PAKISTAN-US SCIENCE AND TECHNOLOGY
COOPERATIVE PROGRAM 2007

Implementation of Superpave Binder and Asphalt Mix Specification to Improve Pavement Performance in Pakistan

Final Report

November 2011

Submitted By

Department of Civil & Environmental Engineering
Michigan State University (MSU)
East Lansing, Michigan

AND

University of Engineering and Technology (UET), Lahore, Pakistan

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Authors

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Annual Report was Compiled and Edited by the US Research Team

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EXECUTIVE SUMMARY

The majority of the road network in Pakistan consists of asphalt concrete (AC) pavements. The total network asset value is estimated at US $45 billion. Inadequate asphalt concrete thickness and deficient bituminous mixture designs accelerate the rate of deterioration of asphalt pavement structures in the forms of premature fatigue cracking and asphalt-bound layer rutting. Due to limited available resources and high road user costs, the high rate of deterioration adversely affects the socio-economic development of the country.

The premature deterioration of the pavement network was investigated by a four year study sponsored by Pakistan-United States Science and Technology Cooperative Program. The study was initiated in 2008 and is being jointly conducted by the University of Engineering and Technology (UET) Lahore and Michigan State University (MSU). The main objective of the study was to introduce the state-of-the-art technology and practices for the design of asphalt concrete mixtures using local materials in Pakistan. The implementation of such practices would enhance the longevity of the pavement infrastructure in a cost-effective manner.

The final report details the methodologies (state-of-the-practice and emerging state-of-the-art) and procedures used to characterize local materials available for road construction in Pakistan. The local construction materials include asphalt binders, aggregates, and HMA mixtures. Based on the results of rigorous laboratory testing and mechanistic-empirical pavement design guide, conclusions and recommendations are documented.

During the study, Pakistan was divided into six temperature zones. The PG 70-10 is the most important asphalt binder that covers more than 70 percent of the country. Unfortunately, the PG 70-10 is currently not produced by any of the two refineries in Pakistan. The closest grade that fulfills the requirements of the PG 70-10 is the PG 76-16 (APMB) which is relatively harder and may be prone to cracking. At present, the commonly used grades in Pakistan are A-60/70 and K-60/70. The corresponding performance-based grades are PG 58-22 and PG 64-22. These softer binders, especially at high temperatures are likely to rut in areas, which require PG 70-10. This may be one of the major reasons of premature rutting failures.

The HMA mixtures with Superpave gradations and polymer modified binders consistently showed better fatigue and rutting performance in the laboratory than those with NHA gradations, especially at intermediate and high stress levels. These results were confirmed by the pavement performance prediction utilizing laboratory data and mechanistic-empirical methods.

It is strongly recommended that the new Superpave methodologies be adopted and implemented immediately in the construction specifications in Pakistan. The adoption and implementation of the recommendation would assist the highway agencies in Pakistan to achieve significant life extension for both newly constructed and rehabilitated road infrastructure.

CHAPTER 1 - INTRODUCTION

1.1 BACKGROUND

The 250,000 kilometers pavement network in Pakistan (150,000 km high class and 100,000 km low class roads) serves millions of automobiles, buses, and heavy trucks on a daily basis. Its estimated asset value is about PKR 2.500 trillion (45 billion USD). This huge asset value is continuously deteriorating at alarming rates due to various causes including:

- Inadequate initial pavement structure
- Deficient asphalt mixture designs
- Lack of construction quality
- Non-uniform local practices
- Repetitive heavy axle loads
- Climatic effects
- Inadequate or lack of maintenance and rehabilitation

In Pakistan, most of the premature deteriorations of the asphalt concrete (AC) pavements are in the form of high severity rutting and fatigue cracking.. The preservation of this huge national investment of road network requires balanced and cost-effective design of pavement rehabilitation and maintenance activities coupled with proper construction practices. This would provide safe and comfortable means of transportation for both private and commercial vehicles. In addition, the implementation of cost-effective preservation of the transportation system would significantly contribute to the socio-economic growth of Pakistan. Such implementation requires good understanding of the pavement structural and asphalt mixture designs that suites Pakistan's local environment and traffic conditions and the available construction materials.

Realizing the importance of an efficient road infrastructure, the highway authorities in Pakistan are spending huge amount of scarce funds to meet the increasing traffic demand by expanding and maintaining the existing road network. Since 1991, the size of the high-class road network has increased from 87,000 to 150,000 km. The costs of the expenditure and maintenance have placed significant burden on the socio-economical development of the country. This magnitude of annual expenditures justifies the application of the best available design procedures and material characterization to optimize the use of highway expenditures. An improvement in the design of new or rehabilitated pavement structures (including asphalt mixture design) would have significant and sizeable implications in reducing the cost of construction and maintenance of highway pavements. If the life of the pavement network is increased by just one or two years because of better design methodologies, it would result in yearly saving of millions of dollars.

It is also recognized that in order to achieve the objective of better roads in Pakistan, proper planning is needed to make improvements in all aspects of highway industry. This does not only require improvement in construction materials by improved methods of asphalt mixture design but efforts should also be concentrated on improving construction practices, technology, and quality. This research study addresses one of these aspects, the establishment of cost-effective asphalt mixture design, and the characterization of the asphalt binders and asphalt mixtures for the implementation of the Superpave methodology. Such implementation would improve the road infrastructure in Pakistan in a cost-effective manner.

The Superpave methodology was developed in 1993 under the Strategic Highway Research Program (SHRP) in the USA. The SHRP program developed new ways to test and specify asphalt binders and to perform asphalt mixture design. Currently, the new Superpave asphalt mixture design methodology has been adopted by most State Highway Agencies in the USA and is being implemented around the world. Laboratory results and field data have clearly shown that the Superpave methodology is superior to the old empirical (Marshall) methodology. Pavement structures designed and constructed using Superpave asphalt mixtures have shown significant improvement in pavement performance and reduced pavement maintenance costs.

Recognizing the importance and the need for research on improving asphalt mixture materials to enhance pavement performance and its impact on the economic growth in Pakistan, a joint research team from Michigan State University (MSU) and the University of Engineering and Technology (UET), Lahore was awarded a three years study by Pakistan-United States Science and Technology Cooperative Program in 2008. This final report covers the activities and the accomplishments of the research team for the entire period of the study March 31, 2008 to November 30, 2011.

1.2 RESEARCH OBJECTIVES

The main objective of this research is to implement the new Superpave mix design technology compatible with the material properties, traffic and environmental conditions in Pakistan and conduct technology transfer to the highway authorities and pavement industries in Pakistan. A systematic and rational approach for implementing Superpave mix design is adopted in this research study, involving both Pakistani and USA institutions. Researchers from the USA shared with their Pakistani counterpart, the state-of-the in the field of material characterization. The Pakistani researchers, on the other hand, enhanced the implementation and usefulness of the research by contributing their local experiences. The specific objectives of this research are to:

- Develop an enhanced asphalt mixture design system for Pakistan based on the state-of-the-art methodologies (such as the Superpave) and the local material properties, traffic needs, and environmental conditions in Pakistan.
- Assist the highway authorities in developing quality assurance/quality control protocol regarding the new asphalt binder and mix characterization methodologies.
- Develop and initiate an integrated pavement research program in cooperation with personnel from the University of Engineering and Technology (UET), the highway authorities and the industry in Pakistan.

- Develop a framework for Pakistan to implement the National Cooperative Highway Research Program (NCHRP) 1-37 [A Design Guide for Mechanistic Empirical Design of New and Rehabilitated Pavement Structures (M-E PDG)].

The accomplishment of the above objectives will precipitate the following socioeconomic benefits:

- Improve the health of the road network and hence reduce pavement maintenance and rehabilitation costs, which would improve the livelihood of ordinary people and enhance the country's economy.
- Conserve natural resources such as energy, aggregate, and asphalt binder.
- Enhance the capacity of the existing academic program at learning institutions.
- Elevate the knowledge of the highway agencies and industries to support the growing traffic demand and to provide reliable road network.
- Improve the capacity of Pakistani institutions to support industry competitiveness, through public/private partnership.

The objectives of the research are in parallel to the main goals of the Science and Technology Cooperative Program. This research will have positive socioeconomic and environmental impacts, it includes technology transfer component and will assist in capacity building of the academic institutions in Pakistan.

1.3 ORGANIZATION OF THE REPORT

This report consists of an executive summary, six chapters, and an appendix. The contents of each chapter are briefly described below.

- Chapter 1 documents the problem statement and objectives of the study.
- Chapter 2 describes the detailed research plan to accomplish the objectives of the research.
- Chapter 3 presents a literature review and summary of current practices used.
- Chapter 4 presents details of the laboratory investigation. It includes information about asphalt binders and mixtures testing conducted during the study.
- Chapter 5 contains the data analysis, results and discussions.
- Chapter 6 documents conclusions and recommendations based on the data analysis and results.

CHAPTER 2 - RESEARCH PLAN

2.1 INTRODUCTION

The main objective of this research study is the development of guidelines for the implementation of Superpave methodology for the design of asphalt mixes in Pakistan. These guidelines for binder and hot-mix asphalt (HMA) mix characterization must be based on extensive pavement materials testing in the laboratory, the properties of the local materials, and the performance of HMA mixtures. To accomplish the objectives of this research and to develop implementable guidelines, a comprehensive and systematic research plan was prepared in 2008 as a result of Pakistan research team visit to the USA in October 2008. Subsequently, the research plan was calibrated and modified during the material testing in tasks 3 and 4. These modifications along with the original research plan are documented in this chapter. The Superpave implementation guidelines will address several topics including:

(a) Methodology for characterizing asphalt binder performance grades (PG) according to climatic needs
(b) Determination of asphalt binder modifications based on the blending of existing binder grades and polymers
(c) Procedures for characterizing HMA mixtures using modified binders
(d) Measures to link HMA mix design and its performance in the laboratory to improve field pavement performance

In order to accomplish the objectives of this research, the overall research plan is divided into various tasks and subtasks:

Task 1: Literature review
Task 2: Review and evaluation of the existing practices and specifications regarding HMA mixes in Pakistan
Task 3: Material evaluation of existing materials
- Subtask 3.1: Asphalt binders
- Subtask 3.2: Coarse and fine aggregates
- Subtask 3.3: Mineral filler
- Subtask 3.4: Asphalt mix design

Task 4: Laboratory testing program to improve material behavior
- Subtask 4.1: Asphalt binder modifications
- Subtask 4.2: Asphalt mix performance evaluation using Superpave approach

Task 5: Training of Pakistani professionals in the USA
Task 6: Training and evaluation of existing HMA technology in Pakistan
Task 7: Reports

Figure 2-1 shows a flowchart presenting all tasks and subtasks along with the overall goals. Figure 2-2 illustrates the detailed of activities for material testing program for evaluating the existing practices and the state-of-the-art material characterization.

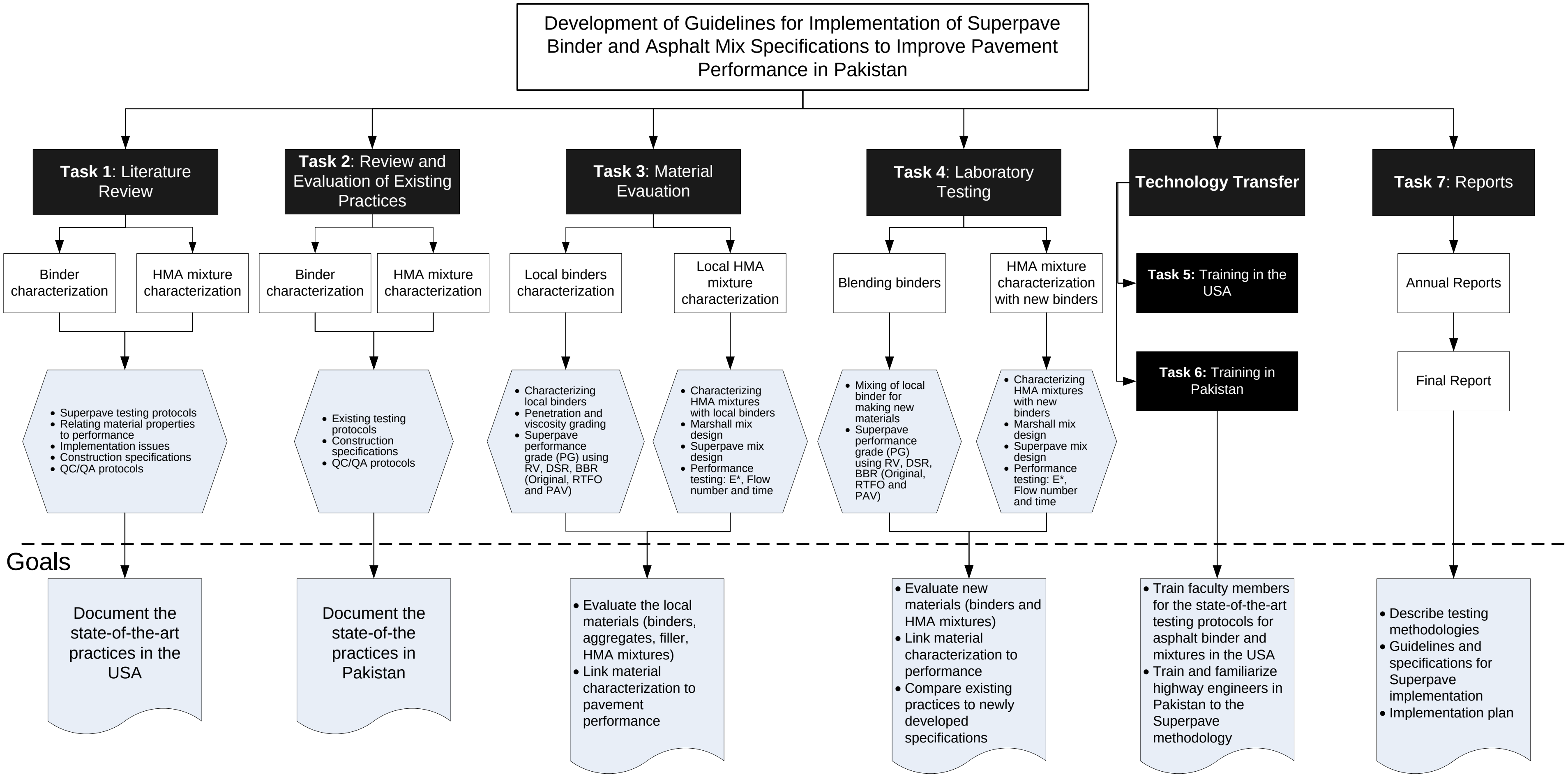

Figure 2-1 Overall research activities and expected goals

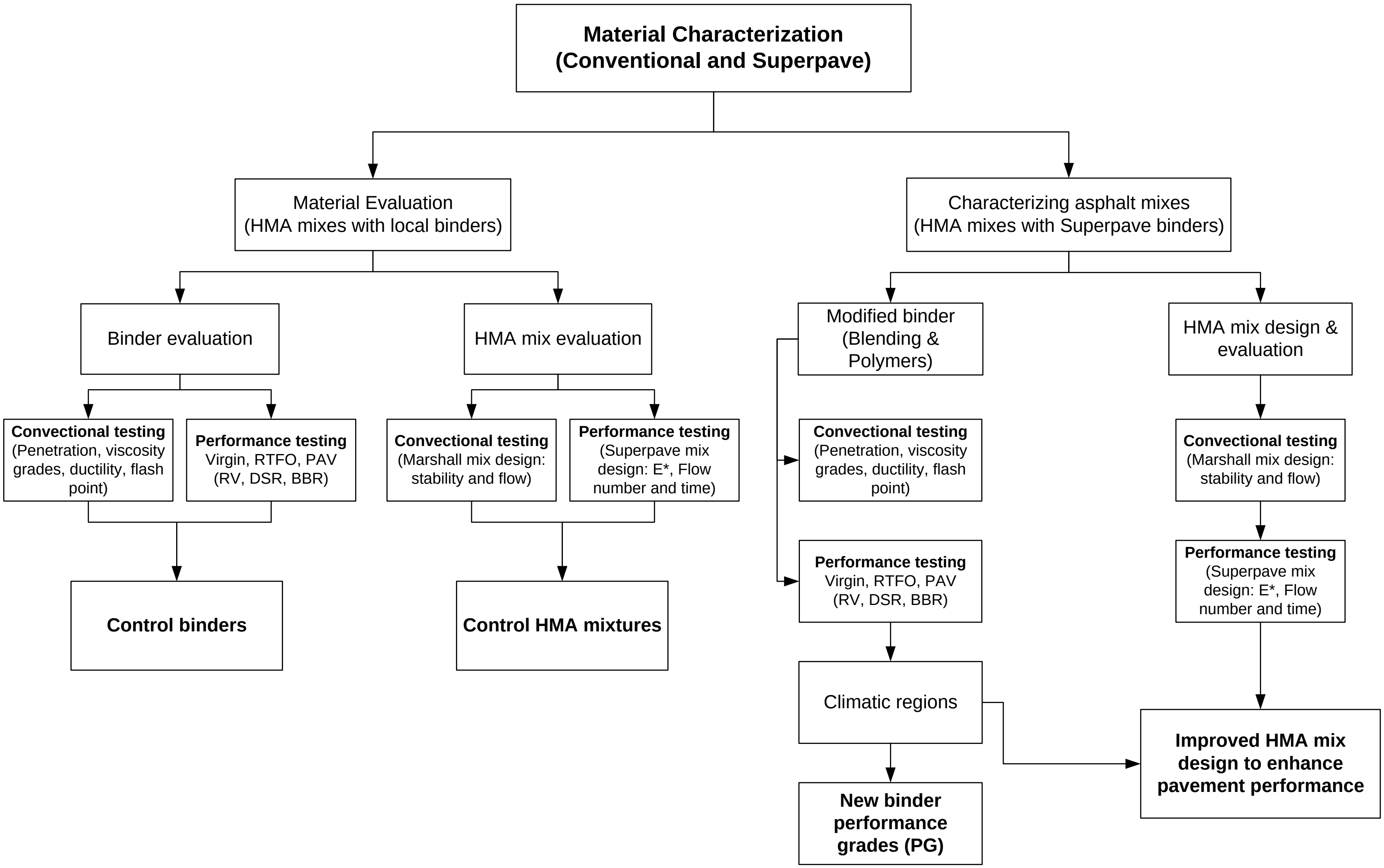

Figure 2-2 Details of material testing and objectives

2.2 RESEARCH APPROACH

The research plan for this study is based on information obtained during the USA research team previous visits to Pakistan (March and September 2008) and the Pakistan team members visit to the USA (October 2008). During their previous visits to Pakistan (under a separate study), the USA and the Pakistani research teams

1. Visited various pavement projects under construction and observed the current construction procedures,
2. Held several meetings to discuss pavement construction practices and related issues with the Pakistani research team,
3. Visited the Attock Oil Refinery (ARL) in Rawalpindi and discussed the asphalt binder grades and production,
4. Discussed with personnel from the Pakistan National Highway Authorities (NHA) various pavement preservation issues including recycling, and
5. Conducted a short course at National Highway Authority (NHA) in Islamabad which was attended by more than 150 people from NHA, the academia and the private sector

The updated research plan along with details in each task is presented below.

2.2.1 Task 1: Literature Review

The review was focused on the latest development in the Superpave technology. These include new test methods, procedures, specifications for characterizing binders and design of asphalt mixes. In addition, information was collected regarding current status and modification of the simple performance tests. These laboratory tests will play a key role in relating the short-term mixture performance in this study. Further, implementation and practical issues related to the adoption of Superpave specification in several states in the USA were documented. It should be noted that the specifications and practices to be reviewed are those of the states which have similar environment to that of Pakistan. As part of this review, several quality control/quality assurance (QC/QA) protocols related to HMA mixtures are documented for the guidance of highway authorities to improve construction practice in Pakistan.

2.2.2 Task 2: Review and Evaluation of the Existing Practices

Under this task, the research team collected information regarding the existing mix design specifications, procedures, and methodologies, regarding all the constituent materials of HMA mixes, that are being used for the provincial and the federally administrated roads in Pakistan. These include specifications and practices relative to the asphalt binders, coarse and fine aggregates, and mineral filler used in Pakistan. Results of the review were used to determine the differences and similarities between the Pakistani practices and the state-of-the-art practices reviewed in Task 1. This information will be assembled from the local agencies, provincial communication and works (C&W) departments and National Highway Authority (NHA). The documentation of existing practices at several levels will be used to highlight limitations, inadequacies and essential areas needing immediate attention for improvements. In addition, information was assembled for the QC/QA practices during construction. The construction specifications are summarized for binder and HMA mixture characterization and design from different completed highway projects. Further, the field performance data for the selected pavements are collected along with their in field HMA mix properties.

2.2.3 Task 3: Material Evaluation

The activities in this task are designed for local material evaluation; the material testing activities are described in Task 4. The local materials include the asphalt binders produced by the two oil refineries (one located in Karachi and the other in Attock, Pakistan), coarse and fine aggregates, and mineral filler. *The main objective of the evaluation is to determine whether or not the binders meet the specifications for high performance HMA mixes.* It should be noted that the properties of the constituent materials in asphalt mixes are function of several variables. For example, the properties of the asphalt binders depend on the source of the crude oil, the refining process, time, storage facilities, and the mixing procedures used in refineries. The appropriate statistical sampling techniques were employed to acquire sample of all constituent materials. The sampling technique will produce representative samples of the asphalt binders, coarse and fine aggregates and the mineral filler used by the various provinces and in the various climatic zones in Pakistan. After developing the sampling techniques, the objectives of this task can be accomplished by the execution of four subtasks as stated below.

2.2.3.1. Subtask 3.1: Asphalt binders

The main objective of this subtask is to evaluate the properties of the local asphalt binders and their impact on the long-term pavement performance. To accomplish this objective, three types of asphalt binder sample will be obtained, tested, and evaluated as mentioned below.

