Linear
Kim et al.	Initial pseudo-stiffness, damage parameter to fatigue failure, material parameter	Exponential
Zhou et al.	Initial stiffness	Power
Yeo et al.	Tensile strain	Power
Xiao et al.	Initial flexural strain, VFA, AV, initial dissipated energy, initial mix stiffness	Artificial neural network
Al-Rub et al.	Fundamental material properties	Finite element model
Salama and Chatti	Axle load and truck configuration	Power
Al-Khateeb and Ghuzlan	Temperature, stress, and loading frequency	Exponential
Ali et al.	Dynamic modulus and phase angle	No model was developed
Mannan et al.	Strain	Power
B. S. Underwood	Strain amplitude	Power

The review of past studies suggests that fatigue characterisation using ITFT is still under the focus of research that needs further investigation and its correlation with other tests such as four-point bending beam is also an important aspect to be looked into in future research endeavours. Moreover, the time consumption for specimen preparation and experimental testing is much higher for bending beam test as compared to ITFT. One more advantage could be the derivation of indirect tensile strength during the fatigue testing that gives an idea of a mix stiffness and strength. Also, previous modelling efforts did not capture the effect of exogenous variables that can help in explaining fatigue behaviour of AC mixtures.

Currently, in Pakistan, empirical procedures are used for both structural and bituminous mix design of flexible pavements that were developed for much lighter traffic and tyre pressure (AASHTO, 1993) compared to the conditions prevailing in Pakistan. Therefore, the Mechanistic-Empirical (M-E) approach seems to be more favourable to produce reliable and cost-effective asphalt mix design that takes into account the effect of climate, load repetition, and material properties in the pavement performance. A series of studies have been conducted to characterise the mixtures using different tests that can help in the implementation of M-E design in Pakistan [(M. Irfan, et al., 2018), (M. Irfan, et al., 2017), (M. Irfan, et al., 2016)].

Owing to need and given resources, this study aimed to characterise AC mixtures using ITFT on the laboratory compacted specimens to determine the effect of varying gradations and type of bitumen grade. It is expected that the implementation of M-E approach would improve the efficacy of design and produces more reliable maintenance and rehabilitation needs over the life of the asphalt pavements.

8.2 Objective and Scope

The objective of this study is twofold: (a) to determine the effect of different aggregate gradation on the fatigue life using ITFT; and (b) to analyse the differential effect of penetration grade on the fatigue life. Under different stress levels, the effect of different factors including the bitumen type, bitumen content, resilient modulus, and initial strain on the number of cycles to fatigue failure is investigated. The study comprises of four wearing course gradations, two bitumen sources 40/50 and 60/70, and single limestone aggregate.

8.3 Methodology

The research methodology adopted in this study is shown in Figure 8.1. Specimens, compacted using Marshall Compactor, were subjected to further testing to determine the performance parameters, e.g., flow and stability, and volumetric parameters including Percent Air Voids (AV%), Percent Voids in Mineral Aggregate (VMA %), and Percent Voids Filled with Bitumen (VFB%). The stress is varied from 2000 N to 5500 N. Note that 75 blows were applied to 100 mm diameter specimens prepared for Marshall testing. On the other hand, the specimens compacted using Gyratory compactor were subjected to 125 number of gyrations (Nd) to prepare 150 mm (6 in.) diameter specimen for ITFT. The results of volumetric and performance parameters are presented in ensuing sub-sections.

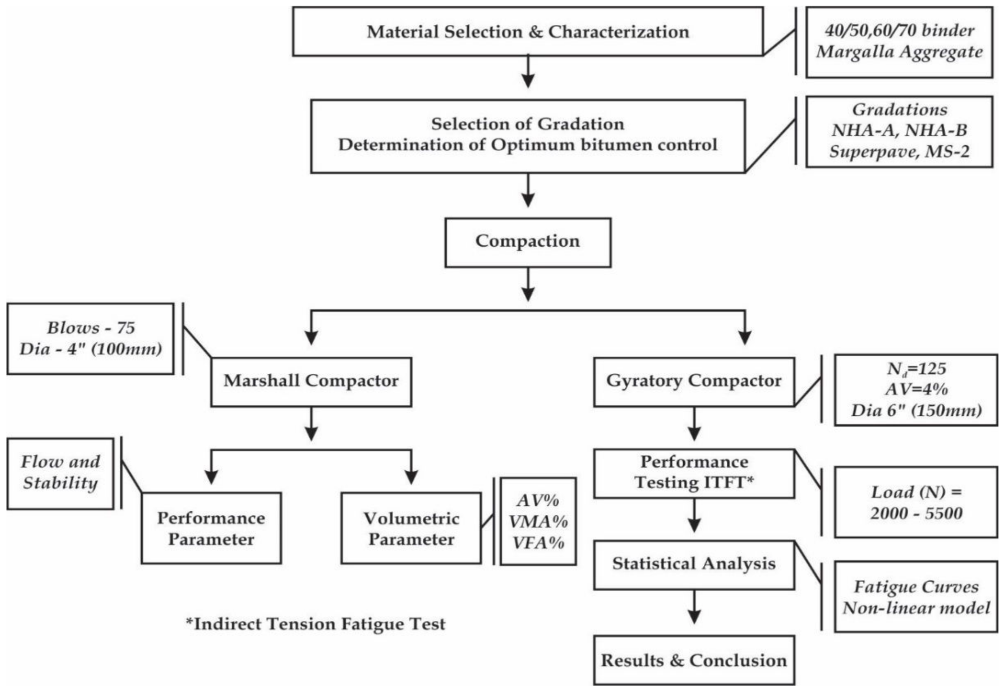


Figure 8.1: Research Methodology

8.3.1 Material Selection

The study presents a complete characterisation of aggregate and bitumen, which is a precursor to performance testing and determination of optimum bitumen content (OBC). The aggregate test results are presented in Table 8.2. The gradations adopted for wearing course in this study are same as used in our preceding studies [(Y. Ali, et al., 2015), (Y. Ali et al. 2016)], which include two local gradations adopted from National Highway Authority of Pakistan, (NHA)'s Class A and B (NHA, 1998) and two internationally practiced gradations, e.g., Superpave mix and asphalt institute manual series (MS)-2 gradation (Asphalt Institute, 1997). The gradations used in this study are presented in Figure 8.2.

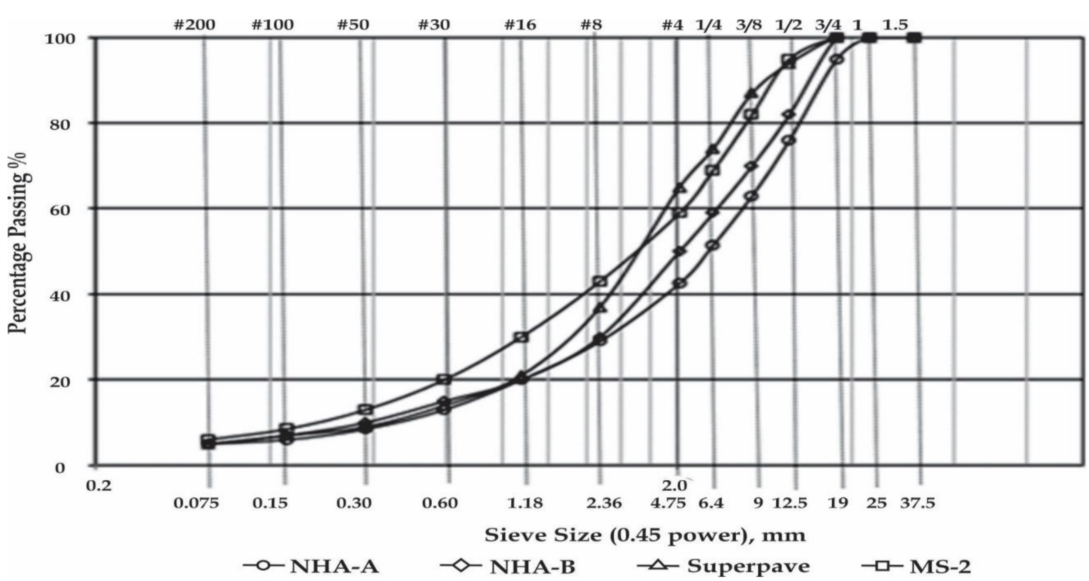


Figure 8.2: Gradation Chart for Wearing Course Mixtures

The rheological properties of asphalt binders largely influence the performance of AC mixtures. There are a variety of tests including consistency and performance tests, which need to be performed before the bitumen is used for performance testing. The performance tests include Penetration test, Flash and Fire point, Softening point, Ductility test, Rotational Viscosity (RV), and Bending Beam Rheometer (BBR); and a summary of these tests is shown in Table 8.3. The penetration grade bitumen 40/50 (i.e., low penetration grade) and 60/70 (i.e., high penetration grade) used in this research are characterised as performance grading of PG 64-16 and PG 58-22, respectively. Similar categorisation is reported in the literature [(Y. Ali et al. 2017), (M. Junaid et al. 2017)].

8.3.2 Optimum Bitumen Content (OBC)

Marshall Mix design method in accordance with ASTM D 6926 was followed to determine the correct aggregate blend along with a proper amount of bitumen to be used. The target air voids were set up to 4 ± 0.5% and volumetric parameters were found in the recommended allowable ranges. The results for the final OBC along with other volumetric parameters are shown in Table 8.4.

8.3.3 Preparation of Specimen

The specimens for ITFT were compacted using Superpave Gyratory compactor (SGC). The cylindrical specimens were subjected to a haversine loading at a loading frequency of 2 Hz and at a loading time period of 100 ms to simulate the movement of vehicles. The standards [(ABF BD, 1997), (B.S. Institution, 2004)] report that a repeated haversine loading should be applied with 0.1 s (100 ms) loading time followed by a rest period of 0.4 s (400 ms). The total loading cycle of 0.5 s (500 ms) corresponds to a frequency of 2 Hz. Triplicate specimens of each mix were prepared and tested for ITFT.

Table 8.2: Summary of Aggregate Test Results.

Summary of Aggregate Test Result:

Standard Test Method		ASTM-C127 & C128				C131		C-88	C-142	D-4791	D-5821	D-4791	D-2419	C-1252	C-29	
Quarry Source	Size of Aggregate (mm)	Specific Gravities				Wear by LA Abrasion %		Soundness%	Clay Lumps%	Elongation %	Fractured Face%	Flakiness%	Sand Equivalent %	Void Content %	Unit Weight Kg/m	
		Bulk	SSD	APP.	Abs.%	Class -A	Class-B								Loose	Rodded
Line Stone Aggregate	20-30	2.64	2.66	2.69	1.14	27.4	-	-	0.48	3.58	100% Crushed	13	76.6	-	1543	1625
	10-20	2.63	2.66	2.7												
	10-5	2.62	2.65	2.71												
	5-0	2.59	2.59	2.69	2.27			-	2.87					39.3	1570	1780
Remarks						40% Max	30%Max	12%Max	1%Max	10% Max	90% Min	10% Max	45%Min	45% Max		

Table 8.3: Summary of Bitumen Test Results.

	Penetration Test		Flash & Fire Point Test		Softening Point Test		Ductility Test		RV Test (Pa.s)			BBR Test (MPa)		
Standard	ASTM D5/ AASHTO T49		ASTM D92/ AASHTO T49		ASTM D36		ASTM D113		AASTO 316			ASTM D6648/ AASHTO T313		
Bitumen Pen Grade	60/70	40/50	60/70	40/50	60/70	40/50	60/70	40/50	Temp.	60/70	40/50	Temp.	60/70	40/50
Results	63	43	328°C & 362°C	335°C & 359°C	48° C	52° C	Greater than 100 mm		135°C	0.225	0.486	-12 °C	232.506	427.410
									160°C	0.073	0.083	-6 °C	86.055	120.65
									-	-	-	0 °C	N/A	58.643

Table 8.4: Mixtures Volumetric and OBC.

Mix Type	Bitumen Type	OBC (%)	Stability (kN)	Flow (mm)	Air Voids (%)	VMA (%)	VFB (%)
NHA – A	Pen grade 60/70	4.0	13.36	12.04	4.0	12.30	66.07
NHA – B		4.1	12.66	12.65	4.0	12.95	65.16
Superpave		5.0	13.96	13.55	4.0	14.70	69.18
MS – 2		4.8	15.24	13.12	4.0	14.52	67.73
NHA – A	Pen grade 40/50	3.9	14.67	12.39	4.0	13.39	70.16
NHA – B		4.4	12.25	8.72	4.0	13.48	63.73
Superpave		5.6	13.56	9.44	4.0	15.10	71.06
MS – 2		4.7	15.55	10.72	4.0	13.62	64.03

8.3.4 Indirect Tensile Fatigue Test

ITFT subjects AC materials to a repeated loading along the vertical diameter of the cylinder-shaped specimens using the Universal Testing Machine (UTM). The jig assembly used for the performance test is shown in Figure 8.3. The vertical loading applied to the specimen produces a vertical compressive stress and a horizontal tensile stress on the diameter of the specimens. Each gradation was subjected to a series of eight load stress levels (2000 N, 2500 N, 3000 N, 3500 N, 4000 N, 4500 N, 5000 N, and 5500 N) at a temperature of 25°C. The stress levels were selected based on the trial and error until the pre-specified criterion of the load repetition for each mix is achieved, which is in consonance with the European standard EN 12697-24 (B.S. Institution, 2004) simulating the loading repetition in the field.

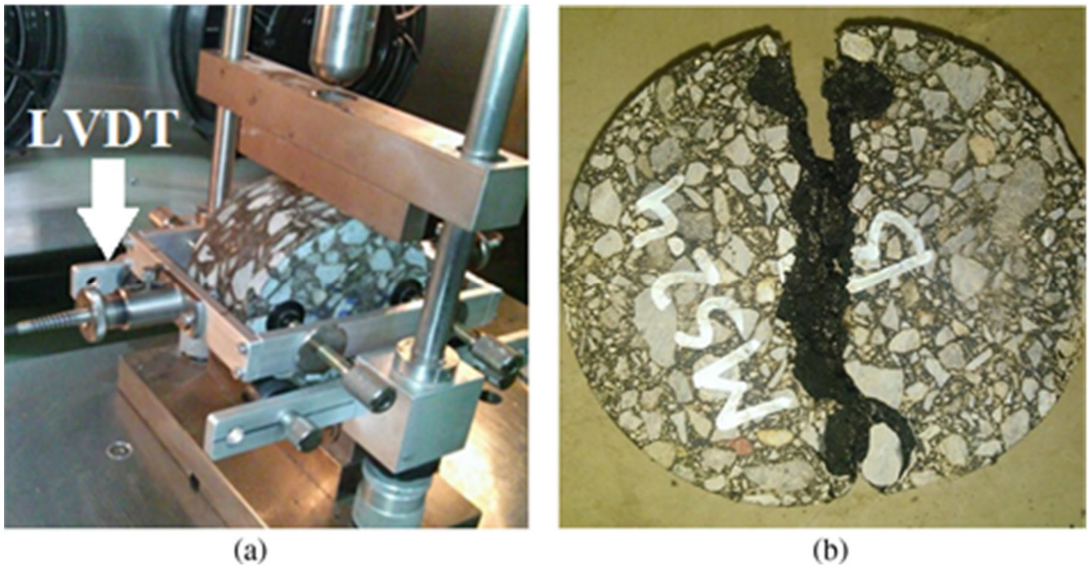


Figure 8.3: (a) Indirect Tensile Jig Assembly (b) Sample After Fatigue Failure

8.4 Results and Analysis

The ITFT software is programmed to acquire about 120 data points from each of the transducers for each loading cycle. The linear variable differential transducers (LVDTs) are attached to the specimen for 150 number of cycles to determine the horizontal deformation values, however, deformations in the specimen are stabilised normally after 60 load applications. In this study, the criterion defined for fatigue failure was the actual breaking of the specimen. Similar criterion has been reported in the literature (Asphalt Institute, 1981). The specimens were closely monitored by the analyst throughout the loading process, and the test was terminated when the cracks manifested on the surface of the specimen. The deformations determined against the number of cycles are plotted as shown in Figure 8.4, and the equation derived from the deformation curve is used to interpolate the horizontal deformations for the required cycles as shown in Table 8.5. The difference between the average of horizontal deformations of 5 load applications from 98 to 102 and the average of the minimum horizontal deformations of 5 load applications from 60 to 64 are used to calculate the initial strain using Equation 8-1 Note that the horizontal deformation (DH) is the difference of average horizontal deformations; X is the diameter, taken as 100 mm; and the Poisson ratio (υ) is assumed as 0.35. The results of the average number of cycles to failure and the average initial strain for the triplicate specimens tested for each loading is shown in Table 8.6.

$$\varepsilon = \left(\frac{2\times\Delta H}{\Omega}\right) \times \left(\frac{1+3\upsilon}{4+(\pi\times\upsilon)-\pi}\right) \qquad 8.1$$

Figure 8.5 represents the number of cycles to failure per strain per 1000 cycles. The number of cycles to failure mentioned on Y-axis (Fig. 8.5) is calculated in three steps that are described as:

Step-1: Mean of # of the cycle to the failure is calculated.

Step-2: Delta (Δ) is computed as the ratio of two differences: the difference between high and low # of cycles to the failure divided by the difference in the corresponding load (N). The corresponding load is then divided by 1000.

Step-3: Percentage of the delta is calculated and is expressed as "number of cycles to the failure/1000 N load (%)". The parameter '# of cycles to the failure/1000 N load (%)' indicates the number of cycles a mixture can resist before failing if 1000 N load is applied to it. A higher value of this ratio indicates that a mix is more fatigue resistant as it takes more number of cycles to reach the same difference in strain and vice versa. Moreover, the term strain here also refers to the ratio of two differences: the difference of two initial strain divided by the corresponding load, and the corresponding load is further divided by 1000 cycles.

For the sake of clarity and easy understanding, "# of cycles to the failure/1000 N load (%)" is calculated for the NHA-A gradation with the bitumen grade of 40/50 as follows:

Step-1: Mean of # of cycles to the failure for the NHA-A (40/50 binder grade):

$$= \frac{\sum_{i=1}^{n} \#\ of\ cycles\ to\ the\ failure}{n} = 3528$$

Step-2: Computation of Delta (Δ)

Difference between high and low cycle to failure = 9844 – 904 = 8940

Corresponding difference between load = 5500 – 3000 = 2500

Divide the corresponding load by 1000 = 2500/1000 = 2.5

$$\Delta = \frac{8940}{2.5} = 3576$$

Step-3: Percentage of the delta is calculated expressed as "# of cycles to the failure/1000 N load (%)".

$$Number\ of\ cycles\ to\ the\ failure/1000\ N\ load\ (\%) = \frac{3576}{100} = 35.76$$

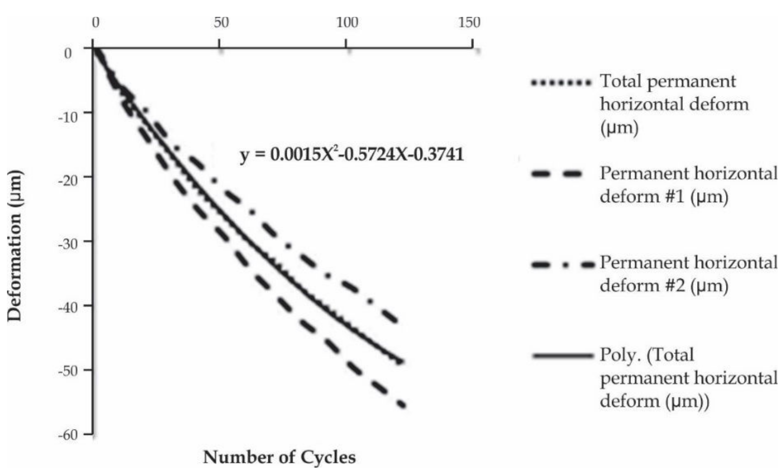


Figure 8.4: Sample Deformation Plot for Interpolation of Horizontal Deformations

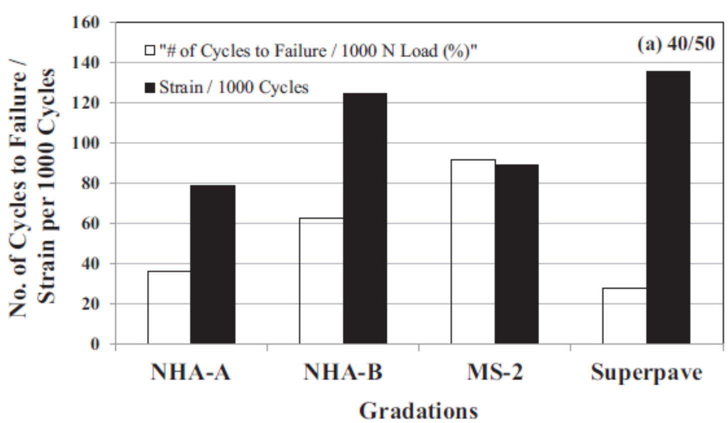
These ratios are calculated to normalise the results in order to evaluate the fatigue resistance of the AC mixtures because the number of cycles to failure and the strain attained by various mixtures at every stress are different, thus, the slope parameter i.e., the ratio of "# of cycles to the failure" and the initial strain were developed to rank the mixtures in this study. From Fig. 8.5, it can be inferred that the maximum strain/1000 cycles were found for the Superpave mix with the least number of cycles for the 40/50 pen grade bitumen, whereas the maximum cycles to failure were achieved by the MS-2 mix. In case of the 60/70 bitumen, it is noted that the MS-2 mix attained the highest cycles to the failure whereas the Superpave has the lowest cycles to the failure under the same testing conditions.

Table 8.5: Sample Horizontal Deformations

Cycle	Total permanent horizontal deform. (mm)	Total permanent horizontal deform. (µm)	Average permanent horizontal deform. (mm)
60	0.029	29.318	0.030
61	0.029	29.709	
62	0.030	30.096	
63	0.031	30.481	
64	0.031	30.863	0.042
98	0.042	42.063	
99	0.042	42.340	
100	0.043	42.614	
101	0.043	42.885	
102	0.043	43.152	
Deformation (mm) =			0.013
Initial Strain =			260.357

Table 8.6: Initial Strain & Cycles to Failure.

Aggregate Gradation Load (N)	NHA-A Cycles to Failure	NHA-A Initial Strain (µε)	NHA-B Cycles to Failure	NHA-B Initial Strain (µε)	MS-2 Cycles to Failure	MS-2 Initial Strain (µε)	Superpave Cycles to Failure	Superpave Initial Strain (µε)
Bitumen Grade	*40/50*							
5500	904	283	–	–	1048	350	–	–
5000	1181	289	898	395	1611	431	–	–
4500	2381	268	1241	680	2548	259	774	591
4000	2876	159	1248	320	3588	203	941	483
3500	3981	131	4138	168	6734	158	1368	375
3000	9844	86	3208	192	–	–	2161	392
2500	–	–	15,676	84	28,451	83	2821	334
2000	–	–	–	–	–	–	7636	252
Bitumen Grade	*60/70*							
5500	–	–	–	–	–	–	–	–
5000	–	–	–	–	–	–	–	–
4500	451	432	–	–	524	702	–	–
4000	751	396	341	1113	861	414	274	588
3500	1341	321	398	946	1417	428	384	503
3000	1998	283	1178	327	3954	253	676	508
2500	4716	211	1534	312	6171	217	3191	309
2000	–	–	5061	109	23,121	143	2578	381



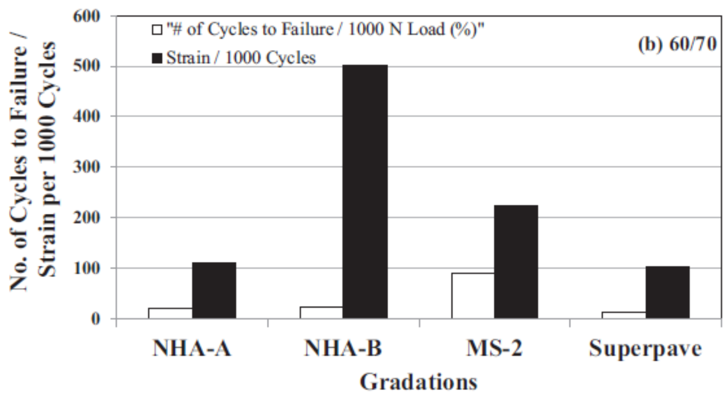


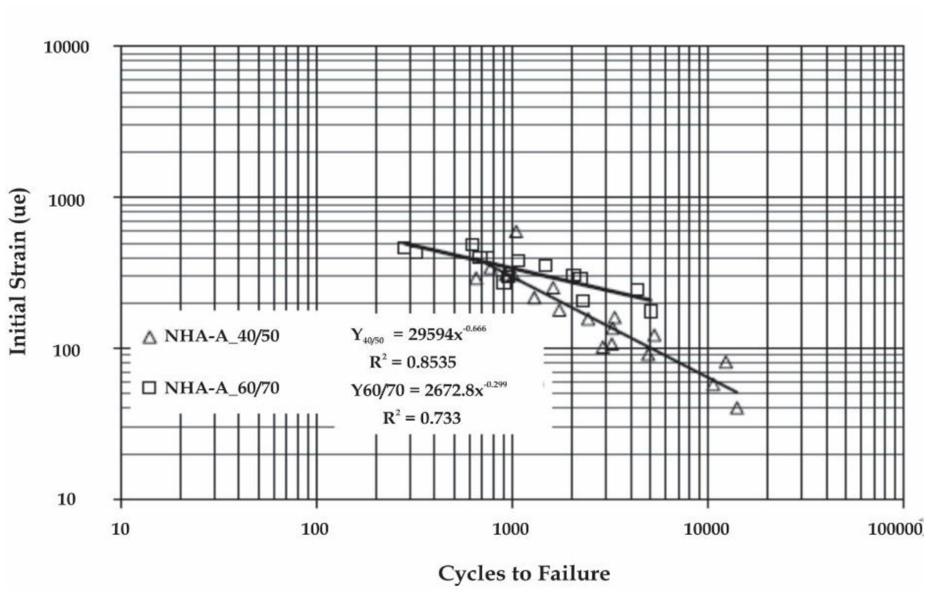
Figure 8.5: Number of Cycles to Failure of AC Mixtures

8.4.1 Fatigue Curves

The performance evaluation of AC mixtures was carried out by developing fatigue curves. The fatigue criterion for an individual bituminous material is determined from the tested specimens. The least-squared regression relationship is fitted to the data of the logarithm of the initial strain (i.e., independent variable) and the data of the logarithm of the fracture life (i.e., dependent variable) [(G.G. Al-Khateeb and K.A. Ghuzlan 2014), (H. Salama and K. Chatti, 2011)]. The laboratory data were utilised to model log of number of cycles to failure and the log of initial strain values using the Equation 8-2 and Equation 8-3. Several models have been proposed using the laboratory data by Asphalt Institute (1983) and Shell (1978), to predict the fatigue lives of pavements using the functional form expressed in Equation 8-2. These models are also used to develop the shift factors based on the field observations to calibrate the laboratory results in order to provide a reasonable estimate of the in-service life of a pavement.

$$N_f = a(\frac{1}{\varepsilon_o})^b \quad \text{8-1}$$

$$logN_f = \log(a) + blog(\varepsilon_o) \quad \text{8-2}$$



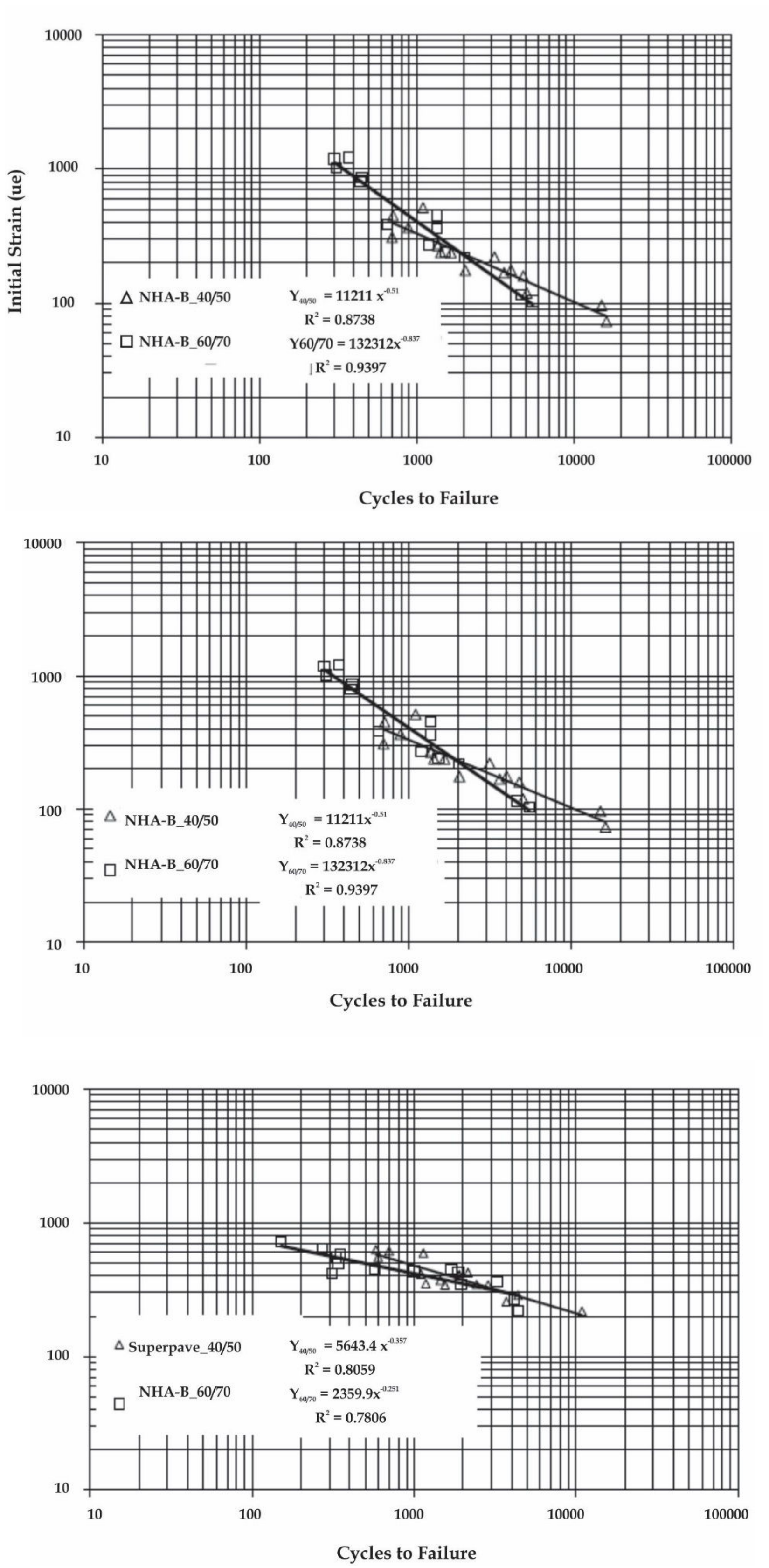


Figure 8.6: Relationship Between Cycles to Failure and Initial Strain

N_f is number of cycles to the fatigue failure, ε_o is the initial strain in μm and a, b are regression coefficients. Note that the model 8.3 is the same as the model 8.2, and the only difference is that it has been transformed to linear specification. The slopes of all the curves are negative that represents the inverse relation of the number of cycles to failure to the initial strain values. The fatigue curves, model equations (y represents initial strain whereas x shows cycles to failure), and coefficient of determination (R^2) are shown in Fig. 8.6 for both the types of bitumen. It can be inferred that the 40/50 bitumen accumulates less initial strain as compared to the 60/70 bitumen and a possible reason could be higher stiffness of a binder that can sustain loading without further deformation (ε), and such trend is observed in all the mixtures for both type of bitumen. Also, the coefficient of determination (R^2) of the 40/50 mixtures is higher, suggesting that more variation in the initial strain is being explained by the change in number of cycles.