- Samples of the *virgin asphalt binders* to be obtained from the two refineries. The physical, engineering, and rheological properties of the samples will be used to determine the long-term performance of the binders. The two major sources of asphalt binders are Attock and Karachi refineries. Both refineries produce limited grades of asphalt. Typical grades produced by these refineries include:
    - Attock Refinery
        - Grade 80/100
        - Grade 60/70
        - Grade 60/70 (PMB)
    - Karachi Refinery
        - Grade 80/100
        - Grade 60/70
        - Grade 40/50

- Samples of the asphalt binders will be obtained by extraction of the binders from freshly made asphalt mixes from two asphalt mixing plants (from the ongoing construction projects). The properties of these samples will be used to study the impact of the mixing plant procedures on the *__short-term aging__* of the asphalt binders.

- Asphalt binder samples will be obtained from the surface of at least four pavement sections located in different climate to determine the *__long-term aging__* properties of the asphalt binders under field conditions.

The above mentioned binders were fully characterized by using both conventional and Superpave testing protocols. Table 2-1 presents the tests conducted for conventional binder evaluation. Superpave testing was carried out to determine the Performance Grade (PG) for each of the binder, the test details are presented in Table 2-2.

Table 2-1 Conventional binder tests

Test	Property	Method	Conditions
Penetration	Penetration	AASHTO T49-03	15°C & 25°C, 100 gm load for 5 seconds
Ring and Ball	Softening point	AASHTO T53-96	Measured temperature
Absolute Viscosity	Absolute viscosity	AASHTO T202-03	60°C
Kinematic Viscosity	Kinematic viscosity	AASHTO T201-03	135°C

Note: Similar data was also collected on another on-going project (RAP study). The same data will also be shared in this study for verification and determining batch to batch variations.

Table 2-2 Superpave binder tests

Test/Equipment	Properties	Method	Conditions	Purpose
Rotational Viscometer (RV)	Rotational viscosity	AASHTO 316-04	60°C to over 200°C	To evaluate viscosity of binders at various temperatures
Dynamic Shear Rheometer (DSR)	Complex modulus (G^*) & phase angle (δ)	AASHTO 315-05	6 to 88°C at 10 rad/sec – dependent upon binder grade	To evaluate visco-elastic properties of binders. The elastic (recoverable) and viscous (unrecoverable) responses to load are measured. The rut and cracking potential for binders is characterized.
Bending Beam Rheometer (BBR)	Flexural creep stiffness (*S*-value) and Slope (m-value)	AASHTO 313-05	Three temperatures; one above low temperature specification limit & two below that limit	To measure the stiffness of binders at very low temperatures. Used to characterize low temperature cracking potential for binders

Note: Similar data was also collected on another on-going project (RAP study). The same data will also be shared in this study for verification and determining batch to batch variations.

All convention testing mention in Table 2-1 was carried out only on the unaged asphalts (virgin binders), extracted binders from the plant mixes and the field cores. However, in the case of Superpave, tests were conducted for three aging conditions for virgin binders:

- Unaged condition
- Short-term aged after RTFO aging
- Long-term aged after PAV aging

Along with the above mentioned aging conditions, testing was carried out at several temperatures. The temperature conditions are detailed in Table 2-2. It should be noted that extracted binders from the plant mixes and the field cores will be already aged; therefore, those will be tested using Superpave tests for unaged conditions.

In summary, the objective of this task is to fully characterize the asphalt binders available in Pakistan using Superpave approach. In addition, comparisons with the conventional characterization are also part of this task.

2.2.3.2. Subtask 3.2: Coarse and fine aggregate

The main objective of this subtask is to determine the physical properties (consensus and source) of coarse and fine aggregates and their impact on the properties and performance of asphalt mixes. Four different sources of crushed coarse and fine aggregates that are commonly used in highway construction in Pakistan will be selected. The major quarries for highway construction in Pakistan include:

- Sargodha
- Margalla
- Dina
- Sekhisarwar

Among the above mentioned sources, Margalla aggregate is most commonly used in the asphalt mixes for both wearing and binder courses. Since this research is mainly geared towards asphalt mixes, only one aggregate source will be used for evaluation purpose. The selected aggregate source is the Margalla aggregate. However, limited testing was also performed on one other type of aggregate for relative comparison. For each aggregate source, aggregate samples were collected from asphalt mixing plants. The samples were obtained after the aggregates have been mixed and heated but before the asphalt binder is added. The samples were then tested to determine whether or not the aggregates meet the specifications relative to gradation, angularity, hardness, and binder absorption capacity. Similar data was also collected from another study (RAP project) for two of the above aggregate types (Margalla and Dina). The data will be shared in this study for evaluation purposes.

2.2.3.3. Subtask 3.3: Mineral filler

Mineral filler (also known as dust) is a material finer than 0.075 mm (passing # 200 sieve size). Mineral filler includes crushed stone dust, lime dust, fly ash, etc. The mass ratio of the mineral filler to the effective mass of the asphalt binder (the mass of the free asphalt binder in the HMA) is called the effective dust ratio. In this task, the impact of the absorption capacity, the surface area, and the dust ratio of the mineral filler from Margalla quarry (from crushed stone) on the long-term asphalt mix performance will be evaluated using the mechanistic-empirical procedures.

2.2.3.4. Subtask 3.4: Asphalt mix design

All constituent materials of asphalt mix were evaluated in above three subtasks. In this subtask, asphalt mixes were designed using aggregates from Margalla and two most common asphalt binders each from Karachi and Attock refineries. Two typical NHA gradations were selected (one for wearing course and another for base course). These two gradations were mixed with two typical binder types used currently by NHA for the preparation of asphalt mixes. The mixes were prepared using the conventional Marshall mix design approach. Table 2-3 shows the matrix of mixes that were evaluated for performance.

Table 2-3 HMA mixture design (State-of-the-practice)

Source/Gradation	Attock			Karachi
	60/70 (A6/7)	60/70 P1 [PA6/7(1.35)]	60/70 P2 [PA6/7(1.70)]	60/70 (K6/7)
Wearing[1]	3[2]	-	-	3
Base[3]	3	-	-	-

Note: For Marshall mix design in each cell, 3 replicates for stability and flow based on NHA specifications (i.e., volumetric) will be prepared. For the optimum mix design make 2 replicates for simple performance tests using gyratory compaction.

For all the asphalt mixes in Table 2-3, two samples were prepared using gyratory compaction for performance testing. The performance testing include dynamic modulus (E^*) testing at five temperatures (10, 40, 70, 100 and 130 ^{o}F) and at each temperature five frequencies (25, 10, 5, 1, and 0.5 Hz). These test results were used to develop master curves for each mix type for a range of temperatures and frequencies. The E^* master curves were utilized to evaluate the long-term performance (cracking and rutting) of these mixes in the field. In addition, Flow Number (FN) and Flow Time (FT) tests were conducted for selected mix type to evaluate their permanent deformation (rutting) propensity at high temperature.

It should be noted that results of performance testing on conventional asphalt mixes in this subtask will be utilized latter in Task 4 to compare asphalt mixes using Superpave mix design. This comparison between the performances of both mix type will highlight the effectiveness and the potential improvement in field performance for asphalt mixes design by using Superpave approach.

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[1] Wearing course gradation according to NHA specification (use highest or typical gradation type)
[2] Number of replicates
[3] Base course gradation according to NHA specification (use highest or typical gradation type)

In addition to laboratory prepared mixes (Table 2-3), gyratory samples will be prepared for two plant prepared mixtures (collected in subtask 2.2.3.1). Similar performance testing will be performed on these samples to evaluate the effects of plant variations on asphalt mixes. This will facilitate in the development of guidelines for QC/QA for construction practices.

2.2.4 Task 4: Detailed Laboratory Testing

Tasks 3 and 4 are the most important activities in this study. Both tasks include the detailed laboratory testing program. In Task 4, a very extensive testing program is proposed for the locally available materials in Pakistan. This includes binder, aggregate, and asphalt mix performance testing. The tests were carried out using conventional methods as well as newly recommended tests under the Superpave technology. While use of conventional methods is only for comparison purposes, it will further elaborate the justification and benefits for adopting Superpave approach. The detailed laboratory testing in Task 4 was carried out in two parts. In the first phase work focused on the asphalt binders while second phase dealt with asphalt mixes.

2.2.4.1. Subtask 4.1: Asphalt binder modifications

The Superpave asphalt binder specifications are intended to improve performance by limiting the potential for the asphalt binder contributions to permanent deformation, low temperature and fatigue cracking in flexible pavements. The specifications provide for this improvement by designating various physical properties that are measured with the equipment purchased during this study. The work in this task will explain how each of the new test parameters relates to pavement performance, and how to select the asphalt binder for a specific project. One important difference between the currently used asphalt specifications and the Superpave specifications is the overall format of the requirements. Unlike in the conventional specifications, the physical properties remain constant in the Superpave specification for all of the performance grades (PG). However, the temperatures at which these properties must be achieved vary depending on the climate in which the binder is expected to serve. As an example, a PG 64-22 grade is designed to sustain the conditions of an environment where the average seven-day maximum pavement temperature is not more than 64 °C and an average one-day minimum pavement surface temperature is not less than -22 °C.

Therefore, for the prevailing environment and traffic conditions in Pakistan, blending of different types of asphalt binders may be needed to produce grades meeting the Superpave requirements. In addition, under certain climatic and loading conditions, polymer modification may be required. In order to optimize the use of polymers in a cost-effective manner, polymer with varying percentage with neat asphalt were examined in this task. Only one polymer type will be used for the design matrix. The selection of the polymer was made in consultation with National Highway Authority (NHA). The design matrices for the blending and polymer modification for the two refineries (Attock and Karachi) are given in Tables 2-4 and 2-5, respectively. The blending matrices were modified and calibrated during the testing because of the following reasons:

- Several combinations were repeated in the blending matrices; therefore, the duplicate cells were eliminated.

- The combinations of blends with polymer modified binder were eliminated because it was not possible to achieve the desired blend properties. The polymer chemical reactivity cannot be reinitiated once a binder is modified.
- The polymer blending needed two types of polymers — reactive and non-reactive, depending on the source of petroleum crude. For reactive polymer, phosphoric acid is added as a catalyst to speeding the chemical reaction. In case of non-reactive polymer heating is used for blending polymer with a binder

Table 2-4 Blending of virgin binders and polymer modification for **Attock** Refinery

Binder type	80/100	60/70	60/70 P	Polymer[5]
80/100	-	-		1.35, 1.7. 2.0
60/70	0, *20*, *50*, 100[4]	-		1.35, 1.7. 2.0
60/70 (P)	-	-	100	-

Note: There are 3 neat and 6 polymer modified binders (a total of 9 binders other than blends)

Table 2-5 Blending of virgin binders and polymer modification for **Karachi** Refinery

Binder type	80/100	60/70	40/50	Polymer[5]
80/100	-	-	0, *20, 50*	2.5, 3.5, 4.5
60/70	0, *20*, *50*, 100	-	-	-
40/50	-	*50*	-	-

Note: There are 3 neat and 3 polymer modified binders (a total of 6 binders other than blends)

The above two updated matrices will result in a total of 22 binder types. However, only 15 binders types will have neat and polymer modification in the above design matrices. The binder coding for all binders to be tested in the study is presented in Table 2-6. All these binders will be fully characterized using conventional testing and Superpave tests as mentioned before in Tables 2-1 and 2-2, respectively. In summary, the asphalt binder testing on the 22 neat, modified and blended binders will at the most result in 22 binder PGs. As a result, the binder performance grades will be recommended for different regions for pavement construction in Pakistan according to the indigenous climatic (high and low temperature) needs.

[4] Percentage of column binder with row binder types for blending. For example, 0 means 0% 60/70 with 100% 80/100 binder.
[5] Percent of polymer

Table 2-6 Standard binder coding for the study

No.	Binder Code	Binder Description
1	APMB	Attock Polymer Modified
2	A6/7	Attock Pen 60/70
3	A8/10	Attock Pen 80/100
4	K4/5	Karachi Pen 40/50
5	K6/7	Karachi Pen 60/70
6	K8/10	Karachi Pen 80/100
7	BA6/7(20)8/10(80)	Blended Attock: Pen 60/70 (20%) and Pen 80/100 (80%)
8	BA6/7(50)8/10(50)	Blended Attock: Pen 60/70 (50%) and Pen 80/100 (50%)
9	PA8/10(1.35)	Polymer Modified Attock 80/100 with 1.35% Elvaloy
10	PA8/10(1.70)	Polymer Modified Attock 80/100 with 1.70% Elvaloy
11	PA8/10(2.00)	Polymer Modified Attock 80/100 with 2.00% Elvaloy
12	PA6/7(1.35)	Polymer Modified Attock 60/70 with 1.35% Elvaloy
13	PA6/7(1.70)	Polymer Modified Attock 60/70 with 1.70% Elvaloy
14	PA6/7(2.00)	Polymer Modified Attock 60/70 with 2.00% Elvaloy
15	BK6/7(20)8/10(80)	Blended Karachi Pen 60/70 (20%) and Pen 80/100 (80%)
16	BK6/7(50)8/10(50)	Blended Karachi Pen 60/70 (50%) and Pen 80/100 (50%)
17	BK8/10(20)4/5(80)	Blended Karachi Pen 80/100 (50%) and Pen 40/50 (80%)
18	BK8/10(50)4/5(50)	Blended Karachi Pen 80/100 (50%) and Pen 40/50 (50%)
19	BK6/7(50)4/5(50)	Blended Karachi Pen 60/70 (50%) and Pen 40/50 (50%)
20	PK8/10(2.5)	Polymer Modified Karachi 60/70 with 2.5% AC
21	PK8/10(3.5)	Polymer Modified Karachi 60/70 with 3.5% AC
22	PK8/10(4.5)	Polymer Modified Karachi 60/70 with 4.5% AC

Note: The shaded cells show the blended binders containing mixtures of two neat binders

2.2.4.2. Subtask 4.2: Asphalt mix performance evaluation using Superpave approach

Considering the climatic conditions in Pakistan and data collected in previous studies; Pakistan can be probably divided into four (4) climatic zones. This will require at least four (4) binder grades. Therefore, in this subtask, two (2) binder grades will be selected from each refinery (from subtask 2.2.41) for asphalt mix performance evaluation.

As mentioned in subtask 2.2.3.4, for the conventional mix testing (Marshall mix design approach) two typical NHA gradations will be selected (wearing and base courses). Similar two gradations will be used in this subtask. Two gradations, one for the wearing and other for the asphalt base layer will be selected using the Superpave gradation criteria. For Superpave testing, as mentioned earlier, four binders (from subtask 2.2.4.1) representing the four climatic zones will be selected for the preparation of asphalt mixes. Table 2-7 summaries the details of the test matrix.

Table 2-7 HMA mixture design (Superpave)

Gradation	Gradation	PG binder types			
		60/70 (A6/7)	60/70 P1 [PA6/7(1.35)]	60/70 P2 [PA6/7(1.70)]	60/70 (K6/7)
Wearing[5]	Fine	2[6]	2	2	2
	Coarse	2	-	-	-
Base[7]	Fine	2	-	-	-
	Coarse	2	-	-	-

Note: The PG binder types will represent four climatic environments in Pakistan. Mix design will be conducted based on Superpave methodology using 3 replicates. For the optimum mix design make 2 replicates for simple performance tests using gyratory compaction.

The above matrices (Tables 2-3 and 2-7) will result in a total of 10 mixes. These mixes will be evaluated for permanent deformation and fatigue cracking using the approach developed under the simple performance testing. For all the asphalt mixes in Table 2-7, two samples will be prepared using gyratory compactor for performance testing. The performance testing will include dynamic modulus (E^*) testing at five temperatures (10, 40, 70, 100 and 130 ^{o}F) and at each temperature five frequencies (25, 10, 5, 1, and 0.5 Hz). These test results will be used to develop master curves for each mix type for a range of temperatures and frequencies. The E^* master curves will be utilized to evaluate the long-term performance (cracking and rutting) of these mixes in the field. In addition, Flow Number (FN) and Flow Time (FT) tests will be conducted for each mix type to evaluate their permanent deformation (rutting) propensity at high temperature.

In summary, the asphalt mixes will be evaluated for their potential performance in the field. A detailed comparison will be made between the performance-related properties (E^*, FN, and FT) of the asphalt mixes using the state-of-the-practice (subtask 2.2.3.4) and Superpave approach (this subtask). The difference in performance-related properties will highlight the expected benefits of adopting the Superpave methodology in Pakistan.

2.2.5 Task 5: Training of Pakistani Professional or Technical Staff in the USA

Two selected professionals from UET will be invited to the USA for two-week training courses at one of the Superpave training center. These training at present are carried out by National Center for Asphalt Technology (NCAT) and Asphalt Institute (TAI) on regular basis. NCAT has developed a training program for college and university civil engineering faculty so that the faculty will be able to develop and offer undergraduate and graduate courses in asphalt technology. The training is designed to allow graduates to have an educational background in HMA (a predominant highway paving material) and

[5] Wearing course gradation according to NHA specification (use highest or typical gradation type)
[6] Number of replicates
[7] Base course gradation according to NHA specification (use highest or typical gradation type)

asphalt technology and to encourage more students to pursue a career in civil engineering and construction. The purpose of this intensive course is to provide a basic understanding of all phases of HMA technology. Upon completion, the participants will be able to make knowledgeable decisions related to HMA pavements and communicate effectively with asphalt specialists when the need arises. NCAT also offers a short course that is tailored to the needs of laboratory technicians for HMA production and testing. The research plan also calls for sending at least ONE technicians/junior lecturers to NCAT for training.

2.2.6 Task 6: Training and Evaluation of Existing HMA Technology in Pakistan

Seminars and training sessions will be conducted at UET and NHA in Pakistan. Drs. Baladi, Bari and Haider from MSU and Dr. Mirza from UET will deliver lectures and the necessary training materials to professionals and academicians in Pakistan. As mentioned in the scope of the project, these sessions will be successively conducted in three important areas of highway engineering including HMA mix design, pavement structure and rehabilitation designs. Their lectures will cover state-of-the-art developments in the designing of HMA mixes, new Mechanistic-Empirical Pavement Design Guide (M-E PDG) for the design and rehabilitation of flexible pavements. Once again, the course participants will include professionals and technicians from educational institutions (Engineering Universities), government agencies (Federal and Local Governments) and private sector (Consultants and Construction industry). While these activities will be helpful in transferring valuable knowledge to Pakistan, they will also assist in knowing the local problems and needs related to asphalt mixes. In addition, the USA research team will visit a maximum of three HMA mix plants, three laboratory facilities and three construction sites in Pakistan to assess the current local practices. This will help the research team to identify the weak links in the highway construction process and will help the team to identify specific needs in the appropriate technology.

2.2.7 Task 7: Reports

Two types of reports will be written and submitted by the research team including:

1. Progress reports, will be submitted on a yearly basis. The progress report will address the findings of the research team, expenditures, and the budget balance.

2. Final report, will be submitted at the conclusion of the study and will include:

    - Description of the testing and analysis methodologies
    - Guidelines and specifications for Superpave binder tests
    - All laboratory and field test results summarized in a format easy to read and reference
    - Detail analysis of test results, and
    - Guidelines for Superpave methodology and the implementation plan

2.3 DATA ANALYSES

A comprehensive and systematic strategy will be adopted to analyze the laboratory data. The laboratory data encompass the outputs from the test matrices mentioned under Tasks

3 and 4. The results from the material testing will be used to design asphalt mixes using Superpave methodology and in determining the differences in expected performance between Marshal and Superpave approaches in order to maximize pavement performance.

Two types of analyses will be performed on the laboratory data:

- Statistical analyses—to study the effects of several factors on the response variables for material test matrices.
- Mechanistic analyses based on material behavior models.

In the statistical analyses, comparison of means (*t*-tests) and analysis of variance (ANOVA) (both univariate and multivariate) will be conducted to determine the statistical significance. The statistical significant results should also pass the test of practical significance. The practical significant threshold will be established through literature search and experience of the research team. Results of the analyses will assist the research team to relate the asphalt pavement performance to the material properties and to determine the threshold values for the proportions of materials. The purpose of the mechanistic, on the other hand, is to determine the strength and moduli of the trial asphalt mixtures. Results of both analyses will be used to establish the asphalt mix design guidelines.

2.4 GUIDELINES FOR ASPHALT MIX DESIGN

Finally, the research team will use the results of the data analyses to develop guidelines for asphalt mix design by using a systematical approach as shown in Figure 2-1. The guidelines will include asphalt mix design specifications and QC/QA protocol. The specifications will address the following issues:

- Develop an enhanced asphalt mixture design system for Pakistan based on the state-of-the-art methodologies (including the new Superpave technology), on the material properties and on the environmental conditions in Pakistan. The developed guideline will include:
    - Methodology for characterizing asphalt binder performance grades (PG) according to climatic needs
    - Determination of asphalt binder modifications based on the blending of existing binder grades and polymers
    - Procedures for characterizing HMA mixtures using modified binders
    - Measures to link HMA mix design and its performance in the laboratory to improve field pavement performance
- Assist the highway authorities in developing quality assurance/quality control protocol regarding the new asphalt mixes and methodologies.
- Develop and initiate an integrated pavement research program in cooperation with personnel from the University of Engineering and Technology (UET), the highway authorities and the industry in Pakistan.

- Develop a framework for Pakistan to implement the National Cooperative Highway Research Program (NCHRP) 1-37A Design Guide for Mechanistic Empirical Design of New and Rehabilitated Pavement Structures (M-E PDG).

CHAPTER 3 - LITERATURE REVIEW AND CURRENT PRACTICES

3.1 INTRODUCTION

From October 1987 through March 1993, the Strategic Highway Research Program (SHRP) conducted a $50 million research effort to develop new methodologies to specify, test and design asphalt materials. At the end of this program, the Federal Highway Administration (FHWA) assumed a role of implementation of SHRP research within the United States. An integral part of FHWA's implementation strategy was to develop a nationally accessible training center aimed at educating both highway agency and industry personnel in the proper use and application of the final SHRP asphalt products referred to as Superpave™. This project was administrated by the FHWA's Office of Technology Applications and the National Asphalt Training Center (NATC).