8.4.2 Statistical Modelling

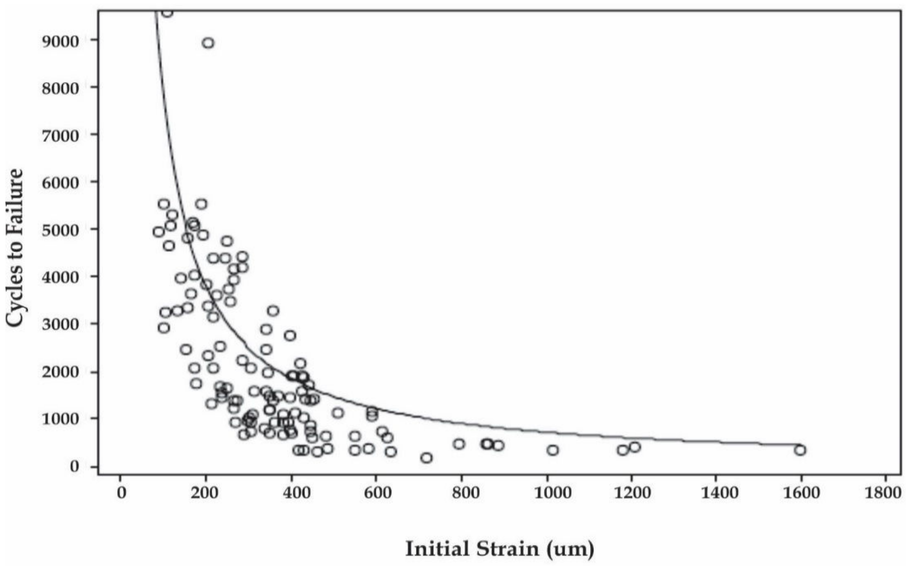
ITFT test results were further employed to develop a statistical model, for which a non-linear model specification was found to be appropriate. Three different models were developed. The first model (Equation 8-4) is a power model to express the number of cycles as a function of the initial strain, while the second model is an intrinsically linear model (Equation 8-5) obtained by applying a transformation on the preceding model. The third model presented in Equation 8-6 is developed by combining dataset of all the gradations and mixtures to get additional variables such as bitumen viscosity, optimum bitumen content, and resilient modulus along with the initial strain. These variables were selected based on the potential theoretical or physical relationship with the number of cycles to failure. The OBC variable is a surrogate measure that captures the effect of mix volumetric properties in addition to mix type. The additional variability in the number of cycles to failure is captured by the addition of the aforementioned variables.

$$N_f = 959994.3(\varepsilon^{-1.044}) \tag{8-3}$$

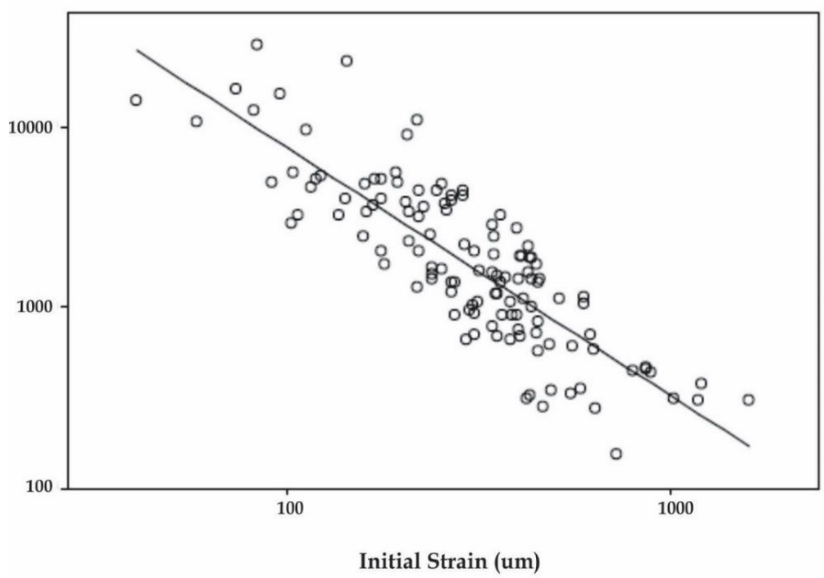
$$LogN_f = 6.629 - 1.374\log(\varepsilon) \tag{8-4}$$

$$N_f = 1.367 \times 10^{-8} \times \varepsilon^{-2.556} \times \eta^{-1.564} \times {v_B}^{9.154} \times E^{2.655} \tag{8-5}$$

Whereas, N_f = Number of cycles to the failure; ε = Initial strain, microstrain; η = Bitumen viscosity (40/50 = 0.486, 60/70 = 0.225); v_B= OBC, percentage; E = Resilient Modulus, MPa. The relationship between the number of cycles to failure and the initial strain is shown by Equation 8-4 with R^2 value of 0.49 (see Fig. 8.7a) that implies that about half of the variation in the response variable (the number of cycles to failure) is explained by the change in an independent variable (initial strain). In addition, the intrinsically linear model shows R^2 of 0.7 (Fig. 8.7b), which suggests that about 70% of the variation in the data is being captured by this relationship. Though the models presented in Fig. 8.7 showed a significant relation, the prediction capability of these models was tested using Mean Absolute Percent Error (MAPE), and results indicate that the power and transformed (intrinsically linear) models have an error of 0.91 and 0.45, respectively. This shows the developed models do not carry any prediction power.



(a) Power model



(b) Intrinsically linear model

Figure 8.7: Model Plots

Table 8.7: Non-linear Model Summary

Parameter	Estimate	S.E	t-stat	p-value	R^2	95% Confidence interval	
						Lower Bound	Upper Bound
Intercept	1.38	0.2	6.90	<0.001	0.86	1.78	0.98
Initial Strain	-2.55	0.11	-24.3	<0.001		-2.76	-2.34
Viscosity	-1.56	0.24	-6.41	<0.001		-2.04	-1.09
OBC	9.15	0.74	12.4	<0.001		7.69	10.62
MR*	2.66	0.82	3.25	0.01		1.04	4.27

***MR** = Resilient modulus

To overcome the prediction issue, a few variables, e.g., bitumen viscosity, optimum bitumen content, and the resilient modulus were added in the model, which not only improve the R^2 value but increase the model prediction as well. A non-linear formulation similar to (Y. Ali, et al., 2015) has been adopted. The model summary is presented in Table 8.7. It can be seen that R^2 of the non-linear model has increased to 0.86. All the parameter estimates are also with the expected sign. Moreover, the developed model was also tested for multi collinearity using Variance Inflation Factor (VIF) and no variable was found to be correlated with any other variable since VIF of different variables was less than 5. The nonlinear regression model was validated by calculating the MAPE, which is 0.15, suggesting that developed model under/over predicts the given laboratory data by 15%. Note that results obtained from this model were found consistent with the fatigue curves presented in Fig. 8.6.

8.4.3 Comparison With the Other Studies

This section compares the findings of the current study with some representative studies from the literature. We compared the fatigue results obtained from earlier study (Y. Ali et al., 2015) and the developed model is compared to (J. Harvey, et al., 1996). These studies were selected because the testing conditions in (Y. Ali, et al., 2015) are similar to the current study's conditions while modelling framework in (J. Harvey, et al., 1996) is closely related to the current study.

Y. Ali et al. (2015) determined the fatigue behavior of four asphalt wearing course mixtures (same mixtures used in the current study) and reported that the Superpave mix has better resistance to fatigue than the other tested mixtures while in the current study, the MS-2 mix is found to be relatively better in fatigue than the other tested mixtures. The difference in the results could be because of the empirical relation used in the (Y. Ali, et al., 2015) that is based on the dynamic modulus and phase angle. Y. Ali et al. (2015) also reported that phase angle is highly sensitive to the bitumen content and in such conditions, fatigue performance tests are required to verify the relationship and to characterize the asphalt mixtures. Harvey and Tsai (1996), proposed a transformed model expressing fatigue life as a function of the initial strain and mix volumetric, and similarly, this study models fatigue behavior as a function of the initial strain, the binder viscosity, the optimum bitumen content (OBC), and the resilient modulus. The two parameters in the current study – initial strain and volumetric parameter (OBC) are similar to Harvey and Tsai (1996)' study. The stiffness parameter in this study's model is also reported in the literature (F. Zhou et al. 2007).

8.5 Conclusions, Limitations, and Future Work

Under the given testing conditions and variables, following conclusions are drawn:

- The 40/50 bitumen provided as much as four times longer fatigue life than the 60/70 bitumen in the stress-controlled mode
- Fatigue curves were derived and characterized by slope and coefficient of determination, suggesting that the MS-2 mix performs relatively better among the tested mixtures for a given set of bitumen grades
- A non-linear functional formulation was adopted to express fatigue resistance as a function of the initial strain, the viscosity of bitumen, the percentage of optimum bitumen content and the resilient modulus

- The developed model performed reasonably well in predicting the fatigue life of the tested mixtures

This study would serve as the basis for selection of AC mixtures depending upon the fatigue life and facilitates the implementation of M-E design procedure in Pakistan. The findings of this study are limited to the scope of this study, however, the testing framework proposed in this study can be utilized in any other part of the world. In addition, the developed model can also be recalibrated to different datasets to describe the fatigue behavior. It is worth mentioning here that this research is limited to the specific gradations, type of aggregate, and bitumen. The loading frequency of the testing regime was 2 Hz with a loading time period of 100 ms. Also, each gradation was subjected to eight different load levels at single temperature of 25°C. However, future studies can perform fatigue testing at varying testing temperatures and loading frequencies. This study considers only conventional mixtures; however, it would be interesting to investigate the performances of AC mixtures containing additive or modifier that can limit the fatigue. Future studies can use the developed parameter to rank the performance of different modifiers.

9 Durability/Moisture Susceptibility

9.1 Introduction

Durability is a significant contributing factor in construction of healthy roads and may be defined as the ability of the pavement materials (bitumen, coarse and fine aggregates) to resist the effects of external conditions (i.e., water/moisture, ageing and temperature variations) while considering traffic loading and keeping deterioration within specified limits for the period of design life of the pavement. Many road authorities around the world experience premature pavement failures since performance tests defining pavement distresses due to moisture damage and or age hardening are utilized only to rank materials, but the damage mechanisms are not clearly understood. In Pakistan, premature failures due to durability aspects are reported largely on national and provincial road network where maintenance needs have escalated to a considerable magnitude. This is largely due to extreme weather effects with high rainfall intensity and prolonged snowy condition in northern regions.

This chapter describes limited laboratory testing on selected asphalt concrete wearing and base course mixtures with a view to evaluating their susceptibility mainly towards moisture damage and durability in general. These tests include Hamburg Wheel Tracking, Modified Lottman, Loss of Marshall Stability, and Indirect Tensile Strength tests. Bitumen from two refineries (i.e., NRL and ARL) with 60/70 & 40/50 penetration grade has been used. Much of the aggregates used are Ubhan Shah while limited numbers of tests have also been carried out on Margalla aggregates.

9.2 Factors Affecting Durability

Durability of bituminous mixtures is affected by a number of factors, but it is generally accepted that the two main factors affecting durability are, ageing of bitumen and moisture damage. Bitumen becomes stiffer with age i.e., its viscosity increases, primarily due to oxidation which occurs initially at the batching plant (i.e., mixing phase). Once laid at site the ageing phenomenon is slow but more robust due to direct exposure to elements (i.e., oxygen, heat-cold cycles and moisture). This process is referred to as oxidative ageing and the bitumen or bituminous mixtures) are prone to age hardening. Ageing is acceptable to a moderate level, but significant ageing can result in embrittlement of the bitumen, which consequently affects the adhesion characteristics of the bitumen and usually results in reduced cracking resistance of the mixture. Thus, age hardening of bitumen can cause premature failure of bituminous mixtures. Factors affecting age hardening generally include oxidation, volatilization and polymerization.

Water also damages the structural integrity of a bituminous mixture. Water damage is usually caused when cohesion is lost in the mixture or when adhesion is reduced between the bitumen and aggregate. As a consequence, the ability of a bituminous mixture to resist the stresses and strains under the traffic loading weakens. Moreover, water ingress and or prolong moisture within the asphalt concrete or on surface causes stripping i.e., physical separation of the aggregate from the bitumen. An outline of the factors which affect the durability of bituminous mixtures is given below particularly, the mechanisms and consequences of age hardening and water sensitivity.

9.2.1 Factors Affecting Ageing

Bitumen is composed of complex organic molecules that further differ in their composition, due to different crude oil sources. However significant advances towards better understanding of the mechanisms of age hardening have been provided by many researchers after investigating age hardening of bitumen and bituminous mixtures. The major contributing factors are exposure to air, heat/temperature, and ultraviolet rays from sun.

9.2.2 Mechanism of Age Hardening

According to (Petersen, 1984), Durability is determined by the chemical composition of the bitumen, which directly relates to its physical properties. Therefore, an understanding of the chemical factors affecting physical properties of bitumen which directly affects durability of asphalt concrete mixtures is important. Following three composition-related factors are identified that could cause hardening of bitumen in pavements:

- Loss of the oily components mainly due to volatilization or absorption by porous aggregates;
- Chemical composition changes from reaction with atmospheric oxygen;
- Thixotropic effects production due to molecular structuring (steric hardening).

The reaction with atmospheric oxygen is identified as probably being the major cause of age hardening. Rapid and irreversible oxidation occurs in pavements where bitumen, existing in thin films is exposed to atmospheric oxygen. This results in the formation of polar bonds due to powerful interacting oxygen; containing chemical functional groups, which greatly increase viscosity and therefore alter complex flow properties i.e., Rheology of bitumen. This phenomenon often leads to embrittlement of the bitumen resulting in different pavement distresses.

9.2.3 Consequences of Age Hardening

Brittle bitumen is usually the outcome of severe ageing, resulting into lesser flow abilities of the bitumen which causes different types of cracking. Cracking is generally of three types i.e., fatigue, thermal, or reflective cracking. The causes of these general three types of cracking are given below

- Fatigue cracking: It is the result of an accumulation of damage, usually arising from repeated stresses (i.e., traffic loading), eventually leading to fracture;
- Thermal cracking: It is the result of thermally induced tensile stresses that exceed the tensile strength of the bitumen. It can occur as a result of the mixture temperature falling below some limiting temperature or as the result of an accumulation of permanent tensile strain arising from repeated thermal stresses;

- Reflective cracking: It generally occurs as a result of stresses developed in the overlay due to traffic loading through differential movement between underlying adjacent cracked portions of asphalt concrete within the existing roadway.

Age hardening reduces the performance and service life of a pavement via embrittlement of the bitumen. It means that age hardened bitumen; under the influence of external loading has a reduced ability to flow, because of increased stiffness.

9.2.4 Factors Affecting Moisture Damage

It has long been recognized that moisture severely damages the strength of bituminous mixtures (Kennedy, 1985). Although many factors contribute to the degradation of bituminous mixtures, moisture appears to play a major role. In general, water can reduce the stiffness or strength of a bitumen-aggregate matrix or cause the bond between bitumen and aggregate to fail, both potentially resulting in significant distress to the bituminous pavement.

9.2.5 Mechanism of Moisture Damage

It is a general consensus that moisture can degrade the structural integrity of bituminous mixtures in the following two ways;

- By causing a reduction in the stiffness of the mixture and cohesive strength, characterized by softening of the mixture;
- By causing failure of the bond (adhesion) between aggregate and bitumen, referred to as stripping (Terrel and Shute, 1989).

However, Lottman in 1982 described the mechanisms of moisture damage in more detailed manner, which included the damage due to repetitions of traffic loading, de-bonding of aggregate and bitumen at variant temperatures, the effect of water vapours on bitumen mastic and finally the clay or fines susceptible to water. Amongst all stripping is considered as most damaging mechanism (Peterson et al., 1982). Recent researches have shown that cohesive failure within the aggregate or the bitumen, or adhesion failures at the bitumen-aggregate interface are major causes of stripping (Jamieson, 1993).

In 1993, Curtis et al. suggested that stripping initiates when water penetrates the aggregate bitumen bond causing pavement distress in following different modes:

- Breaks the bond at the location of the interface;
- Causing the bitumen to fail due to removal of soluble components;
- Cohesive failure within aggregate;
- Phase separation of components when the presence of water (due to hydrogen bonding) accelerates the solubility of polar components of the bitumen.

Water intrusion can occur through cracks in the bitumen or by diffusion through the bitumen film, possibly removing soluble components of the bitumen in the process. Failure can be interfacial or cohesive and can occur in both the bitumen and aggregate.

9.2.6 Consequences of Moisture Damage

Moisture damage happens in various forms and levels of severity. As discussed earlier, the principal consequence of moisture damage is stripping. Stripping is often initially manifested as flushing or bleeding where the bitumen has migrated to the surface of the bituminous layer. This bitumen migration results in an unstable matrix in the lower portions of the bituminous layer which can lead to rutting or shoving as well as the development of potholes and cracking under the action of traffic loading. Subsequent water intrusion into these localized water damaged areas, further degrades the integrity of the pavement layer, and also the underlying layers which, if not repaired, can lead to substantial localized failure of the pavement structure. Raveling can also be caused as a result of stripping phenomenon.

Reduction of strength or stiffness in the bituminous layer is the other major consequence of moisture damage that decreases the load spreading capabilities of the pavement. A pavement, with reduced stiffness; due to water damage, is prone to rutting as a result of increased stresses and strains in the underlying layers. Strength loss in the aggregate-bitumen matrix may also initiate (Plancher et al., 1982).

9.3 Experimental Design

The research study presented here focuses on the evaluation of durability with regard to ageing and moisture susceptibility of limited asphalt concrete mixtures. In this research study material specimens were subjected to Indirect Tensile Strength Test (ITST) to study the effect of age hardening and moisture conditioning. Hamburg Wheel Tracking (HWT) test was performed to evaluate rutting, while Modified Lottman Test (MLT) and Loss of Marshall Stability (LOS) tests were performed to evaluate the moisture susceptibility of asphalt concrete mixtures pertaining to various aggregate gradations. Four gradations, i.e., NHA-A, NHA-B, SP-2 and Asphalt Institute MS-2 were chosen to study the effect of moisture susceptibility.

9.3.1 Sample Preparation for Performance Test

Marshall Mix Design method was used to determine optimum bitumen content (OBC). To achieve the desired air voids, cylindrical specimens were compacted using Superpave Gyratory compactor as per ASTM D6925-09. Specimens were compacted at a constant vertical pressure of 600 ±18kPa using 30.0 ± 0.5 revolutions per minute with an internal angle of 1.16 ± 0.02 degrees maintained in the Superpave gyratory compactor. For aged specimen preparation, blend of aggregate and bitumen was brought to the required mixing temperature and then left in the oven for 2 hours for short term ageing. Once the ageing was complete, the blended mixture was compacted in Superpave gyratory compactor to achieve desired air void content.

The laboratory samples prepared for the indirect tensile strength test were 2 inch high with 4 inch diameter for wearing course and 3 inch high with 6 inch diameter for base course gradations respectively. Slab samples prepared for Hamburg Wheel Tracker were prepared and compacted in roller compactor as per AASHTO T 324-04 while core (cylindrical) samples were prepared through Superpave gyratory compactor. Compacted slab specimens are generally 320 mm (12.6 inch) long, 260 mm (10.24 inch) wide and 38 mm (1.5 in) to 100 mm (4 inch) thick. While the cylindrical specimens prepared in Superpave gyratory compactor were 38 mm (1.5 inch) to 100 mm (4 inch) thick with a 150 mm (6 inch) diameter. The asphalt concrete samples were compacted to 7±2% air voids using Superpave gyratory compactor or roller compactor.

The aim of the Modified Lottman Test is to evaluate moisture sensitivity of the asphalt concrete mixtures. In this test, the specimens are placed in oven for curing at

60°C for 16 hours after mixing and left to cool at room temperature. The samples are then heated at 135°C for two hours, compacted to 7±1% air voids using SGC and stored at room temperature for 72 to 96 hours. A total of six samples were prepared using this method and divided into two subsets; three samples were conditioned and the remaining three were controlled (unconditioned). For Loss of Marshall Stability test, six samples were prepared using Marshall Mix Design method as per ASTM D-1559 for each of the four wearing course gradations.

9.3.2 Laboratory Testing

9.3.2.1 ITST for Moisture Conditioning

The ASTM D4867-96 standard was followed for moisture conditioning of the asphalt concrete mixtures and six (06) samples for each mix type were prepared. Three (03) samples were tested for indirect tensile strength in dry condition while the other three (03) were tested in similar manner after moisture conditioning. The resistance to moisture damage was evaluated in terms of tensile strength ratio, which is ratio of the indirect tensile strength after moisture conditioning to indirect tensile strength of samples in dry condition. Fabricated samples are shown in Figure 9.1 below:

Figure 9.1: Samples Prepared for Moisture Conditioning

The dry samples were directly tested for their indirect tensile strength while the other three samples were placed in a water bath for moisture conditioning at 60°C ± 1.0°C for 24 hrs as shown in Figure 9.2:

Figure 9.2: Moisture Conditioning of Samples in Water Bath

At the end of 24hrs conditioning period, the samples were placed in distilled water at 25°C ± 1°C for 1hr for temperature conditioning. Once the samples were conditioned, they were subjected to indirect tensile strength test. The tensile strength ratio was calculated using equation below:

$$\mathrm{TSR} = \left(\frac{S_{tm}}{S_{td}}\right) * 100 \qquad \text{9-1}$$

Where;

TSR = Computed Tensile Strength Ratio, %
S_{tm} = Average ITS of the moisture conditioned samples, kPa
S_{td} = average ITS of the dry samples, kPa

9.3.2.2 ITST for Age Hardening

Age hardening testing was performed to simulate various stages of binder ageing. Typically, there are two types of binder ageing, short term ageing that determines the ageing during construction (i.e., batch mixing) and is simulated in Rolling Thin Film Oven (RTFO) test; and the long term ageing which determines the ageing of pavement once laid in the field and is simulated in the Pressure Ageing Vessel (PAV) test. In RTFO test, binder in un-agitated form is heated until it begins to flow and the RTFO bottles are filled with 35 g ± 0.5 g of binder. The bottles are then placed in the carousel and rotated at 15 revolutions per minute for 85 minutes, at the end of which these are taken out and residue collected in a container. After the RTFO test, further ageing of the binder is carried out using PAV test for which pre-heated pans are used each containing 50g of aged binder. The pre-heated pans are placed in PAV and ageing of binder is performed at 100°C by sealing the vessel. Once the desired temperature is achieved, pressure of 300 psi is maintained for 20 hours. At the end of the ageing period, pans are taken out of the PAV and collected in container (binder depth not to exceed 40 mm) which is further placed in a vacuum oven at a temperature of 170°C for almost 30 minutes to remove any entrapped air from the binder. For each gradation and binder, nine (09) samples were prepared for indirect tensile strength tests; three of them were tested without ageing, three with RTFO aged binder and three with PAV aged binder.

9.3.2.3 Hamburg Wheel Tracking Test

Hamburg Wheel Tracking (HWT) device was developed in 1970 by Esso A.G. of Hamburg, Germany, (Romero and Stuart, 1998). HWT comprises a steel tyre instead of rubber tyre which has a diameter of 200 mm (8 inch) and 22.87 mm (0.90 inch) of width respectively. It measures the combined effect of rutting and moisture. Test is generally carried out at 40ºC and 50ºC temperatures.

HWT contains a water chamber for maintaining the required test temperature within a range of 25°C to 70°C. Specimen conditioning time is 30 minutes. LVDT (Linear variable differential transducer) measures the rut depth of compacted specimen in millimeters (mm) as the test progresses. The machine is capable of applying 158 lbs (705 N) force (contact stress of 0.73 MPa) with a tyre contact area of 970 mm^2. This contact pressure simulates the effect produced by a rear tyre of a two-axle truck. The contact stress is variable since the area increases with rut depth.

Aschenbrener (1995) specified the test criteria of 10,000 passes or upto 20 mm (0.79 inch) of rutting, whichever occurs first. Wheel moves with an average speed of 1.1 km per hour. The device operates at 50 ± 2 wheel passes per minute.

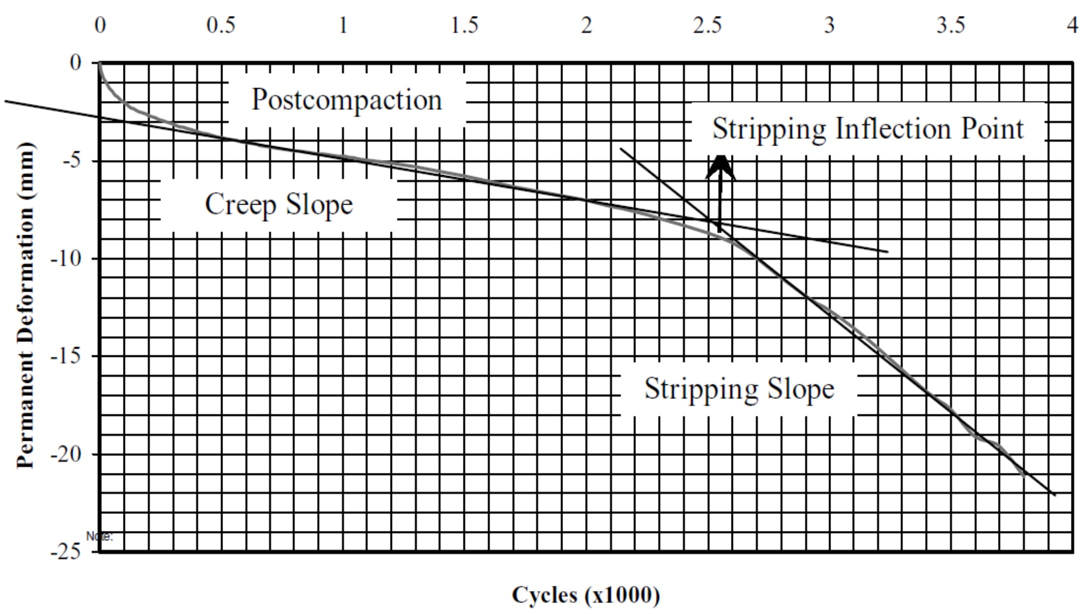


Figure 9.3: General Trends of Results from HWTD

HWT test results comprise post compaction consolidation i.e., initial densification, stripping slope, creep slope and stripping inflection point (SIP). Post-compaction consolidation is the deformation in millimeters at 1000 passes. It is assumed that the wheel is further densifying the mixture within the first 1000 passes. The creep slope is the inverse of the rate of deformation in deformation curve after post-compaction consolidation. It should be noted that the creep slope occurs before the stripping. In this portion of the curve, the deformation occurs due to plastic flow. The stripping slope is the relative measure of damage severity due to moisture. It is the inverse of the rate of deformation after the onset of stripping to the end of the test. The lower the inverse stripping slope, the more severe would be the moisture damage. Stripping inflection point is the representation of number of passes at the intersection of creep slope and stripping slope as shown in Figure 9.3. [Aschenbrener (1995), Miller (1995), and Mohammad et al., (2000)].

The results obtained from HWT are generally reported in terms of maximum rut depth (mm), creep slope, stripping slope and stripping inflection point. Equation 9-2 shows the formula used to evaluate the stripping inflection point:

$$Stripping\ inflection\ Point = \frac{Intercept\ (second\ portion) - Intercept\ (first\ portion)}{Slope(first\ portion) - Slope(second\ portion)} \qquad 9\text{-}2$$

9.3.2.4 Modified Lottman Test

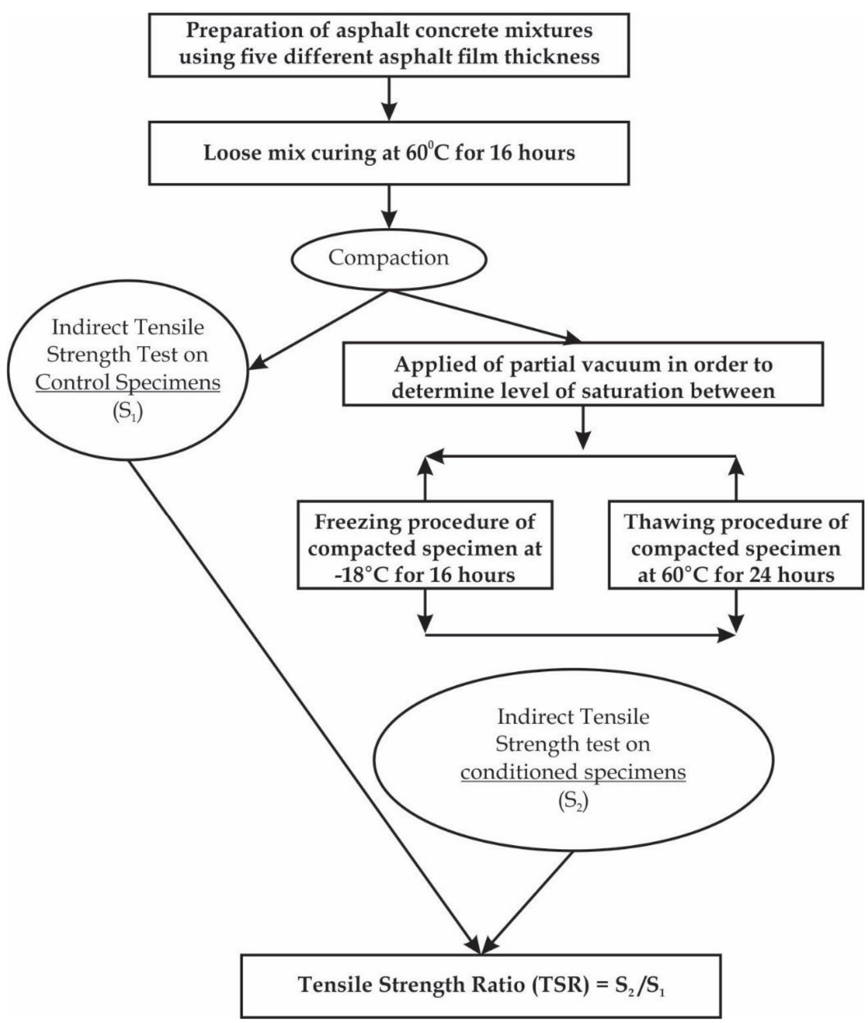


Figure 9.4: Flow Chart of Modified Lottman Test (AASHTO T 283)

The Modified Lottman Test is used to evaluate moisture sensitivity of the asphalt concrete mixtures. In this test, the specimens are placed in oven for curing at 60°C for 16 hours after mixing and left to cool at room temperature. The samples are then heated at 135°C for two hours, compacted to 7±1% air voids using Superpave gyratory compactor and stored at room temperature for 72 to 96 hours. A total of six samples are prepared using this method and divided into two subsets; three samples are conditioned and the remaining three are controlled (unconditioned). A vacuum saturation with water (55%–80% saturation level) is applied. Samples are then placed in a freezer at -18°C for 16hrs and then placed in a water bath at 60°C for 24hrs. After completing freeze and thaw cycles, all the six specimens are placed in water bath at 25°C for at least 2hrs to achieve temperature equilibrium. A constant load at the rate of 50mm/min is then applied for indirect tensile strength test (ITS). Tensile strength of the conditioned specimens is then compared to the unconditioned specimens to determine the tensile strength ratio (TSR). Tensile strength ratio (TSR) expresses the numerical index of resistance of hot mix asphalt concrete to the detrimental effect of water as the ratio of retained strength after moisture and freeze-thaw conditioning to that of the original strength (AASHTO T 283). If the loss of strength is not more than 20%, the specimen is considered as moisture resistant as specified in AASHTO T 283 (1993). However, some agencies accept TSR value up to 70%. The average of tensile strengths of three (03) samples was taken into account. A schematic diagram of the procedure followed is shown in Figure 9.4.

ITS is defined as the stress from a diametrical vertical force that a specimen can withstand and can be expressed by Equation 9-3.

$$S_t = \frac{2000\,P}{Dt\pi} \tag{9-3}$$

Where;

St = Tensile Strength, kPa
P = Maximum Load Carried by the Specimen, N
t = Thickness of Specimen, mm
D = Diameter of Specimen, mm

9.3.2.5 Loss of Marshal Stability

Six (06) samples were prepared using Marshall Mix Design method as per ASTM D-1559 for each of the four wearing courses gradations. Six samples were divided into two subsets of three. Three samples were kept at 60°C in water bath for 24 hours while the other three samples were kept at room temperature. All the samples were tested for stability to compare the stability loss due to moisture damage.

9.4 Experimental Results

9.4.1 Moisture Conditioning Test Results

To evaluate the effect of moisture, ASTM D-4867 was followed. The indirect tensile strength of samples in both the dry state and moisture conditioned state was determined from which tensile strength ratio (TSR) was calculated. The TSR values for both asphalt wearing and base course gradations are shown in Table 9.1 below:

Table 9.1: Moisture Conditioning Test Results

Bitumen	Moisture Conditioning	Asphalt Wearing Course				Asphalt Base Course	
		NHA–A	NHA–B	SP–2	MS–2	SP–2	DBM
NRL 40/50	1 Day	0.93	0.90	0.85	0.84	0.86	0.89
	14 Day	0.63	0.57	0.51	0.52	0.56	0.60
ARL 60/70	1 Day	0.89	0.87	0.85	0.82	0.83	0.86
	14 Day	0.60	0.55	0.48	0.53	0.51	0.56

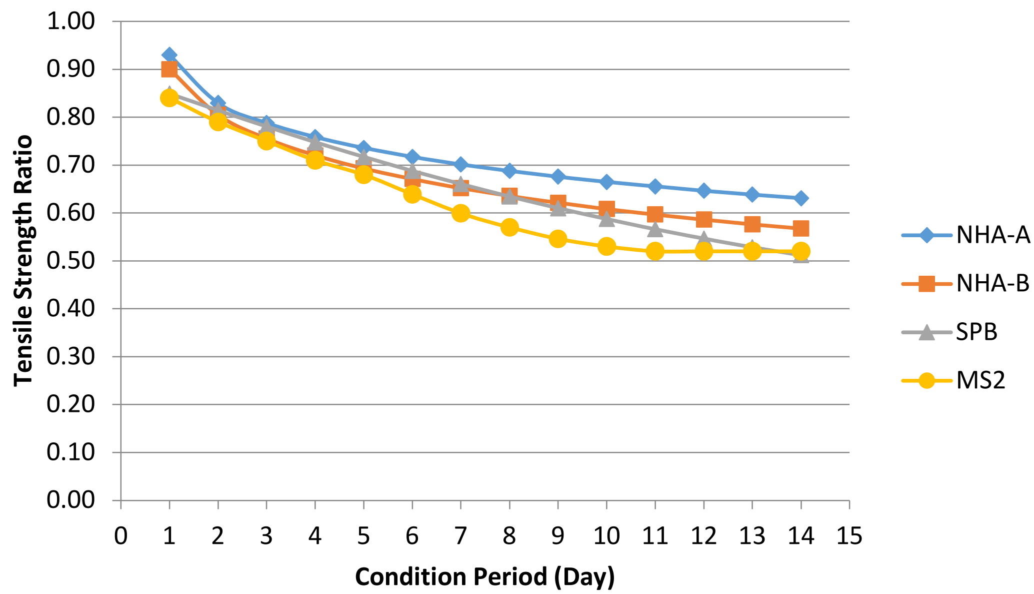


Figure 9.5: TSR Vs. Conditioning Day (Asphalt Wearing Course Gradations With NRL 40-50)

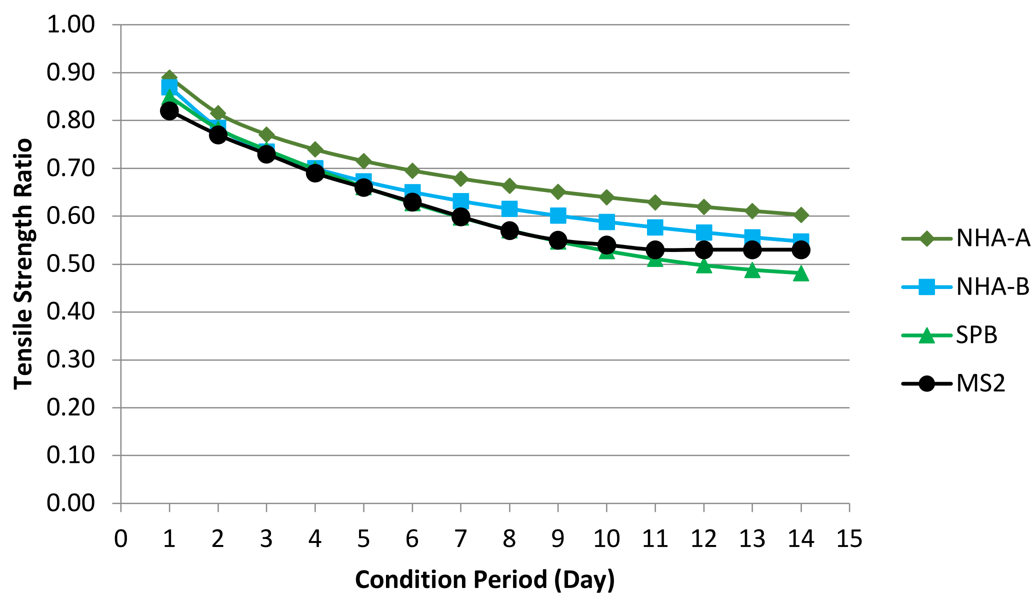


Figure 9.6: TSR Vs. Conditioning Day (Wearing Course Gradations With ARL 60-70)

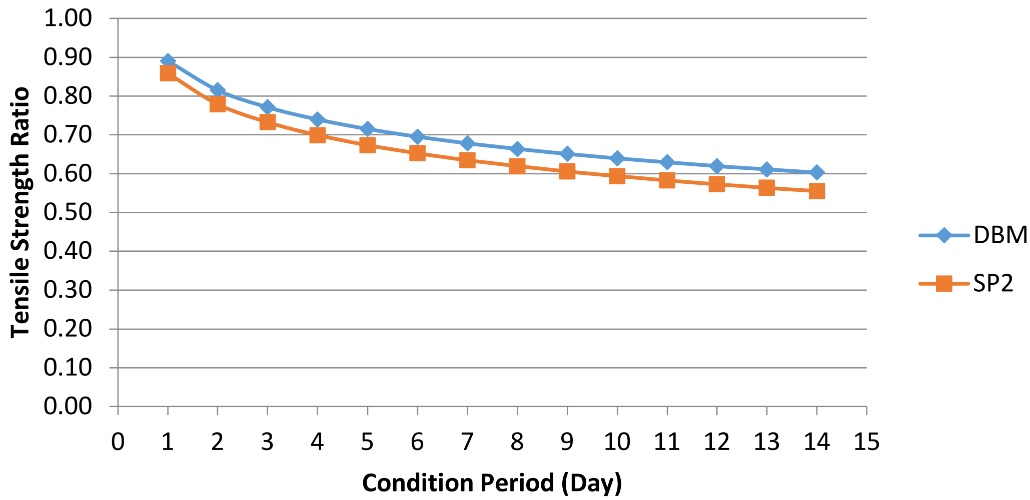


Figure 9.7: TSR Vs. Conditioning Day (Asphalt Base Course Gradations With NRL 40-50)

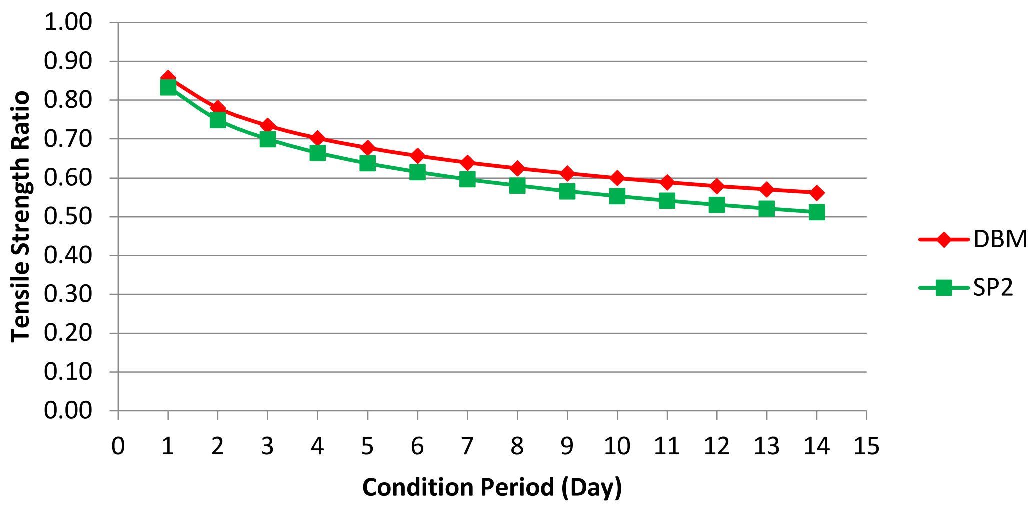


Figure 9.8: TSR Vs. Conditioning Day (Asphalt Base Course Gradations With ARL 60-70)

From table 9.1 it may be concluded that for NHA-A Tensile strength ratio (TSR) has reduced by 7% and 27% for 1 and 14-days moisture conditioning respectively. This gradual decrease in TSR value indicates that moisture is damaging the Marshall compacted specimen specially when it is kept in water bath at 60°C for a period of 1 to 14 days, asphalt being the visco-elasto-plastic material, loses its strength and becomes plastic at higher temperature conditions. The higher the percentage of loss in stability, the greater is the moisture susceptibility, similarly increase in time for conditioning will significantly damage the sample hence lesser will be the TSR value.

Both for asphalt wearing course and base course, TSR decreases gradually according to condition period i.e., highest TSR value at day-1 and lowest at day-14. Moreover, all the gradations are following the same trend with coarser gradations giving better results as compared to the finer ones. The better performance for coarser gradations could be attributed to the fact that coarser aggregate particles provide better aggregate to aggregate interlock and the load is carried and transferred through this particle to particle contact (Hussain et al., 2017). The moisture damage/conditioning in this test has not been able to alter the gradations' tensile strength performance hierarchy. In addition to this, NRL 40/50 gives better TSR values as compare to ARL 60/70. Bitumen Penetration grade is a function of its viscosity which in turn effects the bitumen film thickness or in other words the coating of the bitumen on aggregates. A harder bitumen will have greater film thickness and hence more resistance against moisture damage. (Ahmad. N, 2011)

As typical TSR values range from 0.70 to 0.90, so both the courses (AWC & ABC) and all the gradations, for 1-day conditioning are within range. However, for 14 days conditioning neither the courses nor the gradations (NHA–A, NHA–B, SP–2, and MS–2) are within this range.

9.4.2 Age Hardening Test Results

A comparison of the tensile strength ratio without ageing, after RTFO ageing and after PAV ageing of binders for both asphalt wearing course and base course gradations is summarized in Table 9.2.

Table 9.2: Age Hardening Test Results

Bitumen	Age Hardening	Wearing Course				Base Course	
		NHA–A	NHA–B	SP–2	MS–2	SP–2	DBM
NRL 40/50	Without Ageing	0.93	0.90	0.85	0.84	0.86	0.89
	RTFOAgeing	0.90	0.88	0.82	0.80	0.84	0.87
	PAVAgeing	0.64	0.60	0.58	0.57	0.62	0.70
ARL 60/70	Without Ageing	0.89	0.87	0.85	0.82	0.83	0.86
	RTFOAgeing	0.87	0.85	0.83	0.80	0.81	0.84
	PAVAgeing	0.62	0.60	0.58	0.56	0.61	0.68

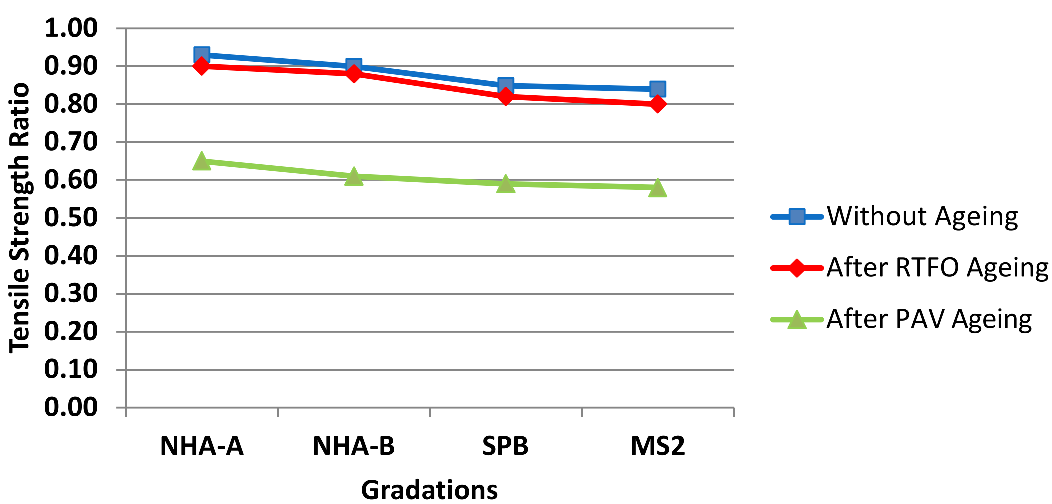


Figure 9.9: Aged TSR Vs. Asphalt Wearing Course Gradations (NRL 40-50)

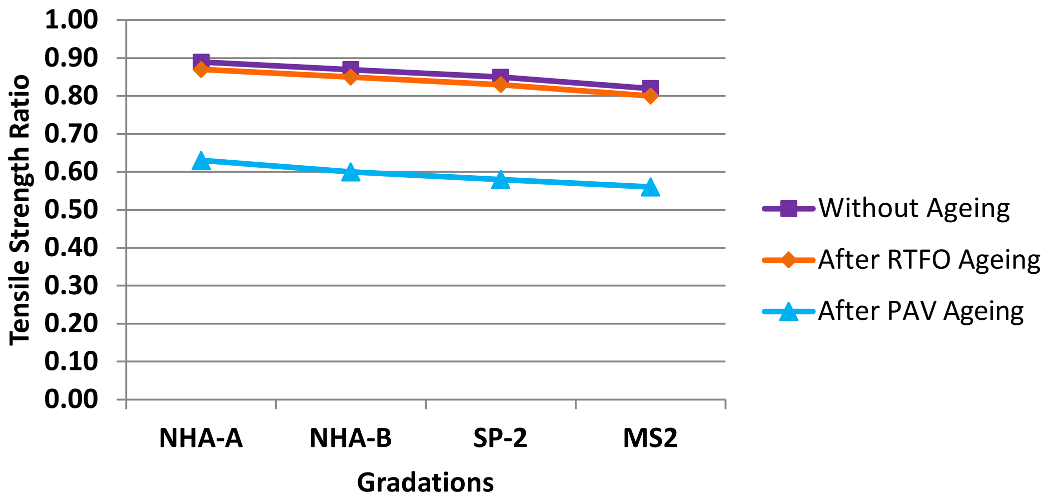


Figure 9.10: Aged TSR Vs. Asphalt Wearing Course Gradations (ARL 60-70)

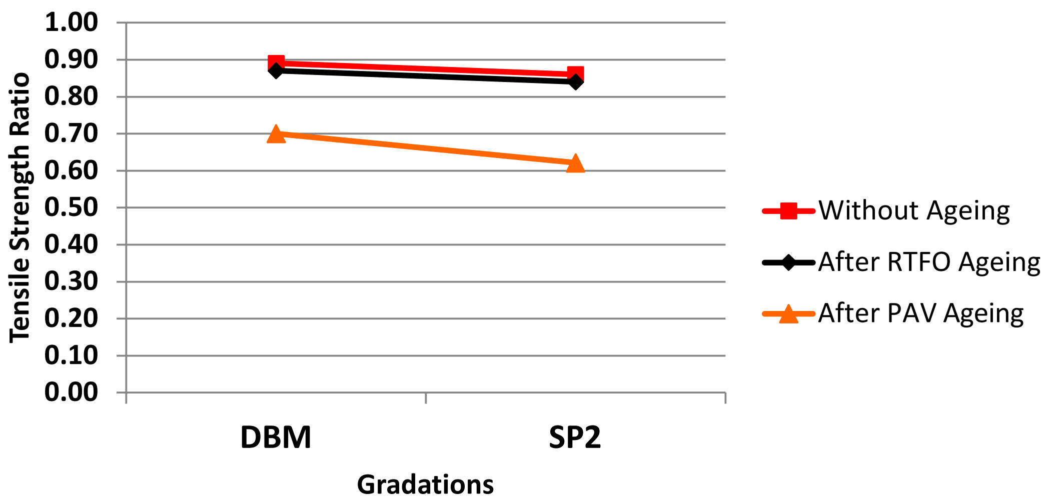


Figure 9.11: Aged TSR Vs. Base Course Gradations (NRL 40-50)

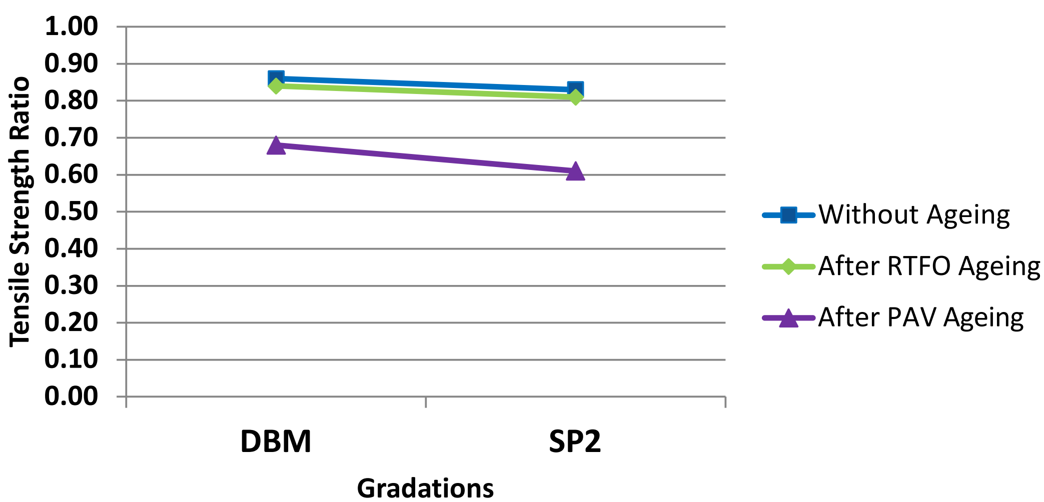


Figure 9.12: Aged TSR Vs. Base Course Gradations (ARL 60-70)

Irrespective of bitumen grade and mix gradation, it may be seen from the graph that ageing adversely effects the TSR value. For wearing course, NHA-A shows least moisture susceptibility resulting in higher value of TSR for both bitumen grades.

For base course, ageing reduces the TSR value more significantly in SP-2 as compared to DBM.

The results also show that short term ageing simulated by RTFO have very less effect on the moisture susceptibility of the mix. While PAV simulating long-term ageing has increased moisture susceptibility significantly which can be attributed to the loss of bond between binder and aggregate due to ageing, hence, making it easier for water to cause damage and reduce strength.

Stiffer bitumen shows better performance than softer grade which can be attributed to higher viscosity and greater film thickness and/or more resistance for the water molecules to flow/move through the binder. (Ahmad. N, 2011)

9.4.3 Hamburg Wheel Tracking Test Results

The results obtained from HWT are generally reported in terms of maximum rut depth (mm), creep slope, stripping slope and stripping inflection point. Samples were tested at 40°C and 50°C and results are reported in Table 9.3 below. Graphical presentation of the rut depth versus the number of passes for various wearing coarse aggregate gradations at 40°C and 50°C is shown in Figure 9.13 and Figure 9.14 below:

Table 9.3: HWT Test Results At 40°C & 50°C

Temperature (ºC)	Gradation	Air Voids (%)	Max. Rut Depth (mm)	Creep Slope	Stripping Slope	Stripping Inflection Point (Passes)
40	NHA-A	6	6.6	2.50E-04	-	-
	NHA-B	6	11.3	1.11E-03	0.00222	5600
	SP-2	6.2	18.1	6.66E-04	0.00125	7200
	MS-2	6.1	20.0	1.00E-03	0.00294	4900
50	NHA-A	5.6	20.0	4.00E-03	0.01000	1310
	NHA-B	5.6	20.0	5.88E-03	0.01000	975
	SP-2	5.6	20.0	5.00E-03	0.01110	1000
	MS-2	6.4	20.0	2.94E-03	0.01300	950

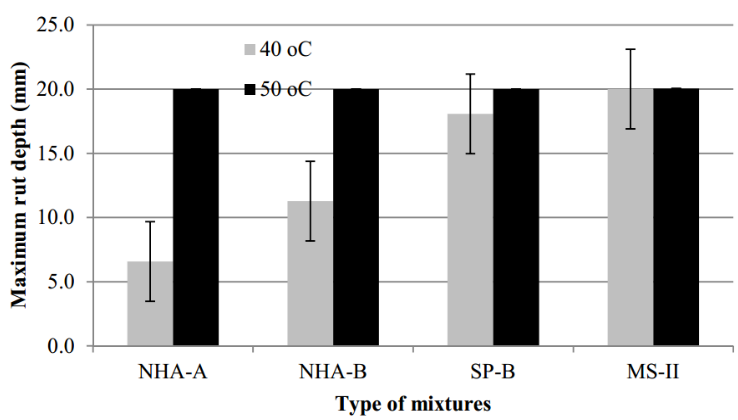


Figure 9.13: Rut Depth Comparison of Various Aggregate Gradations at 40ºC, 50ºC and 10,000 Passes

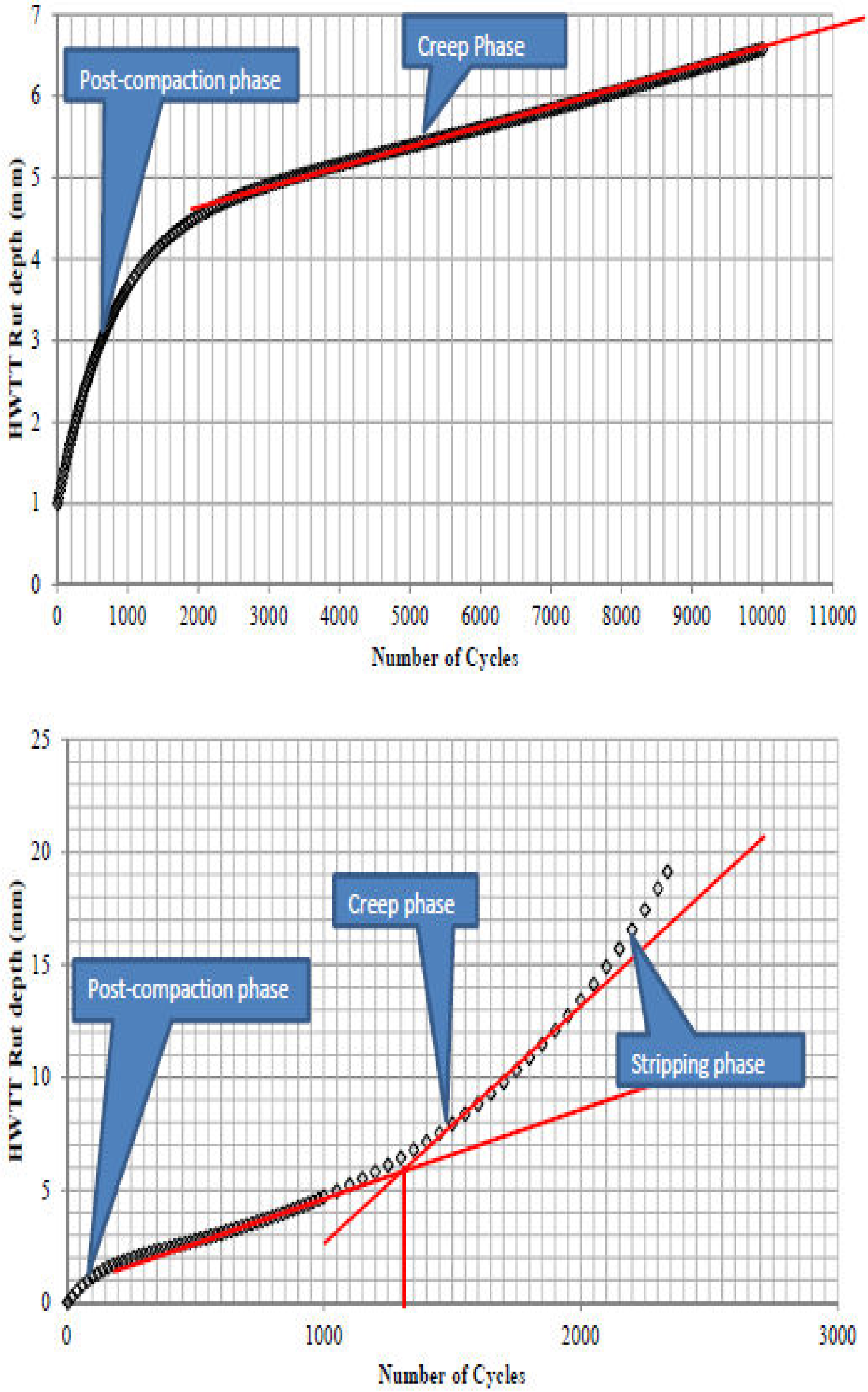


Figure 9.14: HWTT Rutting Development Curves For NHA-A Graded Mixtures At 40°C and 50°C

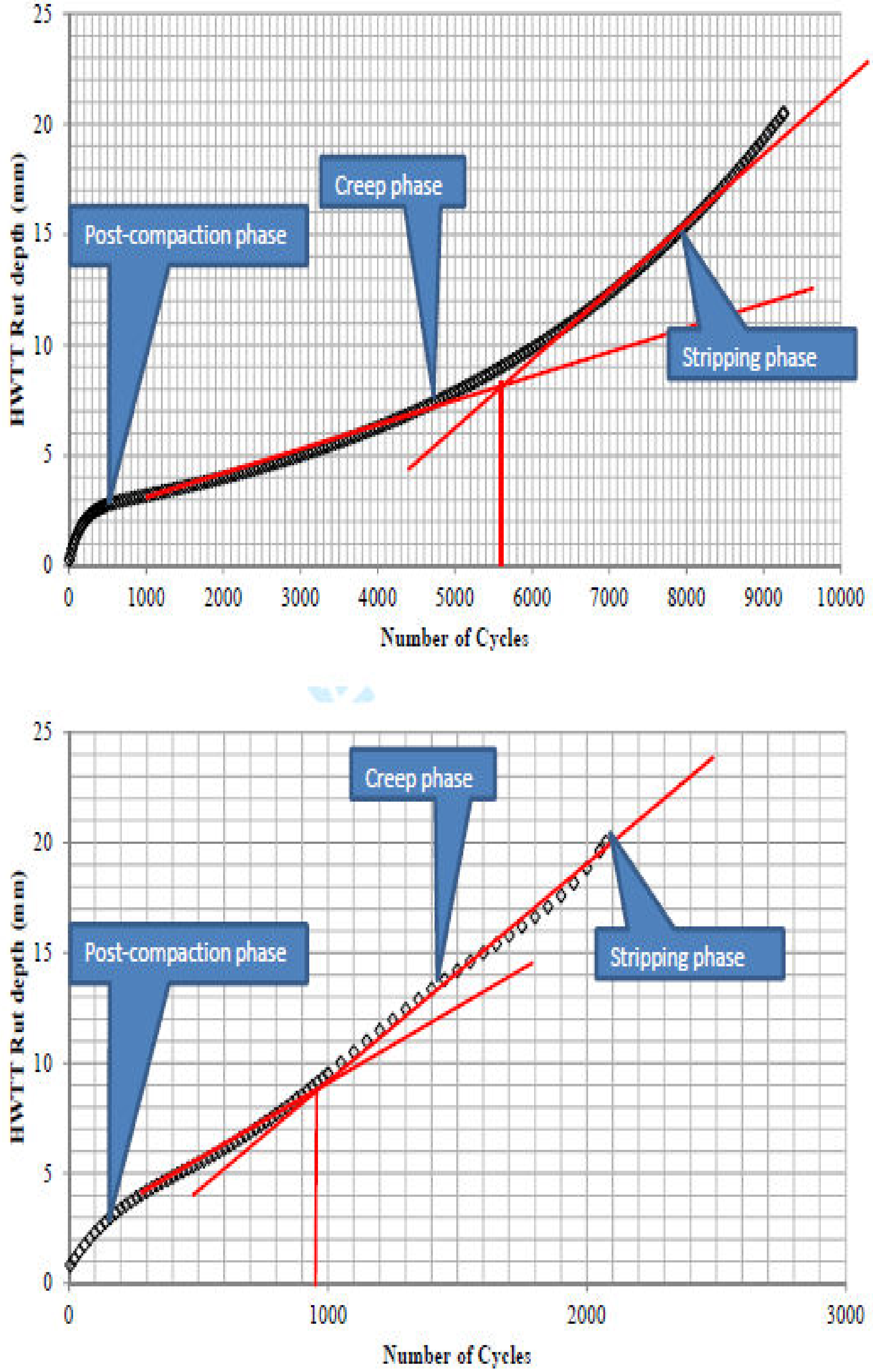


Figure 9.15: HWTT Rutting Development Curves For NHA-B Graded Mixtures At 40°C and 50°C