It is with this objective in mind that the Civil and Environmental Engineering Department at Michigan State University and the Department of Transportation Engineering at the University of Engineering and Technology, Lahore initiated this project. The main objective is to setup a fully functional Superpave laboratory which will then be used for training and implementation of this new Superpave technology in Pakistan. The implementation is aimed at both the provincial and the federally administered roads with a hope that it will improve the pavement performance and result in savings of billions of Rupees.

This chapter documents the literature related to asphalt concrete. The general asphalt binder behavior is presented first. Subsequently, the existing knowledge on material characterization with respect to conventional and the state-of-the-art practices is documented.

3.2 BACKGROUND

Asphalt concrete (sometimes referred to as "hot mix asphalt" or simply "HMA") is a paving material that consists of asphalt binder and mineral aggregate. The asphalt binder—asphalt cement or modified asphalt cement, acts a binding agent to glue aggregate particles into a dense mass. When bounded, asphalt binder and mineral aggregate acts as a stone framework to impart strength and toughness to the system. Because HMA contains both asphalt binder and mineral aggregate, the behavior of the mixture is affected by the properties of the individual components and they interact with each other in the system.

Asphalt binders are valuable because of their strong, adhesive, waterproof and durability characteristics. They provide limited flexibility to mixtures when combined with mineral aggregates. In addition, asphalt binders are also highly resistant to reaction with most acids, alkalis, and salts. Although binders exist in a solid or semi-solid state at ordinary atmospheric temperatures, asphalt binders may be liquefied by applying heat, dissolving in petroleum solvents or emulsifying in water. Asphalt binders are viscoelastic materials and depend on both temperature and rate of loading. Figure 3-1 shows that the asphalt binder flow could be the same for one hour at 60 °C or 10 hours at 25 °C.

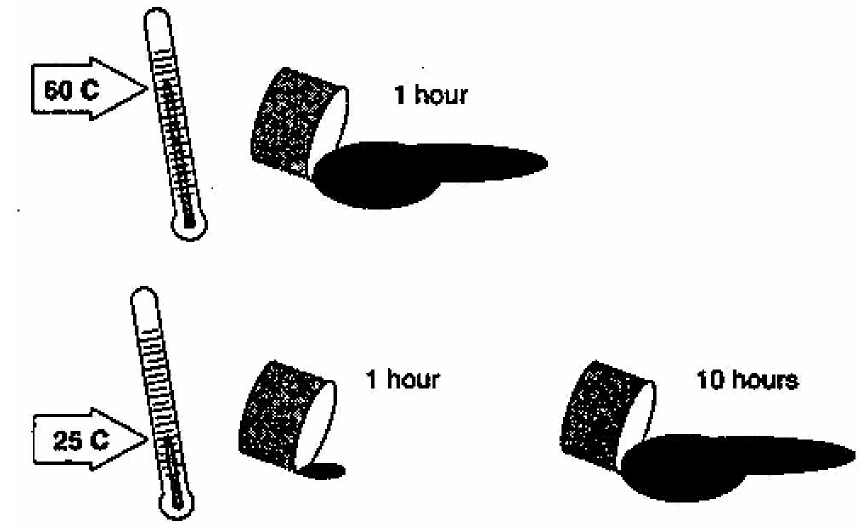


Figure 3-1 Asphalt binder's flow behavior (*1, 2*)

In addition to the temperature and time dependent behavior, another important characteristic is the age hardening property of asphalt cement. Because asphalt cements are composed of organic molecules, they react with oxygen from the environment. As a result the structure and composition of the asphalt molecules are changed, causing oxidative or age hardening resulting in brittleness of asphalt cement. Oxidative hardening occurs at a slow rate, but is faster in warmer climates. Improperly compacted asphalt layers have higher levels of air voids, which allow more oxidative hardening. In practice, a considerable amount of oxidative hardening occurs before the asphalt is placed, especially in a hot mix facility. Other forms of hardening include volatilization during hot mixing and construction, and physical hardening during prolonged exposure in much lower temperatures.

For any aging condition of asphalt cement, the viscosity of the asphalt binder at the temperature of interest is determined from the viscosity temperature relationship defined by Equation 3.1(*3-5*).

$$\log \log \eta = A + VTS \log T_R \tag{0.1}$$

where:

η = viscosity, cP
T_R = temperature, degree Rankine
A = regression intercept
VTS = regression slope (viscosity temperature susceptibility parameter)

This relationship is applicable to virgin asphalt cements and for a wide variety of modified asphalts, when the modification percentages are not excessively high (less than 2 to 3% of polymers). Although Equation 1 is usually used with data from viscosity measurements at 60 ^{o}C and 135 ^{o}C to develop mixing and compaction temperatures, it can be extended to lower temperatures using ring and ball softening point and penetration data. Research by Shell that was later confirmed by Mirza and Witczak indicates that for most unmodified asphalts, the ring and ball softening point corresponds to a viscosity of 13,000 poise (*6*). Penetrations from tests by using 100 g loading for 5 seconds can be converted to viscosity using Equation 3.2.

$$\log \eta = 10.5012 - 2.2601 \log(Pen) + 0.00389(\log(Pen))^2 \qquad (0.2)$$

where:

η = viscosity, P
Pen = measured penetration for 100g, 5 sec loading, 0.10 mm.

Thus, from a combination of penetration, ring and ball softening point, and kinematic and absolute viscosity measurements that are routinely measured to insure compliance with viscosity grading, the viscosity of the binder over a wide range of temperatures can be determined from Equations 1 and 2. Mirza and Witczak also developed equations, which shift the viscosity temperature relationship of the original binder for short-term aging (that occurs during mixing and compaction) and for long term in-situ aging. These equations take into account the aging potential of the binder, the temperature in the pavement, and the time in service (*6*).

If all the above mentioned factors are not taken into account, distress including permanent deformations (rutting) and cracking (fatigue) under repetitive wheel loads due to brittleness is very likely to occur. Therefore, it is imperative to consider both pavement performance and material behavior simultaneously at the design stage. The current state-of-the-practice for designing such materials in Pakistan is based on the old methods (Marshall Method), which do not consider the link between design and performance. Premature rutting in asphalt concrete layers on many of the newly constructed national highways and motorways in Pakistan are testimony to a vital need to resolve this material-related distress issue. Currently, Highway Authorities in Pakistan lack the knowledge, the relevant equipment and the proper specifications to implement the new SUPERPAVE mix design methodology. It is believed that Superpave technology will result in improved road infrastructure producing significant savings in road maintenance costs. In addition, improved infrastructure can indirectly create substantial benefits to the environment and socio-economic conditions.

The following section covers the conventional test methods used to characterize asphalt mixtures.

3.3 CONVENTIONAL ASPHALT MIXTURE DESIGN METHODS

As mentioned before, hot-mix asphalt (HMA) mixtures have two main ingredients—asphalt binders and mineral aggregates. Therefore, in order to characterize HMA, generally, binder properties as well as overall mixture behavior requires assessment in a laboratory before used in construction. These evaluations will determine the adequacy of HMA regarding its performance in the field under local climatic conditions and traffic loads.

3.3.1 Asphalt Binder Characterization

Many tests are available to characterize asphalt binders. Some tests are commonly used by the highway agencies, while others are used for research. Since the properties of the asphalt are highly sensitive to temperature and time of loading, all asphalt binder tests must be conducted at specified temperatures and/or time of loading within very tight tolerance.

Asphalt binder specifications were historically developed around empirical physical property tests. Penetration, ductility, viscosity (absolute and kinematic), and softening point measurements etc. are examples of such tests. The binders are tested at a standard temperature and the test results are used to determine whether the material meets the specification criteria.

There are several limitations in the conventional characterization of asphalt binders. Pavement performance experience is required before the test results yield meaningful information. In addition, these tests do not give information for the entire range of typical pavement temperatures. For example, the viscosity tests only provide information about the behavior of asphalt cement at higher temperatures.. Penetration tests describe only the consistency at a medium temperature. Lower temperature behavior cannot be realistically determined from these tests to predict low temperature performance.

Furthermore, conventional binder specifications may classify different binders within the same grading, when in fact these asphalt binders may have very different temperature and performance characteristics. Figure 3- 2 shows three asphalt binders that meet the viscosity grade because they are within the specific viscosity limits at 60 ^{o}C, meet the minimum penetration at 25 ^{o}C, and reach the minimum viscosity at 135 ^{o}C. While binders *A* and *B* display the same temperature dependency, they have much different consistency at all temperatures. Binders *A* and *C* have the same consistency at low temperatures, but different high temperature consistency. Binders *B* and C have the same consistency at 60 ^{o}C, but share no other similarities. Because these binders meet the same grade specifications, one might erroneously expect the same characteristics during construction and the same performance during hot and cold weather conditions. Therefore the conventional tests alone are not sufficient to properly describe the viscoelastic and failure properties of asphalt cements that are needed to relate asphalt binder properties to mixture properties and to pavement performance.

3.3.1.1 Conventional binder tests

As stated earlier, asphalt binder specifications have been conventionally developed around physical property tests. Binder properties are typically tested at standard temperatures and the test results are used to determine whether the material meets the specification criteria. The conventional binder tests are summarized in Table 3-1 below.

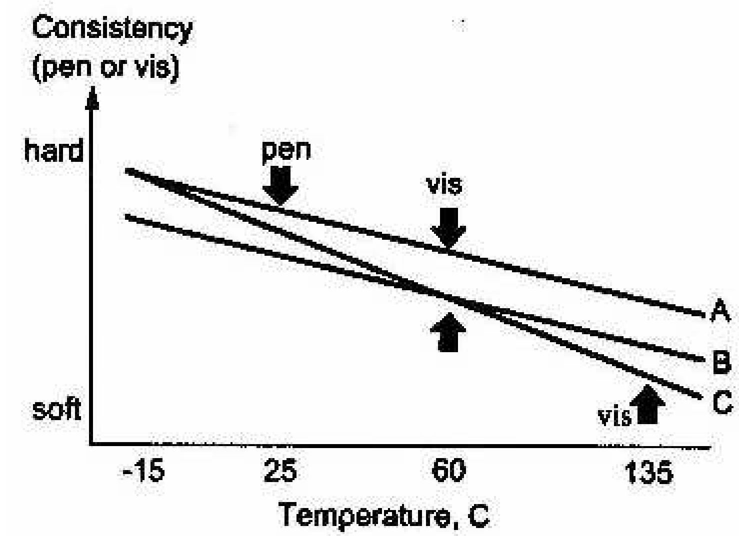


Figure 3-2 Variations of same viscosity-graded binders (*2, 7*)

Table 3-1 Conventional Binder Tests

Test	Property	Method	Conditions
Penetration	Penetration	AASHTO T49-93	15 °C & 25 °C, 100 grams load for 5 seconds
Ring and Ball	Softening point	AASHTO T53-92	Measured temperature
Absolute Viscosity	Absolute Viscosity	AASHTO T202-91	60 °C
Kinematic Viscosity	Kinematic viscosity	AASHTO T201-93	135 °C

Penetration test

This test covers the determination of the penetration of semi-solid and solid asphalt binders. The penetration of an asphalt binder is the distance in tenths of a millimeter that a standard needle penetrates vertically into a binder sample under fixed conditions of temperature, load, and time. The penetration test apparatus consists of a penetration needle, container, water bath, transfer dish for container, thermometers for water bath, timing device, and heater. The binder sample is heated and cooled under controlled conditions. The penetration is measured with a penetrometer using a standard needle under specified conditions. Typically, penetration tests are conducted at 25 ^{o}C using a 100 gm load for 5 seconds. This test is commonly used as a measure of consistency. Higher values of penetration indicate softer consistency and vice versa. Penetrations can be converted to viscosity using Equation 3.2.

Softening point test

This test covers the determination of the softening point of asphalt binders in the range from 30 ^{o}C to 157 ^{o}C (86 ^{o}F to 315 ^{o}F) using the ring-and-ball apparatus. The test apparatus consist of rings, pouring plate, metal balls, ball-centering guides, temperature controlled bath, ring holder and assembly, and thermometers etc. Two horizontal disks of binder, cast in shouldered brass rings, are heated at a controlled rate in a liquid bath, while each support a steel ball. The softening point is reported as the mean of the temperatures at which the two disks soften enough to allow each ball, enveloped in asphalt binder, to fall a vertical distance of 25 mm (1 inch). The softening point is used in the classification of asphalt binders and as one of the elements in establishing the uniformity of shipments or sources of supply. The softening point is indicative of the tendency of the binder to flow at elevated temperatures encountered in service. For most unmodified asphalt binders, the ring and ball softening point corresponds to a viscosity of 13,000 poise (*1, 6*).

Absolute viscosity test

This test covers the determination of the absolute viscosity of asphalt binders by vacuum capillary viscometer at 60 ^{o}C. It is applicable to materials having viscosity in the range of 0.036 to 200,000 poise. This test can measure viscosity of asphalt binder at both Newtonian and non-Newtonian binder conditions. The test apparatus consists of capillary type viscometer, thermometers, bath for immersion of the viscometer, vacuum system, timer, and electrical timing device etc. The time is measured for a fixed volume of asphalt binder to be drawn up through a capillary tube by means of vacuum, under controlled conditions of vacuum and temperature. The viscosity in poise is calculated by multiplying the flow time in seconds by the viscometer calibration factor. The viscosity of asphalt binder at 60 ^{o}C (140 ^{o}F) characterizes flow behavior and may be used for specification requirements for cutbacks and asphalt cements.

Kinematic viscosity test

This test covers the determination of the kinematic viscosity of asphalt binders by capillary viscometer at 135 ^{o}C. It is applicable to materials having kinematic viscosities in the range of 6 cSt to 100,000 cSt. *Kinematic Viscosity* is the ratio of the viscosity to the density of the binder. It is a measure of the resistance to flow of a liquid under gravity. The SI unit of kinematic viscosity is m^2/s. The CGS unit is Stoke (St.) that is equal to 1 cm^2/s. The centistoke (1 cSt = 10^2 St = 1 mm^2/s) is customarily used. The test apparatus consists of viscometer, thermometers, bath for immersion of the viscometer, timer, and electrical timing device etc. The time is measured for a fixed volume of asphalt binder to flow through the capillary of a calibrated glass capillary viscometer under an accurately reproducible and closely controlled temperature. The kinematic viscosity in cSt is then calculated by multiplying the efflux time in seconds by the viscometer calibration factor. The viscosity of asphalt binder at 135 ^{o}C (275 ^{o}F) characterizes pump-ability of an asphalt binder. The viscosity of a Newtonian liquid is calculated from its kinematic viscosity by multiplying the kinematic viscosity by the density of the liquid at the test temperature.

3.3.2 Hot-Mix Asphalt Mixture Characterization

As mentioned before, HMA consists of two basic ingredients, aggregate and asphalt binder. HMA mix design is the process of determining the proper proportioning of aggregate and binder in an optimum manner to achieve the desired performance of the mix in the field. HMA mix design has evolved as a laboratory procedure that requires several critical tests to make key characterizations of each trial HMA blend. Although these characterizations are not comprehensive, they can give the mix designer a good understanding of how a particular mix will perform in the field during construction and under subsequent traffic loading.

HMA is a complex mixture upon which many different, and sometimes conflicting, performance demands are placed. It must resist deformation and cracking, be durable over time, resist water damage, provide a good tractive surface, and yet be inexpensive, readily made and easily placed. The above-mentioned properties can be obtained by changing the three basic variables related to following component materials:

1. Aggregate: Items such as type (source), gradation and size, toughness and abrasion resistance, durability and soundness, shape and texture as well as cleanliness can be measured, judged and altered to some degree.
2. Asphalt binder: Items such as type, durability, rheology, purity as well as additional modifying agents can be measured, judged and changed to some degree.
3. The ratio of asphalt binder to aggregate: Usually expressed in terms of percent asphalt binder by the total weight of HMA, this ratio has a profound effect on HMA pavement performance. Because of the wide differences in aggregate specific gravity, the proportion of asphalt binder expressed as a percentage of total weight can vary widely even though the volume of asphalt binder as a percentage of total volume remains quite constant.

Thus, by manipulation of the above three variables, mix design seeks to achieve the following performance-related qualities in the final HMA product:

3.3.2.1 Deformation resistance (stability)

HMA should not distort (rut) or deform (shove) under traffic loading. HMA deformation is related to one or more of the following properties (*2, 7, 8*):

- <u>Aggregate surface and abrasion characteristics</u>: Rounded particles tend to slip over each other causing HMA distortion under load while angular particles interlock with one another providing a good deformation resistant structure. Softer and brittle particles cause mix distortion because they tend to break apart under agitation or load. Tests for particle shape, texture, durability and soundness can identify problematic aggregate sources. These sources can be avoided, or at a minimum, aggregate with good surface and abrasion characteristics can be blended to provide better overall characteristics.
- <u>Aggregate gradation</u>: Gradations with excessive fines (either naturally occurring or caused by excessive abrasion) cause distortion because the large amount of fine particles tend to push the larger particles apart and act as lubricating ball-bearings between these larger particles. A gradation resulting in low voids in mineral aggregate (VMA) or excessive asphalt binder content can have the same effect. Gradation specifications are used to ensure acceptable aggregate gradation.

- Asphalt binder content: Excess asphalt binder content tends to lubricate and push aggregate particles apart making their rearrangement easier under load. The optimum asphalt binder content as determined by mix design should prevent this phenomenon and form a stable mixture under traffic loads.
- Asphalt binder viscosity at high temperatures: In the hot summer months, asphalt binder viscosity is lowest and as a result, the HMA pavement layer deforms more easily under heavy axle loads. Specifying an asphalt binder with a minimum high temperature viscosity (as can be specified in the Superpave asphalt binder selection process) ensures adequate high temperature viscosity.

3.3.2.2 Fatigue resistance

The HMA pavement layer should not crack when subjected to repeated loads over time. HMA fatigue cracking is related to asphalt binder content and stiffness. Higher asphalt binder contents will result in a mix that has a greater tendency to deform (i.e., more flexible) rather than fracture under repeated load. The optimum asphalt binder content as determined by mix design should be high enough to prevent excessive fatigue cracking and at the same time should resist excessive deformation. The use of an asphalt binder with lower stiffness will increase a mixture's fatigue life by providing greater flexibility.

3.3.2.3 Low temperature cracking resistance

HMA should not crack when subjected to low ambient temperature cycles over time. Low temperature cracking is primarily a function of the asphalt binder low temperature stiffness. Specifying asphalt binder with adequate low temperature properties (as can be done in the Superpave asphalt binder selection process) should prevent, or at least limit, low temperature cracking.

3.3.2.4 Durability

HMA should not suffer excessive aging during production and service life. The aging of asphalt binders will make them stiffer and more brittle. HMA durability is related to one or more of the following properties:

- The asphalt binder film thickness around each aggregate particle: If the film thickness surrounding the aggregate particles is insufficient, the aggregate may become accessible to water through holes in the film. If the aggregate is hydrophilic, water will displace the asphalt film and asphalt-aggregate cohesion is lost. This process is typically called stripping of asphalt mixture. The optimum asphalt binder content as determined by mix design should provide adequate film thickness. In addition thinner asphalt layer around the aggregates is more likely to oxidize quicker resulting in a brittle mix which may be prone to excessive cracking.
- Air voids: Excessive air voids (on the order of 8 percent or more) increase HMA permeability and allow oxygen easier access to more asphalt binder thus accelerating oxidation and volatilization. To address this, HMA mix design seeks

to adjust items such as asphalt content and aggregate gradation to produce design air voids of about 4 percent.

Considering these objectives, the challenge in mix design is then to develop a relatively simple procedure with a minimal amount of tests and samples that will produce a mix meeting all the above HMA qualities.

3.3.3 Conventional HMA Mix Design Methods

To achieve the above-mentioned HMA mixture properties, two commonly used HMA mix design methodologies include:

1. Hveem mix design
2. Marshall mix design

A brief description of each method is presented next.

3.3.3.1 Hveem Mix Design

Francis Hveem of the California Division of Highways originally developed the basic concepts of the Hveem mix design method in the late 1920s and 1930s (*1*). Currently, the Hveem method is used by several western states in the USA. The basic design philosophy for the Hveem method can be summarized in the following three points:

1. HMA requires enough asphalt binder to coat each aggregate particle to an optimum film thickness (allowing for its absorption into the aggregate).
2. HMA requires sufficient stability to resist traffic loading. The stability is generated by internal friction between aggregate particles and cohesion (or tensile strength) created by the binder.
3. HMA durability increases with thicker asphalt binder film thickness.

Based on this philosophy, the design asphalt content is selected such that it results in the highest durability without dropping below a minimum allowable stability. In other words, as much asphalt binder as possible should be used while still meeting minimum stability requirements.

3.3.3.2 Marshall Mix Design

The basic concepts of the Marshall Mix design method were originally developed by Bruce Marshall of the Mississippi Highway Department around 1939 and then refined by the U.S. Army (*1, 9-11*). Currently, the Marshall method is used in some capacity by about 38 states within the USA; however, the Marshall Mix design method is being replaced by Superpave mix design approach. The next section describes the reasons for this current transition.

The Marshall method seeks to select the asphalt binder content at a desired density that satisfies minimum stability and range of flow values. The Marshall method, like other mix design methods, uses several trial aggregate-asphalt binder blends (typically 5 blends with 3 samples each i.e., a total of 15 specimens), each with a different asphalt binder content. Then, by evaluating the performance of each trial blend, optimum asphalt binder content can be selected. The trial blends must contain a range of asphalt contents both above and below the optimum asphalt content. Therefore, the first step in sample preparation is to estimate the optimum asphalt content. Trial blend asphalt contents are then determined from this estimate.

Specimen preparation (compaction) is carried out by making use of the Marshall hammer; a device that applies pressure to a sample through a tamper foot. Some hammers are automatic while some are hand operated. Figure 3-3 shows the hammer used in this research project.