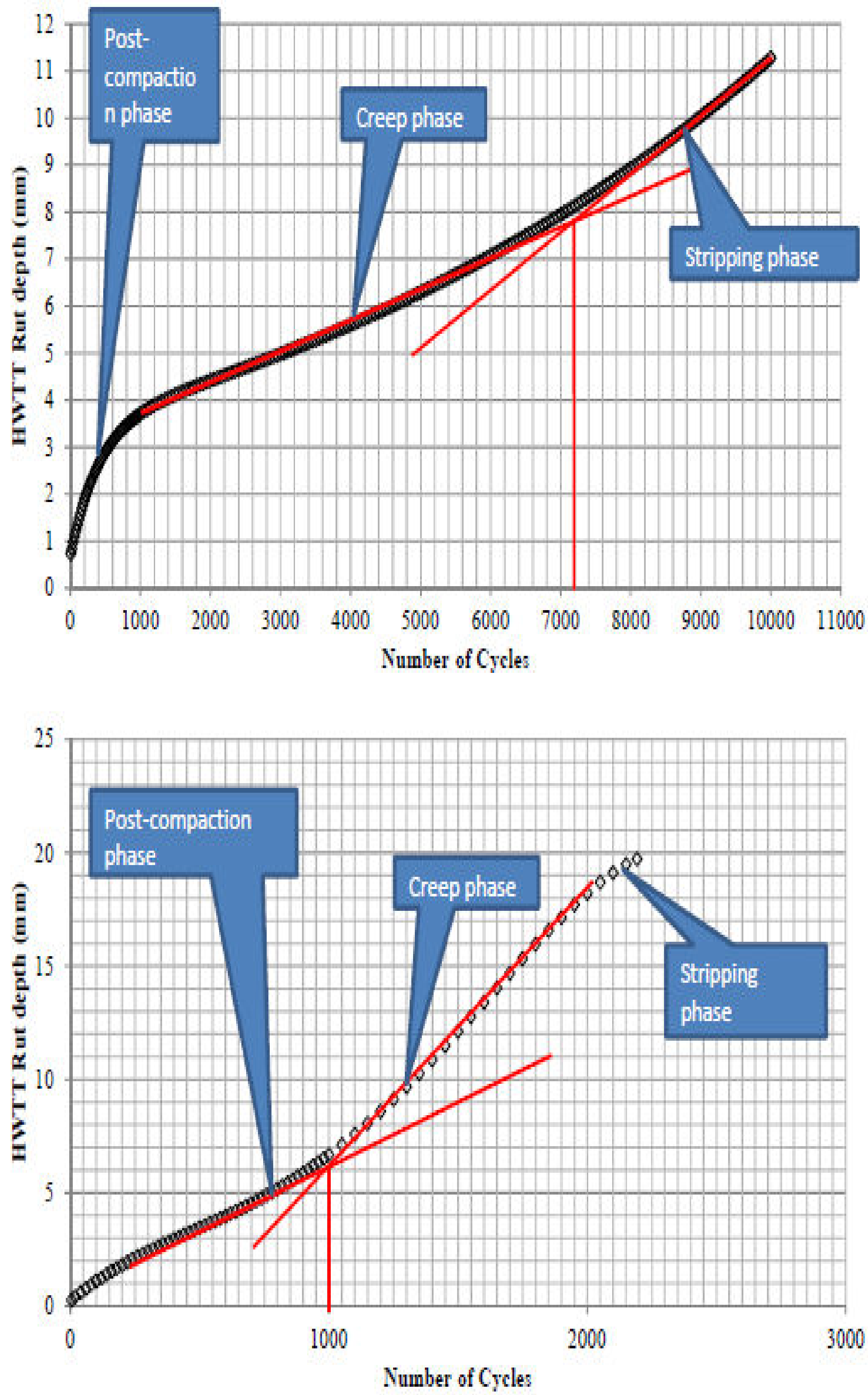


Figure 9.16: HWTT Rutting Development Curves For SP-2 Graded Mixtures At 40°C and 50°C

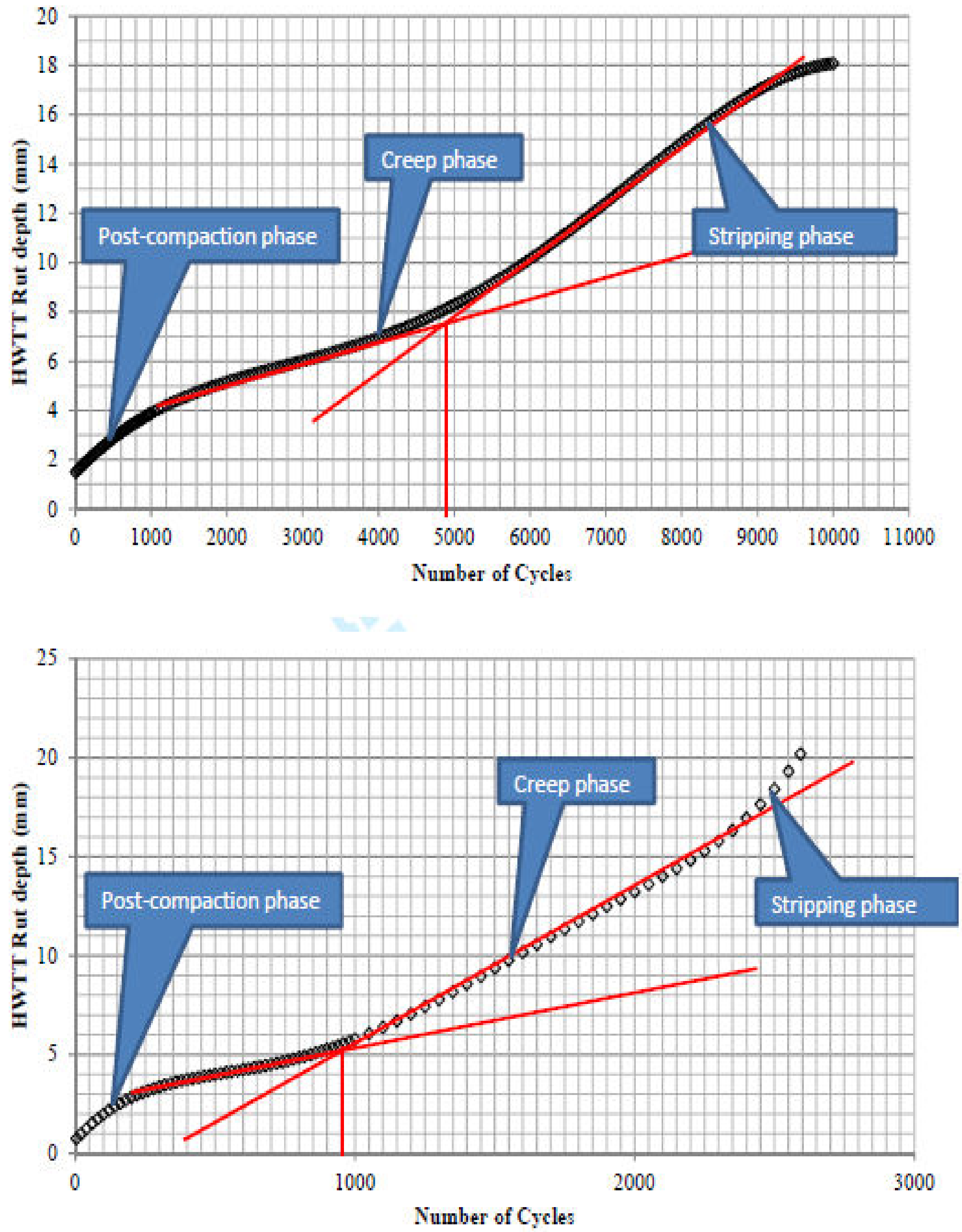


Figure 9.17: HWTT Rutting Development Curves For MS-2 Graded Mixtures At 40°C and 50°C

Figure 9.13 shows that at 40°C a distinct observation may be made in rutting propensity of various aggregate gradations. It can be observed from Table 9.3 and Figure 9.13 that minimum rut depth of 6.5 mm was observed in NHA-A which indicates its maximum resistance against permanent deformation under repeated loading as compared to other gradations. While on the other side MS-2 resulted in maximum rut depth of 20 mm which indicates its less resistance to rutting at 40°C. The difference in rutting behavior between NHA-A (best) and MS-2 (worse) is 13.6 mm and difference in number of passes sustained is approximately 2000. At 50°C samples of all 4 types of gradations reached its threshold point of 20 mm rut depth before the completion of the desired 10000 passes. From Figure 9.14 to 9.17 it is observed that NHA-B required more number of passes to reach maximum rut depth of 20 mm (at 2000 passes, 50°C) while remaining 3 gradations reached its threshold point in less no of load repetitions. Above results indicates that NHA-B showed higher resistance to rutting at higher temperature as compared to the other three gradations.

Coarser gradation (NHA 'A') showed greater resistance against rutting as compared to finer gradation (NHA 'B') at 40°C and same trend was observed by (Hussain et al., 2017), (Guo & Prozzi, 2009) and (Chaturabong & Bahia, 2017). It could be due to more particle-to-particle contact points between large number of coarse aggregate particles which approved its packing and resulted in better performance against rutting under moist conditions.

The rutting curves in Figure 9.14 through Figure 9.17 clearly indicate the three phases of a typical curve between HWTT rut depth and number of cycles i.e., post-compaction phase, creep phase, and stripping phase. It has been observed that due to better resistance to moisture sensitivity, NHA-A graded mixture has not reached the stripping phase at the temperature condition of 40°C even at 10,000 loading cycles. This observation is in line with the ranking of mixtures observed in previous laboratory tests. The creep and moisture susceptibility parameters computed graphically from the above curves has been tabulated in Table 9.4 below.

The creep slope indicates the rutting potential of a particular asphalt mixture. The greater creep slope indicates a higher rutting susceptibility. The creep slope is maximum for MS-2 and minimum for NHA-A graded mixture at both temperature conditions of 40°C and 50°C. Hence MS-2 graded mixture is most rut susceptible mixture while, NHA-A graded mixture has least rutting susceptibility among the tested asphalt mixtures. The same could be translated by terminal rut depth value at temperature condition of 40°C. Stripping slope and stripping inflection point (SIP) are two important parameters related to moisture susceptibility of an asphalt mixture. Evaluation based on stripping slope and SIP follows the similar trend, with maximum stripping slope of 0.00294 and 0.01300 and minimum SIP of 4900 and 950 for MS-2 graded mixture at temperature condition of 40°C and 50°C, respectively (Table 9.3).

9.4.4 Modified Lottman Test Results

Results obtained from MLT have been reported in the form of Tensile Strength and Tensile Strength ratio (TSR) in Table 9.4, and the graphs are plotted in Figure 9.18 to 9.26.

Table 9.4: Modified Lottman Test Results

Gradation	Tensile Strength (kPa)		TSR	Ranking
	Conditioned	Unconditioned		
NHA–A	1384	1706	81.19	Good
NHA–B	1130	1412	80.04	Good
SP–2	1062	1383	76.76	Poor
MS–2	1078	1397	77.24	Poor

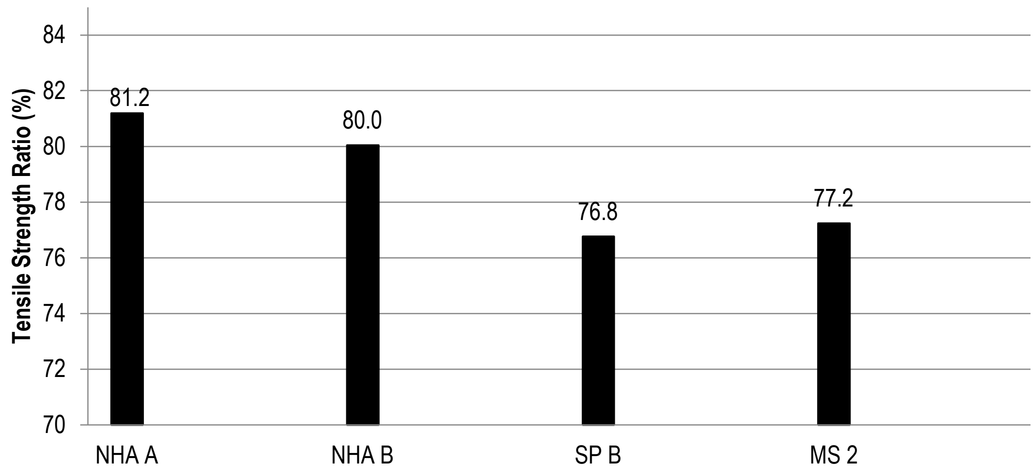


Figure 9.18: TSR Obtained from Ubhan Shah Aggregate &NRL 60/70 Binder

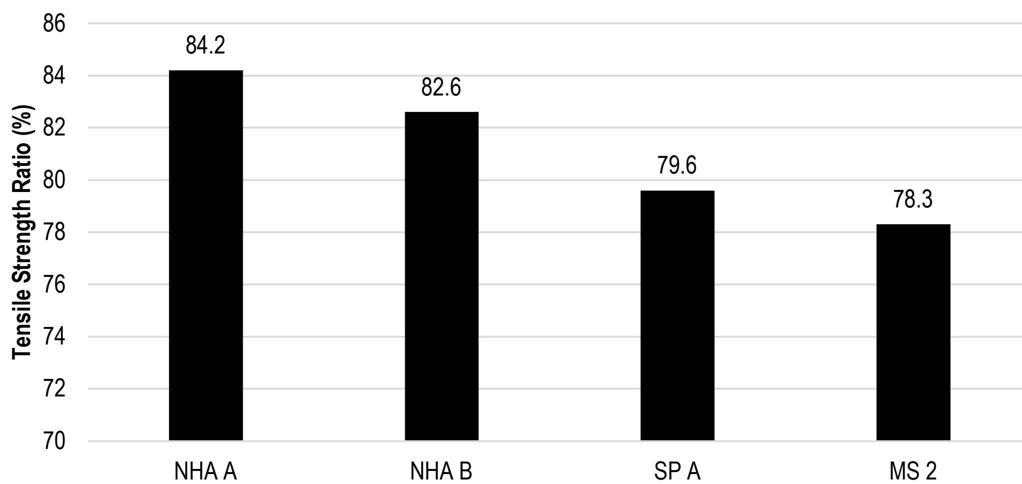


Figure 9.19: TSR Obtained from MLT Margalla Aggregate &ARL 60/70 Binder

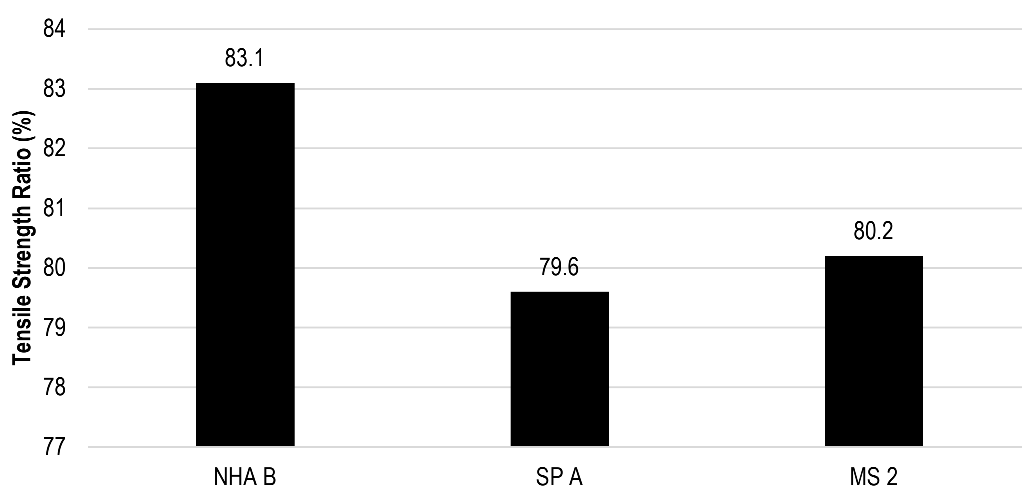


Figure 9.20: TSR Obtained from MLT Margalla Aggregate & NRL 40/50 Binder

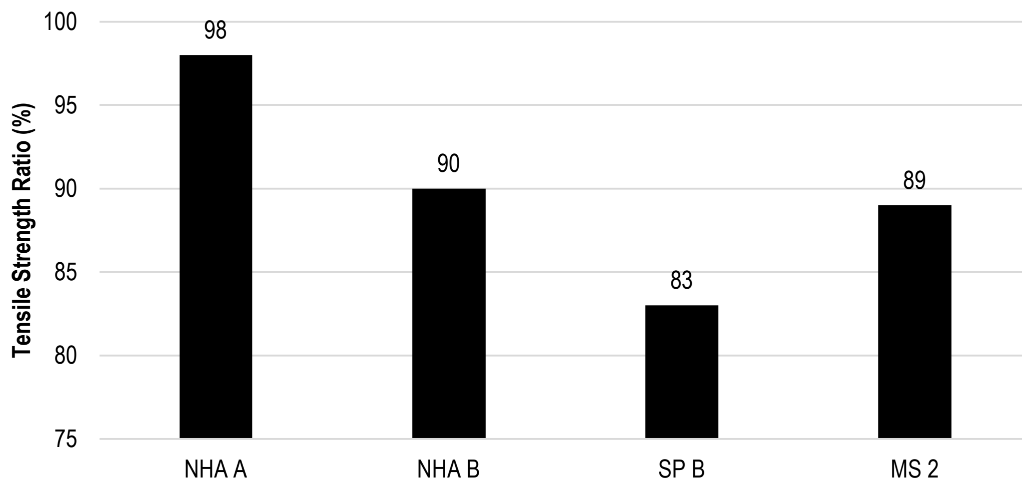


Figure 9.21: TSR Obtained from MLT Ubhan Shah Aggregate & NRL 40/50 Binder

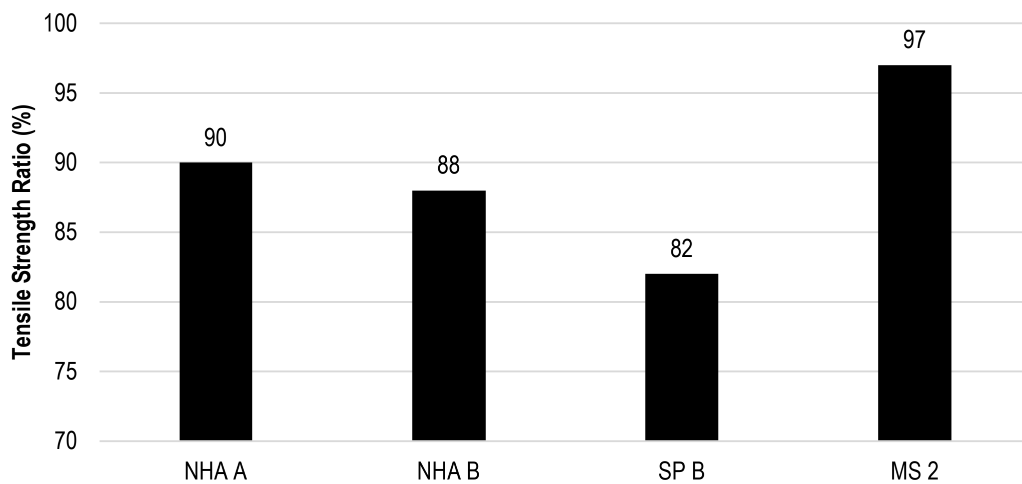


Figure 9.22: TSR Obtained from MLT Ubhan Shah Aggregate & ARL 60/70 Binder

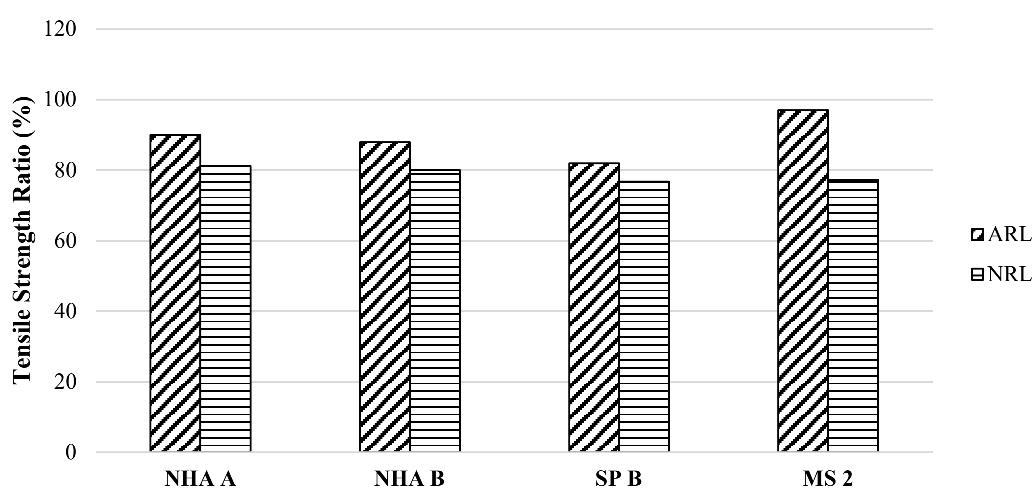


Figure 9.23: TSR Obtained from MLT Ubhan Shah Aggregate With ARL 60/70 And NRL 60/70

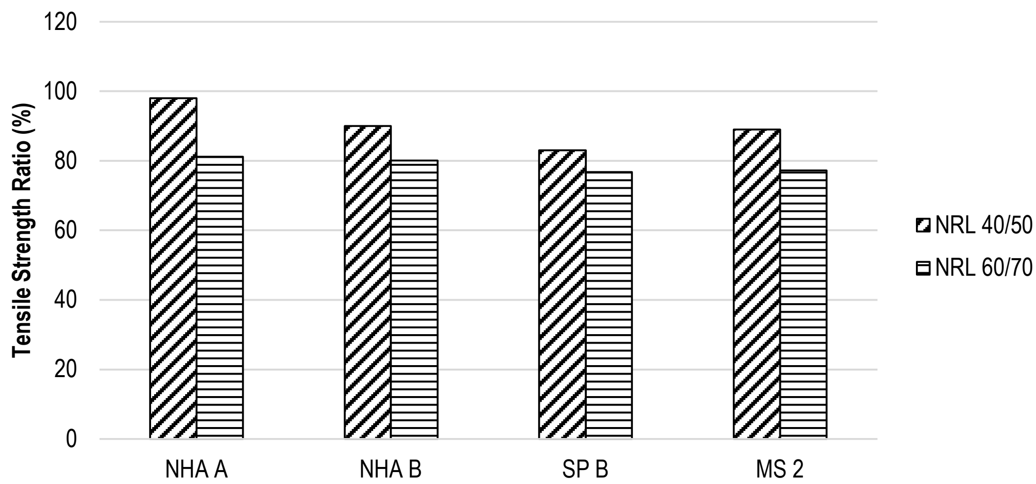


Figure 9.24: TSR Obtained from MLT Ubhan Shah Aggregate with NRL 40/50 And 60/70

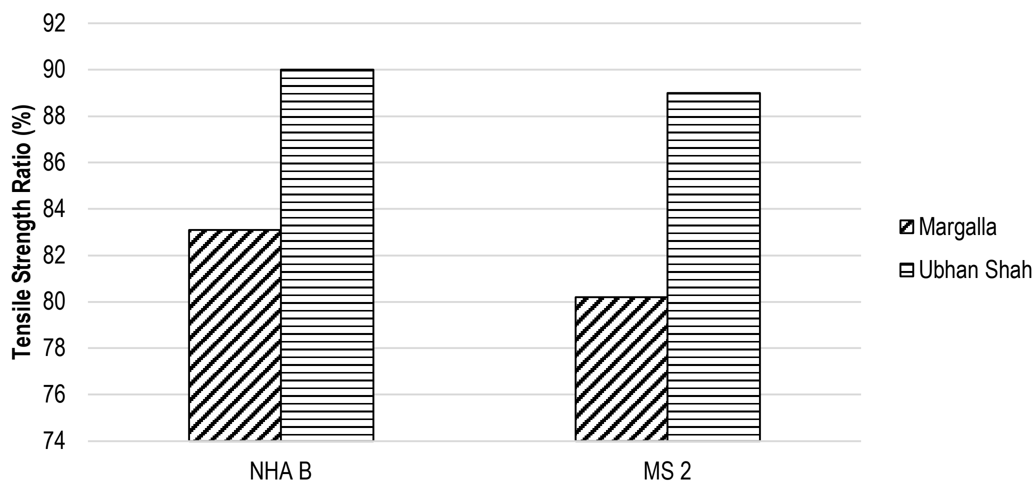


Figure 9.25: TSR Obtained from MLT for Margalla and Ubhan Shah Aggregates with NRL 40/50

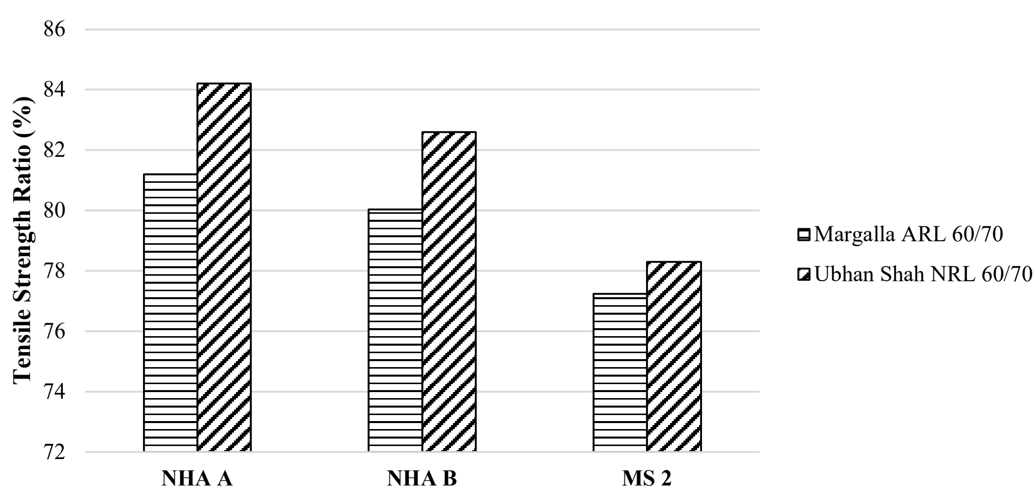


Figure 9.26: TSR Obtained from MLT Margalla Aggregates with ARL 60/70 And Ubhan Shah Aggregates with NRL 60/70

The MLT just like the loss of stability test follows the same trend with NHA-A being the best of all gradations. Here again, the TSR value decreases with finer gradations just like the trend observed in the loss of stability test. (Hussain et al., 2017).

The results from MLT indicates that NHA-A and NHA-B are the only two gradations types which justifies the minimum requirement of TSR i.e., TSR must be greater than 80%. This can be attributed to the coarser nature of NHA-A & B than SP-2 and MS-2. This could be explained by the greater particle to particle contact that exists in the coarse aggregate gradation. (Hussan et al., 2017).

Figure 9.18 to 9.26 shows that even both are limestone, still Ubhan Shah aggregate is less susceptible to moisture than Margalla aggregates. Hence further detailed chemical and physical analysis is recommended to study the reason behind better performance of Ubhan Shah aggregates.

All the gradations follow the same trend before and after conditioning, that might be indicating less effectiveness of conditioning on the mix types.

Results indicate that ARL binder shows better performance against moisture than NRL binder. Also, just like the results from other tests the stiffer grade bitumen shows less susceptibility to moisture than softer grade which can be attributed again to film thickness.

9.4.5 Loss of Marshall Stability

Six (06) samples were prepared using Marshall Mix Design method as per ASTM D 1559 for each of the gradations i.e., NHA ‘A’, NHA ‘B’, Superpave ‘2’ and MS-2 with NRL 60/70 and 40/50 binders. Samples were divided in two subsets of three. Three samples were kept at 60°C in a water bath for 24 hours while the other three samples were kept at room temperature. All the samples were tested for stability to compare the stability loss due to moisture damage. Results are shown graphically in Figure 9.27:

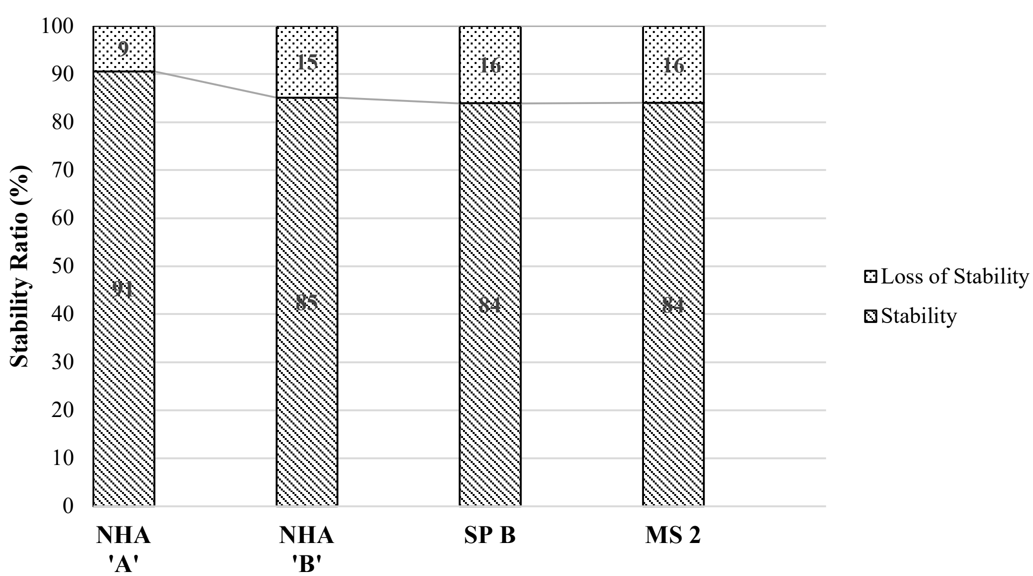


Figure 9.27: TSR obtained by Loss of Stability for Ubhan Shah with NRL 60/70

Indirect tensile strength testing was performed to evaluate the effect of moisture conditioning and age hardening on the selected mixtures. The bituminous mixture of NHA-A using binder NRL 40-50 is the most durable mixture showing 7.5% more TSR and 10% more indirect tensile strength among all the wearing course mixtures. On the other hand, DBM using binder NRL 40-50 is the most durable mixture showing 4.5% more TSR and 18% more indirect tensile strength among all the base course mixtures. All the evaluated bituminous mixtures showed good resistance to moisture susceptibility with TSR values typically falling below 1. Among all the wearing course gradations, the bituminous mixture of NHA-A gradation with binder NRL 40/50 showed highest TSR of 0.93 while DBM gradation with binder NRL 40/50 showed highest TSR of 0.89 for base course gradations. At the end of 14 days, only the bituminous mixture of NHA-A with binder NRL 40/50 resulted in TSR above 0.6 among all the wearing course mixtures while DBM with binder NRL 40-50 resulted in TSR above 0.6 among all the base course mixtures.

After RTFO ageing, bituminous mixture of NHA-A with binder NRL 40-50 showed highest TSR of 0.9 among all the wearing course mixtures while DBM with binder NRL 40-50 showed highest TSR of 0.87 among all the base course mixtures. For all evaluated wearing course and base course mixtures, 2% and 3% reduction in TSR value was noted after RTFO ageing whereas 30% and 20% reduction in TSR value was noted after PAV ageing respectively.

NHA-A was the only gradation that fulfilled the rut depth criteria (12.5mm) at 40ºC and also did not show any sign of stripping at 40ºC at 10,000 passes. Samples prepared for wearing course gradations (NHA-A, NHA-B, SP-2, MS-2) were not able to bear more than 3000 passes at 50ºC temperature. Still at 50ºC, NHA-A performed reasonably well among other gradations. Although, it was difficult to isolate the creep behavior from stripping in MS-2 samples at 50ºC, all the other gradations showed a smooth rutting curve. The gradations containing more percentage of fines tend to lose fine aggregates after stripping inflection point and hence failure of samples occurred rapidly. Out of the selected gradations, NHA-A showed good resistance against moisture as compared to other selected gradations as per MLT. While in LOS tests, NHA-A performed better with 60/70 pen. binder and MS-2 performed better with 40/50 pen. bitumen.

9.4.6 Permeability Analysis

According to recent researches, permeability of asphalt mixtures depends upon mixture's gradation and specimen's air voids. The same observation has been made in this research. Liu et al., (2017) concluded that nominal maximum aggregate size (NMAS) of various aggregate gradations effects the permeability of asphalt mixtures. Xu et al., (2016) established that open graded asphalt mixtures exhibited higher susceptibility to moisture damage due to freeze-thaw cycles. The results in Figure 9.28 indicate that coarse graded mixture with NHA-A gradation exhibits maximum permeability, while fine graded mixture with MS-2 gradation exhibits least permeability in units of cm/sec. The higher permeability allows the water to flow through a specific mixture in shorter span of time; hence, the moisture damage is reduced, as stripping phenomenon aggravates with moisture entrapped within the asphalt layer. This could be the probable reason of least moisture susceptibility of relatively coarser grade i.e., NHA-A mixture performed better in all the laboratory tests conducted in this study.

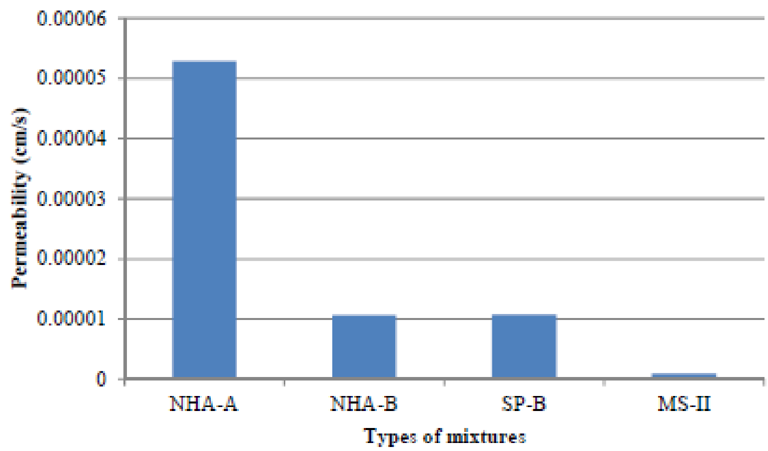


Figure 9.28: Evaluation of mixtures in permeability test

Table 9.5: Test Conditions for Permeability Test

Test condition	Value
Water Pressure (kPa)	150
Lateral Pressure (kPa)	170
Time (hrs.)	24
Collection Time (s)	600
Test Temperature (ºC)	15
Average specimen thickness (cm)	5.16
Average specimen diameter (cm)	9.81
Height of water (cm)	1500

9.5 Conclusions

Conclusions drawn from the study are summarized below:

- All gradation except MS-2 fulfilled the rut depth criteria (20 mm) at 40°C and did not show any sign of stripping at 10,000 passes.
- Samples prepared for wearing course gradations (NHA-A, NHA-B, SP-2, MS-2) were not able to bear more than 3000 passes at 50°C temperature.
- At 50°C, NHA-B performed reasonably well among other gradations.
- It was difficult to isolate the creep behavior from stripping in MS-2 samples at 50°C while all the other gradations showed a smooth rutting curve.
- The gradations containing more percentage of fines were observed to lose fine aggregate after stripping inflection point which resulted in rapid failure of samples.
- NHA-A and NHA-B showed good resistance against moisture as compared to SP-2 and MS-2 as per MLT and LOS tests.
- Bituminous mixture of NHA-A using binder NRL 40/50 is the most durable mixture among all the wearing course mixtures.
- Among all the wearing course gradations, the bituminous mixture of NHA-A gradation with binder NRL 40-50 showed highest TSR of 0.93.

- Moisture susceptibility of all the evaluated wearing course mixtures reduced the TSR by 12% initially for first 3 days of conditioning whereas further reducing TSR by 27% for up to 14 days of conditioning.
- Moisture susceptibility of all the evaluated base course mixtures reduced the TSR by up to 15% initially for first 3 days of conditioning whereas further reducing the TSR by 24% for up to 14 days.
- At the end of 14 days, only the bituminous mixture of NHA-A with binder NRL 40/50 resulted in TSR above 0.6 among all the wearing course mixtures.
- At the end of 14 days, only the bituminous mixture of DBM with binder NRL 40/50 resulted in TSR above 0.6 among all the base course mixtures.
- After RTFO ageing, bituminous mixture of NHA-A with binder NRL 40/50 showed highest TSR of 0.9 among all the wearing course mixtures.
- After RTFO ageing, bituminous mixture of DBM with binder NRL 40/50 showed highest TSR of 0.87 among all the base course mixtures.
- For all evaluated wearing course mixtures, 2% reduction in TSR value was noted after RTFO ageing whereas 30% reduction in TSR value was noted after PAV ageing.
- For all evaluated base course mixtures 3% reduction in TSR value was noted after RTFO ageing whereas 20% reduction in TSR value was noted after PAV ageing.
- After PAV ageing, bituminous mixture of NHA-A with binder NRL 40/50 showed highest TSR of 0.64 among all the wearing course mixtures.
- After PAV ageing, bituminous mixture of DBM with binder NRL 40/50 showed highest TSR of 0.7 among all the base course mixtures.
- Higher number of pores/voids in asphaltic concrete results in increased moisture sensitivity. However, it is only true if these pore/voids are not interconnected (impermeable).
- Once these voids get interconnected the asphalt becomes permeable allowing water to flow through it leading to a lesser amount of moisture damage to the asphalt.
- It is pertinent to mention here that permeable asphalt would perform poorly in the field (unless porous asphalt is required) as the water would penetrate down to the lower layers and would cause damage to the pavement structure.
- Coarser gradations are more sensitive to the amount of air voids in order to keep them impermeable the amount of air voids should be kept to a level where they remain impermeable.
- NHA-A gradation may have been performing well in the laboratory because of lack of dead-end pores and higher number of permeable pores but would not perform well in the field as lower pavement layers are not permeable and would retain water (unlike Porous Pavement) and would retain water leading to structural damage.
- Permeability rate in asphalt mixtures is sensitive to the type of aggregate gradation and Nominal Maximum Aggregate Size (NMAS) of the aggregate gradation. Permeability not only depends on the total amount of air voids present in the asphalt mixture, but also the interconnectivity of air voids. Use of Bailey-modified gradation provides better control on permeability as compared to Conventional gradation. A study based on laboratory experimentation concludes that the maximum recommended permeability value in wearing course asphalt mixture may not exceed 1.5×10^{-2} mm/s (Mahmood and Imran, 2021).

10 Discussion & Recommendations

10.1 Introduction

The stated objective of this study was to improve guidance on providing asphalt mix designs with lesser susceptibility towards rutting and premature cracking in conditions existent in Pakistan. Similarly, the mix should be flexible enough to sustain high tensile strains and be adequately impermeable to resist water ingress with good durability against the elements. Besides asphalt mixture ranking, this research considered elements pertaining to mixture design philosophy which are of significance e.g., the control of composition parameters i.e., aggregate grading and packing characteristics, volumetric composition and method of preparation of laboratory test specimen (closely resembling site compaction), binder properties and moisture susceptibility.

This chapter presents a brief discussion on critical mix design parameters influencing the field performance of asphalt concrete mixtures. Finally, based on laboratory evaluation of selected asphalt mixtures and binders, recommendations are presented for application of best ranked asphalt mixtures on in-service pavements, suiting specific climate regions in Pakistan. In addition, the research study highlighted certain revisions to be incorporated in NHA's General Specifications of 1998.

10.2 Compositional Controls

10.2.1 Aggregate Grading

Currently NHA's General Specifications of 1998 provides limited guidance on aggregate grading and is restricted mainly to the continuous dense graded mixtures, namely; NHA "A" (coarse graded) and NHA-B (comparatively fine graded); for both asphalt base and wearing course mixtures, respectively. These gradations have been employed for nearly three (03) decades now on national highways and motorways with performance track record of both; success and failures. Dense or continuous grading is usually favored because of its aggregate interlocking behavior to resist shear deformations in heavy loading conditions. In addition, shape, texture and hardness of aggregates are critically important in achieving deformation resistant asphalt mixtures.

A number of newer aggregate gradations (tried and in-service across various continents) were incorporated in this research to develop understanding of their applicability in different temperature and geographic zones within Pakistan. These included range of coarse, dense and finer gradations to address rutting, cracking and permeability problems. As expected, coarser gradations performed better over the testing regimes included in the research project. Better rankings for NHA-A, NHA-B and Dense Bitumen Macadam (DBM) mixtures were achieved through Dynamic Modulus, Resilient Modulus, Wheel tracking and Asphalt Pavement Analyzer tests. The results reveal better interlocking behavior; in terms of resistance to permanent deformation. Although to mitigate rutting, the ranking of mixtures may be useful but in real terms their applicability to specific project conditions needs to be evaluated in terms of moisture susceptibility. This is of significance for asphalt wearing course, where coarser mixtures may be difficult to handle in terms of mix design and laydown, whereas; durability is dependent to greater degree on binder content and fines to reduce permeability levels through lesser air void content.

The Bailey procedure, investigated in this research, presents a systematic tool of aggregate blending that emphasize on the importance of aggregate interlock and uniform gradation in an asphaltic mixture. To yield better aggregate blends with maximum packing characteristics, Bailey method uses the relationship of aggregate gradation and volumetric composition to develop a blend of coarser and finer aggregate to achieve maximum interlock within a mix.

10.2.2 Binder Properties

Determination of appropriate binder content for the designed or selected aggregate grading is critical for any asphalt mixture design. Binder plays a key role in terms of resistance against rutting, cracking, stiffness and thermal distresses. These properties are normally expressed in terms of penetration, softening point, viscosity and stiffness modulus. The laboratory tests for these properties form baseline for the binder selection. The most comprehensive specification for binder properties is presented by SHRP Superpave system developed by USA. One of the major products of SHRP research was a suite of tests which are used for classification of binders by their performance characteristics, introducing an asphalt specification which categorizes binder by performance grade (PG). The required grade of binder is determined according to the climate in which it is to be used, and this may be assessed by the geographical location or from pavement or air temperature data. The grading of the binder is based on properties which provides resistance to fatigue and low temperature cracking as well as its susceptibility to permanent deformation at higher temperature.

The NHA's General Specifications of 1998 include only the base line empirical tests for binder such as penetration grade, softening point, flash point and ductility etc. It ignores the viscosity specifications which gained much preference after SHRP. Similarly, in binder specific testing, it does not include any performance tests recommended by SHRP, as yet. The NHA specifications defines a reference mean annual temperature (of any geographical location) as 24°C and recommends using 40/50 penetration grade if mean annual temperature of project area is higher than 24°C, and 60/70 penetration grade if mean annual temperature is less than 24°C to mitigate rutting and fatigue distresses, respectively. However, Over the last three (03) decades, 60/70 penetration grade is mostly used in Pakistan mainly for its availability from the limited oil refineries located in the country. The use of 40/50 penetration grade diminished in general mostly due to the monopoly of the refineries suiting their refining and marketing preferences. It would not be unjust to say that the NHA specifications were largely ignored with continuous use of 60/70 penetration grade in hot climatic conditions prevalent in almost two third of the country's road network.

10.2.3 Volumetric Composition & Specimen Compaction

The volumetrics within the mix, is a function of the chosen gradation and its quarry source, which is determined by the grading of the individual and relative proportion of components. One of the building stone for evaluation of volumetrics is determination of Voids in Mineral Aggregate (VMA) which depends on the nominal maximum size of the aggregates (NMAS), where a potential variation in reported values may arise. NHA's general specifications of 1998 refers Asphalt Institute Manual Series MS-2 for determination of minimal requirement of VMA based on nominal maximum aggregate size and air void content. Other factors such as bitumen absorption in aggregates is also critical. With the onset of asphalt concrete base course and wearing course mixtures on national highways in the early 1990's, widespread rutting phenomenon occurred as a result of shear flow of asphalt concrete due to high ambient temperatures and axle loads. This led to removal of Voids Filled with Asphalt (VFA) parameter (ranges as proposed in MS-2) from the NHA's General Specifications of 1998. Many view this as a main cause of widespread cracking of pavements afterwards across national highways, with visible durability issues on newer and rehabilitation projects.

VMA is a function of binder content and air voids and influences the resultant mix performance. Road temperatures in Pakistan are high in summers with heavy monsoon, which demands that the binder content to be at the optimum level to overcome rutting while restricting air voids to the lower allowable limits, to reduce permeability. It may be noted that the current NHA's General Specification of 1998 provides a much wider envelope of 4% to 8% for air voids in asphalt mixtures compared to Asphalt Institute MS-2, which suggest this to be 3% to 5%. Whereas, the air void content further inflates in the in-place asphalt concrete, due to the compaction criteria of 97%, provided in the NHA's General Specification of 1998. Studies carried out by National Centre for Asphalt Technology show that mixtures having a Nominal Maximum Aggregate Size (NMAS) of 25mm become permeable at 4.4 per cent air voids. Similarly, for 19mm aggregate; the air voids at which the pavement becomes permeable is 5.5%. With no testing provision in NHA general specifications to mitigate permeability, it may be perceived as one of the reasons of premature distress in terms of cracking on national highways and motorways in Pakistan.

One of the major outcomes of the SHRP Superpave Program was to adopt gyratory compaction to replace Marshall method of laboratory compaction. The gyratory process provides a closer simulation to the field compaction processes and hence the influence of differences in the packing processes of aggregates on the final VMA and air voids. It also enables variation of the number of compaction cycles as an improved relationship to traffic densities. The gyratory compaction tends to increase laboratory compactive effort, whereas the current NHA General Specifications 1998 allows the contractor to compact to a minimum of 97% of laboratory (marshall density), in the field. This may be the cause of a great deal of the wheel path rutting, observed on thin (50mm) asphalt overlays. In Pakistan, this compaction specification needs upward revision. However, the requirement must be to set laboratory standards that can be achieved in the field by direct comparison. Instead of setting a target of a percentage of Gyratory compaction, it may be better to set a range of air voids (say 3 to 5%) that must be attained after compaction and a minimum of one month of bedding down under traffic.

Despite of good deal of favorable arguments for gyratory compactor in the literature, many road engineers still prefer Marshall Method, as a simple robust and repeatable process, though recognizing the superiority of gyratory compaction, being both, a research as well as a general design tool. It is suggested that it may be valuable to proceed with field trials of promising mixtures, as the laboratory compaction selection may be time consuming and potentially yield expensive and impractical procedures for routine testing on project.

10.2.4 Bitumen Film Thickness

It is recommended that bitumen film thickness (i.e., the nominal thickness of non-absorbed bitumen coating the aggregate particles) is calculated, as given in Overseas Road Note 19 of United Kingdom (UK), and used to assist in the design process. If the bitumen film thickness is less than 7 microns it is recommended that the determination of VMA be reviewed.

10.2.5 Durability/Moisture Susceptibility

Air voids content play a significant role in defining the moisture damage performance of asphalt mixture. Recent studies into the permeability of asphalt concrete have shown that most of the asphalt mixtures become permeable at air void values of as low as 6% while the coarse graded mixtures will generally have higher permeability values than the fine graded mixtures for same level of air voids (Julie E. Nodes, 2004). A similar study conducted (Khan, 2017) using the same gradations with lower air void contents (4%) showed coarser gradation performing poorer as compared to the finer ones.

This study has shown that selecting higher air void contents could be detrimental to the pavement structure. As coarser gradations are more sensitive to the percentage of air voids, selection of design air voids is of great significance. It can thus be inferred that for coarser gradation such as NHA-A, lower percentage air void criterion should be followed for its better performance. Likewise, permeability rate in asphalt mixtures is sensitive to the type of aggregate and Nominal Maximum Aggregate Size (NMAS) of the aggregate gradation. Permeability not only depends on the total amount of air voids present in the asphalt mixture, but also the interconnectivity of air voids. Use of Bailey-modified gradation provides better control on permeability as compared to conventional gradation. A study based on laboratory experimentation concludes that the maximum recommended permeability value in wearing course asphalt mixture may not exceed 1.5×10^{-2} mm/s (Mahmood and Imran, 2021).

NHA general specifications regarding mix design air voids criterion are required to be adjusted/revised in the light of the above findings.

10.2.6 Higher Axle Loading and its Enforcement

The control of heavy axle loading and its enforcement needs no emphasis. The permissible load limits for different types of freight vehicles (Trucks) on national highways in Pakistan were subject to series of revisions since the year 2000. Each revision entails approximately 20% of increase in loading limits on national highways. The damaging effect has been phenomenal as compared to standard axle load of 8 tons, to which our roads are being designed and is an internationally accepted norm. The damage caused by excessive load relative to the standard axle follows a damaging effect which is exponential.

The exponent of the damaging effect is 4.5, which means that an axle weighing 16 tons (which is double to a standard axle of 8 tons) would damage the pavement 22 times compared to standard axle. The enforcement of legal axle loads on highways need to be regulated as is being done on the motorway network. This is a major challenge for this study in future, where field performance of selected asphalt mixtures will be evaluated.

10.3 Recommendations

10.3.1 Recommendations for Long-Term Pavement Evaluation Study (Phase-2)

- Selected asphalt concrete gradations used in this research study to be evaluated through test sections on upcoming road projects are given in Table 10.1 to Table 10.3. The evaluation of selected asphalt pavement for in-service pavement may be termed as un-controlled pavement test sections.

Table 10.1: Recommended Combination of AWC and ABC (Cold, Mean Annual Air Temp. ≤ 7°C)

Asphalt Base Course	Asphalt Wearing Course					Binder (Pen. Grade)	
	NHA-A	NHA-B	SP-1	SP-2	MS-2	60/70	40/50
NHA-A							
NHA-B					✓	✓	
SP-2			✓	✓	✓	✓	
DBM			✓	✓	✓	✓	

Table 10.2: Recommended Combination of AWC and ABC (Warm, Mean Annual Air Temp. Between 7°C and 24°C)

Asphalt Base Course	Asphalt Wearing Course					Binder (Pen. Grade)	
	NHA-A	NHA-B	SP-1	SP-2	MS-2	60/70	40/50
NHA-A		✓		✓	✓	✓	✓
NHA-B		✓		✓	✓	✓	✓
SP-2		✓		✓	✓	✓	✓
DBM	✓	✓		✓	✓	✓	✓

Table 10.3: Recommended Combination of AWC and ABC (Hot, Mean Annual Air Temp. ≥ 24°C)

Asphalt Base Course	Asphalt Wearing Course					Binder (Pen. Grade)	
	NHA-A	NHA-B	SP-1	SP-2	MS-2	60/70	40/50
NHA-A	✓	✓		✓	✓		✓
NHA-B		✓		✓	✓		✓
SP-2							
DBM	✓	✓		✓	✓		✓

- The recommended combinations of asphalt concrete mixtures and binders given in Table 10.1 to Table 10.3 are applicable for defined temperatures zones of Pakistan.
- It is recommended that the selected asphalt concrete mixtures as evaluated through this research study may also constitute construction of new controled-instrumented test sections at the newly acquired land for HRTC, situated 5km short of Burhan Interchange along the Northbound Carriageway of motorway M-1. This would help validate the Mechanistic Empirical Pavement Design Guide (MEPDG) for Pakistan i.e., Phase-III of Strategic Pavement Research Study.
- It is suggested that while the long-term performance evaluation is in process the asphalt mixtures and binders given in Table 10.1 to Table 10.3 which were evaluated to be the best; following laboratory performance-based testing; can be employed on the upcoming projects across Pakistan. This will provide immediate and tangible benefits to the industry.
- The test sections shall be constructed along the terrain conditions in an unbiased manner to mitigate the superiority of a poor performing asphalt mixture over a better one.
- It is also recommended that the field evaluation of selected asphalt concrete mixtures shall also be taken up with cement stabilized aggregate bases. It will be interesting to incorporate measures towards mitigation of reflective cracking in the asphalt surfacing, this will be a value addition in Phase-II.
- The dynamic and resilient modulus testing of asphalt mixtures in the laboratory have yielded promising results and correlations. These results shall be incorporated in formulation of Mechanistic Empirical Pavement Design Guide (MEPDG) for Pakistan.
- Robust control of legal axle loading shall be enforced on national highways and motorways. This is a major challenge for this study in future, where field performance of the selected asphalt mixtures will be evaluated.

10.3.2 Recommendations specific to NHA's General Specifications of 1998

This research study on selected asphalt mixtures prompted attention towards gaps/improvements in NHA's General Specifications of 1998. The improvements were targeted towards aggregates, binder and asphalt concrete mixtures; namely asphalt base course and wearing course, respectively. The gaps/improvements were largely deliberated by the research group both from academia and industry where a consensus was reached keeping in view the practical implications of these recommendations. Following recommendation (Table 10.4. to Table 10.8) specific to NHA's General Specifications may bring tangible results on ongoing and forthcoming new, maintenance and rehabilitation projects.

Table 10.4: Schedule for Sampling and Testing of Asphalt Base Course Plant Mix (Item No. 203)

Spec Page No.	Material	Current Text & Test Designation	Amended Text, Test & Designation (As recommended by all officers/research partners)	Remarks
G-22	Coarse Aggregate	Flat & Elongated Particle…………. Visual	Flat & Elongated Particle…**<u>BS 812 or ASTM D4791</u>**	-
	Fine Aggregate	Sand Equivalent ……AASHTO T-176 or Plasticity Index………. AASHTO T-89 and T-90	Sand Equivalent…AASHTO T-176 **<u>&</u>** Plasticity Index………. AASHTO T-89 and T-90	“or” has been replaced with “&”.
	Mixture	Bulk Sp. Gr……AASHTO T-166 Method B	Bulk Sp. Gr……AASHTO T-166 Method B. **<u>Aggregate water absorption should not be more than 2 percent.</u>**	-

Table 10.5: Schedule for Sampling and Testing of Asphaltic Wearing Course Plant Mix (Item No. 305)

Spec Page No.	Material	Current Text, Test Designation, Sampling and Testing Frequency	Amended Test, Designation, Sampling and Testing Frequency (As recommended by all officers/research partners)	Remarks
G-26	Coarse Aggregate	Stripping, AASHTO T-182, 3/Source plus 1/5000 CM	BS test BS- EN 12697-11 to be included (Rolling Bottle Test) 3/Source plus**<u>2/5000 CM</u>**	BS test to be included (Rolling Bottle Test) as Stripping Test being obsolete.
	Fine Aggregate	Sand Equivalent…AASHTO T-176 or Plasticity Index………. AASHTO T-89 and T-90; 3/Source plus as required base on visual observation 1/1000 CM	Sand Equivalent…AASHTO T-176 **<u>&</u>** Plasticity Index ………. AASHTO T-89 and T-90;3/Source plus **<u>2/1000 CM</u>**	“or” has been replaced with “&”.
		Specific Gravity, AASHTO T-84, 2/Source	Specific Gravity, AASHTO T-84, **<u>4/Source</u>**	-
		Friable Particles, AASHTO T-112, 1/5000 CM	Friable Particles, AASHTO T-112, **<u>2/5000 CM</u>**	-
	Premix Asphalt	Bulk Sp. Gr…AASHTO T-166	Bulk Sp. Gr... AASHTO T-166 **<u>(aggregate water absorption should not be more than 2%)</u>**	-
		-	**<u>Max. Sp Gravity……AASHTO T-209</u>**	New test/spec added.

Table 10.6: Asphaltic Base Course Plant Mix (Item No. 203)

Spec Page No.	Sr./Line No.	Current Text	Amended/Additional Text (As recommended by all officers/research partners)	Remarks
203-2	203.2.1 a)	The percentage of wear by the Los Angeles Abrasion test (AASHTO T - 96) shall not be more than forty (40).	The percentage of wear by the Los Angeles Abrasion test (AASHTO T - 96) shall not be more than **thirty (30).**	-
	203.2.1 d)	Fine aggregates shall have a liquid limit not more than twenty five (25) and a Plasticity Index of not more than six (6) as determined by AASHTO T 89 and T-90.	Fine aggregates shall have a Liquid Limit not more than twenty five (25) and a Plasticity Index of not more than **four (4)** as determined by AASHTO T-89 and T-90**. Non-Plastic (N.P) material is preferably recommended.**	-
	203.2.1 e)	The portion of aggregate retained on the 9.5 mm (3/8 inch) sieve shall not contain more than 15 percent by weight of flat and/or elongated particles (ratio of maximum to minimum dimensions = 2.5:1).	**Flakiness/Elongation Test shall be carried out as per BS 812 or ASTM D 4791. For ASTM D 4791, its maximum value shall be 10 percent with ratio 2.5: 1.**	-
	203.2.1 i)	-	**Moisture Sensitivity Test shall be performed on compacted asphalt mixtures as described under AASHTO T-283.The minimum Tensile Strength Ratio (TSR) value shall be 80 percent (0.8).**	New text/spec under bullet 203.2.1 i) & j) added.
	203.2.1 j)	-	**Un-compacted Voids Test shall be performed as per AASHTO TP-56 and ASTM C1252 for coarse and fine aggregates respectively. The minimum value for coarse and fine aggregate shall be 45% and 40% respectively.**	
	203.2.2	Asphalt binder to be mixed with the aggregate to produce asphaltic base shall be asphalt cement having penetration grade 40-50, 60-70 or 80-100 as specified by the Engineer. Generally, it will meet the requirements of AASHTO M - 20.	Asphalt binder to be mixed with the aggregate to produce asphaltic base shall be asphalt cement having penetration grade 40-50, 60-70 or 80-100 as specified by the Engineer. Generally, it will meet the requirements of AASHTO M – 20, **Softening Point of bitumen (ASTM D36) & Specific Gravity of bitumen (ASTM D70).**	-
203-3	Table 203-1	Asphalt Content weight percent of total mix: - Class A: 3 (Minimum) Class B: 3 (minimum)	Asphalt Content weight percent of total mix: - **Class A:3.5 (Minimum)** **Class B:3.5 (minimum)**	-
	Line 7	Percent air voids in mix....4-8	Percent air voids in mix....**3-6**	-
	Line 10	Loss in Stability ... 25 percent (Max)	Loss in Stability... **20 percent (Max) (AASHTO T-245)**	-
		-	**Percent VFA in the mix..........55-65 %**	New text/specs added in

Spec Page No.	Sr./Line No.	Current Text	Amended/Additional Text (As recommended by all officers/research partners)	Remarks
			Bulk specific gravity …AASHTO T-166 (Method B) **Bailey's method of aggregate gradation of asphalt mix is recommended for adoption for better packing characteristics within the binder aggregate matrix. Asphalt literature regarding Bailey's method is attached at Appendix I (Reference: Transportation Research Board, USA Circular, October 2002). Though, adoption of this procedure is not obligatory, its adoption for mix aggregate gradation will have better packing characteristics & field performance.** **The thickness of bitumen film (coating the aggregate) shall be 7 microns minimum. Calculation method to determine film thickness is attached at Appendix II (Reference: Overseas Road Note 19 of TRL Limited, UK).**	Marshall Test Criteria for asphalt concrete levelling/ base course mixture.
203-6	203.2.4, Line 6 under Heading: Asphalt Content	Weight percent of total mix:±0.3%	Weight percent of total mix: **±0.2% (without violating specified limits mentioned in Table 203-1)**	-
	Line 13	Loss of Marshall stability by immersion of specimen in water at sixty (60) degree centigrade for 24 hours as compared with stability measured after immersion in water at 60 degrees centigrade for 20 minutes shall not exceeds twenty five (25) percent.	Loss of Marshall stability by immersion of specimen in water at sixty (60) degree centigrade for 24 hours as compared with stability measured after immersion in water at 60 degrees centigrade for 20 minutes shall not exceeds **twenty (20)** percent.	-

Table 10.7: Asphaltic Materials (Item No. 301)

Spec Page No.	Table No.	Current Text	Added Text (as recommended by all officers/research partners)	Remarks
301-a	301-2	-	**Softening Point (ASTM D – 36):** **Grade 40-50: 50 Min, 60 Max** **Grade 60-70: 46 Min, 57 Max** **Grade 80-100: 43 Min, 54 Max**	New text/tests/specs added.
		-	**Specific Gravity (ASTM D - 70):** **Grade 40-50: 1.01 Min, 1.06 Max** **Grade 60-70: 1.01 Min, 1.06 Max**	

Spec Page No.	Table No.	Current Text	Added Text (as recommended by all officers/research partners)	Remarks
			Grade 80-100: 1.01 Min, 1.06 Max	
		-	**Mixing and compaction temperatures, shall be determined based on Rotational Viscometer Test shall be performed as per AASHTO T-316 and as per procedure outlined in Asphalt Institute MS-2.**	
		-	**Projects (wherein asphalt binder usage is 1000 tons minimum), complete performance testing shall be carried out following SHRP testing protocols for binders Appendix III (Reference: ASTM Designation: D 6373-99).**	

Table 10.8: Asphalt Concrete Wearing Course - Plant Mix (Item No. 305)

Spec Page No.	Sr./Line No.	Current Text	Amended/Added Text (As recommended by all officers/research partners)	Remarks
	305.2.1 i)	-	**Moisture Sensitivity Test shall be performed on compacted asphalt mixture as described under AASHTO T-283. The minimum Tensile Strength Ratio (TSR) value shall be 80 percent (0.8).**	New text/spec under bullet 305.2.1 i) & j) added.
	305.2.1 j)	-	**Un-compacted Voids Test shall be performed as per AASHTO TP-56 and ASTM C1252 for coarse and fine aggregates respectively. The minimum value for coarse and fine aggregate shall be 45% and 40% respectively.**	
	305.2.2	Asphaltic binder to be mixed with the aggregate to produce asphaltic base shall be asphalt cement penetration grade 40-50, 60-70 or 80-100 as specified by the Engineer. Generally, it will meet the requirement of AASHTO M-20.	Asphaltic binder to be mixed with the aggregate to produce asphaltic base shall be asphalt cement penetration grade 40-50, 60-70 or 80-100 as specified by the Engineer. Generally, it will meet the requirement of AASHTO M-20, **Softening Point of bitumen (ASTM D36) & Specific Gravity of bitumen (ASTM D70).**	-
305-3	Table 305-1	Asphalt Content weight percent of total mix:- Class A: 3.5 (min) Class B: 3.5 (min)	Asphalt Content weight percent of total mix:- **Class A: 4 (min)** **Class B: 4 (min)**	-

Spec Page No.	Sr./Line No.	Current Text	Amended/Added Text (As recommended by all officers/research partners)	Remarks
		Stability.... 1000 Kg (Min.)	Stability.... 1000 Kg (Min.) **AASHTO T-245**	-
		Flow, 0.25 mm (0.01 in.) ….8-14	Flow, 0.25 mm (0.01 in.) ….8-14 **AASHTO T-245**	-
		Percent air voids in mix....4-7	Percent air voids in mix....**3-6**	-
		Loss of Stability ... 20 percent (Max)	Loss of Stability**... 20 percent (Max) (AASHTO T-245)**	-
		-	**VFA ……..…60-70 % (MS-2)** **Refusal Density air voids…… 2% min air Voids (BS 598, Marshall Compactor Method or Gyratory Compactor)** **Bulk specific gravity …AASHTO T-166 (Method B).** **Bailey's method of aggregate gradation of asphalt mix shall be adopted for better packing characteristics within the binder aggregate matrix. Asphalt literature regarding Bailey's method is attached at Appendix I (Reference: Transportation Research Board, USA Circular, October 2002). However, adoption of this procedure is not obligatory but its adoption for mix aggregate gradation will causes better packing characteristics & field performance.** **The thickness of bitumen film (coating the aggregate) shall be 7 microns minimum. Calculation method to determine film thickness is attached at Appendix II (Reference: Overseas Road Note 19 of TRL Limited, UK).** **Frequency of Testing of Cores: One core shall be taken for each 100 linear meters of each lane of Asphaltic Wearing Coarse, or fraction thereof, in special cases. If the core so taken is failed against the specified 98% density, then two (2) additional cores shall be taken in the longitudinal alignment of the road at an interval of three (3) meters on either side with respect to the failing core and shall be tested against field density. If all the three cores give an average of 98% compaction, and the individual compaction of the core is not less than ninety six (96) percent, then the compaction is acceptable. If average of the cores further fails against compaction, then retake the cores at a distance of fifteen (15) meters on either side and compaction shall be checked for all the five cores in the same fashion. If average of**	New text/specs added in Marshall Test Criteria for asphalt concrete wearing course mixture. Note: To compensate this additional text/ amendments, a new page bearing no. 305-3(a) is required to be inserted.