Figure 3-3 Marshall drop hammer

The standard Marshall method sample preparation procedure is contained in AASHTO T 245: "*Resistance to Plastic Flow of Bituminous Mixtures Using the Marshall Apparatus*". A sample prepared using the Marshall hammer is tested for Marshall stability and flow values. The Marshall stability and flow test provides the performance prediction measures for the Marshall mix design method. The stability portion of the test measures the maximum load supported by the test specimen at a loading rate of 50.8 mm/minute (2 inches/minute). The load is increased until it reaches a maximum and when the load begins to decrease, the loading is stopped and the maximum load is recorded. Figure 3-4 shows the Marshall test apparatus that is being used on this research project. The readings of stability are taken manually.

Figure 3-4 Marshall testing apparatus

Typical Marshall design stability and flow design criteria are shown in Table 3-2.

Table 3-2 Typical Marshall design criteria (Asphalt Institute, 1979)

Mix Criteria	Light Traffic ($< 10^4$ ESALs)		Medium Traffic (10^4 - 10^6 ESALs)		Heavy Traffic ($> 10^6$ ESALs)	
	Min.	Max.	Min.	Max.	Min.	Max.
Compaction (No. of blows on each end of the sample)	35		50		75	
Stability (minimum)	2224 N (500 lb)		3336 N (750 lb)		6672 N (1500 lb)	
Flow (0.25 mm)	8	20	8	18	8	16
Air Voids (%)	3	5	3	5	3	5

3.4 CURRENT PRACTICES FOR HMA MIX DESIGN IN PAKISTAN

The objective of this task is to review and evaluate the existing Pakistani procedures, specifications, and practices regarding all the constituent materials of HMA mixes. These include specifications and practices relative to the asphalt binders, coarse and fine aggregates, and mineral filler used in Pakistan. The results of this review will be used to determine the differences and similarities between the Pakistani practices and the state-of-the-art practices reviewed in Task 1.

At present, Pakistan has total of 250,000 kilometers of road network (150,000 km high class and 100,000 km low class) that carries millions of automobiles, buses, and heavy trucks every day. Approximately 8,000 kilometers are maintained by the National Highway Authority (NHA) and falls under the category of National Highway System (NHS). The remaining roads are maintained by the four provinces and are called the provincial roads. The NHA which is a federal authority has its own specifications for the construction and maintenance of road under the NHS. Similarly, the four provinces have developed their own specifications for the maintenance and construction of provincial roads. However, most of the provincial specifications are developed from the NHA General Specification. A summary of items from NHA specifications relevant to this project are discussed below.

3.4.1 NHA Specifications

The NHA General Specifications cover the following types of activities:

1. Construction of new highways, bridges and allied works
2. Rehabilitation and improvement of existing road network
3. Maintenance of existing roads and structures

The NHA Specifications describe the requirements and procedures for execution of work items to achieve required workmanship and quality. The materials to be used shall conform to specifications and testing procedures as per the American Association of State Highway and Transportation Officials (AASHTO), the American Society for Testing and Materials (ASTM) or British Standard (B.S.) as indicated in their latest editions. Samples of materials for laboratory tests and their subsequent approval shall be utilized according to these references. A brief description of items that are relevant to this project are presented below. This includes specification related to:

- Asphalt binders
- Coarse and fine aggregates
- Mineral filler
- Asphalt mix design

3.4.1.1 Asphalt binders

Asphalt binder to be mixed with the aggregate for producing asphaltic wearing (or surface) and base courses shall be asphalt cement having penetration grade 40-50, 60-70 or 80-100 as specified by the Engineer. Generally, the asphalt binder will meet the requirements of AASHTO M 20. The general requirements for the required binder grades are given in Table 3-3.

Table 3-3 Requirement for asphalt cement (AASHTO M 20), NHA General Specs

Description	40-50		60-70		80-100		120-150	
	min	max	min	max	min	max	min	max
Penetration at 77 °F (25 °C) 100 g, 5 sec	40	50	60	70	80	100	120	150
Flash point, Cleveland Open Cup, °F (°C)	450 (232)	- -	450 (232)	- -	450 (232)	- -	425 (218)	- -
Ductility at 77 °F (25 °C) 5 cm per min, cm	100	-	100	-	100	-	100	-
Solubility in trichloroethylene, %	99	-	99	-	99	-	99	-
Thin-film oven test, 1/8 in (3.2 mm), 325 °F (163 °C) 5 hours loss on heating, %	-	0.80	-	0.80	-	1.0	-	1.3
Penetration, of residue, % of original	58	-	54	-	50	-	46	-
Ductility of residue at 77 °F (25 °C) 5 cm per min, cm	-	-	50	-	75	75	100	-

The above requirements are in terms of penetration grade. However, the grade selection for a specific location is based upon the environmental condition. In this situation, the viscosity grading system is used as is given in Table 3-4.

Table 3-4 Selection of asphalt grade based on environmental temperature, NHA General Specification (Item 301)

Temperature Condition	Asphalt Grade
Cold, mean annual air temperature ≤ 7 °C (45 °F)	AC-10 AR-4000 80 / 100 pen
Warm, mean annual air temperature between 7 °C (45 °F) and 24 °C (75 °F)	AC-20 AR-8000 60 / 70 pen
Hot, mean annual air temperature > 24 °C (75 °F)	AC-40 AR-8000 40 / 50 pen

It is worth mentioning that requirements relating to the viscosity grading system are given in the NHA General Specifications.

3.4.1.2 Coarse and fine aggregates

The typical testing requirements as specified in the General Specification are given in Table 3-5.

Table 3-5 Testing requirements for coarse and fine aggregates
(NHA General Specifications: Item No. 203)

Material	Test	Designation
Coarse Aggregate	Gradation	AASHTO T 27
	Abrasion	AASHTO T 96
	Sodium Sulphate Soundness	AASHTO T 104
	Stripping	AASHTO T 182
	Fractured faces	Visual
	Flat and Elongated Particles	Visual
	Specific Gravity and Absorption	AASHTO T 85
Fine Aggregate	Sand Equivalent	AASHTO T 176
	Plasticity Index	AASHTO T 89 and T 90
	Specific Gravity	AASHTO T 84
	Friable Particles	AASHTO T 112

Mineral aggregate for bituminous (asphaltic) base course shall consist of coarse aggregate, fine aggregate and filler material, if required. All of these materials should conform to the following requirements:

1. Coarse aggregate that is the material retained on the AASHTO No. 4 sieve shall consist of crushed rock or crushed boulder. It shall be clean, hard, tough, sound, durable and free from decomposed stones, organic matter, shale, clay lump or other deleterious substances. Rocks or boulders, from which coarse aggregate is obtained, shall be of uniform quality throughout the quarry.
2. The crushing shall be so regulated such that at least ninety-five (95) % by weight of material retained on the AASHTO No. 4 sieve shall consist of pieces with at least two (2) mechanically fractured faces and when tested for stability of bituminous mix shall show satisfactory stability.
3. Fine aggregate, material passing the No. 4 sieve, shall consist of one hundred (100) % crushed material from rock or boulder. No natural sand will be allowed in the mix.

In addition, the coarse and fine aggregates shall meet the following applicable requirements:

1. The percentage of wear by the Los Angeles Abrasion test (AASHTO T 96) shall not be more than forty percent (40%).
2. The loss when subject to five (5) cycles of the Sodium Sulphate Soundness test (AASHTO T 104) shall be less than twelve percent (12%).

3. The Sand Equivalent (AASHTO T 176) determined after all processing except for addition of asphalt cement shall not be less than forty-five (45).
4. Fine aggregates shall have a liquid limit not more than twenty-five percent (25%) and a Plasticity Index of not more than six percent (6%) as determined by AASHTO T89 and T90.
5. The portion of aggregate retained on the 9.5 mm (3/8 in) sieve shall not contain more than twenty (20) % by weight of flat and/or elongated particles (ratio of maximum to minimum dimensions = 3:1).
6. Stripping test shall be performed on coarse aggregates as described under AASHTO T 182 and only material that satisfies the test shall be allowed.
7. The coarse aggregates shall be checked if desired by the Engineer for cationic and anionic behavior so that their affinity with the bitumen to be used is verified.
8. Petrographic examination of the coarse aggregate shall be conducted if so directed by the Engineer.

The gradation requirements of the asphaltic concrete paving mixtures for base and wearing courses for Class A and/or Class B are presented in Tables 3-6 and 3-7.

Table 3-6 Combined aggregate grading requirements for asphaltic base layer, NHA Specifications (Item 203-1)

Mix Designation	Class A	Class B
Use	Leveling / Base	Leveling / Base
Compacted Thickness	70~90 mm	50 ~ 80 mm
U.S. Standard Sieve Size	Percent Passing by Weight	
2" (50 mm)	100	–
1½" (38 mm)	90~10	100
1" (25 mm)	–	75~90
3/4" (19 mm)	56~75	65~80
1/2" (12.5 mm)	–	55~70
3/8" (9.5 mm)	–	45~60
No. 4 (4.75 mm)	23~40	30~45
No. 8 (2.38 mm)	15~30	15~35
No. 50 (0.300 mm)	4~10	5~15
No. 200 (0.075 mm)	3~16	2~7
Asphalt Content weight percent of total mix	3 (minimum)	3 (minimum)

Table 3-7 Combined aggregate grading requirements for wearing course layer, NHA Specifications (Item 305-1)

Mix Designation	Class A	Class B
Compacted Thickness	50~80 mm	35~60 mm
Asphalt Content weight, percent of total mix	3.5 (min)	3.5 (min)
Combined Aggregate Grading Requirements		
Sieve Designation	Percent Passing by Weight	
1" (25 mm)	100	-
3/4" (19 mm)	90~100	100
1/2" (12.5 mm)	-	75~90
3/8" (9.5 mm)	56~70	60~80
No. 4 (4.75 mm)	35~50	40~60
No. 8 (2.38 mm)	23~35	20~40
No. 50 (0.300 mm)	5~12	5~15
No. 200 (0.075 mm)	2~8	3~8

Some of the general observations are:

1. For both the gradations, Class B gradation is finer than the Class A gradation. This is because Class A is generally recommended for high traffic areas and vice versa.
2. In the case of Asphalt Base, Class A as well as Class B are very close to the maximum density line resulting in very little space for the air voids. These types of mixes are relatively more prone to rutting. None of the two gradations pass through the control points as is recommended in the SUPERPAVE gradation requirements.
3. In the case of the wearing course, both gradations meet the SUPERPAVE grading requirements. Each gradation passes through the control points and not through the restricted zone.

3.4.1.3 Mineral filler

When the combined grading of the coarse and fine aggregates is deficient in material passing the No. 200 sieve, additional filler material shall be added. The filler material shall consist of finely divided rock dust, hydrated lime, hydraulic cement or other suitable minerals. However, in case the coarse aggregates are of quartzite nature, then hydrated lime or a better material shall be allowed. At the time of use, it shall be sufficiently dry to flow freely. Filler material shall conform to following gradation:

US Standard Sieve	Percent Passing by Weight
No. 30	100
No. 50	95~100
No. 200	70~100

In the case of the Superpave Mix Design approach, this is defined as the dust proportion and is computed as the ratio of the percentage by weight of aggregate finer than the 0.075 mm sieve to the effective asphalt content expressed as a percent by weight of the total mix. Effective asphalt content is the total asphalt used in the mixture less the percentage of absorbed asphalt. Dust proportion is used during the mixture design phase as a design criterion. An acceptable dust proportion is in the range from 0.6 to 1.6, inclusive for all mixtures. Low dust proportion values are indicative of mixtures that may be unstable and high dust proportion values indicate mixtures that lack sufficient durability.

3.4.1.4 Asphalt mix design

The composition of the asphaltic concrete paving mixtures for base and wearing courses shall conform to Class A and/or Class B shown in Tables 3-6 and 3-7. Separate requirement relative to mix design are specified for the base layer and the wearing course. A brief description of the specifications relative to the base and wearing course is given below:

Asphalt base layer

The asphalt concrete leveling / base course mixture is based on the Marshall Mix design and shall meet the following Marshall Test criteria:

Compaction, number of blows each end of specimen	75
Stability.............…..........……………………..........	1000 kg (min)
Flow, 0.25 mm (0.01 in)...........................………	8~14
Percent voids filled with bitumen…………………	55~65
Percent air voids in mix ....……….	4~8
Percent voids in mineral aggregates ……………..	According to Table 5.3, MS-2, Asphalt Institute, sixth edition 1993.
Loss in Stability ....………	25 % (max)

Mixes composed of larger size aggregates with maximum size up to 38 mm ($1^1/_2$ in) will be prepared according to modified Marshall method as per the MS-2 Asphalt institute, sixth edition, 1993 or the latest edition. The combined gradation should produce a smooth curve approximately paralleling grading band limits for the designated mix. The job mix formula (JMF) with allowable tolerances for a single test then becomes the job control grading band. If application of job mix tolerances results in a job control grading band outside the master grading band, the full tolerances shall still apply. Mineral filler and asphalt should be used to produce a final mix that, when compared to JMF shall be within the following limits:

Maximum Variation of Percentage of Materials

Retained No. 4 and larger	+ 7.0%
Passing No. 4 to No. 100 sieve	+ 4.0%
Passing No. 200	+ 1.0%

Asphalt Content

Weight percent of total mix	+ 0.3%

Asphalt wearing layer

The asphalt concrete wearing mixture is based on the Marshall Mix design and shall meet the following Marshall Test criteria:

Compaction, number of blows each end of specimen	75
Stability..	1000 kg (min)
Flow, 0.25 mm (0.01 in)..	8~14
Percent voids filled with bitumen................................	65~75
Percent air voids in mix	3.5~5.5
Percent voids in mineral aggregates	According to Table 5.3, MS-2, Asphalt Institute, sixth edition 1993.
Loss in Stability	20 % (max)

The major difference between the asphalt base and the wearing courses is in air voids and loss of stability. After the JMF is established, all mixtures furnished for the project represented by samples taken from the asphalt plant during operation, shall conform to the following ranges of tolerances:

Combined aggregates gradation

Retained No. 4 and larger	± 7.0%
Passing No. 4 to No. 100 sieves	± 4.0%
Passing No. 200	± 1.0%

Asphalt Content

Weight percent of total mix	± 0.3%

3.5 THE STATE-OF-THE-ART ASPHALT MIX DESIGN METHOD

Asphalt mixtures have typically been designed with empirical design procedures (for example, the Marshall Mix design method), meaning that field experience is required to determine if the laboratory test results correlate with the pavement performance. However, even with proper adherence to these procedures, good pavement performance could not be assured. Therefore, in 1987, the Strategic Highway Research Program (SHRP) began developing a new system for specifying asphalt materials (*7*). The final product of the SHRP asphalt research program is a new system called "Superpave", an abbreviation for *Su*perior *Per*forming Asphalt *Pave*ments. The Superpave includes a new system for selecting and specifying asphalt binder and has detailed mineral aggregate requirements.

The Superpave binder specification and mix design system includes various test equipment, test methods, and criteria. The unique feature of the Superpave system is that it is a performance-based specification system because the tests and analyses have direct relationships to the field performance. In addition, the Superpave asphalt binder tests measure physical and engineering properties that can be related to the expected field performance through engineering principles. These properties are determined at temperatures that are encountered by the in-service pavements. The new characterization tests and properties for asphalt binders and HMA mixes are described below.

3.5.1 Asphalt Binder Characterization

Under Superpave, SHRP researchers developed performance-based specification to control three major distress modes in asphalt pavements: rutting, fatigue cracking, and thermal cracking. These new specifications required a new set of test equipment and procedures. The SHRP Superpave binder specification was the ultimate product of the asphalt binder research work. It was hypothesized that this specification would be applicable for modified as well as unmodified asphalts. A unique feature of the Superpave binder specification is that the specified criteria remain constant, but the temperature at which the criteria must be achieved changes for various grades.

The central theme of the Superpave binder specification is its reliance on testing asphalt binders in conditions that simulate the three critical stages during the binder life. These three stages are as follows:

1. Original or tank condition that includes transportation, storage, and handling.
2. The second stage represents the asphalt aging, which occurs during the blending and agitation in the hot mixing facility and during construction. Aging the asphalt binder in a Rolling Thin Film Oven (RTFO) simulates this condition. This procedure exposes the thin binder films to heat and air, and simulates aging of this phase [AASHTO T240, ASTM D 2872].
3. The third stage represents service phase aging. For example, aging over a long period as a part of the hot-mix asphalt pavement layer. This stage is simulated by a Pressure Aging Vessel (PAV) [AASHTO PP1].

In addition to the three aging conditions, the tests need to be conducted at several temperatures for binder characterization. A brief description of binder testing is given below.

3.5.1.1 Asphalt binder testing

As stated earlier, asphalt binder specifications have conventionally been developed around physical property tests. Binder properties are conventionally tested at standard temperatures and the test results are used to determine whether the material meets the specification criteria. On the other hand, the most recent Superpave tests measure physical properties that can be related to the field performance by engineering principles. The Superpave binder tests are also conducted at temperatures that are encountered by in-service pavements. A summary of Superpave binder tests are shown in Table 3-8. All of these tests, including the RTFO and PAV tests are described briefly in the following paragraphs.

Table 3-8 Superpave binder tests

Test	Properties	Method	Conditions
Rotational Viscometer (RV)	Rotational viscosity	AASHTO TP-48	60°C, 80°C, 100°C, 121.1°C, 135°C and 176.7°C
Dynamic Shear Rheometer (DSR)	Complex Modulus & Phase Angle	AASHTO TP5-98	15,25,35,45,60,70,80,95,105 and 115°C 1,10,100 rad/sec
Bending Beam Rheometer (BBR)	Flexural creep stiffness and m-value	AASHTO TP1-98	Three temperatures; one above low temperature specification limit & two below that limit

Rolling thin film oven (RTFO) test

This test simulates the asphalt aging that occurs during the blending and agitation in the hot mixing facility and during construction (*2*). This procedure exposes thin binder films to heat and air in a designated test environment.

A double-walled electrically heated convection type oven is used and is shown in Figure 3-5. Other apparatus that are used are the flow meter, thermometer, glass container, and balance etc. The test procedure includes heating of asphalt to a fluid at or below 163 ^{o}C (325 ^{o}F). Thirty five (35 $\pm$ 0.5) gm of this liquefied binder is poured into each container, cooled to room temperature and the containers are arranged in the carriage so that the carriage is balanced. The carriage assembly is then rotated at 15 $\pm$ 0.2 rpm with air flow rate of 4000 $\pm$ 300 mile per min. Thus the moving film of asphalt binder is heated and oxidized i.e. aged in the oven for 75 minutes at 163 ^{o}C (325 ^{o}F). The effects of aging are determined from changes in physical test values as measured before and after the oven treatment.

This test simulates the asphalt aging that occurs during the blending and agitation in the hot mixing facility and during construction. The test indicates approximate change in properties of asphalt binder during conventional hot mixing at about 150 ^{o}C (302 ^{o}F) as indicated by viscosity measurements. It yields a residue that approximates the binder condition as incorporated in the pavement.

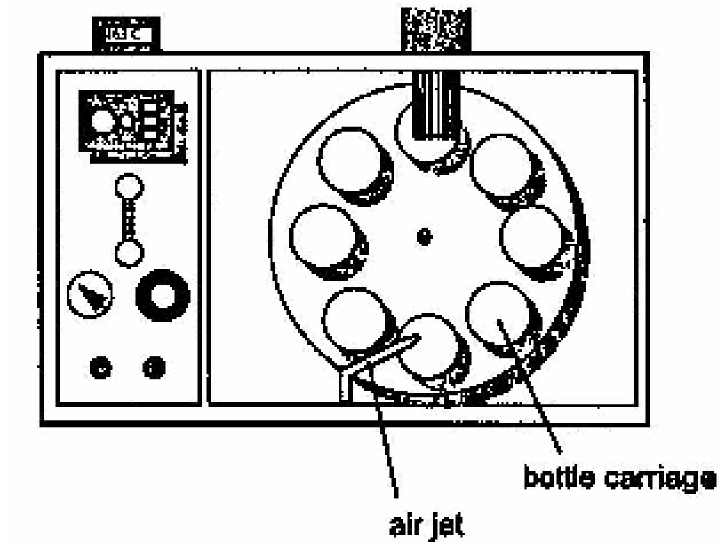


Figure 3-5 Rolling thin film oven (*2*)

Pressure aging vessel (PAV) test

This test covers the accelerated aging (oxidation) of asphalt binders by means of pressurized air and elevated temperature (*2, 8*). It simulates the in-service oxidative aging of asphalt binders that occurs in the pavement as a result of the combined effects of time, traffic, and environment. This test is intended for use with residue from RTFO test. The PAV test system is shown in Figure 3-6. This system consists of a pressure vessel, pressure and temperature controlling device, and a vacuum system.

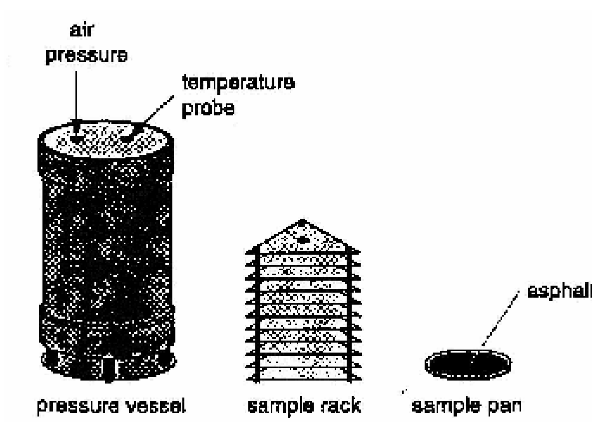


Figure 3-6 Pressure aging vessel (*2, 8*)

The asphalt binder is first aged using the RTFO (T240). A specified thickness of residue from the RTFO is then placed in Thin Film Oven Test (T179) stainless steel pans.

It is then aged at the specified aging temperature for 20 hours in a vessel pressurized with air to 2.10 MPa. The aging temperature is selected according to the grade of the asphalt binder. At completion of the PAV process, the asphalt binder residue is vacuum degassed.