Spec Page No.	Sr./Line No.	Current Text	Amended/Added Text (As recommended by all officers/research partners)	Remarks
			five cores is 98%, the area will be accepted. In case average is ninety seven (97%) or more, then Engineer may withhold the payment in full or partly and observe behavior during maintenance period, for the release of payment or otherwise. In case of failure of the average of these five cores giving average compaction of less than 97%, the failed area shall be removed and subsequently be replaced by specified mix in an approved manner at the expense of contractor.	
305-4	305.2.4, Line 22 under Heading: Asphalt Content	Weight percent of total Mix: $\pm$ 0.3%	Weight percent of total Mix: **+0.2% (without violating the limits specified in Table 305-1 of general specifications.**	-

References

A. Hartman, M.D. Gilchrist, G. Walsh, “Effect of mixture compaction on indirect tensile stiffness and fatigue”, J. Transp. Eng. 2001.

AASHTO M-20, “Standard Specification for Penetration-Graded Asphalt Cement,” AASHTO Washington, DC, USA, 2004.

AASHTO PP1, “Standard Practice for Accelerated Ageing of Asphalt Binder using a Pressurized Ageing Vessel (PAV),” AASHTO Washington, DC, USA, 1998.

AASHTO T 316, “Standard Method of Test of Viscosity Determination of Asphalt Binder using Rotational Viscometer,” American Association of State Highway and Transportation Officials (AASHTO), Washington, D.C, 2010.

AASHTO TP 62: “Standard Test Method for Determining the Dynamic Modulus of Hot Mix Asphalt (HMA),” American Association of State Highway and Transportation Officials, D.C, 2009.

AASHTO TP 62-07, “Standard Test Method for Determining the Dynamic Modulus of Hot Mix Asphalt (HMA),” American Association of State Highway and Transportation Officials, Washington D.C., USA, 2007.

AASHTO TP 79, “Standard Method of Test for Determining the Dynamic Modulus and Flow Number for Hot Mix Asphalt using Asphalt Mixture Performance Tester,” American Association of State Highway and Transportation Officials, Washington D.C., USA, 2010.

AASHTO, “AASHTO Guide for Design of Pavement Structures,” AASHTO, Washington. D.C., 2002.

AASHTO, American Association of State Highway Transportation Officials (AASHTO) “Guide for Design of Pavement Structures”, 1993: AASHTO, 1993.

AASHTO, TP8-94, “Standard Test Method for Determining the Fatigue Life of Compacted Hot Mix Asphalt (HMA) Subjected to Repeated Flexural Bending”, 1994.

ABF BD, “Method for Determination of the Fatigue Characteristics of Bituminous Mixtures Using Indirect Tensile Fatigue”, British Standard Institution, London, UK, 1997.

Abo-Qudais, S.A., Al-Shweily, H., “Effect of Antistripping Additives on Environmental Damage of Bituminous Mixtures,” Building and Environment Journal, 2005.

Abu Abdo, A.M., “Sensitivity Analysis of a New Dynamic Modulus (|E*|) Model for Asphalt Mixtures,” Road Materials and Pavement Design 13 (3), 549–555, 2012.

Adedare, S., Adedimila and Kennedy, T. W., “Fatigue and Resilient Characteristics of Asphalt Mixtures by Repeated-Load Indirect Tensile Test: Research Report No. 183-5,” Centre for Highway Research, University of Texas at Austin, 1975.

Adedimila, A.S., and Kennedy, T.W., “Repeated-Load Indirect Tensile Fatigue Characteristics of Asphalt Mixture,” Transportation Research Record, no. 595, pp. 25–33, 1976.

Agalbjorn, S., Koncar, N., and Jones. A. J., “A note on the gamma test. Neural Computing and Applications”. 5(3), 131–133, 1997. doi: 10.1007/bf01413858.

Ahmad, N. “Asphalt mixture moisture sensitivity evaluation using surface energy parameters”. PhD thesis, University of Nottingham, 2011.

Ahmed M. A., and Attia M. I. E., “Impact of aggregate gradation and type on hot mix asphalt rutting in Egypt”. International Journal of Engineering Research and Applications (IJERA) ISSN: 2248-9622, 3 (4): 2249-2258, 2013.

Ahmed, A. W., & Erlingsson, S., “Evaluation of a Permanent Deformation Model for Asphalt Concrete Mixtures Using Extra-Large Wheel-Tracking and Heavy Vehicle Simulator Tests,” Road Materials and Pavement Design, 15(4), 1-18, 2014.

Ahmed, A. W., and Erlingsson, S., “Evaluation of a permanent deformation model for asphalt concrete mixtures using extra-large wheel-tracking and heavy vehicle simulator tests”. Road Materials and Pavement Design, 16(1), 154-171, 2015.

Aho, B., Vavrik, W., and Carpenter, S., “Effect of flat and elongated coarse aggregate on field compaction of hot-mix asphalt”. Transportation Research Record. 1761, Transportation Research Board, National Research Council, Washington D.C., 26-31, 2001.

Airey, G. D., & Collop, A. C., “Mechanical and structural assessment of laboratory- and field-compacted asphalt mixtures”. International Journal of Pavement Engineering, 17(1), 50-63, 2016.

Alatas, T., and Kizirgll, M. E., “The Effects of using Styrene-Butadiene-Styrene and Fly Ash Together on the Resistance to Moisture-Induced Damage, Permanent Deformation and Fatigue of Hot Mixture Asphalt”. KSCE Journal of Civil Engineering, 17(5), 1030-1039, 2013.

Al-Khateeb, G. G., Khedaywi, T. S., Obaidat, T. I. A., and Najib, A. M., "Laboratory Study for Comparing Rutting Performance of Limestone and Basalt Superpave Asphalt Mixtures". J. Mater. Civ. Eng., 25(1), 21-29, 2013.

Al-Khateeb, G.G., and Ghuzlan, K.A., "The Combined Effect of Loading Frequency, Temperature, and Stress Level on the Fatigue Life of Asphalt Paving Mixtures Using IDT Test Configuration," International Journal of Fatigue Volume 59, pp.254-261, 2014.

Al-Khateeb, G.G., Khedaywi, T.S., Obaidat, T.I.A., and Najib, A.M., "Laboratory Study for Comparing Rutting Performance of Limestone and Basalt Superpave Asphalt Mixtures," J. Mater. Civ. Eng., 25(1), 21-29, 2013.

Al-Mosawe, H., Thom, N. H., Airey, G. D., and Al-Bayati, A. H., "Effect of different variables on asphalt mechanical properties". Proceedings of the 4th Chinese-European Workshop on Functional Pavement Design, CEW 2016, Delft; Netherlands; 29 June 2016 through 1 July 2016.

Al-Mosawe, H., Thom, N., Airey, G. D., and Al-Bayati, A.,"Effect of Aggregate Gradation on the Stiffness of Asphalt Mixtures". International Journal on Pavement Engineering & Asphalt Technology, 16(2), 39-49, 2015.

Almudaiheem, J., A., and Al-sugair, F., H., "Effect of Loading Magnitude on Measured Resilient Modulus of Asphalt Concrete Mixtures," Journal of Transportation Research Board, No. 1317, pp. 139-144, 1991.

Almudaiheem, J.A., and Al-sugair, F.H., "Effect of Loading Magnitude on Measured Resilient Modulus of Asphalt Concrete Mixtures," Journal of Transportation Research Board, No. 1317, pp. 139-144, 1991.

Al-Qadi, I., Yoo, P., Elseifi, M., and Nelson, S. "Creep Behavior of Hot-Mix Asphalt due to Heavy Vehicular Tire Loading". J. Eng. Mech., 135(11), 1265–1273. 2009

Al-Qadi, I.L., and Nassar, W.N., "Fatigue Shift Factors to Predict HMA Performance," International Journal of Pavement Engineering, Volume 4, No. 2, pp. 69-77, 2003.

Al-Qadi, I.L; Son, S.; Zehr, T.; "Development of an economical, thin, quiet, long-lasting, high friction surface layer, Volum-I: Mix Design and Laboratory Performance Testing" Rep. No. FHWA-ICT-13-001. Illinois centre for transportation. Univ. Of Illinois at Urbana-Champaign, Rantoul, IL, 2013.

Alshamsi, K. S.,"Development of a Mix Design Methodology for Asphalt Mixtures with Analytically Formulated Aggregate Structures". MSc. Dissertation submitted to the graduate faculty of the Louisiana State University, 2006.

Ameri, M., Mansourian, A., and Sheikhmotevali, A. M., "Laboratory evaluation of ethylene vinyl acetate modified bitumens and mixtures based upon performance related parameters". Construction and Building Materials 40, 438-447, 2013.

American Association of State Highway and Transportation Officials (AASHTO). "Standard Test Method for Density of Semi-Solid Bituminous Materials", AASHTO Designation T228, 2002.

American Association of State Highway and Transportation Officials (AASHTO). "Softening Point of Bitumen (Ring and Ball Apparatus)", AASHTO Designation T53, 2000.

American Society for Testing and Materials (ASTM). "Standard Test Method for Penetration of Bituminous Materials". ASTM Designation D5, 2004.

American Society for Testing and Materials (ASTM). "Standard Test Method for Soundness of Aggregates by Use of Sodium Sulfate or Magnesium Sulfate. ASTM Designation C88, 2004.

American Society for Testing and Materials (ASTM). "Standard Test Method for Resistance to Degradation of Small-Size Coarse Aggregate by Abrasion and Impact in the Los Angeles Machine". ASTM Designation C131, 2004.

American Society for Testing and Materials (ASTM). "Standard Test Method for Clay Lumps and Friable Particles in Aggregates". ASTM Designation C142, 2004.

American Society for Testing and Materials (ASTM). "Standard Test Method for Ductility of Bituminous Materials". ASTM Designation D113, 2004.

American Society for Testing and Materials (ASTM). "Standard Test Method for Relative Density (Specific Gravity) and Absorption of Fine Aggregate". ASTM Designation C128, 2004.

American Society for Testing and Materials (ASTM). "Standard Test Method for Density, Relative Density (Specific Gravity), and Absorption of Coarse Aggregate". ASTM Designation C127, 2004.

American Society for Testing and Materials (ASTM). "Test Method for Theoretical Maximum Specific Gravity and Density of Bituminous Paving Mixtures". ASTM Designation D2041, 2004.

American Society for Testing and Materials (ASTM). "Test Method for determining the Percentage of Plastic Fines in Graded Aggregate and Soils by Use of the Sand Equivalent Test". ASTM Designation D241, 2004.

American Society for Testing and Materials (ASTM). "Test Method for determining the Percentage of Fractured Faces in Coarse Aggregate". ASTM Designation D5821, 2004.

American Society for Testing and Materials (ASTM). "Standard Test Method for Sieve Analysis of Fine and Coarse Aggregates". ASTM Designation C136, 2014.

Amirkhanian, S. N., Douglas, K., and James, B., "Effects of Los Angeles abrasion test values on the strength of laboratory prepared Marshall specimens". Transportation Research Record. 1301 Transportation Research Board, National Research Council, Washington D.C., 77-86, 1991.

Anthony, D.S.; Bahia, H.U. "The effect of fine aggregate angularity, asphalt content and performance graded asphalts on hot mix asphalt performance". WisDOT Highway research study Rep. No. 0092-45-98. University of Wisconsin, Madison W.I, 2003.

Apeagyei, A. "Rutting as a Function of Dynamic Modulus and Gradation". J. Mater. Civ. Eng., 23(9), 1302–1310, 2011.

Apeagyei, A.K., "Flow Number Predictive Models from Volumetric and Binder Properties," Construction and Building Materials. 64 (14), pp: 240-245, 2014.

Apeagyei, A.K., "Rutting as a Function of Dynamic Modulus and Gradation," Journal of Materials in Civil Engineering, Vol. 23, No. 9, pp: 1302-1310, 2011.

Apeagyei, A.K., Diefenderfer, B.K. and Diefenderfer, S.D., "Development of Dynamic Modulus Master Curves for Hot-Mix Asphalt with Abbreviated Testing Temperature," International Journal of Pavement Engineering, 13(2), 98-109, 2012.

Archilla, A. R., and Diaz, L. G., "Effects of Asphalt Mixture Properties on Permanent Deformation Response. Transportation Research Record. 2210, Transportation Research Board, National Research Council, Washington D.C., 1-8, 2011.

Archilla, A., Diaz, L., and Carpenter, S. "Proposed Method to Determine the Flow Number in Bituminous Mixtures from Repeated Axial Load Tests". J. Transp. Eng., 133(11), 610–617, 2007.

Archilla, A.R., "Use of Superpave Gyratory Compaction Data for Rutting Prediction," J. Transp. Eng., 132(9), 734-741, 2006.

Archilla, A.R., and Madanat, S., "Development of a Pavement Rutting Model from Experimental Data," J. Transp. Eng., 126(4), 291-299, 2000.

Archilla, A.R., Diaz, L.G., and Carpenter, S.H., "Proposed Method to Determine the Flow Number in Bituminous Mixtures from Repeated Axial Load Tests," Journal of Transportation Engineering, Vol. 133, No. 11, pp: 610-617, 2007.

Aschenbrener, T., “Evaluation of Hamburg Wheel-Tracking Device to Predict Moisture Damage in Hot Mix Asphalt,” Transportation Research Record 1492, TRB, National Research Council, Washington, D.C., pp. 193-201, 1995.

Asphalt Institute, “Mix Design Methods,” Manual Series No.2 (MS-2), Pennsylvania State University, 1989.

Asphalt Institute, “Thickness Design-Asphalt Pavements for Highways and Streets,” Manual Series No.1, Pennsylvania State University, 1981.

Asphalt Institute, “Mix Design Methods for Asphalt Concrete and other Hot-Mix types”, Manual Series No. 2, Lexington, KY, 1997.

Asphalt Institute, “Thickness design–asphalt pavements for highways and streets”, Asphalt Institute, 1981.

ASTM C 127, “Standard Test Method for Density, Relative Density (Specific Gravity), and Absorption of Coarse Aggregate,” American Society of Testing Materials (ASTM) International, West Conshohocken, PA, 2012.

ASTM C 131, “Standard Test Method for Resistance to Degradation of Small-Size Coarse Aggregate by Abrasion and Impact in Los Angeles Machine,” American Society of Testing Materials (ASTM) International, West Conshohocken, PA, 2006.

ASTM C 136, “Standard Test Method for Sieve Analysis of Fine and Coarse Aggregates,” American Society of Testing Materials (ASTM) International, West Conshohocken, PA, 2010.

ASTM C 142, “Standard Test Method for Cay Lumps and Friable Particles in Aggregates,” American Society of Testing Materials (ASTM) International, West Conshohocken, PA, 2010.

ASTM C 29, “Standard Test Method for Bulk Density (Unit Weight) and Voids in Aggregate,” American Society of Testing Materials (ASTM) International, West Conshohocken, PA, 2009.

ASTM C 88, “Standard Test Method for Soundness of Aggregate Using Sodium Sulphate or Magnesium Sulphate,” American Society of Testing Materials (ASTM) International, West Conshohocken, PA, 2005.

ASTM D 113, “Standard Test Method for Ductility of Bituminous Materials,” American Society of Testing Materials (ASTM) International, West Conshohocken, PA, 2007.

ASTM D 1252, “Standard Test Method for Un-compacted Void Content of Fine Aggregate (as influenced by Particle Shape, Surface Texture and Grading),” American Society of Testing Materials (ASTM) International, West Conshohocken, PA, 2006.

ASTM D 2419, "Standard Test Method for Sand Equivalent Value of Soils and Fine Aggregate," American Society of Testing Materials (ASTM) International, West Conshohocken, PA, 2009.

ASTM D 2872, "Standard Test Method for Effect of Heat and Air on a Moving Film Asphalt (Rolling Thin-Film Oven Test)," American Society of Testing Materials (ASTM) International, West Conshohocken, PA, 2012.

ASTM D 36, "Standard Test Method for Softening Point of Bitumen (Ring and Ball Apparatus)," American Society of Testing Materials (ASTM) International, West Conshohocken, PA, 1995.

ASTM D 4791, "Standard Test Method for the Flat Particle, Elongated Particles, or Flat and Elongated Particles in Coarse Aggregate," American Society of Testing Materials (ASTM) International, West Conshohocken, PA, 2010.

ASTM D 5, "Standard Test Method for Penetration of Bituminous Materials," American Society of Testing Materials (ASTM) International, West Conshohocken, PA, 2013.

ASTM D 5821, "Standard Test Method for Determining the Percentage of Fractured Particles in Coarse Aggregate," American Society of Testing Materials (ASTM) International, West Conshohocken, PA, 2013.

ASTM D 6521, "Standard Practice for Accelerated Ageing of Asphalt Binder Using Pressure Ageing Vessel (PAV)," American Society of Testing Materials (ASTM) International, West Conshohocken, PA, 2013.

ASTM D 6648, "Standard Test Method for Determining the Flexural Creep Stiffness of Asphalt Binder Using Bending Beam Rheometer (BBR)," American Society of Testing Materials (ASTM) International, West Conshohocken, PA, 2008.

ASTM D 6926, "Standard Practice for Preparation of Bituminous Specimens Using Marshall Apparatus," American Society of Testing Materials (ASTM) International, West Conshohocken, PA, 2010.

ASTM D 6927, "Standard Test Method for Marshall Stability and Flow of Asphalt Mixtures," American Society of Testing Materials (ASTM) International, West Conshohocken, PA, 2010.

ASTM D 92, "Standard Test Method for Flash and Fire Points by Cleveland Open Cup Tester," American Society of Testing Materials (ASTM) International, West Conshohocken, PA, 2012.

ASTM D113, "Standard Test Method for Ductility of Bituminous Materials," ASTM International, West Conshohocken, PA, USA, 1999.

ASTM D2872, "Standard Test Method for Effect of Heat and Air on a Moving Film of Asphalt (Rolling Thin-Film Oven Test)," ASTM International, West Conshohocken, PA, USA, 2012.

ASTM D5, "Standard Test Method for Penetration of Bituminous Materials," ASTM International, West Conshohocken, PA, USA, 1997.

ASTM D88, "Standard Test Method for Saybolt Viscosity," ASTM International, West Conshohocken, PA, USA, 2013.

ASTM D92, "Standard Test Method for Flash and Fire Points by Cleveland Open Cup Tester," ASTM International, West Conshohocken, PA, USA, 2013.

Awanti, S.S., Amarnath, M.S., and Veeraragavan, A., "Influence of Rest Period on Fatigue Characteristics of SBS Polymer Modified Bituminous Concrete Mixtures," International Journal of Pavement Engineering, Volume 8, No. 3, pp. 177-186, 2007.

Azarhoosh, A. R., Hamedi, G. H., and Abandansari, H. F., "Providing Laboratory Rutting Models for Modified Asphalt Mixtures with Different Waste Materials". Periodica Polytechnica: Civil Engineering, 2017 doi:10.3311/PPci.10684.

B. Smith, S. Hesp, "Crack pinning in asphalt mastic and concrete: regular fatigue studies", Transp. Res. Rec. 1728.75–81. 2000.

B.S. Institution, "Bituminous mixtures—test methods for hot mix asphalt—resistance to fatigue", BSI London, 2004.

B.S. Underwood, "A continuum damage model for asphalt cement and asphalt mastic fatigue", Int. J. Fatigue 82, 387–401. 2016.

Bassett, C.E., "Effects Of Maximum Aggregate Size on Rutting Potential And Other Properties of Asphalt-Aggregate Mixtures," Journal of Transportation Research Board, No. 1259, pp. 107-119, 1990.

Bayat, A.R., and Knight, M., "Investigation of Hot-Mix Asphalt Dynamic Modulus by Means of Field-Measured Pavement Response," Journal of the Transportation Research Board, 2154, 138-145, 2010.

Bessa, I. S., Branco, V. T. F. C., Soares, J. B., and Neto, J. A. N., "Aggregate Shape Properties and Their Influence on the Behavior of Hot-Mix Asphalt". J. Mater. Civ. Eng., 27(7), 04014212, 2015.

Bhasin, A., Button, J., and Chowdhury, A., "Evaluation of Simple Performance Tests on Hot-Mix Asphalt Mixtures from South Central United States," Transportation Research Record. 1891, Transportation Research Board, Washington, D.C., 174–181, 2004.

Biligiri, K.; Said, S. "Prediction of the remaining fatigue life of flexible pavements using laboratory and field correlations." J. Mater. Civ. Eng, 2014. DOI: 10.1061/(ASCE)MT.1943-5533.0001161 , 04014201.

Biligiri, K.P., and Way, G.B., "Predicted E* Dynamic Moduli of the Arizona Mixtures Using Asphalt Binders Placed Over a 25-year period," Construction and Building Materials 54 (2), pp: 520–532, 2014.

Birgisson B, Roque R, Kim J, Pham L. The Use of Complex Modulus to Characterize the Performance of Asphalt Mixtures and Pavements in Florida.‖ Final Report. Report; 2004.

Bodin, D., Grenfell, J. R., & Collop, A. C., "Comparison of Small and Large Scale Wheel Tracking Devices". Road Materials and Pavement Design, 10 (sup 1), 295-325, 2009.

Breitling, R., Armengaud, P., Amtmann, A., and Herzyk, P., "Rank products: a simple, yet powerful, new method to detect differentially regulated genes in replicated microarray experiments". FEBS Letters, 573 (1-3), 82-92, 2004.

British Standard Institution, BS 812-105.1:1989. "Testing aggregates. Methods for determination of particle shape". Flakiness index", 1989

British Standard Institution, BS 812-105.2:1990. "Testing aggregates. Methods for determination of particle shape. Elongation index of coarse aggregate", 1990

British Standard Institution, EN 12697-22. "Bituminous mixtures- Test methods for hot mix asphalt – Part 22 Wheel tracking test", 2005.

British Standard Institution, EN 12697-25. "Bituminous mixtures- Test methods for hot mix asphalt – Part 25 Cyclic compression test", 2005.

Brown, S., F., "Material Characterisation for Analytical Pavement Design," Developments in Highway Pavement Engineering -1, Applied Science Publishers, London, pp 42-92, 1978.

BS DD ABF, "Method for Determination of the Fatigue Characteristics of Bituminous Mixtures using Indirect Tensile Fatigue", British Standard Institution, London, UK, 1997.

BS EN 12697-22. "Test methods for hot mix asphalt" Part-22, wheel tracking. European Standard, U.K. 2002.

Buchanan, M. S., "Evaluation of the Effect of Flat and Elongated Particles on the Performance of Hot Mix Asphalt Mixtures". Report No. 2000-03, National Center for Asphalt Technology, Auburn University, Alabama, 2000.

C. Monismith, Y. Salam, "Distress Characteristics of Asphalt Concrete Mixtures". Association of Asphalt Paving Technologists Proc, 1973.

C.L. Monismith, J.A. Deacon, "Fatigue of asphalt paving mixtures", J. Transp. Eng. Div. 95 (2) (1969) 317–346.

Cao, W., Liu, S., Li, Y., and Xue, Z., "Effect of Aggregate Gradation on Volumetric Parameters and the High Temperature Performance of Asphalt Mixtures". Proceedings of 4th Geo-China International Conference, 25 - 27 July, Shandong, China, 2016.

Cao,W. Liu, S. and Feng, Z., "Comparison of Performance of Stone Matrix Asphalt Mixtures Using Basalt and Limestone Aggregates," Construction and Building Material, 41, 474-479, 2013.

Capitão, S. and Picado-Santos, L., "Assessing Permanent Deformation Resistance of High Modulus Asphalt Mixtures". J. Transp. Eng., 132(5), 394–401, 2006.

Cavazzuti, M., "Optimization Methods: From Theory to Design Scientific and Technological Aspects in Mechanics", ISBN: 978-3-642-31186-4, Springer International Publishing, Switzerland, 2013.

Chatti, K., and El Mohtar, C.S., "Effect of Different Axle Configurations on Fatigue Life of Asphalt Concrete Mixtures," Journal of the Transportation Research Board, No. 1891, pp. 121-130, 2004.

Chaturabong, P., & Bahia, H. U. "Mechanisms of asphalt mixture rutting in the dry Hamburg Wheel Tracking test and the potential to be alternative test in measuring rutting resistance". Construction and Building Materials, 146, 175–182, 2017.

Chen, F., Chen, and Bao, H., "Comprehensive Analysis of High Temperature Performance of SBS-modified Asphalt Mixture". Journal of Wahan University of Technology, 25(6), 1038-1043, 2010.

Chen, J. S., Chang, M. K., and Lin K. Y., b. "Influence of coarse aggregate shape on the strength of asphalt concrete mixtures". Journal of the Eastern Asia Society for Transportation Studies, 6 (1): 1062 – 1075, 2005.

Chen, J. S., Wong, S. Y., and Lin, K. Y., a. "Quantification of movements of flat and elongated particles in hot mix asphalt subject to wheel load test". Materials and Structures, 3(8), 395-402, 2005.

Chen, M., Lin, J., Wu, S., and Liu, C., "Utilization of recycled brick powder as alternative filler in asphalt mixture". Construction and Building Materials 25, 1532-1536, 2011.

Chen, X., Huang, B., and Xu, Z., "Comparison between Flat Rubber Wheeled Loaded Wheel Tester and Asphalt Pavement Analyzer". Road Materials and Pavement Design, 8(3), 595-604, 2007.

Cheng, D., "Surface Free Energy of Asphalt-Aggregate System and Performance Analysis of Asphalt Concrete Based on Surface Energy," Ph.D. dissertation, Texas University, 2002.

Cheung, L. W., and Dawson, A., "Effects of Particle and Mix Characteristics on Performance of Some Granular Materials. Transportation Research Record. 1787, Transportation Research Board, National Research Council, Washington D.C, 2003.

Cho, Y.-H., Park, D.W., and Hwang, S.D.A., "Predictive Equation for Dynamic Modulus of Asphalt Mixtures used in Korea," Construction and Building Materials, 24(1), pp. 513-519, 2010.

Choubane, B., Page, G. C., and Musselman, J. A., "Suitability of Asphalt Pavement Analyzer for Predicting Pavement Rutting". Transportation Research Record. 1723, Transportation Research Board, National Research Council, Washington D.C., 107-115, 2000.

Christensen, Jr., D.W., Pellinen, T. K. and Bonaquist R. F., "Hirsch Model for Estimating the Modulus of Asphalt Concrete," J. Assoc. Asphalt Paving Technologists, 72, 97-121, 2003.

Cleff, T., "Exploratory Data Analysis in Business and Economics: An Introduction Using SPSS, Stata, and Excel", ISBN: 978-3-319-01517-0, Springer International Publishing, Switzerland, 2014.

Clyne, T.R., Li, X, Marasteanu, M.O., & Engene, K., "Dynamic modulus and Resilient Modulus of Mn/Dot Asphalt Mixtures," Report: MN/RC–2003-09: Minnesota Department of Transportation, St. Paul, MN, USA, 2003.

Collop, A., C., and Cebon, D., "A Theoretical Analysis of Fatigue Cracking in Flexible Pavements," Proceedings of Institution of Mechanical Engineers, Volume 209, pp 345-361, 1995.

Contreras, J.N., Fresno, D.C., & Zamanillo, A.V., "Dynamic Modulus of Asphalt Mixture by Ultrasonic Direct Test," NDT&E International, 43(1), 629-634, 2010.

Cooper, K.,E., and Pell, P.,S., "The Effect of Mix Variables on the Fatigue Strength of Bituminous Materials," TRRL Laboratory Report 633, Crowthorne, Berkshire, 1974.

Curtis, C., W., Ensley, K., and Epps, J., "Fundamental Properties of Asphalt-Aggregate Interactions Including Adhesion and Adsorption," SHRP-A-341, Strategic Highway Research Program, National Research Council, Washington, D.C., 1993.

Daniel, J.; Kim, Y. ”Laboratory evaluation of fatigue damage and healing of asphalt mixtures.” J. Mater. Civ. Eng., 13(6), 434–440, 2001.

Darabadi, B. K., and Taherkhani, H., “An investigation on the effects of flaky particles on the properties of asphaltic mixtures”. Canadian Journal of Civil Engineering, 42(11), 26-31, 2015.

Dehnad, M.H., Khodaii, A., and Moghadas. F., “Moisture Sensitivity of Asphalt Mixtures Under Different Load Frequencies and Temperatures,” J. Cons. and Building Mater. v. 48, p. 700-707, 2013.