Residue from this test may be used to estimate the physical or chemical properties of asphalt binders after five to ten years of in-service aging in the field.

Rotational viscometer test

Rotational viscosity tests determine the viscosity (flow characteristics) of asphalt binders at higher temperatures. In this test, the Brookfield™ (or similar) rotational coaxial viscometer is used with a Thermosel™ (or similar) temperature control system (*8*). The rotational viscometer automatically calculates the viscosity at the test temperature. The rotational viscosity is determined by measuring the torque required to maintain a constant rotational speed of a cylindrical spindle while submerged in a binder at a constant temperature. This torque is directly related to the binder viscosity. The rotational viscometer can measure viscosity of asphalt binder both at Newtonian and non-Newtonian binder conditions.

The test apparatus consists of a rotational viscometer system with a variety of spindles and a temperature control system as shown in Figure 3-7.

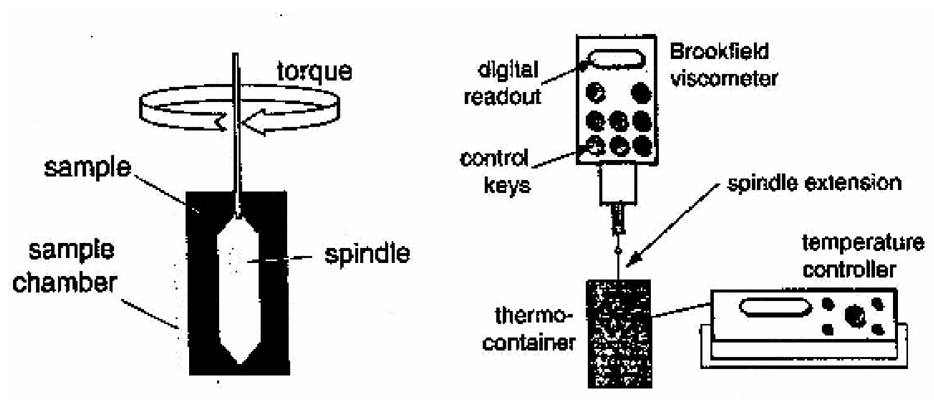


Figure 3-7 Rotational viscometer operation [5]

The apparatus consists of two main parts: the viscometer and the temperature control system. The temperature control system is set at the desired test temperature. The viscometer is then set to zero torque. The binder sample is heated and poured in the sample chamber, which is placed in the thermo-container. The amount of binder poured in the sample chamber is determined by the specific spindle to be used. Again the spindle should be chosen according to the assumed or known grading and viscosity of the binder. An appropriate spindle is heated, attached to the viscometer, and lowered into the sample. The spindle is rotated at an appropriate speed. For specification testing, the motor is set to operate at 20 rpm. This operation can be fixed manually or by setting a computer program. The units of viscosity are centipoise (cP).

The rotational viscometer calculates the viscosity of asphalt binders at intermediate to high temperatures. Thus, one is able to determine the flow characteristics of the asphalt binder to provide some assurance that it can be pumped and handled at the hot mix facilities. Unlike capillary tube viscometers, rotational viscometers have larger clearances between the components and, therefore, are applicable to modified as well as unmodified asphalt binders. The viscosity at different shear rates and different temperatures can be used to determine the viscosity-temperature susceptibility of asphalt binders.

Dynamic shear rheometer test

This test covers the determination of the dynamic shear modulus, G^* and the phase angle, δ of asphalt binder when tested in dynamic (oscillatory) shear using parallel plate test geometry (*2, 8*). It is applicable to asphalt binders having dynamic shear modulus values in the range from 100 Pa to 10 MPa. This range in modulus is typically obtained between 5 °C and 85 °C. Complex shear modulus (G^*) is the ratio calculated by dividing the absolute value of the peak-to-peak shear stress (τ) by the absolute value of the peak-to-peak shear strain (γ). Phase angle (δ) is the angle in radians between a sinusoidal applied strain and the resultant sinusoidal stress in a controlled-strain testing mode or between the applied stress and the resultant strain in a controlled-stress testing mode. Figure 3-8 shows the schematic diagram of dynamic shear rheometer (DSR).

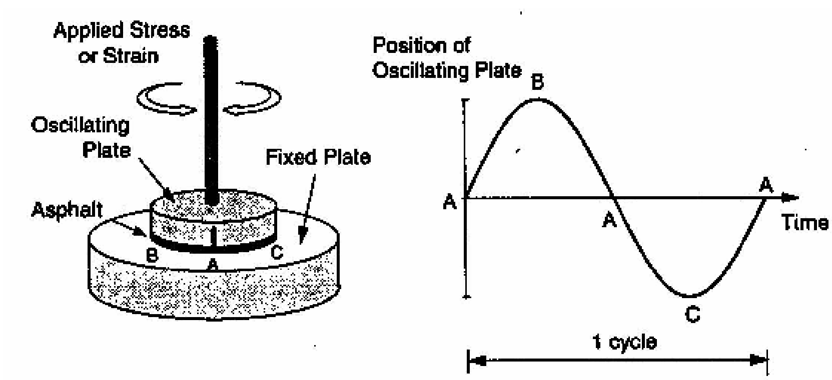


Figure 3-8 Dynamic shear rheometer operation (*1, 2*)

Superpave binder tests are conducted in the controlled stress mode. In the DSR operation asphalt is "sandwiched" between two parallel plates, one that is fixed and one that oscillates (see Figure 3-8). As the plate oscillates, the centerline of the plate at point A (indicated by the dark vertical line) moves to point B. From point B, the plate centerline moves back and passes point A to point C. From point C the plate centerline moves back to point A. This oscillation is one cycle and is continuously repeated during the DSR operation. The speed of oscillation is termed as frequency. All Superpave DSR binder tests are performed at a frequency of 10 radians per second, which is equal to approximately 1.59 Hz (cycle per second).

The DSR is used to characterize both viscous and elastic behavior by measuring the complex modulus (G^*) and phase angle (δ) of asphalt binders. G^* is a measure of the total resistance of a material to deformation when exposed to repeated pulses of shear stress. It consists of two components: elastic (recoverable) and viscous (non-recoverable).

δ is an indicator of the relative amounts of recoverable and non-recoverable deformation. The value of G^* and δ for asphalt binders are highly dependent on the temperature and frequency of loading. At high temperatures, asphalt binders behave like a viscous fluid with no capacity for recovery or rebound. In this case, the asphalt could be represented by the vertical axis (viscous component only) in Figure 3-9; there would be no elastic component of G^*, since $\delta = 90°$. Again at very low temperatures, asphalt behaves like an elastic solid that rebounds from deformation completely. This condition is represented by the horizontal axis (elastic component only) in Figure 3-9. In that case, there is no viscous component of G^*, since $\delta = 0°$. Under normal pavement temperature and traffic loading, asphalt binders act with the characteristics of both viscous liquids and elastic solids. By measuring G^* and δ, the DSR provides a more complete picture of the behavior of asphalt binder at pavement service temperatures.

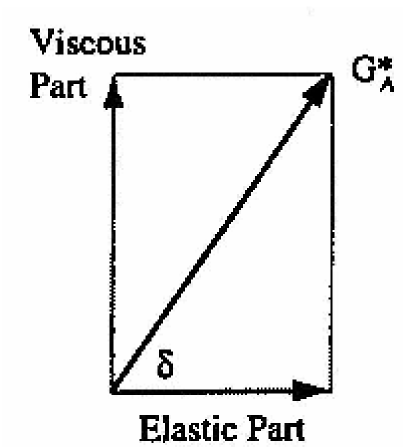


Figure 3-9 Determination of asphalt behavior by DSR (*1*)

The DSR measures the rheological properties (complex shear modulus and phase angle) at intermediate to high temperatures experienced by the pavement in the geographical area for which the asphalt binder is intended. The DSR test provides stiffness behavior of asphalt binders over a wide range of temperatures. Two forms of G^* and δ are used in the binder specification. Permanent deformation is governed by limiting the $G^*/\sin\delta$ at the test temperature to values greater than 1.00 kPa for original binder and 2.20 kPa after RTFO aging. Fatigue cracking is governed by limiting $G^*\sin\delta$ of PAV aged material to values less than 5000 kPa at the test temperature.

Bending beam rheometer test

The Bending Beam Rheometer (BBR) test is used to evaluate binder properties accurately at low temperatures (*1*). The test measures how much a binder deflects or creeps under a constant load and at a constant temperature. This temperature is related to the lowest service temperature of the pavement. The test method uses beam theory to calculate the flexural creep stiffness or compliance of an asphalt binder under a creep load.

This test is applicable to materials having creep stiffness values from 20 MPa to 1 GPa and can be used with asphalt binder having aging conditions of Tank (unused), RTFO, or PAV. The test apparatus is designed for testing within a temperature range from –36 °C to +22 °C. This test gives the Flexural Creep Stiffness, S(t), and m-values at different loading time and temperatures. Flexural Creep Stiffness is the ratio obtained by dividing the maximum bending stress in the beam by the maximum bending strain. The m-value is the absolute value of the slope of the logarithm of stiffness curve versus the logarithm of time.

The BBR test system, shown in Figure 3-10, includes a loading frame, cell and LVDT, controlled temperature fluid bath, data acquisition system, temperature measuring equipment, test beam molds, stainless steel beams, standard masses, calibrated thermometers, and thickness gauge etc.

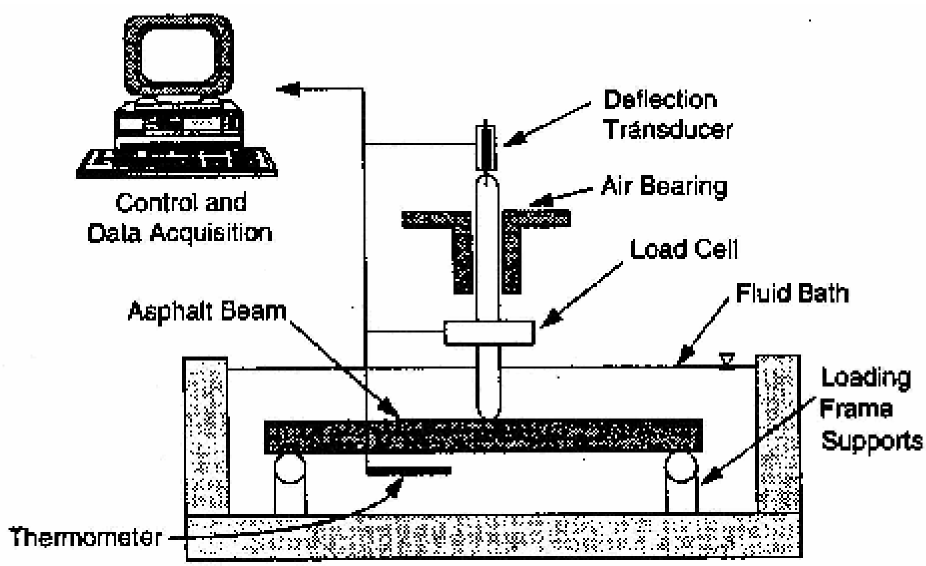


Figure 3-10 Schematic diagram of BBR test (*1*)

The test temperature is related to the lowest temperature experienced by the pavement in the geographical area for which the asphalt binder is intended. The flexural creep stiffness determined from this test describes the low temperature stress-strain-time response of asphalt binder at the test temperature within the linear viscoelastic response range. The low temperature thermal cracking performance of paving mixtures is related to the creep stiffness and the m-value. These stiffness and m-values are used as performance-based specification criteria for asphalt binders.

3.5.1.2 Sources of asphalt binders in Pakistan

The major source of asphalt binder in Pakistan is from Karachi and Attock refineries. At present, these refineries have limited capabilities to produce grades that meet the specifications for different environmental regions in Pakistan. The four asphalt binder grades produced by these two refineries are Pen 40/50, Pen 60/70, Pen 60/70 (PMB) and Pen80/100 which may not be sufficient to meet the grading requirements for the different environments in Pakistan. In addition, variability/quality of binders produced at refineries, with time and batch, is a concern that needs to be addressed.

3.5.2 Hot-Mix Asphalt Characterization

Two basic components of HMA mix design procedure are the mix design methodology and simple performance tests. Currently, the simple performance tests are still under research and are not formally part of the mix design. However, these tests were employed in this research to compare the performance of both (Marshall and Superpave) mix design methodologies. The simple performance tests are documented in the later chapters; however, Superpave mix design procedure is briefly discussed below.

3.5.2.1 Superpave mix design

As stated previously, one of the major contributions of the Strategic Highway Research Program (SHRP) was the Superpave mix design method. The Superpave mix design method was designed to replace the Hveem and Marshall methods. The volumetric analysis common to the Hveem and Marshall methods provides the basis for the Superpave mix design method. The Superpave system ties asphalt binder and aggregate selection into the mix design process, while also considering traffic and climate. In addition, a gyratory compactor has replaced the specimen compaction devices from the Hveem and Marshall procedures and the compaction effort in mix design process has been tied to the expected traffic. The Superpave mix design method consists of seven basic steps:

1. Aggregate selection
2. Asphalt binder selection
3. Sample preparation (including compaction)
4. Performance tests
5. Density and voids calculations
6. Optimum asphalt binder content selection
7. Moisture susceptibility evaluation

The aggregate and binder requirements have been discussed previously. However, one of most important aspect of Superpave mix design method is the sample preparation. Superpave method, like other mix design methods, creates several trial aggregate-asphalt binder blends, each with different asphalt binder content. Then, by evaluating the performance of each trial blend, optimum asphalt binder content can be selected. The Gyratory compactor (see Figure 3-11) is used for the sample preparation. Key parameters of the gyratory compactor are:

- Compacted sample size is 150 mm (6-inch) in diameter and approximately 115 mm (4.5 inches) in height (corrections can be made for different sample heights). This sample size is larger than those used for the Hveem and Marshall methods. The advantage of using a larger compacted sample is that larger aggregate sizes can be used in the Superpave mix design.
- The load is applied at the top of the sample and covers almost the entire sample top area. The sample is inclined at 1.25 degrees and rotates at 30 revolutions per minute as the load is continuously applied. This helps achieve a sample particle

orientation that is somewhat similar to the field orientation that is achieved by using roller compaction.

Figure 3-11 Superpave gyratory compactor at UET, Lahore

As in the case of Marshall mix design, the traffic level is accounted by the number of blows. In the case of the Gyratory compactor, the traffic level is the number of gyrations that are used to achieve the desired compaction and density. The typical criteria for the number of gyrations are given in Table 3-9.

Table 3-9 Number of gyrations from AASHTO, 2001

20-yr Traffic Loading (in millions of ESALs)	Number of Gyrations $N_{initial}$	N_{design}	N_{max}
< 0.3	6	50	75
0.3 to < 3	7	75	115
3 to < 10*	8 (7)	100 (75)	160 (115)
10 to < 30	8	100	160
>30	9	125	205

* When the estimated 20-year design traffic loading is between 3 and < 10 million ESALs, the agency may, at its discretion, specify $N_{initial} = 7$, $N_{design} = 75$ and $N_{max} = 115$.

In summary, the Superpave mix design method was developed to address specific mix design shortcomings in the Hveem and Marshall methods. Superpave mix design is a rational method that accounts for traffic loading and environmental conditions. Although not yet fully complete (the performance tests have not been implemented), Superpave mix design produces quality HMA mixtures. As of 2000, 39 states in the United States have adopted, or are planning to adopt, Superpave as their mix design system (*1*). The salient aspects of the Superpave method are:

- The use of formal aggregate evaluation procedures (consensus requirements)
- The use of the PG asphalt binder grading system and its associated asphalt binder selection system
- The use of the gyratory compactor to simulate field compaction
- Traffic loading and environmental considerations
- Volumetric approach to mix design

Despite many differences when compared to the Hveem or Marshall method, Superpave still uses the same basic mix design steps and still strives for an optimum asphalt binder content that results in 4 percent design air voids. Thus, the method is quite different but the ultimate goals remain consistent.

CHAPTER 4 – LABORATORY INVESTIGATION

4.1 INTRODUCTION

The details of material characterizations including types of materials, and test methods used in this research are presented in this chapter. Chapter 5 includes results and discussion of the data analysis.

Asphalt concrete (sometimes referred to as "hot mix asphalt (HMA)" is a paving material that consists of asphalt binder and mineral aggregates. The asphalt binder, which can be asphalt cement or modified asphalt cement, acts as a binding agent to glue the aggregate particles into a dense and stable mass capable of withstanding the effects of traffic loads. The mineral aggregate consists of combination of different sizes of aggregates along with mineral fillers. The binder and aggregates are mixed together uniformly at elevated temperatures to achieve the desired properties. The characteristics of the binder and aggregates define the overall performance of the mix. Thus, before mixing the individual components, it is imperative to obtain the desired quality and properties of the finished mix.

In this study, the aggregates were obtained from Margalla quarry (a local source) i and the asphalt cement from Attock & National refineries as noted in the research plan (Chapter 2). In addition to virgin/neat asphalt binders, the effects of polymer modification to enhance the HMA performance were studied. Elvaloy reactive polymer and AC non-reactive polymers were obtained from Dupont Industries. These polymers were mixed with the available binders in different proportions to characterize its effect on pavement performance.

4.2 ASPHALT BINDERS

In order to accomplish the objectives of this study, conventional and Superpave testing was carried out on 22 binders. These binders included five neat, ten modified and seven blended asphalts. As mentioned before, the modification was carried out using reactive and non-reactive polymers. The types of binders obtained from the two refineries are:

- Attock Refinery
  - Grade 80/100
  - Grade 60/70
  - PMB (Polymer modifed)
- Karachi Refinery
  - Grade 80/100
  - Grade 60/70
  - Grade 40/50

As detailed in the research plan, the original binders were blended in different proportions to obtain binders of different grades. In addition the Attock refinery binders, Pen 80/100 and Pen 60/70, were modified with the reactive polymer Elvaloy at the percentages of 1.35, 1.70 and 2.0 by weight of the asphalt cement. The binder from Karachi refinery, Pen 80/100, was modified with a non-reactive polymer at the percentages of 2.5, 3.5 and 4.5 by weight of asphalt cement. A summary of the 22 evaluated binders are given in Table 4-1.

Table 4-1 Binder code and description for the 22 binders used in the evaluation

No.	Binder code	Binder description	No.	Binder code	Binder description
1	APMB	Attock Polymer Modified	12	PA6/7(1.35)	Polymer Modified Attock Pen 60/70 with 1.35% Elvaloy
2	A6/7	Attock Pen 60/70	13	PA6/7(1.70)	Polymer Modified Attock Pen 60/70 with 1.70% Elvaloy
3	A8/10	Attock Pen 80/100	14	PA6/7(2.00)	Polymer Modified Attock Pen 60/70 with 2.00% Elvaloy
4	K4/5	Karachi Pen 40/50	15	BK6/7(20)8/10(80)	Blended Karachi Pen 60/70 (20%) and Pen 80/100 (80%)
5	K6/7	Karachi Pen 60/70	16	BK6/7(50)8/10(50)	Blended Karachi Pen 60/70 (50%) and Pen 80/100 (50%)
6	K8/10	Karachi Pen 80/100	17	BK8/10(20)4/5(80)	Blended Karachi Pen 80/100 (50%) and Pen 40/50 (80%)
7	BA6/7(20)8/10 (80)	Blended Attock: Pen 60/70 (20%) and Pen 80/100 (80%)	18	BK8/10(50)4/5(50)	Blended Karachi Pen 80/100 (50%) and Pen 40/50 (50%)
8	BA6/7(50)8/10 (50)	Blended Attock: Pen 60/70 (50%) and Pen 80/100 (50%)	19	BK6/7(50)4/5(50)	Blended Karachi Pen 60/70 (50%) and Pen 40/50 (50%)
9	PA8/10(1.35)	Polymer Modified Attock Pen 80/100 with 1.35% Elvaloy	20	PK8/10(2.5)	Polymer Modified Karachi Pen 80/100 with 2.5% AC
10	PA8/10(1.70)	Polymer Modified Attock Pen 80/100 with 1.70% Elvaloy	21	PK8/10(3.5)	Polymer Modified Karachi Pen 80/100 with 3.5% AC
11	PA8/10(2.00)	Polymer Modified Attock Pen 80/100 with 2.00% Elvaloy	22	PK8/10(4.5)	Polymer Modified Karachi Pen 80/100 with 4.5% AC

4.2.1 Polymer Modification

Two types of elastomeric polymers from DuPont were used in the modification of locally available asphalt binders. The two polymers are:

1. DuPont™ Elvaloy® 4170
2. DuPont ™ Elvaloy® AC 3427

The DuPont Elvaloy® 4170 is a reactive elastomeric terpolymer that uses resins for asphalt modification; Elvaloy is typically used in small quantities to reduce rutting and cracking susceptibility. Unlike most other plastomers and elastomers that are simply mixed with the asphalt binder, Elvaloy® has an active ingredient that chemically reacts with asphalt and hence, it yields permanently modified binders with improved elastomeric properties. The chemical reaction results in a mixture of asphalt and modifier, which is stable, elastically improved, and more resilient. In this study, the polymer is added to the asphalt binders using three percentages by the asphalt binder weight; 1.35%, 1.70% and 2.0%. This polymer was found to be compatible with the asphalts from Attock refinery. However, it did not work very well with the asphalt from Karachi refinery because of significantly different nature of the crude oil in both refineries. Therefore, Elvaloy 4170 was blended with binders from Attock refinery only.

The second polymer DuPont Elvaloy AC 3427 is a non-reactive copolymer of ethylene and butyl acrylate. It is available in pellet form and can be processed at melt temperatures ranging from 160 to 285 °C (320 to 545 °F). In this study, the polymer is added using percentages by the asphalt binder weight; 2.5%, 3.5% and 4.5%.

4.2.2 Blending Process for Polymers

Elvaloy® 4170 chemically reacts with asphalt binder; however, the second polymer Elvaloy AC 3427 is non-reactive. The blending process for Elvaloy® 4170 Elvaloy AC 3427 is very similar. The only exception is the use of phosphoric acid with Elvaloy 4170, which acts as a catalyst during chemical reactivity.

A successful blending of polymer and asphalt binder is highly dependent upon the blending duration, temperature and the frequency of the shear mixer. In this study, these parameters were selected based on the recommendation of the DuPont representative in Pakistan. One of the major concerns with the blending operation is that neat asphalt consists of volatile material, which may be lost during the blending process resulting in increase in the neat binder viscosity. However, every effort was made to make sure that blending process is carried out in a controlled environment. The systematic procedure for preparation of the binder blends is given below.

1. Carefully heat the asphalt sample in its original container in an oven set at 140 °C (350 °F) until it is fluid enough to pour.
2. Stir the sample until it is homogeneous and pour into suitable blending containers (half gallon paint cans work well).
3. Place the container of asphalt in the heating mantle and heat to 165 °C (330 °F). Stir with a lab mixer set fast enough (usually 200-rpm) to create a small vortex, without whipping excessive air.
4. Maintain the asphalt at the desired blending temperature for 10 minutes before adding the polymer.
5. Slowly add the desired amount of polymer to the heated asphalt (about 10 g/min.), while stirring.

6. Blend for 2 hours, at that time the increase in viscosity of he blend stabilizes, and the reaction is considered to be complete for the Elvaloy® AC 3427 polymer. Hence, the remaining steps below applies to Elvaloy® 4170 polymer only.
7. Add to the mixture 0.2% by weight of the asphalt binder phosphoric acid, and stir for 15 to 30 minutes. Phosphoric acid is added to act as catalyst to expedite the chemical reaction. Adding phosphoric acid prior to fully dissolving the Elvaloy® RET 4170 may result in formation of lumps.
8. Pour the remaining blend into two containers and cover tightly with a lid. Place each container in an oven at the desired curing temperature, of 165 °C (330 °F). The curing time used for this research is one hour.
9. Remove each container after one hour cure time; allow the container to cool off for 5-10 min., and then open carefully. Warning: Heating asphalt in a sealed container can lead to pressure buildup inside the container, due to volatile components in the asphalt. Use caution when opening the container. A lid with a built-in valve can be used to manually relieve the pressure before opening the container.

4.2.3 Conventional Binder Testing

As stated earlier, asphalt binder's specifications have conventionally been developed their physical properties. Binder properties are conventionally tested at standard temperatures and the test results are used to determine whether the material meets the specification criteria. The conventional binder tests are summarized in Table 4-2 below.

Table 4-2 Conventional binder tests

Test	Property	Method	Conditions
Penetration	Penetration	AASHTO T49-93	15 °C & 25 °C, 100 gm load for 5 seconds
Ring and Ball	Softening point	AASHTO T53-92	Measured temperature
Absolute Viscosity	Absolute Viscosity	AASHTO T202-91	60 °C
Kinematic Viscosity	Kinematic viscosity	AASHTO T201-93	135 °C

4.2.4 Superpave Tests

The most recent Superpave tests measure physical properties that can be related to field performance based on engineering principles. The Superpave binder tests are also conducted at temperatures that are expecte in the field. A summary of Superpave binder tests are shown in Table 4-3. All tests including the RTFO and PAV tests were described briefly in Chapter 3.

Based on the tests described above the following testing criteria were adopted for each binder:

Table 4-3 Superpave binder tests

Test	Properties	Method	Conditions
Rotational Viscometer (RV)	Rotational viscosity	AASHTO TP-48	60°C, 80°C, 100°C, 121.1°C, 135°C and 176.7°C
Dynamic Shear Rheometer (DSR)	Complex Modulus & Phase Angle	AASHTO TP5-98	15,25,35,45,60,70,80,95 ,105 and 115°C 1,10,100 rad/sec
Bending Beam Rheometer (BBR)	Flexural creep stiffness and m-value	AASHTO TP1-98	Three temperatures; one above low temperature specification limit & two below that limit

- Rotational Viscometer (RV) at 20 RPM at seven temperatures (125^{o}C to 185^{o}C).
- Dynamic Shear Rheometer (DSR) tests were cnduvted at temperature from 7 – 82^{o}C at a frequency of 1.59 Hz (10 rad/sec). The testing was carried at three aging conditions; original, RTFO aged and PAV aged. The results were used to define the grade of asphalt.
- Bending Beam Rheometer (BBR) tests were conducted on PAV aged samples at four temperatures (-12,-18, -24 and -30 degree Centigrade) to define the low temperature grade.

4.3 HOT-MIX ASPHALT

In the United States, since the 1940s, most hot mix asphalt (HMA) mixtures were designed using the Marshall and Hveem methods. Since the mid-1990s Superpave technology was gradually implemented. Presently, most State Highway Agencies in United States have adopted Superpave technology for the design of asphalt mixes. In Pakistan, the current state of the practice for the design of asphalt mixes is still based on the Marshall methodology. As mentioned earlier, the main objective of this research study was the implementation of Superpave technology in Pakistan. In order to achieve this objective, the effects of binder type, gradation and compaction methods were compared by making specimen using the Marshall and Superpave approaches. For the purpose of comparison, several mixes were prepared and the details of the mix volumetric properties are given in the following sections. The properties of the mixes are compared at the optimum design volumetric.

4.3.1 Aggregate

A wide variety of mineral aggregates have been used to produce HMA in Pakistan. Some materials are referred to as natural aggregate because they are simply mined from river or glacial deposits and are used without further processing to manufacture HMA. These are often called “bank-run” or “pit-run” materials. Processed aggregate can include natural aggregate that has been separated into distinct size fractions, washed, crushed, or otherwise treated to enhance certain performance characteristics of the finished HMA. To ensure a strong aggregate blend for HMA, engineers typically have specified aggregate properties that enhance the internal friction portion of the overall shear strength. Generally, this is accomplished by specifying a certain percentage of the coarse portion of an aggregate blend be crushed. Finally, because the natural sand particles tend to be rounded (low friction), the amount of natural sand in a blend is often limited.

In this study, the aggregates were obtained from Margalla quarry. The Margalla hills (located north of Islamabad, Pakistan) are at the foothills of the Himalayas and vary in height from 685 meters at the western end to 1,604 meters at the eastern end. The area of the Margalla range is about 12,605 hectares consisting mainly of hard limestone rocks.

4.3.1.1 Aggregate strength properties

To establish the suitability of an aggregate as a road making material, the aggregate should satisfy certain specifications (some of the UK or USA specifications are generally adopted in Pakistan). In Pakistan, the properties of selected aggregates often exceed the minimum requirements of the specification; this is often the case because the best available materials are used for road construction. Little consideration is given to the suitability of an aggregate or cost savings in using an aggregate of appropriate quality. However; some of the general properties of the aggregate obtained from the Margalla quarry are presented below.

In Pakistan, aggregates of a particular grading are specified or classified on the basis of the following properties.

- Hardness
- Toughness
- Specific gravity and porosity
- Particle shape

Hardness is a measure of the resistance to crushing and abrasion of aggregates. Aggregates are subjected to crushing and abrasive wear during production, placing, and compaction of pavement layers. In addition, aggregates are also subjected to abrasion under traffic loads. The hardness of the aggregate is defined by the Aggregate Crushing Value (ACV) (BS 812 Part 110 1990). ACVs vary from about 5% for strong aggregates to 30% for soft aggregates. General requirement for ACV is less than 25%.

The Los Angles Abrasion (LAA) test measures the resistance of aggregates to a combination of abrasion and impact. It is performed in accordance with AASHTO T96 or ASTM C 131, 2003, standard test procedures. Hardness of aggregates ensures resistance against the abrasive action of the wheels on the surface. Low values of LAA indicate 'harder' aggregates. For aggregates used in a base course material (or in bituminous mixes), it is generally specified that the LAA value should not exceed 30% (e.g., ASTM D-693-03a, 2003).

The 10% fine value (TFV test defines the load requirements to produce 10% fines from a sample of the aggregate held in a steel mold. This test was performed in accordance with BS 812 Part 111 (1990). A high value indicates aggregates that are difficult to crush. In order to meet typical US and UK specifications (Specification for Highways Works, Clause 803, HMSO, 1986), the TFV should not be less than 50 kN. Toughness is the ability of an aggregate to resist impact forces and is defined by the Aggregate Impact Value (AIV). The AIV test, measures the resistance of aggregates to impact. The AIV test is performed in accordance with BS 812, Part 112 1990. A lower AIV indicates greater resistance to impact. The AIV less than 10% is exceptionally high, AIV between 10% and 20% is good while an aggregate with an AIV greater than 35% is suitable for lower layers only.

The Specific gravity (SG) of aggregates is often considered to be a strong indicator of the quality of the minerals and aggregate type. Higher values of SG indicate a better aggregate. This test is performed in accordance with AASHTO T84-D1193. Generally, the SG of aggregates used in pavements is between 2.6 and 2.7, with a maximum value of 2.9.

Another important parameter that is considered to define the aggregate property is the porosity. The porosity of an aggregate is a measure of the interconnected air voids in an aggregate particle and is measured by assessing the amount of water a particle absorbs when soaked in water. A certain degree of porosity is desirable, as it permits the aggregate particle to absorb bitumen, which forms a mechanical linkage between the bitumen film and the stone particle. Aggregates having low specific gravity or high water absorption are generally considered unsuitable unless they have acceptable hardness, toughness and strength properties. Higher porosity generally leads to a lower value of SG and vice versa. Water absorption is determined as part of SG tests (AASHTO T84D11-1993). A value less than 0.75 is considered to be excellent for surface asphalt mixes.

Typical values of various indicators mentioned above and observed during the testing for the Margalla aggregate are given in Table 4-4. It can be seen that the aggregate qualifies for construction of flexible pavements meet all requirements to be used in surface layer of flexible pavements.

Table 4-4 General aggregate strength properties

Property	Value
ACV (%)	21.98
LAAV (%)	16.30
TFV (kN)	74
AIV (%)	13.38
Specific Gravity	2.70
Porosity	0.58

4.3.1.2 Aggregate Gradation

In addition to strength properties, one of the most important aggregate characteristic required is the gradation. The purpose of aggregate gradation is to provide a dense stable mass of the aggregate structure. Several gradations have been proposed by different agencies as in the case of National Highway Authority (NHA) Pakistan. NHA has gradations defined for both the asphaltic wearing and base course layers. In addition, Superpave methodology specifies requirements for the gradation of aggregate structure. It uses the 0.45 power gradation chart to define a permissible gradation. An important feature of the 0.45 power gradation is that it produces the maximum density gradation. This gradation plots as a straight line from the maximum aggregate size through the origin.

Superpave uses a standard set of ASTM sieves and the following definitions with respect to aggregate size:

- Maximum Size: One sieve size larger than the nominal maximum size.
- Nominal Maximum Size: One sieve size larger than the first sieve to retain more than 10 percent.

To specify aggregate gradation, two additional features are added to the 0.45 power chart—control points and restricted zone. Control points function as master ranges through which gradations must pass. The control points specify the acceptable ranges at three aggregate sizes; the nominal maximum size, an intermediate size of 2.36 mm, and the dust size of 0.075 mm. Figure 4-1 Illustrates the control points and restricted zone for a 12.5 mm Superpave mixture.

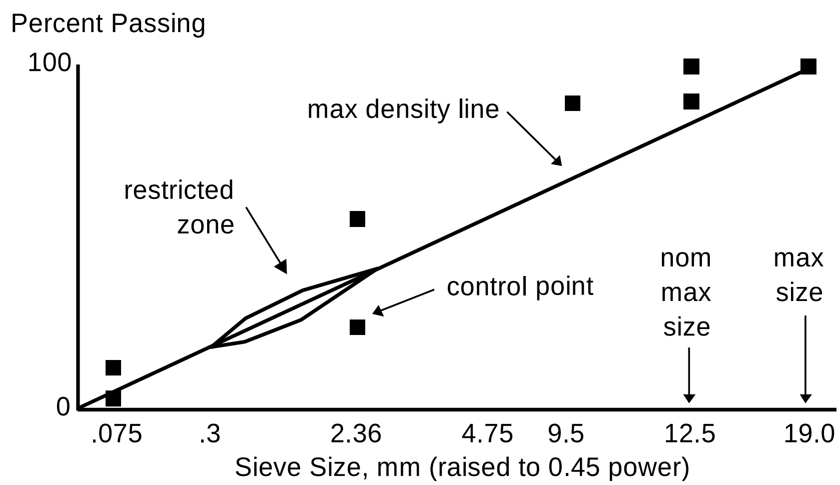


Figure 4-1 Gradation control chart for 12.5 mm nominal size

The restricted zone resides along the maximum density gradation between the intermediate sizes, either 4.75 or 2.36 mm and the 0.3 mm size. It forms a band through which gradations should not pass. Gradations that pass through the restricted zone have often been called “humped gradations” because of the characteristic hump in the grading curve that passes through the restricted zone. In most cases, a humped gradation indicates a mixture that possesses too much fine sand in relation to total sand. This gradation practically always results in tender mix behavior, which is manifested by a mixture that is difficult to compact during construction and offers reduced resistance to permanent deformation during its performance life.

In this research, the NHA recommended wearing and base course gradations were selected for the purpose of comparison with the Superpave recommendations. Two gradations, one on the fine side and the other on the coarse side were selected for the wearing and base layers. This resulted in a total of six gradations, three for the binder course and three for base layer. Table 4-5 shows the gradations for the asphaltic wearing course (NHA and Superpave), whereas Table 4-6 presents the gradations for the asphaltic base layer. The graphical display of the six gradations is shown in Figures 4-2 and 4-3, respectively.

Table 4-5 Gradation properties of asphalt wearing course

Gradation	NHA Gradation (Wearing Course)	SUPERPAVE Gradation (Fine Wearing Course)	SUPERPAVE Gradation (Coarse Wearing Course)
	Cumulative Percentage Passing, %		
37.5 mm (1.5 in)	100	100	100
25.4 mm (1 in)	100	100	100
19 mm (3/4 in)	100	100	100
12.5 mm (1/2 in)	82	94	95
9.0 mm (3/8 in)	70	87	84
6.4 mm (1/4 in)	58	74	57
4.75 mm (No. 4)	50	65	45
2.36 mm (No. 8)	30	37	30
1.18 mm (No. 16)	20	21	20
0.6 mm (No. 30)	14	14	15
0.3 mm (No.50)	10	9	10
0.15 mm (No. 100)	7	7	6
0.075 mm (No.200)	5	5	4

Table 4-6 Gradation properties of asphalt base course

Gradation	NHA Gradation (Base Course)	SUPERPAVE Gradation (Fine Base Course)	SUPERPAVE Gradation (Coarse Base Course)
	Cumulative Percentage Passing, %		
37.5 mm (1.5 in)	100	100	100
25.4 mm (1 in)	82	94	95
19 mm (3/4 in)	73	86	85
12.5 mm (1/2 in)	60	73	60
9.0 mm (3/8 in)	52	65	47
6.4 mm (1/4 in)	44	53	35
4.75 mm (No. 4)	37	44	30
2.36 mm (No. 8)	25	25	20
1.18 mm (No. 16)	19	16	15
0.6 mm (No. 30)	14	11	12
0.3 mm (No.50)	10	7	8
0.15 mm (No. 100)	8	5	6
0.075 mm (No.200)	4	4	4

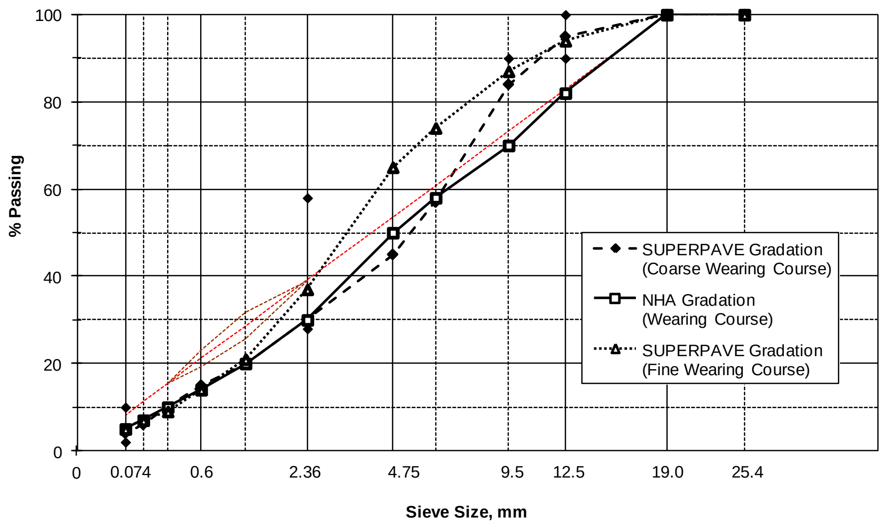


Figure 4-2 Gradation chart for wearing course mixes

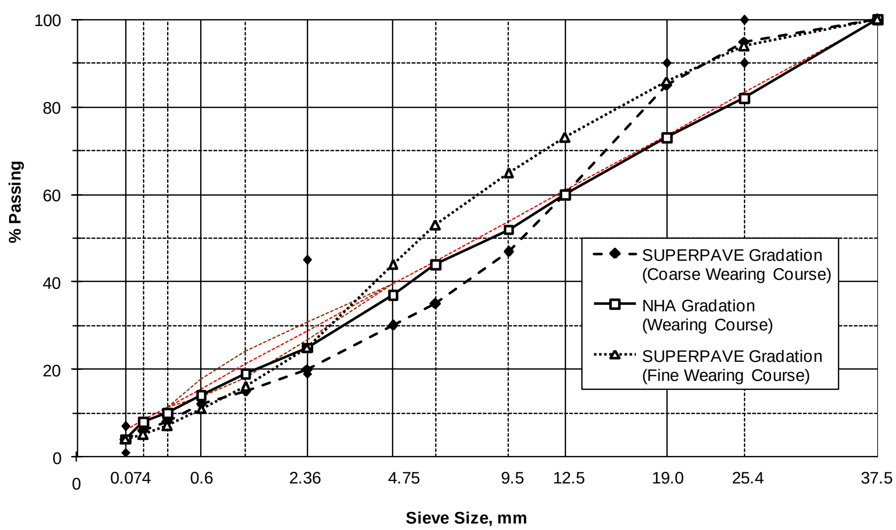


Figure 4-3 Gradation chart for base course mixes

The NHA gradations in the case of both wearing and base courses are more close to the maximum density line. That is, space available for binder is less compared to the Superpave mixes. This may be one of the reasons for excessive rutting behavior observed in Pakistan. The methodologies adopted to prepare HMA mixes in this study are presented next.

4.3.2 Marshall Mix Design (75 blows)

The main purpose of HMA mix design is to determine the optimum asphalt content for a specific gradation. To obtain the optimum asphalt content, several percentages (3.5, 4.0, 4.5, 5.0, 5.5 and 6.0 percent) of asphalt content by weight of dry aggregate were blended with the heated aggregate. Marshall testing was only carried out for wearing course gradations because of the 4-inch size of Marshall specimen, which is not considered to be appropriate for the size of aggregate used in the base course mixes. The nominal aggregate sizes are 12.5 and 25 mm for wearing and base courses, respectively. The 25.0 mm aggregate size requires a mold size of six inches. therefore, the Marshall evaluation was only carried out for the wearing course mixes.

The appropriate percentages of asphalt as mentioned earlier were mixed with the heated aggregates. The aggregates were heated at the mixing temperature for at least 4 hours prior to mixing. The asphalt binder was also heated to achieve the appropriate mixing temperature. The aggregate and the binder are then mixed in a portable mixer until uniform coating of aggregate is achieved. The mixed sample is then compacted using manual Marshall Compactor with 75 blows on each side. The prepared sample is then used for the estimation of the volumetric properties, which were then used to establish optimum binder content.

Volumetric testing included determining the bulk, theoretical specific gravities, air voids, Marshall stabilities, flow, voids in mineral aggregate (VMA), and the percent of voids filled with asphalt (VFA). The results of testing are shown in Tables 4-7 through 4-11. Table 4-7 shows the bulk specific gravity values for the aggregate retained on and passing the number 4 sieve for the three gradations. Table 4-8 presents the combined bulk specific gravities for the three gradations. These are based upon the proportion of material retained and passed from the number 4 sieve. Equation (1) is used to computed the combined SG.

$$G_{sb} = \frac{P_1 + P_2}{\left[\frac{P_1}{G_1} + \frac{P_2}{G_2}\right]} \tag{4.1}$$

where P_n are the percent by weight of each component aggregate in blend and G_n are the specific gravities for the respective proportions

Table 4-7 Summary of aggregate bulk specific gravity for wearing/base course

Aggregate Size	Gradation	Aggregate Average Bulk SG (Gsb)
Retained #4 Sieve	NHA	2.662
	SUPERPAVE Fine	2.706
	SUPERPAVE Coarse	2.645
Passing #4 Sieve	NHA	2.606
	SUPERPAVE Fine	2.583
	SUPERPAVE Coarse	2.606

Table 4-8 Combined aggregate bulk specific gravity for wearing and base course

Mix Type	**Gradation**	**Aggregate Average Bulk SG (Gsb)**
Wearing Course	NHA	2.634
	SUPERPAVE Fine	2.625
	SUPERPAVE Coarse	2.627
Base Course	NHA	2.641
	SUPERPAVE Fine	2.646
	SUPERPAVE Coarse	2.633

No significant difference in specific gravities was observed for the three gradations. The bulk specific gravities are used for the estimation of the optimum asphalt content. The optimum asphalt content was selected at the percentage that would produce 4 percent air voids. Based on this criterion and data presented in Tables 4-10 and 4-11, the optimum binder contents for each mix was estimated and are presented in Table 4-12.