Di Benedetto H, Partl M. N, Francken L, De La Roche Saint Andre C, “Stiffness Testing for Bituminous Mixtures,” Materials Structure, 34(2), pp: 66-70, 2001.

Doyle, J. D., and Howard, I. L., “Rutting and moisture damage resistance of high reclaimed asphalt pavement warm mixed asphalt: loaded wheel tracking vs. conventional methods”. Road Materials and Pavement Design, 14(sup2), 148-172, 2013.

Drescher A, Newcomb D, Zhang W. “Interpretation of indirect tension test based on viscoelasticity. Transportation Research Record”: Journal of the Transportation Research Board. (1590):45-52, 1997.

Ekblad, J., Lundstrom, R., and Simonsen, E., “Water Susceptibility of Asphalt Mixtures as Influenced by Hydraulically Active Fillers,” J. Materials and Structures, 2013. DOI 10.1617/s11527-013-0220-4.

El-Badawy, S, Bayomy, F, and Awed, A., “Performance of MEPDG Dynamic Modulus Predictive Models for Asphalt Concrete Mixtures: Local Calibration for Idaho,” Journal of Material in Civil Engineering, 24(11), 1412–1421, 2012.

Epps, A. L., Glover, J. C., and Barcena, R., “Performance graded binder specification for surface treatment,” Journal of Transportation Research Board, FHWA Texas Transportation Institute, Texas Publisher and place .11810(1):63-71, 2001.

Erlingsson, S., and Ahmed, A., “Performance prediction modelling of flexible pavement structures”. Presented in Transportation Research Arena, Paris, France, 2014.

F. Khodary Moalla Hamed, “Evaluation of Fatigue Resistance for Modified Asphalt Concrete Mixtures Based on Dissipated Energy Concept”, TU Darmstadt, 2010.

F. Wu, “Assessment of Residual Life of Bituminous Layers for the Design of Pavement Strengthening”, Polytechnic of Wales, 1992.

F. Xiao, S. Amirkhanian, C.H. Juang, "Prediction of fatigue life of rubberized asphalt concrete mixtures containing reclaimed asphalt pavement using artificial neural networks", J. Mater. Civ. Eng. 21 (6) 253–261, 2009

F. Zhou, S. Hu, T. Scullion, D.-H. Chen, X. Qi, G. Claros, "Development and verification of the overlay tester based fatigue cracking prediction approach (With discussion)", J. Assoc. Asphalt Paving Technol. 76, 2007.

Federal Highway Administration (FHWA), "Strategic Highway Research Program", Washington, D.C. USA, 1998.

Feyissa, B. A., "Analysis and Modeling of Rutting for Long Life Asphalt Concrete Pavement". PhD Thesis, Department of Civil Engineering and Geodesy, Technische Universität Darmstadt, Germany, 2009.

Fini, E. H., Al-Qadi I. L. and Masson, J. F., "A New Blister Test to Measure Bond Strength of Asphaltic Materials". Asphalt Paving Technologists, 76, pp. 275-302, December 01, 2007.

Fini, E.H., Al-Qadi, I.L., Dessouky, S. H., and Masson, J.F. "Adhesion of Bituminous Crack sealants to Aggregates". 85th Annual Meeting of the Transportation Research Board, Washington, D.C., 2006.

Flintsch, G.W., Loulizi, A, & Diefenderfer, B.K., "Asphalt Material Characterization in Support of Implementation of Mechanistic Empirical Pavement Design Guide," Virginia Transportation Research Council: Final Report, Report No. VTRC 07-CR10, 2007.

G.G. Al-Khateeb, K.A. Ghuzlan, "The combined effect of loading frequency, temperature, and stress level on the fatigue life of asphalt paving mixtures using the IDT test configuration", Int. J. Fatigue 59, 254–261, 2014.

Gandhi, T. S., "Effects of Warm Asphalt Additives on Asphalt Binder and Mixture Properties," Ph.D. dissertation, Clemson University, Clemson. Paper 203, 2008.

Gandomi, A.H., Alavi, A.H., MirzaHosseini, M.R., and Nejad, F.M., "Nonlinear Genetic-Based Models for Prediction of Flow Number of Asphalt Mixtures," Journal of Materials in Civil Engineering, Vol. 23, No. 3, pp: 248-263, 2011.

Garba, R., "Permanent Deformation Properties of Asphalt Concrete Mixtures. PhD Thesis, Norwegian University of Science and Technology NTNU, NO-7491, Trondheim, Norway, 2002.

Gedafa, D. S. Hossain,M. Romanoschi, S. and Gisi, A. J., "Field Verification of Superpave Dynamic modulus," J Material Civil Eng, 22(5), 485-494, 2009.

Ghosh, A. Padmarekha. A. and Krishnan, J. M., "Implementation and Proof-Checking of Mechanistic-Empirical Pavement Design for Indian Highways Using

AASHTOWARE Pavement ME Design Software," 2nd Conference of Transportation Research Group of India, Procedia - Social and Behavioral Sciences,104, 119 – 128, 2013.

Ghuzlan, K., A., "Fatigue Damage Analysis in Asphalt Concrete Mixtures Based upon Dissipated Energy Concepts," Ph.D. dissertation. University of Illinois at Urbana-Champaign, 2001.

Golalipour, A., Jamshedi, E., Niazi, Y., Afsharikia, Z., and Khadem, M., "Effect of Aggregate Gradation on Rutting of Asphalt Pavements". Procedia-Social and Behavioral Sciences, 53, 440-449, 2012.

Gul, W. A., "Effect of Recycled Cement Concrete Content on Rutting Behavior of Asphalt Concrete". MSc Thesis submitted to Graduate School of Natural And Applied Sciences, Middle East Technical University, Turkey, 2008.

Guo, R., & Prozzi, J. A. "A Statistical analysis of Hamburg wheel tracking device testing results". Road Materials and Pavement Design, 10(January 2015), 327–335, 2009. https://doi.org/10.1080/14680629.2009.9690249.

H. Salama, K. Chatti, "Evaluation of fatigue and rut damage prediction methods for asphalt concrete pavements subjected to multiple axle loads", Int. J. Pavement Eng. 12 (1) 25–36, 2011

H.-J. Lee, J.S. Daniel, Y.R. Kim, "Continuum damage mechanics-based fatigue model of asphalt concrete", J. Mater. Civ. Eng. 12 (2) 105–112, 2002.

H.-J. Lee, Y.R. Kim, "Viscoelastic constitutive model for asphalt concrete under cyclic loading", J. Eng. Mech. 124 (1) 32–40, 1998.

Hafeez, I., 'Impact of Hot Mix Asphalt Properties on its Permanent Deformation Behavior". Thesis (PhD). University of Engineering and Technology, Taxila, Pakistan, 2009.

Hafeez, I.; Kamal, M.A. "An experimental based approach to predict asphalt mixtures permanent deformation behaviour". The Arabian Journal of Sciences and Engineering. 39(12):8681¬–8690, 2014. DOI:10.1007/s13369-014-1453-4.

Hafeez, I.; Kamal, M.A. "Creep compliance: a parameter to predict rut performance of asphalt binder and mixtures". Arabian Journal of Sciences and Engineering. 39 (8): 5971-5978, 2014. DOI: 10.1007/s13369-014-1216-2.

Hafeez, I.; Kamal, M.A.; Mirza, M.W.; Barkatullah; Bilal, S. "Laboratory fatigue performance evaluation of different field laid asphalt mixtures". J. Constr. and Build. Mater. 44(2013) 792-797. 2013. DOI: 10.1016/j.conbuildmat.2013.03.083

Hafeez, I.; Kamal, M.A.; Mirza. M.W. “An experimental study to select aggregate gradation of stone mastic asphalt”, J. Chinese Institute Eng, 2014. DOI:10.1080/02533839.2014.953242.1-8.

Hafeez, I.; Kamal, M.A.; Waseem, M.W. “Evaluation of Rutting in HMA Mixtures Using Uniaxial Repeated Creep & Wheel Tracker Tests.” Pak. J. Engg. & Appl. Sci. 7 (2010): 55–64, 2010.

Hafeez, I.; Ozer, H.; Al-Qadi, I.L. “Performance characterization of hot in-place recycled mixtures.” Journal of Transportation Engineering, 140(8);, 2014. DOI: 10.1061/(ASCE)TE.1943-5436.0000679.

Hamed, F.K.M., “Evaluation of fatigue resistance for modified asphalt concrete mixtures based on dissipated energy concept,” Ph.D. Thesis, Technische University Darmstadt, Darmstadt, Germany, 2010.

Haritonovs, V.; Zaumanis, M.; Brencis, G.; Smirnovs, J.; “Performance of asphalt concrete with dolomite sand waste and basic oxygen furnace steel slag aggregate.” The Baltic J. Road Bridge Eng. 8 (2):91–97, 2013. DOI: 10.3846/bjrbe.2013.12.

Hartman, A.M., Gilchrist, M.D. and Walsh, G., “Effect of Mixture Compaction on Indirect Tensile Stiffness and Fatigue,” Journal of Transportation Engineering, September/ October, pp. 370-378, 2001.

Hassan HF, & Al-Jabri K., “Laboratory Evaluation of Hot-Mix Asphalt Concrete Containing Copper Slag Aggregate,” Journal of Material in Civil Engineering, 23(6), 879-885, 2011.

Hesami, E., Birgisson, B., and Kringos, N., “Effect of mixing sequence on the workability and performance of asphalt mixtures”. Road Materials and Pavement Design, 16(sup2), 197-213. 2015. Doi: 10.1080/14680629.2015.1077008.

Hicks, R.G. NCHRP Synthesis of Highway Practice 175: “Moisture Damage in Asphalt Concrete”. Transportation Research Board, National Research Council. Washington, D.C, 1991.

Hofko, B., Blaboratory, R., and Alisov, A., “Influence of compaction direction on performance characteristics of roller-compacted HMA specimens”. International Journal of Pavement Engineering 17(1), 39-49, 2016.

Hossain, M. M., Majumder, A. K., and Basak, T., “An Application of Non-Linear Cobb-Douglas Production Function to Selected Manufacturing Industries in Bangladesh”. Open Journal of Statistics, 2, 460-468, 2012. Doi: 10.4236/ojs.2012.24058.

Hossain, N., Singh, D., and Zaman, M. “Dynamic Modulus-Based Field Rut Prediction Model from an Instrumented Pavement Section,” Procedia - Social and Behavioral Sciences 104, pp: 129 – 138, 2013.

Hu, S., Zhou, F., and Scullion, T., “Development, Calibration, and Validation of a New M-E Rutting Model for HMA Overlay Design and Analysis,” J. Mater. Civ. Eng., 23(2), 89-99, 2011.

Huang, X., and Zhang, Y., “A New Creep Test Method for Asphalt Mixtures”. Road Materials and Pavement Design, 11(4), 969-991, 2010.

Huang, Y.H., “Pavement Analysis and Design,” Pearson Education, Inc. 2004.

Hughes, D., Kamal, M. A., and Hafeez, I., “Time Dependent Volumetric Behaviour of Flexible Pavements under Heavy Loading and High Temperature”. Proceedings of first International Conference on Transportation Geotechics, Nottingham, United Kingdom, 523-527, 2008.

Hughes, D., Kamal, M. A., and Sulehri, M. A., “Engineering Characteristics of Road Aggregates from Northern Pakistan and the Development of a Toughness Index”. Geotechnical and Geological Engineering, 24(4), 819-831, 2006.

Hussain A, Tahir HB, Sheikh MZ, Waheed A, Khurshid MB. “Evaluation of resilient modulus of HMA mixtures from highway construction projects in Pakistan”. Journal of Basic and Applied Research International; 17(4):290-301, 2016.

Hussan, S., Kamal, M. A., Hafeez, I., and Ahmad, N., b. “Comparing and correlating various laboratory rutting performance tests”. International Journal of Pavement Engineering, 2017. doi: 10.1080/10298436.2017. 1402591.

Hussan, S., Kamal, M. A., Hafeez, I., Farooq, D., Ahmad, N., & Khanzada, S. “Statistical evaluation of factors affecting the laboratory rutting susceptibility of asphalt mixtures”. International Journal of Pavement Engineering, 8436(March), 1–15. 2017. https://doi.org/10.1080/10298436.2017.1299527.

Hussan, S., Kamal, M. A., Hafeez, I., Farooq, D., Ahmad, N., and Khanzada, S., a. “Statistical evaluation of factors affecting the laboratory rutting susceptibility of asphalt mixtures”. International Journal of Pavement Engineering, 2017. doi: 10.1080/10298436.2017.1299527.

I. Yeo, Y. Suh, S. Mun, “Development of a remaining fatigue life model for asphalt black base through accelerated pavement testing”, Constr. Build. Mater. 22 (8) 1881–1886, 2008.

Irfan, M., Ali, Y., Iqbal, S., Ahmed, S., and Hafeez, I., “Rutting Evaluation of Asphalt Mixtures Using Static, Dynamic, and Repeated Creep Load Tests”.

Arabian Journal of Science and Engineering. 2017. Doi: 10.1007/s13369-017-2982-4.

Irwin, L., H., "Use of Fracture Energy as a Fatigue Failure Criterion," Proceedings of the Association of Asphalt Paving Technologists (AAPT), Volume 46, pp 41, 63, 1977.

Iwama, M., "Influence of Specimen Size and Orientation on the Mechanical Properties of Laboratory Compacted Asphalt Specimens". MSc Thesis, Department of Civil Engineering Nottingham Transportation Engineering Centre, The University of Nottingham, Nottingham, UK, 2009.

J. Harvey, B.-W. Tsai, "Effects of asphalt content and air void content on mix fatigue and stiffness", Transp. Res. Rec. 1543, 38–45, 1996.

J.A. Epps, C.L. Monismith, "Fatigue of asphalt concrete mixtures-summary of existing information. Fatigue of compacted bituminous aggregate mixtures", ASTM STP 508, 19–45, 1972.

J.M. Read, "Fatigue Cracking of Bituminous Paving Mixtures", University of Nottingham England, 1996.

Jackson, N. M., and Baldwin, C. D., "Assessing the Relative Rutting Susceptibility of HMA in the Laboratory with the Asphalt Pavement Analyzer". International Journal of Pavement Engineering, 1(3), 203-217, 2000.

Jahromi, S., G., and Khodaii, A., "Compairing Factors Affecting Resilient Modulus in Asphalt Mixtures," Journal of Scientia Iranic, Vol. 6, pp. 1302-1312, 2009.

Jamieson, I., L., Jones, D., R. and Moulthrop, J., S., "Advances in the Understanding of Binder-Aggregate Adhesion and Resistance to Stripping," Highways and Transport, January, 1993.

Jeong, K., Hwang, S., Lee, S., and Kim, K. W., "Investigation of Rutting Potential of Open Graded Friction Course OGFC Mixtures Using Asphalt Pavement Analyzer". KSCE Journal of Civil Engineering, 15, 1259-1262, 2011.

Ji, X., Zheng, N., Niu, S., Meng, S., and Xu, Q. "Development of a rutting prediction model for asphalt pavements with the use of an accelerated loading facility". Road Materials and Pavement Design, 17(1), 15-31, 2016.

Jia, X., Huang, B., Moore, J. A., and Zhao, S., "Influence of Waste Engine Oil on Asphalt Mixtures Containing Reclaimed Asphalt Pavement". J. Mater. Civ. Eng., 27(12), 04015042, 2015.

JULIE E. NODES. "Relationship of Air Voids, Lift Thickness, and Permeability in Hot-Mix Asphalt Pavements", 2004. https://doi.org/10.17226/13777.

K. Ghuzlan, S. Carpenter, “Energy-derived, damage-based failure criterion for fatigue testing”, Transp. Res. Rec. 1723, 141–149, 2000.

K.A. Ghuzlan, “Fatigue Damage Analysis in Asphalt Concrete Mixtures Based Upon Dissipated Energy Concepts”, University of Illinois at Urbana-Champaign, 2001.

Kaloush, K.E., “Simple Performance Test for Permanent Deformation of Asphalt Mixtures,” Ph.D. dissertation, Department of Civil and Environmental Engineering, Arizona State University, Tempe, AZ, 2001.

Kamal, M., A., Shazib, F., and Yasin, B., “Resilient Behaviour of Asphalt Concrete under Repeated Loading & Effects of Temperature,” Journal of the Eastern Asia Society for Transportation Studies, Vol. 6, pp. 1329-1343, 2005.

Kandhal, P. S., Parker, F., and Manglorkar, D. A., “Evaluation of Alabama limestone aggregates for asphalt wearing courses’. Transportation Research Record 1418, 12-21, 1993.

Kanitpong, K., and Bahia, H. “Relating Adhesion and Cohesion of Asphalts to Effect of Moisture on Asphalt Mixtures Laboratory Performance”. 84th Annual Meeting of the Transportation Research Board, Washington, D.C., 2005.

Kennedy, T., W., “Prevention of Water Damage in Asphalt Mixtures," STP 899, American Society for Testing and Materials, pp 119-133, Philadelphia, 1985.

Kennedy, T.W., “Characterization of Asphalt Pavement Materials Using the Indirect Tensile Test,” Proceedings of the Association of Asphalt Paving Technologists (AAPT), Volume 46, pp 132-150, 1977.

Khan, M. K. “Performance Evaluation of Moisture Damage in Asphalt Mixtures”, 2017.

Khan, M. K. “PERFORMANCE EVALUATION OF MOISTURE DAMAGE IN ASPHALT MIXTURES”, a thesis submitted in partial fulfillment of the requirements for the degree of Master of Science in, 2017.

Khan, M.B., Khattak, A., Irfan, M., and Ahmed S., “Factorial Design Approach to Investigate the Effect of Different Factors on the Resilient Modulus of Bitumenous Paving Mixtures,” ARPNJournal of Science and Technolology, Vol. 2, No.11, 2012.

Khattab, A., M. El-Badawy, S., M. Al Hazmi, A., Elmwafi, M., “Evaluation of Witczak E* Predictive Models for the Implementation of AASHTOWare-Pavement ME Design in the Kingdom of Saudi Arabia,” Construction and Building Materials 64, 360–369, 2014.

Khattak, M., J., Baladi, G., Y., “Fatigue and Permanent Deformation Models for Polymer-Modified Asphalt Mixtures,” Journal of the Transportation Research Board 1767, pp. 135-145, 2001.

Khedaywi, T. S. and E. Tons. "Aggregate Rugosity and Size Effect on Bituminous Mixes." Transportation Research Record 1619(1): 26-36, 1998.

Khedr, S. A., and Breakah, T. M., “Rutting parameters for asphalt concrete for different aggregate structures”. International Journal of Pavement Engineering, 12(1), 13-23, 2011.

Khodaii, H.F., Haghshenas, H., Kazemi, T., and Khedmati. M., “Application of Response Surface Methodology to Evaluate Stone Matrix Asphalt Stripping Potential. Korean Society of Civil Engineers (KSCE) Journal of Civil Engineering v. 17, p. 117-121, 2013.

Kim Y, Seo Y, King M, Momen M. “Dynamic modulus testing of asphalt concrete in indirect tension mode”. Transportation Research Record: Journal of the Transportation Research Board. (1891):163-73, 2004

Kim, Y. K., Park, H. M., Aragao, F. T. S., and Lutif, J. E. F., “Effects of aggregate structure on hot-mix asphalt rutting performance in low traffic volume local pavements”. Construction and Building Materials 23, 2177-2182, 2009.

Kim, Y. R, “Modelling of Asphalt Concrete”, 1st ed. ASCE press USA: McGraw-Hill Construction, (2009

Kringos, N., and Scarpas, A., “Ravelling of Asphalt Mixes due to Water Damage: computational identification of controlling parameters”. Transportation Research Record: Journal of the Transportation Research Board, No. 1929, Transportation Research Board of the National Academics, Washington, D.C., 2005, pp. 79-87, 2005.

Kvasnak, A., Robinette, C.J., and Williams, R.C., “Statistical Development of a Flow Number Predictive Equation for the Mechanistic-Empirical Pavement Design Guide,” TRB 86th Annual Meeting Compendium of Papers, Paper No. 07-1000 (CD-ROM), Transportation Research Board, Washington, DC, 18, 2008.

Lai, J. S., and Shami, H., “Development of rolling compaction machine for preparation of asphalt beam samples”. Transportation Research Record 1492, 18-25, 1995.

Lee, K. Kim, H. Kim, N. and Kim, Y. "Dynamic Modulus of Asphalt Mixtures for Development of Korean Pavement Design Guide," ASTM J Test Eval, 35(2), 28-59, 2002.

Lenz,R.W., “Pavement Design Guide.” 2011 <http://onlinemanuals.txdot.gov/txdotmanuals/pdm/index.htm>

Li, Q., Ni, F., Gao, L., Yuan, Q., and Xiao, Y., “Evaluating the rutting resistance of asphalt mixtures using an advanced repeated load permanent deformation test under field conditions”. Construction and Building Materials 61, 241-251, 2014.

Li. J., Zofka. A., and Yut. I., “Evaluation of Dynamic Modulus of Typical Asphalt Mixtures in Northeast US Region,” Road Materials and Pavement Design. 13 (2), 249–265, 2012.

Lim, C., T., Tan, S., A., and Fwa, T., F., “Specimen Size Effects on the Diametrical Mechanical Testing of Cylindrical Asphalt Mixtures,” Journal of Testing and Evaluation (JTE), Vol. 23, pp. 436-441, 1995.

Little, D., Button, J., Jayawickrama, P., Solaimanian, M., and Hudson, B., “Quantify shape, angularity and surface texture of aggregates using image analysis and study their effect on performance”. Report 0-1707-4, Texas Department of Transportation Research and Technology, Implementation Office, Austin, Texas, 2003.

Liu, H., Hao, P., and Xu, J., “Effects of Nominal Maximum Aggregate Size on the Performance of Stone Matrix Asphalt”. Applied Sciences, 7(2): 126, 2017.

Liu, J., Zhao, S., Li, P., Saboundjian, S. “Variability of composition, volumetric, and mechanic properties of hot mix asphalt for quality assurance”. J. Mater. Civ. Eng. 29 (3). D4015004, 2017.

Lottman, P.R., “Laboratory Test Method for Predicting Moisture – Induce Damage to Asphalt Concrete,” J Transport Res. Rec. p. TRB 843, 1982.

Loulizi A, Flintsch G, Al-Qadi I, Mokarem D. “Comparing resilient modulus and dynamic modulus of hot-mix asphalt as material properties for flexible pavement design”. Transportation Research Record: Journal of the Transportation Research Board. (1970):161-70, 2006.

Loulizi, A., Al-Qadi, I., L., Lahouar, S., and Freeman, T.E., “Measurement of Vertical Compressive Stress Pulse in Flexible Pavements: Representation of Dynamic Loading Tests,” Journal of Transportation Research Board, No. 1816, pp. 125-136, 2002.

Loulizi, A., Flintsch, G.W., Al-Qadi, I.L., and Mokarem, D., “Compairing Resilient Modulus and Dynamic Modulus of Hot-Mix Asphalt as Material Properties for Flexible Pavement Design,” Journal of Transportation Research Board, No. 1970, pp. 161-170, 2006.

Lu, Q., and Harvey, J.T. “Inclusion of Moisture Effect in Fatigue Test for Asphalt Pavements”. 86th Annual Meeting of the Transportation Research Board, Washington, D.C., 2007.

Lytton R.L., Uzan J., Fernado E.G., Rogue R. Hiltumen D and Stoffels S.M., “Development and Validation of Performance Prediction Model and Specifications for Asphalt Binders and Paving Mixtures,” Report Number A357, Strategic Highway Research Program (SHRP), 1994.

M. Irfan, A.S. Waraich, S. Ahmed, Y. Ali, “Characterization of various plantproduced asphalt concrete mixtures using dynamic modulus test, Adv. Mater. Sci. Eng, 2016.

M. Irfan, M. Saeed, S. Ahmed, Y. Ali, “Performance evaluation of elvaloy as a fuel-resistant polymer in asphaltic concrete airfield pavements”, J. Mater. Civ. Eng. 29 (10) 04017163, 2017.

M. Irfan, Y. Ali, S. Ahmed, I. Hafeez, “Performance evaluation of crumb rubbermodified asphalt mixtures based on laboratory and field investigations”, Arab. J. Sci. Eng. 43 (4) 1795–1806, 2018.

M. Irfan, Y. Ali, S. Iqbal, S. Ahmed, I. Hafeez, “Rutting evaluation of asphalt mixtures using static, dynamic, and repeated creep load tests”, Arab. J. Sci. Eng. 1–13, 2017.

M. Junaid, M. Irfan, S. Ahmed, Y. Ali, “Effect of binder grade on performance parameters of asphaltic concrete paving mixtures’, Int. J. Pavement Res. Technol, 2017.

Ma, T., Zhang, D., Zhang, Y., Zhao, Y., and Huang, X., “Effect of air voids on the high-temperature creep behavior of asphalt mixture based on three-dimensional discrete element modeling”. Materials and Design 89, 304-313, 2016.

Mahmood Ahmad, Imran Hafeez “Permeability of asphalt mixtures with Bailey and Conventional aggregate gradations”. Arabian Journal for Science and Engineering, 2021. https://doi.org/10.1007/s13369-020-05300-0."

Mahmood, M., Rahman, M., and Mathavan, S., “Distress Based Pavement Performance Prediction Models”. Proceedings of 8th International Conference on Maintenance and Rehabilitation of Pavements (MAIREPAV8), 27 - 29 July, Singapore, 2016.

Mahmud, M.Z.H., et al., “Laboratory investigation on the effects of flaky aggregates on dynamic creep and resilient modulus of asphalt mixtures”. Jurnal Teknologi, 70 (4), 107–110, 2014.

Manual Series No. 24 (MS-24), "Moisture Sensitivity: Best Practices to Minimize Moisture Sensitivity in Asphalt Mixtures", Asphalt Institute, USA, 2007.

Marasteanu, M., Li, X., Clyne, T.R., Voller, V.R., Timm, D. H., & Newcomb, D.E., "Low Temperature Cracking of Asphalt Concrete Pavements: Final Report (Report No. MN/RC-2004-23)," Retrieved From Local Road Research Board: 2004 http://www.lrrb.org/PDF/200423.pdf.

Mehrara, A., and Khodaii., A., "A Review of State of the Art on Stripping Phenomenon in Asphalt Concrete," J. Cons. Building Mater. v.48, p.700-707, 2012.

Miller and Bellinger, "Distress Identification Manual for the Long-Term Pavement Performance Program," Strategic Highway Research Program (SHRP), Report No. 645, 2003.

Mirzahosseini, M., Najjar, Y. N., Alavi, A. H., and Gandomi, A. H., "ANN-Based Prediction Model for Rutting Propensity of Asphalt Mixtures". presented in Transportation Research Board 92nd Annual Meeting, Washington D.C. 2013.

Mirzahosseini, S., Aghaeifar, A., Alavi, A. H., Gandomi, A. H., and Seyednour, R., "Permanent deformation analysis of asphalt mixtures using soft computing techniques". Expert Systems with Applications 38, 6081-6100, 2011.

Mishra, D.; Tutumuler, E. "Aggregate physical properties affecting modulus and deformation characteristics of unsurfaced pavement." J. Mater. in civil eng. 24 (9):1144–1152, 2012. DOI:10.1061/(ASCE)MT.1943-5533.0000498.

Mitchell, M.; Link, R.; Punith, V.; Raju, S.; Kumar, K.K.; Boes, S.; Veeraragaven, A. "Laboratory evaluation of stone matrix asphalt mixtures with polyethylene and cellulose stabilizers." J. Testing and Evaluation , 2011. DOI. 10.1520/JTE102919, 102919.

Mohammad, L., Abadie, C., Gokmen, R., and Puppala, A., "Mechanistic Evaluation of Hydrated Lime in Hot-Mix Asphalt Mixtures," Transportation Research Record 1723. Washington D.C.: National Academy Press, 2000.

Mohammad, L., Wu, Z., Obularreddy, S., Copper, S., and Abadie, C., "Permanent Deformation Analysis of Hot-Mix Asphalt Mixtures with Simple Performance Test and 2002 Mechanistic-Empirical Pavement Design Software," Journal of Transportation Research Record, pp-142, 2006.

Monismith C.L., Epps J.A., and Finn F.N., "Improved Asphalt Mix Design," Asphalt Pavement Technology, Volume 54, pp 347–406, 1985.

Monismith, C., L., and Salam, Y., M., "Distress Characteristics of Asphalt Concrete Mixtures," Proceedings of the Association of Asphalt Paving Technologists (AAPT), Volume 42, pp 320-349, 1979.

National Cooperative Highway Research Program (NCHRP), "Guide for mechanistic-empirical design of new and rehabilitated pavement structures," NCHRP Final report 1-37A, TRB; Washington DC, USA, 2004.

National Cooperative Highway Research Program (NCHRP), "Ruggedness Testing of the Dynamic Modulus and Flow Number Tests with the Simple Performance Tester," Report 629, Transportation Research Board, Washington, D.C., 2008.

NCHRP, "Validating the Fatigue Endurance Limit for Hot Mix Asphalt (HMA)," National Cooperative Highway Research Program, Transportation Research Board, Washington, D.C., 2010.

NHA, General Specification, NHA Headquarters 27-Mauve Area, G-9/1, Islamabad, Pakistan, 1998.

Pan, T., Tutumluer, E., and Carpenter, S.H., "Effect of Coarse Aggregate Morphology on the Resilient Modulus of Hot Mix Asphalt," Journal of Transportation Research Board, No. 1929, pp. 1-9, 2005.

Parker, F., and Peng, J., "An evaluation of Alabama aggregate properties for asphalt mixtures". Final Report Project Number 930-464, National Center for Asphalt Technology Auburn, Alabama, 2004.

Petersen, J., C., "Chemical Composition of Asphalt as Related to Asphalt Durability: State of the Art," TRR 999, pp 13-30, Transportation Research Board, Washington D.C., 1984.

Petersen, J., C., Plancher, H., Ensley, E., K., Venable, R., L. and Miyake, G., "Chemistry of Asphalt-Aggregate Interaction: Relationship with Pavement Moisture-Damage Prediction Test," TRR 843, pp 95-104, Transportation Research Board, Washington, D.C., 1982.

Ping WV, Xiao Y. "Empirical correlation of indirect tension resilient modulus and complex modulus test results for asphalt concrete mixtures". Road Materials and Pavement Design. 2008. 9(sup1):177-200.10.1080/14680629.2008.9690165.

PutraJaya, R., AbdulHassan, N., Mahmud, M.Z.H., Aziz, M.A., Hamzah, M.O., and Chewan, C.N., "Effect of aggregate shape on the properties of asphaltic concrete AC14". Jurnal Teknologi, 71 (3), 69-73, 2014.

Qasim, Z. I., Abbas, A. S., and Qasim, Z. I., "Effect of Filler Content on Properties of Asphaltic Mixtures for Marshall and Superpave Gyratory compactor". Al-Nahrain Journal for Engineering Sciences, 20(1), 183-193, 2017.

Qiao, Y., Dawson, A., Huvstig, A., and Tanttu, L. K., "Calculating rutting of some thin flexible pavements from repeated load triaxial test data". International Journal of Pavement Engineering, 16(6), 467-476, 2015.