Table 4-9 Coding scheme for mixes

Mix Type	Binder Code	Aggregate Gradation	Mix Code
Wearing Course	A6/7	NHA	A6/7NHAW
		SPF	A6/7SPFW
		SPC	A6/7SPCW
	K6/7	NHA	K6/7NHAW
		SPF	K6/7SPFW
	PA6/7(1.35)	SPF	PA6/7(135)SPFW
	PA6/7(1.70)	SPF	PA6/7(170)SPFW
Base Course	A6/7	NHA	A6/7NHAB
		SPF	A6/7SPFB
		SPC	A6/7SPCB

In order to carry out the comparison of the NHA mix design practices and the Superpave recommendations, several mix designs were prepared. The objective of these mix design were to establish the optimum asphalt content. Performances testing at optimum binder content were then carried out and are discussed in the next chapter. A total of four binders were selected for establishing the mixes to compare NHA and Superpave gradations. In addition, the comparison also includes comparing the Marshall compaction methodology to Superpave gyratory compaction. The selected four binders are:

1. Attock 60/70, A6/7
2. Karachi 60/70, K6/7
3. Attock 60/70 with 1.35% polymer, PA6/7(1.35)
4. Attock 60/70 with 1.60% polymer, PA6/7(1.70)

Table 4-10 summarizes the volumetric properties for the selected mixes used for wearing course, whereas, Table 4-11 is for the base course mixes. Finally Table 4-12 shows the volumetric properties at optimum binder content. Samples for performance evaluation were made at optimum values.

Table 4-10 Marshall mix design results for wearing course mixes

Mix Code	Asphalt Content (%)	Air Voids (%)	VMA (%)	VFA (%)	Marshall Stability, (kg.)	Flow, 0.01 (in.)
A6/7NHAW	3.5	4.4	14.3	69.4	1384	11.5
	4.0	3.8	15.1	74.7	1510	9.5
	4.5	1.1	13.7	91.8	1646	12.3
	5.0	0.2	13.0	98.4	1745	13.3
	5.5	0.1	13.1	99.0	1564	15.7
	6.0	0.2	13.5	98.2	1401	14.8
A6/7SPFW	3.5	7.5	17.0	56.1	1244	12.7
	4.0	6.7	17.2	61.0	1484	12.2
	4.5	5.9	16.7	65.0	1637	12.2
	5.0	3.5	16.0	78.0	1678	14.3
	5.5	2.0	14.6	86.2	1580	12.7
	6.0	0.5	15.1	96.5	1289	13.5
A6/7SPCW	3.5	7.3	16.9	57.1	1062	9.3
	4.0	6.0	15.4	61.4	1329	11.2
	4.5	4.1	15.4	73.4	1623	12.0
	5.0	1.4	13.9	89.7	1422	14.5
	5.5	0.6	13.9	95.4	1353	13.3
	6.0	0.6	14.3	96.1	1347	16.7
K6/7NHAW	3.5	8.9	15.1	41.0	1434	13.3
	4.0	2.2	14.9	84.9	1601	13.7
	4.5	2.0	14.3	86.2	1638	14.7
	5.0	0.6	13.6	95.3	1558	14.7
	5.5	0.2	14.1	98.3	1438	15.7
K6/7SPFW	4.0	5.3	17.1	68.8	1234	13.7
	4.5	3.8	16.1	76.6	1521	15.0
	5.0	1.5	15.6	90.1	1687	12.0
	5.5	0.3	15.5	98.2	1555	13.0
	6.0	0.2	15.4	98.4	1746	13.2
PA6/7(135)SPFW	3.5	6.3	17.4	63.6	1745	10.7
	4.0	5.3	17.3	69.1	2174	16.5
	4.5	2.5	15.7	84.0	2357	13.5
PA6/7(170)SPFW	3.5	4.2	18.1	77.1	2025	15.3
	4.0	6.3	17.6	64.0	1654	13.3
	4.5	3.7	17.8	79.3	2050	15.0

Table 4-11 Marshall mix design results for base course mixes

Mix Code	Asphalt Content (%)	Air Voids (%)	VMA (%)	VFA (%)	Marshall Stability, (kg.)	Flow, 0.01 (in.)
A6/7NHAB	3.5	3.8	12.1	68.4	1391	15.3
	4.0	3.6	12.0	70.3	1689	12.3
	4.5	2.7	11.7	77.1	1830	15.8
	5.0	1.9	12.1	84.3	1474	19.3
	5.5	1.7	12.3	86.6	1135	20.8
	6.0	0.1	12.7	99.1	1149	24.0
A6/7SPFB	3.5	3.6	14.3	75.0	1662	17.0
	4.0	2.0	14.1	85.6	1606	16.2
	4.5	1.1	14.1	92.4	1371	13.3
	5.0	0.4	14.0	96.9	1434	13.2
	5.5	0.6	14.7	96.2	1388	14.7
	3.0	5.8	15.5	62.2	1121	12.8
A6/7SPCB	3.8	7.4	12.9	42.6	1291	11.7
	4.0	4.8	13.2	63.6	1156	13.5
	4.5	3.8	12.5	69.9	1221	15.3
	5.0	1.8	12.2	85.3	1221	14.3
	3.0	3.5	12.9	72.9	1278	16.3

Table 4-12 Mix properties at optimum asphalt content

Mix Code	Optimum Asphalt Content (%)	Air Voids (%)	VMA (%)	VFA (%)	Marshall Stability, (kg.)	Flow, 0.01 (in.)
A6/7NHAW	3.80	4.0	14.8	72.6	1459	10.3
A6/7SPFW	4.89	4.0	16.1	75.1	1669	13.9
A6/7SPCW	4.52	4.0	15.3	74.0	1615	12.1
K6/7NHAW	3.86	4.0	15.0	72.6	1534	13.5
K6/7SPFW	4.44	4.0	16.2	75.7	1487	14.8
PA6/7(135)SPFW	4.23	4.0	16.6	76.0	2258	15.1
PA6/7(170)SPFW	4.20	4.0	18.1	77.6	2031	15.3
A6/7NHAB	3.16	4.0	12.2	67.1	1188	17.4
A6/7SPFB	3.40	4.0	14.5	72.5	1554	16.2
A6/7SPCB	4.38	4.0	12.6	68.4	1205	14.9

When comparing the optimum binder content, NHA mixes usually result into lower binder content compared to Superpave mixes. It seems, the existing aggregate structure proposed in the NHA specifications are built around to minimize the binder content. Although, the mixtures produced using NHA gradation meets the existing Stability requirements but are significantly less that the Stability values from Superpave mixes. The Highway authorities in Pakistan at present faces a significant pavement rutting problems and increase in stability is likely to reduce the rutting potential. This can also be verified using the performance testing which is discussed in the next chapter.

4.3.3 State-of-the-Art Mix Performance Testing

Indirect testing was utilized for evaluating the resilient modulus, fatigue and permanent deformation characteristics of the asphalt mixes. These tests were performed by applying a repetitive load to a cylindrical specimen through two-diametrally opposite arc-shaped rigid loading platens. The test specimen is placed with its axis parallel to the loading platens and its diameter between the platens. After placing the specimen, a haversine loading pulse was utilized for the estimation of resilient modulus. In the diametral compression test induces compression along the vertical axis, which produces a nearly uniform tensile stress over a significant portion of the diametral plane containing applied load. Because applying a compressive load is technically much easier than tensile, the test became is ideal for evaluating the tensile and Resilient properties of paving and other materials. In addition, because of simplicity of testing, this approach was used in this project.

4.3.3.1 Testing methodologies

Indirect testing theoretical background

Within the pavement industry, several test methods such as (1) direct tension, (2) beam flexure, (3) indirect diametral, and (4) triaxial compression are used to characterize the material properties such as tensile strength, fracture and permanent deformation. Of these four, the indirect diametral and beam flexural are used for estimating the resilient modulus of the asphalt mixes along with Poisson's ratio. Among these two procedures, diametral testing is most widely used because of the following reasons:

- Considered to be the simplest and most cost effective test.
- Requires relatively small samples obtained from the field.
- Yields information regarding tensile strength, elastic modulus, Poisson's ratio, fatigue life, permanent deformation and moisture susceptibility characteristics of the material.
- Can be used for both static as well as dynamic loading.

The word diametral is used because the specimen is loaded along its diameter that produces compressive stresses in the vertical direction and tensile stresses in the horizontal direction along the vertical diameter. The word "indirect" is used because direct tensile force is not used to produce the tensile stresses used in determining the material properties.

As mentioned earlier, the test can be used for determining the tensile strength along with other material properties such as the elastic modulus and the Poisson's ratio. For determination of Poisson's ratio it is important to measure the specimen deformations in both directions. Hence, horizontal and vertical Linear Variable Differential Transducers (LVDTs) are installed on the sample to reduce the end-effect of loading strips. In this study; only the horizontal LVDT was used for estimatiing the resilient modulus. Poisson ratio was estimated using the relationship developed as a function of temperature and is discussed later in this chapter.

Several researchers that include Wright (1955), Olszak Kajfasz and Pietrzykowski (1957), Timoshenko (1934), Frocht (1948), Muskhelishvili (1953) and Sokolnikoff

(1956) solved the stress equations of the circular element subjected to concentrated forces. Hondros (1959) took the next step and developed a closed form solution of stresses along the horizontal and vertical diametral axis due to a strip loading. Stresses along the principal diameters are given by the following equations.

Stresses along the Horizontal Diameter (OX) - Hondros Solution

$$\sigma_{\theta x} = \sigma_{yx} = -\frac{2P}{\pi at}\left[\frac{(1-r^2/R^2)\sin 2\alpha}{(1+2r^2/R^2\cos 2\alpha + r^4/R^4)} + \tan^{-1}\frac{(1-r^2/R^2)}{(1+r^2/R^2)}\tan\alpha\right] \tag{4.2}$$

$$\sigma_{rx} = \sigma_{xx} = +\frac{2P}{\pi at}\left[\frac{(1-r^2/R^2)\sin 2\alpha}{(1+2r^2/R^2\cos 2\alpha + r^4/R^4)} - \tan^{-1}\frac{(1-r^2/R^2)}{(1+r^2/R^2)}\tan\alpha\right] \tag{4.3}$$

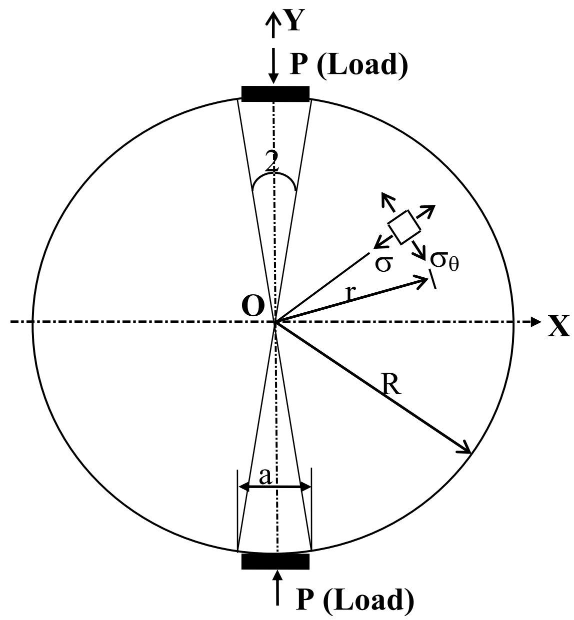
Stresses along the Vertical Diameter (OY) - Hondros Solution

$$\sigma_{\theta y} = \sigma_{xy} = +\frac{2P}{\pi at}\left[\frac{(1-r^2/R^2)\sin 2\alpha}{(1-2r^2/R^2\cos 2\alpha + r^4/R^4)} - \tan^{-1}\frac{(1+r^2/R^2)}{(1-r^2/R^2)}\tan\alpha\right] \tag{4.4}$$

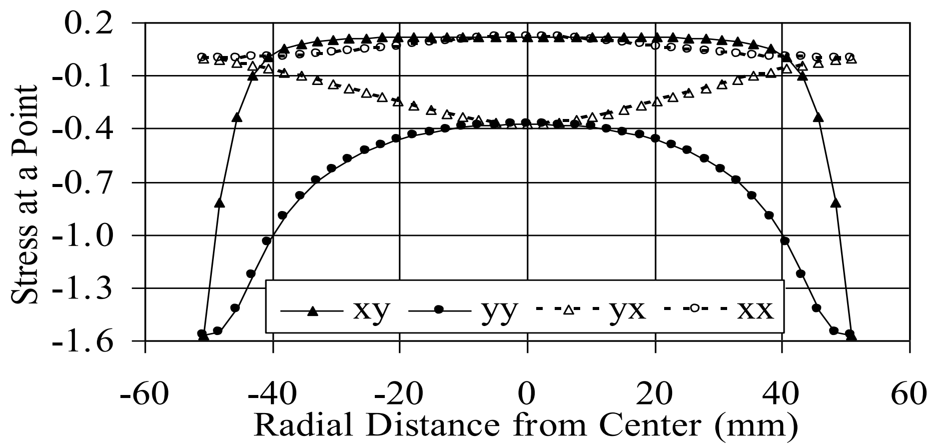
$$\sigma_{ry} = \sigma_{yy} = -\frac{2P}{\pi at}\left[\frac{(1-r^2/R^2)\sin 2\alpha}{(1-2r^2/R^2\cos 2\alpha + r^4/R^4)} + \tan^{-1}\frac{(1+r^2/R^2)}{(1-r^2/R^2)}\tan\alpha\right] \tag{4.5}$$

In the above equations, a double subscript notation is employed for stresses, the first indicating the direction of the stress and the second refers to the axis. The variable "*t*" represents thickness of the circular element, the remaining variables are shown graphically in Figure 4-4a.

The graphical representation of the stresses along the principal planes corresponding to the diameters through the OX and OY axes for a loading strip width of 0.5 inch are shown in Figure 4-4b. The data in the figure are normalized to $2P/\pi at$. The figure shows that the tangential (xy) and radial (yy) stresses along the y-axis converge to a high compressive stress values at the strips, equal to the applied pressure. Tangential (yx) and radial (xx) stresses along x-axis reach a value of zero at the boundary. In addition, the tangential (xy) stresses along the vertical diameter are approximately constant from the center to r/R = 0.60, whereas, other stresses suffer relatively greater changes near the center of the element.



(a) Schematic diagram



(b) Stress distribution

Figure 4-4 Theoretical representation of Hondros solution

Knowledge of the stress values as is defined in Equations 4.2 through 4.5, the corresponding strains are estimated using the three dimensional elastic theory equations:

$$\varepsilon_x = \frac{1}{E}[\sigma_x - \mu(\sigma_y + \sigma_z)] \tag{4.6}$$

$$\varepsilon_y = \frac{1}{E}[\sigma_y - \mu(\sigma_x + \sigma_z)] \tag{4.7}$$

$$\varepsilon_z = \frac{1}{E}[\sigma_z - \mu(\sigma_x + \sigma_y)] \tag{4.8}$$

For the purpose of simplification, the three dimensional analysis can be reduced to two dimensional analysis for special element size and loading conditions. For the case of a circular disk, the two dimensional analysis can be categorized as (a) plane stress and (b) plane strain. The applicability of these two types of analysis are functions of the size of the element. In the case of a thin plate, the stress σ_z along the thickness is zero on both faces of the plate, and it may be assumed tentatively that they are zero also within the plate. Additionally, the strain in the z direction (ε_z) may be neglected for further simplification. The state of stress is then specified by σ_x and σ_y only, and is called plane stress. The above relationships can be simplified and reduced to:

$$\varepsilon_x = \frac{1}{E}(\sigma_x - \mu\sigma_y) \tag{4.9}$$

$$\varepsilon_y = \frac{1}{E}(\sigma_y - \mu\sigma_x) \tag{4.10}$$

For a circular disk, the total deformation, δ, in the *x* and *y* directions can be obtained by integrating the plane stress equation over the radius from -r to +r. This will result in the following equations:

$$\delta_{yy} = \int_{-r}^{+r} \varepsilon_{yy} = \int_{-r}^{+r} \frac{1}{E}(\sigma_{yy} - \mu\sigma_{xy}) \tag{4.11}$$

$$\delta_{xx} = \int_{-r}^{+r} \varepsilon_{xx} = \int_{-r}^{+r} \frac{1}{E}(\sigma_{xx} - \mu\sigma_{yx}) \tag{4.12}$$

As stated earlier, in this study, only horizontal measurements of deformation were made. so prediction of Poisson's ratio was determined from the following regression equation (Mirza & Witczak).

$$\mu = 0.15 + \frac{0.35}{1 + \exp(3.1849 - 0.04233xTemp)} \tag{4.13}$$

Temperature in the above equation is expressed in degree Fahrenheit. After obtaining Poisson's ratio, the modulus can be obtained from Equations 11 and 12 for a known corresponding deflection value. The deflection values can be obtained from the laboratory indirect tensile test. The deflection in the horizontal direction, δ_{xx}, is tensile and is positive. The modulus value can then be obtained from the following equation:

$$\text{Resilient Modulus} = E = M_R = \frac{P}{t\delta_{xx}}(a + b\mu) \tag{4.14}$$

In the above equation "a" and "b" are the integration constants and their value depends upon the gage length or the length over which the deflection measurements were made. For this specific study a gage length of 1.7-inch (43-mm) was used. The resulting integration constant used are: a = 0.2128, b = 0.6938. Figure 4-5 is a schematic of the indirect tensile strength and fatigue equipment setup. The load is applied on the specimen through the top-loading strip along the diametral axis of the specimen. The specimen used has a diameter of 100 mm with a thickness of 65 mm. Servo-hydraulic testing machine capable of applying haversine load for Resilient modulus estimation.

The above setup was used for estimating the resilient modulus and fatigue properties of the asphalt mixes. In addition, permanent deformation parameters along with the flow properties were also evaluated. Testing was carried out on 5 to 6 replicate specimens for each mix type. Testing was carried out at 2 Hz, with pulse time of 0.1 second and a rest period of 0.4 seconds resulting in a total cycle time of 0.5 seconds. The testing was carried out at a single temperature of 25 ^{o}C. Cooper servo hydraulic test system was used to load the specimens. At temperatures 25 ^{o}C temperature, dynamic horizontal stress varies between 200 and 450 kPa. The variation in stress level was needed to establish the fatigue properties as well as the flow properties for the selected mixes.

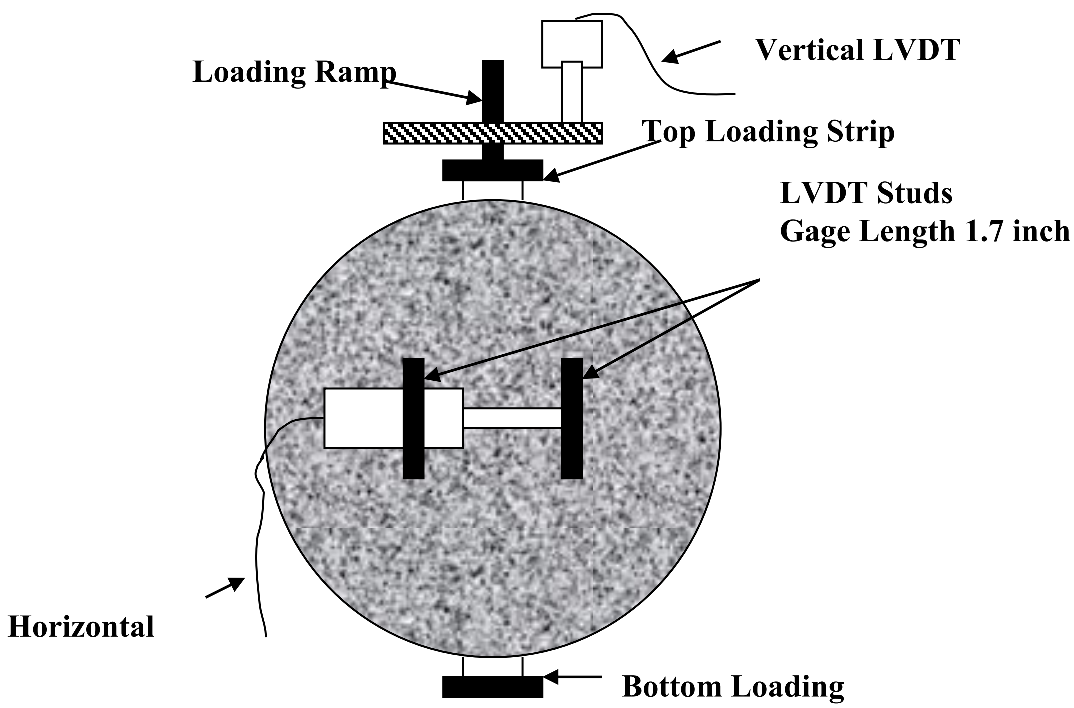


Figure 4-5 Tensile and fatigue test setup

For all measurements of dynamic strain (displacement), two spring-loaded LVDTs clamped horizontally on diametrically opposite specimen sides were used. Mounting arrangement was used for LVDTs to have a gage length of 43 mm. All tests were conducted within an environmentally controlled chamber throughout the testing sequence (i.e. temperatures was held constant within the chamber to ±0.5 ^{o}C throughout the test).

Uniaxial dynamic modulus testing

The complex (dynamic) modulus test is conducted to determine both the linear visco-elastic and elastic properties of pavement materials. The complex modulus, E^*, is defined as a complex number that relates stress to strain for a linear visco-elastic material subjected to a sinusoidal loading. The absolute value of the complex modulus, $|E^*|$, is commonly referred as the dynamic modulus. Dynamic modulus values of asphaltic materials are normally conducted on unconfined specimen using a uniaxial applied sinusoidal stress pattern as shown in Figure 4-6. The applied stress and the resulting strains are measured continuously throughout the test. Mathematically, the dynamic modulus is defined as:

$$E^* = \left|E^*\right| = \frac{\sigma_o}{\epsilon_o} \tag{4.15}$$

where σ_o and ε_o are the peak stress and strain amplitudes.

Figure 4-6 Dynamic modulus setup for uniaxial testing

The dynamic modulus tests were conducted on two replicate specimens for each mix type. For each mixture tested, a full factorial test frequencies (0.1, 0.5, 1, 5, 10 and 25 Hz) and temperatures (10 ^{o}C, 25 ^{o}C, and 40 ^{o}C). Cooper servo hydraulic test system was used to load specimens. At temperatures 10 ^{o}C temperature, dynamic (peak) stress between 30 and 35 psi was used, while peak dynamic stress between 16 and 17.5 psi was used at temperatures of 25 ^oC and 40 ^oC (a load cell was used to record actual dynamic stresses applied on each test specimen). The dynamic moduli were measured by applying a dynamic sinusoidal stress (continuous wave) on unconfined specimens.