Qiao, Y., Flintsch, G., Dawson, A., and Parry, T., "Examining Effects of Climatic Factors on Flexible Pavement Performance and Service Life". Transportation Research Record. 2349, Transportation Research Board, National Research Council, Washington D.C., 100-107, 2013.

R. Hicks, F. Finn, C. Monismith, R. Leahy, "Validation of SHRP binder specification through mix testing (with discussion)", J. Assoc. Asphalt Paving Technol. 62, 1993.

R. Reese, "Properties of aged asphalt binder related to asphalt concrete fatigue life", J. Assoc. Asphalt Paving Technol. 66,1997.

R.K.A. Al-Rub, M.K. Darabi, D.N. Little, E.A. Masad, "A micro-damage healing model that improves prediction of fatigue life in asphalt mixes", Int. J. Eng. Sci. 48 (11), 966–990, 2010.

R.L. Lytton, J. Uzan, E.G. Fernando, R. Roque, D. Hiltunen, S.M. Stoffels, "Development and validation of performance prediction models and specifications for asphalt binders and paving mixtures": Strategic Highway Research Program, 1993.

R.M. Rodrigues, "A model for fatigue cracking prediction of asphalt pavements based on mixture bonding energy", Int. J. Pavement Eng. 1 (2), 133–149, 2000

Ramli, I., Yaacob, H., Abdul Hassan, N., Ismail, C. R., and Hainin, M. R., "Fine Aggregate Angularity Effects on Rutting Resistance of Asphalt Mixture". Jurnal Teknologi, 65 (3), 105–109, 2013.

Read, J. and Whiteoak, D., "The Shell Bitumen Handbook, 5th ed. London: Thomas Telford Ltd, 2003.

Remisova, E., "Study of mineral filler effect on asphalt mixtures properties". Proceedings of 6th International Conference 'Bituminous Mixtures and Pavements', 10-12 June, Thessaloniki, Greece, 2015.

Robert, N.H., "Asphalts in Road Construction." Thomas Telford Publishing", London, 2000.

Roberts, F. L.; Kandhal, P.S.; Brown, E.R.; Lee, D.Y.; Kennedy, T.W. "Hot mix-asphalt materials, mixture design, and construction", 2nd Edition, NAPA Research and Education Foundation, Lanham, M.D, 1996.

Roberts, F.E., “Hot Mix Asphalt Materials,” Mixture Design, and Construction.Lanham, Maryland: National Asphalt Pavememnt Association Research and Education Foundation. v. 21, p. 809-815, 2009.

Rodeznò, M.C., Kaloush, K.E., and Corrigan, M.R., “Development of a Flow Number Predictive Model,” Transportation Research Board of the National Academies, No.2181, Transportation Research Record, Washington, DC, 79–87, 2010.

Rodrigues, R.M., “A Model for Fatigue Cracking Prediction of Asphalt Pavements Based on Mixture Bonding Energy,” International Journal of Pavement Engineering, Volume 1, No. 2, pp. 133-149, 2000.

Romero, P,. and Stuart, K., “Evaluating Accelerated Rut Testers,” Public Roads, V. 62, Federal Highway Administration, U.S. Department of Transportation, July/August, 1998.

Roy, N., Veeraragavan, A., & Krishnan, J. M., “Influence of confinement pressure and air voids on the repeated creep and recovery of asphalt concrete mixtures”. International Journal of Pavement Engineering, 17(2), 133-147, 2016.

Rushing, J. F., Little, D. N., and Garg, N., “Using the Asphalt Pavement Analyzer to Assess Rutting Susceptibility of HMA Designed for High Tire Pressure Aircraft”. Presented in Transportation Research Board 91st Annual Meeting, Washington D.C. 2012.

Rushing, J. F., Little, D. N., and Garg, N., “Selecting a rutting performance test for airport asphalt mixture design”. Road Materials and Pavement Design, 15(sup1), 172-194, 2014.

Rushing, J. and Little, D. “Static Creep and Repeated Load as Rutting Performance Tests for Airport HMA Mix Design”. J. Mater. Civ. Eng., 26(9), 04014055, 2014.

Rushing, J., and Little, D., “Static Creep and Repeated Load as Rutting Performance Tests for Airport HMA Mix Design.” J. Mater. Civ. Eng., 26(9), 04014055, 2014.

S. Brown, Material characteristics for analytical pavement design, Dev. Highway Pav. Eng. 1 (1978) 41–92.

Saal, R.N.J., and Pell, P.S., “Kolloid Zeitschrift (Darmstadt),” KZZPA, Vol. 171, 1960.

Salama, H. Chatti, K., “Evaluation of Fatigue and Rut Damage Prediction Methods for Asphalt Concrete Pavements Subjected to Multiple Axle Loads,” International Journal of Pavement Engineering, Volume 12, No. 1, pp. 25-36, 2011.

Saleh, M. F., and Ji, S.J., "Experimental Analysis of Factors Affecting Resilient Modulus of the Hot Mix Asphalt," Australian Road Research Board, 12p, 2006.

Saleh, M. F., and Jian, S.J., "Investigation of Factors Affecting Resilient Modulus for Hot Mix Asphalt," University of Canterbury, Civil Engineering, 2006.

Santucci. L. "Moisture Sensitivity of Asphalt Pavements": technology transfer program. Institute of Transportation Studies, UC Berkeley Institute of Transportation Studies, USA, 2002.

Scholz, T.V., Terrel, R.L., Joaib, A., and Bea, J., "Water Sensitivity: Binder Validation National Research Council. SHRP-A-402. Washington (DC), 1994.

Sel, I., Yildirim, Y., and Ozhan, H. "Effect of Test Temperature on Hamburg Wheel-Tracking Device Testing". J. Mater. Civ. Eng., 26(8), 04014037, 2014.

Shafiei, A., and Namin, M. L., "Experimental Investigation on the Effect of Hydrated Lime on Mechanical Properties of SMA," Construction and Building Materials 70, 379-387, 2014.

Shahadan, Z, Hamzah, M. O, Yahya, A. S. and Jamshidi, A, "Evaluation of the dynamic modulus of asphalt mixtures incorporating reclaimed asphalt pavements", Indian J Eng. Mater. Sci., 20 376-384, 2013.

Shang G.T.; Takahashi, O.; Maekawa, R. "Recommended combination of the bailey parameters in Superpave gradation design for Japanese airfield pavements". Int. J. Pavement Res. Technol. 6(6):704-713. 2013. DOI. 10.6135/ijprt.org.tw/2013.6(6).704.

Shaw, P.S., "Stress-Strain Relationships for Granular Materials under Repeated Loading," PhD Thesis, Department of Civil Engineering, University of Nottingham, 1980.

Shell SPDM-A, pavements and Overlays for Road Traffic, Shell International Petroleum, London, 1978

Shen,S.; Yu, H. "Analysis of aggregate gradation and packing for easy estimation of hot-mix-asphalt voids in mineral aggregate." J. Mater. Civ. Eng., 23: 664–672, 2011.

Shu, X and Huang, B., "Micromechanics-based dynamic modulus prediction of polymeric asphalt concrete mixtures," Composites: Part B. 39(4), 704-174, 2008.

Singh, D.; Zaman, M.; Commuri, S. "Inclusion of aggregate angularity, texture, and form in estimating dynamic modulus of asphalt mixtures." Road Mater. Pavement Des. , 2012. DOI:10.1080/14680629.2011.650088, 327-344.

Suh, Y. C., and Choand, N. M., "Development of a Rutting Performance Model for Asphalt Concrete Pavement based on Test Road and Accelerated Pavement Test Data". KSCE Journal of Civil Engineering, 18(1), 165-171, 2014.

Suh, Y., Cho, N., and Mun, S., "Development of mechanistic-empirical design method for an asphalt pavement rutting model using APT". Construction and Building Materials 25, 1685-1690, 2011.

T.W. Kennedy, "Characterization of Asphalt Pavement Materials Using the Indirect Tensile Test", Association of Asphalt Paving Technologists Proc, 1977.

Tapkin, S., and Keskin, M., "Rutting analysis of 100 mm diameter polypropylene modified asphalt specimens using gyratory and Marshall compactors". Materials Research, 16(2), 2013.

Tapkin, S., Cevik, A., and Usar, U., "Accumulated strain prediction of polypropylene modified marshall samples in repeated creep test using artificial neural networks". Expert Systems with Applications 36, 11186-11197, 2009.

Tapkin, S., Cevik, A., and Uzcan, S., "Utilising neural networks and closed form solutions to determine static creep behaviour and optimal polypropylene amount in bituminous mixtures". Materials Research, 15(6), 865-883, 2012.

Tapkin, S., Cevik, A., Usar, U., and Gulsan, E., "Rutting Prediction of Asphalt Mixtures Modified by Polypropylene Fibers via Repeated Creep Testing by Utilising Genetic Programming". Materials Research, 16(2), 277-292, 2013.

Tarefder, R. A., & Ahmad, M. "Evaluation of pore structure and its influence on permeability and moisture damage in asphalt concrete". International Journal of Pavement Engineering, 18(3), 274–283. 2017. https://doi.org/10.1080/10298436.2015.1065995.

Tayfur, S., Ozen, H., and Aksoy, A., "Investigation of rutting performance of asphalt mixtures containing polymer modifiers". Construction and Building Materials 21, 328-337, 2007.

Terrel, R., L., and Shute, J., W., "Summary Report on Water Sensitivity," SHRP-A/IR-89-003, Strategic Highway Research Program, National Research Council, Washington, D.C., 1989.

Tjan, A., and Napitupulu, R., "Verification of Resilient Modulus Prediction of Asphalt Mixtures," Proceeding of the Eastern Asian Society for Transportation Studies, Vol. 9, 2013.

Topal, A., and Sengoz, B., "Determination of fine aggregate angularity in relation with the resistance to rutting of hot-mix asphalt". Construction and Building Materials 19, 155-163, 2005.

Transportation Research Board. “A mix design manual for hot-mix asphalt”. NCHRP Rep. No. 673, National Research Council, Washington, DC, 2009.

Transportation Research Board: “Bailey method for gradation selection in hot-mix asphalt mixture design”. Transportation Research Circular No. E-C044, Washington, D.C, 2002.

U.A. Mannan, M.R. Islam, R.A. Tarefder, “Effects of recycled asphalt pavements on the fatigue life of asphalt under different strain levels and loading frequencies”, Int. J. Fatigue 78, 72–80, 2015.

Ullidtz, P., “Pavement Analysis,” Elsevier, New York, N.Y, USA, 1987.

Uzarowski, L., Paradis, M., and Lum, P., “Accelerated Performance Testing of Canadian Asphalt Mixtures Using Three Different Wheel Rut Testers”. Presented at the Accelerated Field and Laboratory Pavement Testing Session of the Annual Conference of the Transportation Association of Canada, Quebec City, Canada, 2004.

Van der Poel., C., “A General System Describing the Viscoelastic Properties of Bitumen and its Relation to Routine,” Test Journal of Applied Chemistry Volume 4, pp.221-232, 1954.

Vasconcelos, K.L.; Bhasin, A.; Little, D.N.; Lytton, R.L. “Experimental measurement of water diffusion through fine aggregate mixtures.” J. Mater. in civil eng. 23(4):445–452. 2011. DOI:10.1061/(ASCE)MT.1943-5533.0000190.

Vavrik, W.R.; Huber, G.; Pine, W.J.; Carpenter, S.H.; Bailey, R. “Bailey method for gradation selection in hot-mix asphalt mixture design”. Transportation research E-circular, report No: E-C044, 2002.

Walubita, L. F., Faruk, A. N. M., Zhang, J., Hu, X., Lee, S. I., “The Hamburg rutting test – Effects of HMA sample sitting time and test temperature variation”. Construction and Building Materials 108, 22-28, 2016.

Walubita, L. F., Zhang, J., Alvarez, A. E., and Hu, X., “Exploring the flow number FN index as a means to characterize the HMA permanent deformation response under FN testing”. Journal of the South African Institution of Civil Engineering, 55(3), 103-112, 2013.

Walubita, L. F., Zhang, J., Das, G., Hu, X., Mushota, C., Alvarez, A. E., and Scullion, T., “Comparison of the Hamburg, dynamic modulus, and repeated load tests for evaluation of HMA permanent deformation”. Presented in Transportation Research Board 91st Annual Meeting, Washington D.C, 2012.

Wang, H., Tan, H., Qu, T., and Zhang, J., “Effects of Test Conditions on APA Rutting and Prediction Modeling for Asphalt Mixtures”. Advances in Materials Science and Engineering. 2017. Doi: 10.1155/2017/2062758.

Wang, H., Zhang, Q., and Tan, J., “Investigation of Layer Contributions to Asphalt Pavement Rutting,” J. Mater. Civ. Eng. 21, SPECIAL ISSUE: China: Innovative Use of Materials for Highway Construction, 181–185, 2009.

Washington, D. “Factors affecting HMA Permeability”. Washinton State Department of Transportation, (February), 1–3, 2005.

White, T. D., Haddock, J. E., and Rismantojo, E., “Aggregate tests for hot-mix asphalt mixtures used in pavements”. NCHRP (National Cooperative Highway Research Program) Report 557, American Association of State Highway and Transportation Officials, Washington, D. C, 2006.

Whiteoak, D., “The Shell Bitumen Handbook,” Surrey, UK, 1990.

Williams, R. C., Hill, D., Hofmann, K., Zelenock, M., and Bausan, J., “Development of Laboratory Performance Test Procedures and Trial Specifications for Hot Mix Asphalt”. Final Report Number RC-1410, Department of Civil and Environmental Engineering, Michigan Technological University prepared in co-operation with Michigan Department of Transportation Construction and Technology Division, Michigan, USA, 2004.

Witczak M. " A Comparison of the Dynamic (Complex) Modulus Test (E*) and Indirect Diametral Test (Mr) for AC Mixtures". White Paper” Report, NCHRP, 1999.

Witczak, M., W., Fonseca, O., A., “Revised Predictive Model for Dynamic (Complex) Modulus of Asphalt M`ixtures,” Transportation Res. Rec, 1540, 15-23, 2007.

Witczak, M.W., and Kaloush, K., “Simple Performance Test for Superpave Mix Design,” National Cooperative Highway Research Program, NCHRP Report 465, Arizona State University, Tempe, AZ, 2002.

Wu, F., “Assessment of Residual Life of Bituminous Layers for the Design of Pavement Strengthening,” PhD. Thesis, Council for National Academic Awards, London, March 1992.

Wu, S., Zhang, K., Wen, H., DeVol, J., and Kelsey, K., “Performance Evaluation of Hot Mix Asphalt Containing Recycled Asphalt Shingles in Washington State”. J. Mater. Civ. Eng., 28(1), 04015088, 2016.

Xiao, F., Amirkhanian, S., and Juang, C. "Rutting Resistance of Rubberized Asphalt Concrete Pavements Containing Reclaimed Asphalt Pavement Mixtures". J. Mater. Civ. Eng., 19(6), 475–483, 2007.

Xu HN, Guo W, Tan YQ. "Permeability of asphalt mixtures exposed to freeze–thaw cycles. Cold Regions Science and Technology 123:99–106, 2016.

Xu, T., Wang, H., Li, Z., and Zhao Y., "Evaluation of permanent deformation of asphalt mixtures using different laboratory performance tests". Construction and Building Materials 53, 561-567, 2014.

Y. Ali, M. Irfan, M. Zeeshan, I. Hafeez, S. Ahmed, "Revisiting the relationship of dynamic and resilient modulus test for asphaltic concrete mixtures", Constr. Build. Mater. 170, 698–707, 2018.

Y. Ali, M. Irfan, S. Ahmed, "Can asphaltic concrete permanent deformation test methods be used as surrogate to each other?" 96th Transportation Research Board Annual Meeting, Transportation Research Broad, Washington, DC. 2017.

Y. Ali, M. Irfan, S. Ahmed, S. Ahmed, "Empirical correlation of permanent deformation tests for evaluating the rutting response of conventional asphaltic concrete mixtures", J. Mater. Civ. Eng. 29 (8), 04017059, 2017.

Y. Ali, M. Irfan, S. Ahmed, S. Ahmed, "Permanent deformation prediction of asphalt concrete mixtures–a synthesis to explore a rational approach", Constr. Build. Mater. 153, 588–597, 2017.

Y. Ali, M. Irfan, S. Ahmed, S. Khanzada, T. Mahmood, "Investigation of factors affecting dynamic modulus and phase angle of various asphalt concrete mixtures", Mater. Struct. 49 (3), 857–868, 2016.

Y. Ali, M. Irfan, S. Ahmed, S. Khanzada, T. Mahmood, "Sensitivity analysis of dynamic response and fatigue behaviour of various asphalt concrete mixtures", Fatigue Fract. Eng. Mater. Struct, 2015.

Y.-R. Kim, D. Little, R. Lytton, "Effect of moisture damage on material properties and fatigue resistance of asphalt mixtures", Transp. Res. Rec. 1891, 48– 54, 2004.

Y.-R. Kim, D. Little, R. Lytton, "Fatigue and healing characterization of asphalt mixtures", J. Mater. Civ. Eng. 15 (1), 75–83, 2003.

Y.R. Kim, H.-J. Lee, D.N. Little, "Fatigue characterization of asphalt concrete using viscoelasticity and continuum damage theory (with discussion)", J. Assoc. Asphalt Paving Technol. 66, 1997.

Yan, J.; Zhang, Z.; Charmot, S.; Ding, W.; Li, F. "Investigation and prediction model for the dynamic modulus of asphalt emulsion cold recycled mixtures." J.

Mater. Civ. Eng., 2014. DOI: 10.1061/(ASCE)MT.1943-5533.0001048, 04014113.

Yildirim, Y., Jayawickrama, P. W., Hossain, M. S., Alhabshi, A., Yildirim, C., Smit, A. F., and Little, D., "Hamburg Wheel-Tracking Database Analysis". Report FHWA/TX-05/0-1707-7 prepared in cooperation with the Texas Department of Transportation and the Federal Highway Administration, 2007.

Yilmaz, I., and Kaynar, O., "Multiple regression, ANN (RBF, MLP) and ANFIS models for prediction of swell potential of clayey soils". Expert Systems with Applications 38, 5958-5966, 2011.

Yu, H., and Shen, S., "Impact of Aggregate Packing on Dynamic Modulus of Hot Mix Asphalt Mixtures Using Three-Dimensional Discrete Element Method," Construction and Building Materials, 26(3), 302-309, 2011.

Zhang W, Drescher A, Newcomb D. "Viscoelastic analysis of diametral compression of asphalt concrete. Journal of Engineering Mechanics". 123(6):596-603, 1997

Zhang, J., Alvarez, A. E., Lee, S. I., Torres, A., and Walubita, L. F., "Comparison of flow number, dynamic modulus, and repeated load tests for evaluation of HMA permanent deformation". Construction and Building Materials 44, 391-398, 2013.

Zhang, J., Yang, J., and Kim, Y. R., "Characterization of mechanical behavior of asphalt mixtures under partial triaxial compression test". Construction and Building Materials 79, 136-144, 2015.

Zhang, Y. Luo, R. and Lytton, R. L., "Characterizing Permanent Deformation and Fracture of Asphalt Mixtures by Using Compressive Dynamic Modulus Tests," J Material Civil Eng, 24(7), 898-906, 2012.

Zhao, Y, Tang, J. and Liu H., "Construction of triaxial dynamic modulus master curve for asphalt mixtures," Construction & Building Material, 37, 21-26, 2012.

Zhu, H.; Sun, L.; Yang, J.; Chen, Z.; Gu, W."Developing master curves and predicting dynamic modulus of polymer-modified asphalt mixtures." J. Mater. Civ. Eng., 23(2), 131–137, 2011.

Ziari, H. and Khabiri, M., M., "Effect of Bitumen Content on Resilient Modulus of Asphalt Concrete", Proceedings of the International Symposium on Pavement Recycling, Sao Paulo, Brazil, 2005.

Zulkati, A., Diew, W., and Delai, D., "Effects of Fillers on Properties of Asphalt-Concrete Mixture," J. Transp. Eng., 138(7), 902–910, 2012.

List of Publications

Ali, Y., Irfan, M., Ahmed, S., and Ahmed, S. “Empirical Correlation of Permanent Deformation Tests for Evaluating the Rutting Response of Conventional Asphaltic Concrete Mixtures”. Journal of Materials in Civil Engineering, ASCE, 2017. ISSN 0899-1561. Volume 28, No. 8, Impact Factor: 1.644.

Ali, Y., Irfan, M., Ahmed, S., and Ahmed, S. “Permanent Deformation Prediction of Asphalt Concrete Mixtures - A Synthesis to Explore a Rational Approach. Construction & Building Materials”. Elsevier, 2017. ISSN: 0950-0618. Volume 153, Pages 588-597, Impact Factor: 3.429.

Ali, Y., Irfan, M., Ahmed, S., Khanzada, S., and Mahmood, T. “Investigation of Factors Affecting Dynamic Modulus and Phase Angle of Various Asphalt Concrete Mixtures”, Materials and Structures, 49, 857-868, 2016. ISSN: 1359-5997 (Print) 1871-6873 (Online). Impact Factor: 2.69.

Ali, Y., Irfan, M., Ahmed, S., Khanzada, S., and Mahmood, T. “Sensitivity Analyses of Dynamic Response and Fatigue Behavior of Various Asphalt Concrete Mixtures”. Fatigue and Fracture of engineering Material and Structures, 38, 1181–1193, 2015. Online ISSN: 1460-2695. Impact Factor: 1.68.

Ali, Y., Irfan, M., Zeeshan, M., Ahmed, S., and Hafeez, I. “Revisiting the Relationship of Dynamic and Resilient Modulus Test for Asphaltic Concrete Mixtures”. Construction & Building Materials. Elsevier. ISSN: 0950-0618. Volume 170, Pages 698-70, 2018. Impact Factor: 3.429.

Gul, M. A., Irfan, M., Ahmed, S., Ali, Y., and Khanzada, S. “Modelling and Characterising the Fatigue Behaviour of Asphaltic Concrete Mixtures”. Construction & Building Materials, Elsevier. ISSN: 0950-0618. Volume 184, Pages 723-732, 2018. Impact Factor: 3.429.

Hussan S., Kamal M. A., and Hafeez I., “Evaluation and modelling of permanent deformation behavior of asphalt mixtures using dynamic creep test in uniaxial mode.” International Journal of Pavement Engineering. 2017. doi.org/10.1080/10298436.2017.1380805 (Impact Factor: 1.832).

Hussan S., Kamal M. A., and Hafeez I., Ahmad N. “Correlating and comparing different rutting performing laboratory tests” International Journal of Pavement Engineering (IJPE). 2017. http://dx.doi.org/10.1080/10298436.2017.1380805. I.F 1.832.

Hussan, S., Kamal, M. A., and Hafeez, I., “Modeling and correlating rut depth observed in different asphalt mixture performance tests”. Mehran University

Research Journal of Engineering and Technology, 37(3), 553-570. 2017. doi: 10.22581/muet1982.1803.10,

Hussan, S., Kamal, M. A., Hafeez, I., Ahmad, N., Khanzada, S., and Ahmed, S., "Modeling Asphalt Pavement Analyzer (APA) rut depth using different statistical techniques". Journal of Road Materials and Pavement Design. 2017. (Published online; doi:10.1080/14680629.2018.1481880).

Hussan, S., Kamal, M. A., Hafeez, I., and Ahmad, N., "Comparing and correlating various laboratory rutting performance tests". International Journal of Pavement Engineering, 2017. doi: 10.1080/10298436.2017. 1402591.

Hussan, S., Kamal, M. A., Hafeez, I., and Ahmad, N., "Evaluation and modeling of permanent deformation behavior of asphalt mixtures using dynamic creep test in uniaxial mode". International Journal of Pavement Engineering, 2017. doi: 10.1080/10298436.2017.1380805.

Hussan, S., Kamal, M. A., Hafeez, I., Farooq, D., Ahmad, N., and Khanzada, S., "Statistical evaluation of factors affecting the laboratory rutting susceptibility of asphalt mixtures". International Journal of Pavement Engineering, 2017. doi: 10.1080/10298436.2017.1299527.

Imran Hafeez, "Investigating the creep response of asphalt mixtures under waveform loading". Journal of Road Materials and Design, ASCE. 1–18, 2017. DOI:10.1080/14680629.2017.1279071. I.F 1.401.

Imran Hafeez, M.A. Kamal, M. A Ishaq, N. Ahmad, S. Khanzada. "A Laboratory Based Research Study to Investigate the Aggregate Packing Characteristics and Its Influence on Asphaltic Mixtures Performance." Arabian Journal of Science and Engineering. 40(11):3119–3134, 2015. (DOI) 10.1007/s13369-015-1804-9. (I.F 0.367).

Irfan, M., Ali, Y., Iqbal, S., Ahmed, S., and Hafeez, I. "Rutting Evaluation of Asphalt Mixtures Using Static, Dynamic, and Repeated Creep Load Tests". Arabian Journal for Science and Engineering, Springer, Volume 43, Issue 10, pp 5143–5155, 2018. Impact Factor: 0.865. ISSN: 2193-567X (Print)

M. Irfan, Y. Ali, S. Ahmed, I. Hafeez "Rutting evaluation of asphalt mixtures using static, dynamic and repeated creep load tests" Arabian Journal for Science and Engineering. 1–13, 2017. DOI 10.1007/s13369-017-2982-4. I.F 0.867.

Muhammad Irfan, Yasir Ali, Sarfraz Ahmed and Imran Hafeez. "Performance Evaluation of Crumb Rubber Modified Asphalt Mixtures Based on Laboratory and Field Investigations" Arabian Journal for Sciences and Engineering, 2017. DOI 10.1007/s13369-017-2729-2. I.F 0.867.

Sabahat Hussan, M.A. Kamal. Imran Hafeez. "Statistical evaluation of factors affecting the laboratory rutting susceptibility of asphalt mixtures". International Journal of Pavement Engineering, ASCE. 2017. doi.org/10.1080/10298436.2017.1299527. I.F 1.832

Sabahat Hussan, Mumtaz Ahmad Kamal, Imran Hafeez, "Modelling and correlating rut depth observed in different asphalt mixture performance tests". Mehran University Research Journal of Engineering and Technology, [S.l.], v. 37, n. 3, p. 553-570, July 2018. ISSN 2413-7219.

Sabahat Hussan, Mumtaz Ahmad Kamal, Imran Hafeez, Naveed Ahmad. "Modelling asphalt pavement analyser rut depth using different statistical techniques". Journal: Road Materials and Pavement Design (TRMP). 2018. Doi: 10.1080/14680629.2018.1481880. I.F 1.832

Yasir Ali, Muhammad Irfan, Imran Hafeez, Muhammad Zeeshan Shafeeq Ahmed, "Revisiting the relationship of dynamic and resilient modulus test for asphaltic concrete mixtures". Construction and Building Materials 170 (2018) 698–707. I.F 3.169, 2018.

Hafeez, I., Shan, A., and Hussan, S., "Predicting the Laboratory Rutting Response of Asphalt Mixtures using different Neural Network Algorithms". International Journal of Pavement Engineering, (Under Review), 2017.

Maqbool, S., Khan, A., Inam, A., Sultan, T., Rashid, M., & Rizvi, M. (2015). Effect of Fines Content on Dry and Saturated Indirect Tensile Strength of Hot Mix Asphalt Mixtures. University of Engineering and Technology Taxila. Technical Journal, 20(1), 54.

Maqbool, S., Khan, A. H., Rizvi, M. A., Inam, A., & Kashmiri, F. A. (2022). Modelling and evaluating moisture susceptibility of laboratory prepared asphalt concrete mixtures. Ain Shams Engineering Journal -Elsevier, 13(1), 101512.

Rizvi, M. A., Khan, A. H., Inam, A., & Masoud, Z. (2021). Evaluation of Linear Deformation and Unloading Stiffness Characteristics of Asphalt Mixtures Incorporating Various Aggregate Gradations. Sustainability - Multidisciplinary Digital Publishing Institute (MDPI), 13(16), 8865.

Rizvi, M. A., Khan, A. H., Masoud, Z., & Inam, A. (2021). Effect of Fractured Aggregate Particles on Linear Stress Ratio of Aggregate and Resilience Properties of Asphalt Mixes - A Way Forward for Sustainable Pavements. Sustainability - Multidisciplinary Digital Publishing Institute (MDPI), 13(15), 8630.

Conference Papers

Presented at 1st International Seminar: Improvement of Asphalt Mix Design Technology for Pakistan, National Highway Authority, Islamabad, May 14-15, 2014.

Khanzada, S. "Improvement of Asphalt Mix Design Technology for Pakistan – An Overview".

Hafeez, I and Mohsin, M. "Packing Characteristics of different Aggregate Gradations for Pakistan".

Ammad, H and Mujassim, A."Influence of performance based binder properties on the distress behaviour of asphalt mixtures".

Irfan, M. and Ali, Y. "Laboratory Characterization of Asphalt Concrete Mixtures using Dynamic Modulus Test".

Kamal, M. A. and Sabahat, H. "Development of Laboratory Rutting Transfer/Shift Functions of Bituminous Mixtures for Pakistan".

Sarfraz, A. and Aniq, M. "Development of Laboratory Fatigue Transfer/Shift Functions of Bituminous Mixtures for Pakistan".

Naveed, A. and Aakif, M "Durability Assessment of Bituminous Mixtures for Pakistan".

Shafeeq, A."Damage Impact Analysis of Asphalt Concrete Pavements due to over loading in Pakistan".

Hutschenreuther, J."In-Process Compaction Control/In-Plant Recycling Technologies".

Hutschenreuther, J."Asphalt Concrete Recycling Technology".

Larsen, B. "Asphalt Mix Design in Hot Climates".

Asim, I. "Brief Concept of Superpave Mix Design".

Biedermann, A. "Asphalt Production Process".

In collaboration with Academic Institutes

This research study report is concerned with improving Asphalt Concrete (AC) mix design technology for Pakistan. The research herein looks into the laboratory characterization of selected aggregate gradations for AC mixtures; practiced in Pakistan and in different parts of the world, with a view to identifying performance and ranking of selected AC mixtures over a range of temperature and stress regimes.

The reserach project is part of a larger R & D initiative namely "Strategic Pavement Research Study (SPRS)" programme, launched by National Highway Authority (NHA) in collaboration with Academia. The aim is to adopt AC mix design methodology for development of rut and fatigue resistant longer performing pavements in Pakistan.

SPRS is envisaged to be conducted in three phases:-

Phase I: Characterization of asphalt mixtures through Performance based Laboratory Experiments - "Improvement of Asphalt Mix Design Technology for Pakistan".

Phase II: Validation of rut & fatigue resistant AC mixtures through Accelerated Pavement Testing (APT) tracks and in-service pavement test sections.

Phase III: Continuous improvement through field data collection, data analysis and development of local pavement design system and standards for Pakistan.

Highway Research and Training Centre (HRTC)
Motorway (M-1) Camp, Burhan, Tehsil Hassanabdal
District Attock, Pakistan
Postal Code: 43710
Phone: +92 572 362030
Fax: +92 572 362036
www.hrtc.com.pk