For all measurements of dynamic strain (displacement), two spring-loaded LVDTs clamped vertically on diametrically opposite specimen sides were used. Mounting arrangement was used for LVDTs to have a gage length of 57 mm. All tests were conducted within an environmentally control chamber throughout the testing sequence (i.e. temperatures was held constant within the chamber to $\pm 0.5^oC$ throughout the entire test). After testing at a given temperature was completed the new temperature was adjusted in the chamber for the next day test and specimens stored within the chamber to reach new equilibrium temperature. A minimum of 6 hours equilibrium temperature was established before each test.

CHAPTER 5 - DATA ANALYSIS, RESULTS AND DISCUSSIONS

5.1 INTRODUCTION

The main objective of this research is to implement the new Superpave mix design technology compatible with the material properties, traffic and environmental conditions in Pakistan and conduct technology transfer to the highway authorities and pavement industries in Pakistan. A systematic and rational approach for implementing Superpave mix design is adopted in this research study as described in Chapter 2. In this chapter, the data analysis and results from the laboratory testing mentioned in Chapter 3 are documented. For convenience, the chapter is divided into two main parts: (a) asphalt binders, and (b) HMA. In each part, the test results of the specific material type are presented, discussed and then summarized.

5.2 ASPHALT BINDERS

The asphalt binder data analyses and results from conventional and Superpave testing are presented first. Later, the analyses and results of required binder grades to suit the local climate in Pakistan are documented.

5.2.1. Conventional and Superpave Binder Characterization

Asphalt binder is a viscoelastic material i.e., its behavior depends on both temperature and loading frequency. As asphalt is a byproduct of petroleum crude, the source of crude has a significant impact on the complex chemical nature of the binders. Therefore, considering the intricate chemical composition and variability involved in characterizing an asphalt binder, its properties are characterized into two broad groups: physical and rheological (*12-14*). Traditionally, the physical and rheological characterization of binders is accomplished by conducting conventional tests (CT) such as penetration, softening point, viscosity (absolute and kinematic) and ductility etc. Lately, the rheological characterization of binders has gained a widespread acceptance in practice due to new developments e.g., Superpave testing protocols (*15, 16*). The rheological properties of binders include temperature and shear susceptibility, rate of loading dependence and stiffness. These properties are determined by performing several Superpave tests including dynamic shear rheometer (DSR) and bending beam rheometer (BBR) tests after preparing samples by using rolling thin film oven (RTFO) and pressure aging vessel (PAV) to simulate short- and long-term binder aging, respectively. In addition, rotational viscometer (RV) is used to characterize its temperature-viscosity behavior over a wide range of temperature (*15*). These binder properties significantly impact the short- and long-term flexible pavement distress and performance. Consequently, asphalt binder characterization plays a vital role in predicting flexible pavement performance. While conventional tests (absolute and kinematic viscosity) can be used to measure viscosity of asphalt binders at limited temperatures, RV can determine temperature-viscosity relations at a wide range of temperatures and loading rates. The measure of binder's resistance to flow under external forces is termed as the binder viscosity (η). Also, viscosity is determined as the ratio of shear stress (τ) to shear strain (γ) rate at a given temperature. From rheological perspective, binders behave as Newtonian liquids at very high temperatures—the ratio is nearly constant, while at low or intermediate temperatures those behave like non-Newtonian fluids—the ratio varies.

The Superpave asphalt binder specifications are intended to improve pavement performance by limiting the potentials for plastic deformation and cracking of the asphalt binders. As the asphalt binder rheological properties control the viscoelastic behavior of hot-mix asphalt (HMA); the HMA behavior depends on the binder viscosity temperature susceptibility and the rate of loading. The mechanistic-empirical pavement design guide (M-E PDG) uses a dynamic modulus (E^*) master curve to characterize HMA behavior for different temperatures and loading rate (*17*). The binder viscosity temperature susceptibility (VTS) characteristics play a major role in determining the HMA master curve. Three test methods are specified to determine the asphalt binder VTS: (i) dynamic shear rheometer (DSR) for determining G_b^* and δ; (ii) conventional tests such as penetration, kinematic and absolute viscosities, and ring and ball softening point; and (iii) rotational viscosities. In this study, the three test methods were employed to characterize 22 original, modified, and blended binders. The overall goal of the study is to facilitate the implementation of Superpave specification in Pakistan. Consequently, appropriate binder performance grades (PG) will be recommended for pavement construction in different regions according to the indigenous climatic (high and low temperature) conditions. Recall that the main objectives of this study are to: (a) compare the differences in the binders VTS parameters obtained from three test methods, and (b) evaluate the impact of these differences (if any) on E^* and flexible pavement performance predictions using the M-E PDG.

5.2.1.1. Viscosity-temperature susceptibility of binders

The binder viscosity temperature susceptibility (VTS) characteristics play a major role in determining the HMA master curve, especially when design levels 2 and 3 of the M-E PDG are used. For these two input levels, E^* could be predicted by the Witczak equation (*13*), whereas, three test methods are specified to determine the asphalt binder VTS. The ASTM model given by Equation (5.1) is used to define the viscosity-temperature relationship.

$$\log\log\eta = A + VTS\log T_R \tag{5.1}$$

where:

η = viscosity, cP
A= regression intercept
VTS= regression slope (viscosity, temperature susceptibility parameter)
T_R= temperature, degree Rankine

Results of the three laboratory test methods mentioned above, can be utilized to determine the A and VTS parameters as discussed below.

Conventional testing: For some practical situations or when data are available, simple binder characterization without conducting a full range of binder tests could be conducted. In these scenarios, existing relationships can be used to determine viscosities at different temperatures. For unmodified asphalt binders, the ring and ball softening point corresponds to a viscosity of 13,000 Poise and penetration test results can be converted to viscosity using Equation (5.2) (*18*).

$$\log \eta = 10.5 - 2.26 \log(Pen) + 0.00389 \left[\log(Pen)^2 \right] \tag{5.2}$$

where:
Pen= Penetration value using 100 g loading for 5 seconds

Traditionally, conventional tests are used to measure absolute and kinematic viscosities of binders at 60 and 135 ^oC, respectively. From these conventional tests, the binder viscosity-temperature susceptibility can be determined for a temperature range of 25 to 135 ^oC. Once, the viscosities are determined at different temperatures, Equation **Error! Reference source not found.** could be used to determine the A and the VTS parameters empirically.

Rotational viscometer: A rotation viscosity test is used to determine the flow characteristics of the asphalt binder at high temperatures (80 to 185 ^oC). The resulting temperature-viscosity relationship is used for determining mixing and compaction temperatures of the hot-mix asphalt (HMA). The measured viscosities from the test can also be used to determine the A and VTS parameters empirically using Equation **Error! Reference source not found.**.

Dynamic shear rheometer (DSR): In the Superpave binder characterization methods, the binder complex shear modulus (G_b^*) and the phase angle (δ) are obtained from the DSR test. The DSR data can be used to quantify the effects of both temperatures and loading rates on binder shear stiffness. The measured binder G_b^* and δ data could be used to estimate the viscosities using Equation (5.3) (*12, 17*)

$$\eta = \left(\frac{\left| G_b^* \right|}{\omega} \right) \left(\frac{1}{\sin \delta_b} \right)^{a_0 + a_1\omega + a_2\omega^2} \tag{5.3}$$

where:
η = viscosity, cP
$|G_b^*|$= binder shear modulus, Pa
δ_b= binder phase angle, degree
ω= angular frequency, rad/s
a_0, a_1, a_2= fitting parameters, 3.639, 0.1314, and -0.0009, respectively

Equation (5.3) is reduced to Equation (5.4) for $\omega = 10$ rad/s, which is the specified test frequency for temperature sweep in the Superpave performance grading system

$$\eta = \left(\frac{\left| G_b^* \right|}{10} \right) \left(\frac{1}{\sin \delta_b} \right)^{4.8628} \tag{5.4}$$

where; all variables are the same as defined above

It should be noted the binder viscosity obtained through the above mentioned methods is independent of loading frequency, meaning that a constant binder viscosity (Newtonian behavior) is assumed for different frequencies (*12*). The DSR test can be used for characterizing binder behavior at intermediate temperatures. Due to measurement limitations, a DSR temperature range between 7 and 82 ^{o}C is typically used. Brief details of binders and laboratory testing results are presented in the next section.

5.2.1.2. Laboratory testing results

To evaluate the locally available materials in Pakistan, samples of the virgin asphalt binders produced by the two major oil refineries were obtained. It should be noted that the properties of the asphalt binders are function of several variables including the source of the crude oil, the refining process, storage facilities, and the mixing procedures used by the refineries. Both refineries produce limited grades of asphalt binders. Typical binder grades produced and obtained from the Attock Refinery are Pen grades 80/100, 60/70 and 60/70 polymer modified binder (PMB), and from the Karachi Refinery, Pen grades 80/100, 60/70, and 40/50. The Superpave specifications are intended to improve performance by limiting the potential of the asphalt binder to contribute to permanent deformation, and low temperature and fatigue cracking in asphalt pavements. The evaluation of local binders is intended to explain how each of the new test parameters relates to the observed pavement performance and how to select the asphalt binder grade for a specific project. Consequently, the evaluation data will indicate whether or not the local binders meet the Superpave performance grading specifications in the local climate.

Further, considering the prevailing environment and traffic conditions in Pakistan, blending of different types of asphalt binders may be needed to produce grades meeting the Superpave requirements. In addition, under certain climatic and loading conditions, polymer modification may be required. In order to optimize the use of polymers in a cost-effective manner, polymer with varying percentages mixed with the neat asphalt were examined in this study. Only two polymer types were used in the testing. Reactive polymer was mixed with binders from Attock refinery and non-reactive polymer was mixes with binders from Karachi refinery. The test matrices for polymer blending and modification for the two refineries are given in Tables 5-1 and 5-2, respectively.

The two test matrices resulted in a total of 22 binder types. The codes for all binders tested and analyzed are presented in Table 5-3. All these 22 binders (neat and modified) were individually characterized using conventional (penetration, softening point and viscosity) and Superpave (DSR, BBR and RV) tests. For each binder type, three replicates were tested and the average results were considered in the analyses.

Table 5-1 Blending of original Attock Refinery binders and polymer modification

Binder type	80/100	60/70	60/70 P	Polymer*
80/100	-	-		1.35, 1.7, 2.0
60/70	0, 20, 50, 100[1]	-		1.35, 1.7, 2.0
60/70 (P)	-	-	100	-

* Elvaloy® RET, a reactive elastomeric terpolymer with the chemical designation EGA (ethylene/glycidyl/acrylate).

Table 5-2 Blending of original Karachi Refinery binders and polymer modification

Binder type	80/100	60/70	40/50	Polymer**
80/100	-	-	0, 20, 50	2.5, 3.5, 4.5
60/70	0, 20, 50, 100[1]	-	-	-
40/50	-	50	-	-

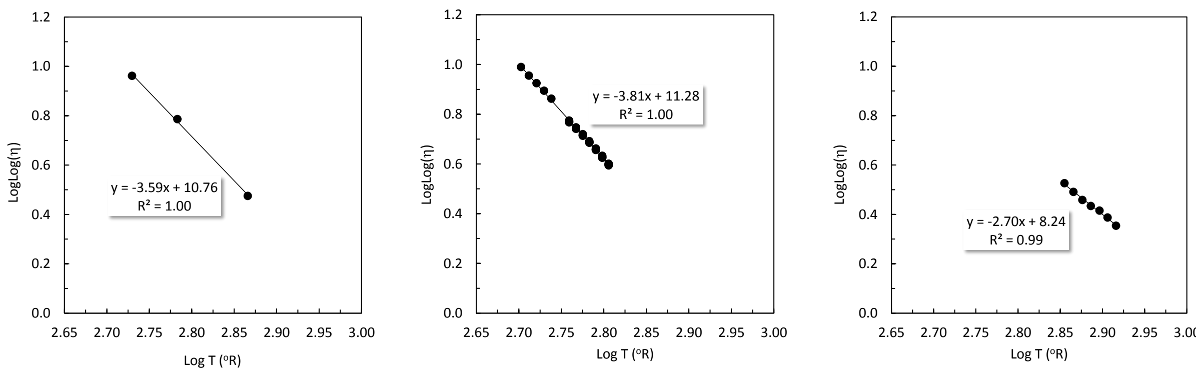
** Elvaloy® AC, a non-reactive polymer, [1] Percentage of column binder with row binder types for blending. For example, 0 means 0% 60/70 with 100% 80/100 binder.

Table 5-3 Binder coding for 22 binders used in this study

No.	Binder Code	Binder Description
1	APMB	Attock Polymer Modified
2	A6/7	Attock Pen 60/70
3	A8/10	Attock Pen 80/100
4	K4/5	Karachi Pen 40/50
5	K6/7	Karachi Pen 60/70
6	K8/10	Karachi Pen 80/100
7	BA6/7(20)8/10(80)	Blended Attock: Pen 60/70 (20%) and Pen 80/100 (80%)
8	BA6/7(50)8/10(50)	Blended Attock: Pen 60/70 (50%) and Pen 80/100 (50%)
9	PA8/10(1.35)	Polymer Modified Attock 80/100 with 1.35% Elvaloy
10	PA8/10(1.70)	Polymer Modified Attock 80/100 with 1.70% Elvaloy
11	PA8/10(2.00)	Polymer Modified Attock 80/100 with 2.00% Elvaloy
12	PA6/7(1.35)	Polymer Modified Attock 60/70 with 1.35% Elvaloy
13	PA6/7(1.70)	Polymer Modified Attock 60/70 with 1.70% Elvaloy
14	PA6/7(2.00)	Polymer Modified Attock 60/70 with 2.00% Elvaloy
15	BK6/7(20)8/10(80)	Blended Karachi Pen 60/70 (20%) and Pen 80/100 (80%)
16	BK6/7(50)8/10(50)	Blended Karachi Pen 60/70 (50%) and Pen 80/100 (50%)
17	BK8/10(20)4/5(80)	Blended Karachi Pen 80/100 (50%) and Pen 40/50 (80%)
18	BK8/10(50)4/5(50)	Blended Karachi Pen 80/100 (50%) and Pen 40/50 (50%)
19	BK6/7(50)4/5(50)	Blended Karachi Pen 60/70 (50%) and Pen 40/50 (50%)
20	PK8/10(2.5)	Polymer Modified Karachi 80/100 with 2.5% AC
21	PK8/10(3.5)	Polymer Modified Karachi 80/100 with 3.5% AC
22	PK8/10(4.5)	Polymer Modified Karachi 80/100 with 4.5% AC

Note: Shaded binders were used for the M-E PDG analysis.

Comparison of A-VTS from different methods: According to Equation**Error! Reference source not found.**, the respective dependent [loglog(η)] and independent [logT in Rankine] variables were plotted for each binder type and test method. The intercept and slope of the linear line represent the A and VTS parameters of Equation**Error! Reference source not found.**. Figure 5-1 shows an example of the linear relationship for binder #1 for the three test methods. It should be noted that test temperature range is dependent on the test method. The temperature ranges of 25 to 135 ^{o}C, 100 to 185 ^{o}C, and 7 to 82 ^{o}C are for conventional, RV, and DSR tests, respectively.



(a) Conventional (b) DSR (c) RV

Figure 5-1 The A-VTS parameters plots for binder #1

Figure 5-2 shows the VTS parameter for all binders tested by all methods. In general, the VTS parameters from RV test is the lowest than those measured from the other two test types (CT and DSR). The VTS parameters calculated based on CT are slightly lower than those measured by the DSR test for all binder types used in the study. The descriptive statistics shown in Table 5-4 confirm these general observations. The 95% confidence mean intervals for VTS parameter by test type are also given in the table. The statistical comparison of multiple means using one way analysis of variance (ANOVA) showed that the mean of parameter A obtained using CT and DSR tests data is significantly different from that of RV while the overall means between CT and DSR test methods are not different at 5% significance level. Similar results were obtained when VTS parameters were compared among different methods.

Figure 5-3 shows the mean comparisons of A-VTS parameters among different test methods. The 95% confidence intervals are also shown in the figure. The means between test methods are not significantly different from each other if the 95% confidence intervals overlap. On the other hand, non-overlapping intervals show a significant statistical difference between the means.

In order to further investigate the reasons for the A-VTS mean differences between test methods, the binders were stratified according to type i.e., neat vs. modified. The A-VTS mean comparisons were made again within each binder type. Figure 5-4 shows the results of those comparisons. Figure 5-4a shows that, for neat binders, the A-VTS parameters are significantly different among all test methods while, for modified binders, the difference is only between RV and the other two test types (see Figure 5-4b).

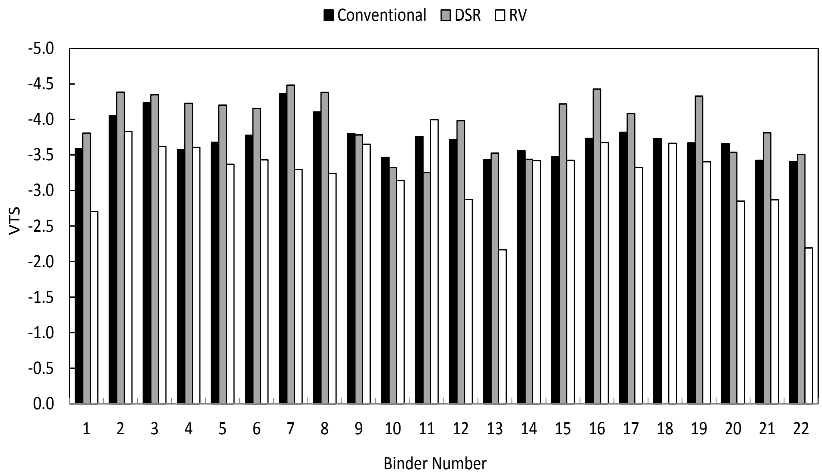
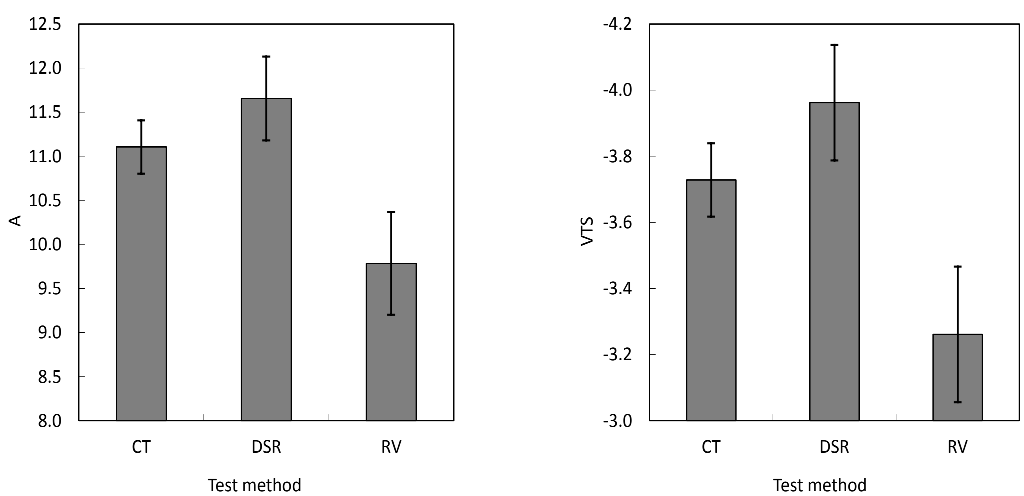


Figure 5-2 The VTS parameters for all binders by test method

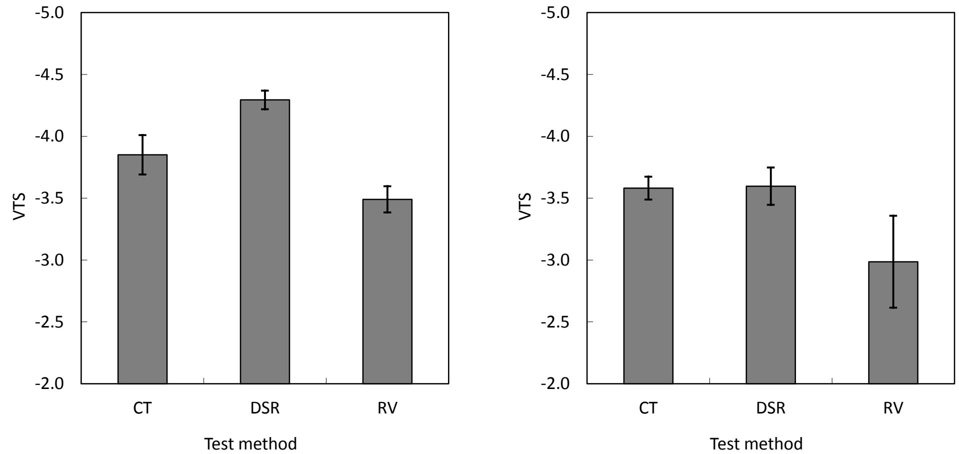
Table 5-4 Descriptive statistics of VTS by test method

Test Type	n	Mean	Std. Dev.	Std. Error	95% CI	
					Lower	Upper
CT	22	-3.73	0.26	0.055	-3.84	-3.62
DSR	21	-3.96	0.40	0.087	-4.14	-3.79
RV	22	-3.26	0.48	0.103	-3.47	-3.06



(a) A (b) VTS

Figure 5-3. Average A-VTS parameters measured by different test methods



(a) Neat binders (b) Modified binders

Figure 5-4 Average A-VTS parameters measured by binder type

The above results were based on the comparison of the A and VTS parameters means by test methods from the statistical standpoint. However, in order to identify the practical differences among test methods, the impact of A-VTS parameters on E^* master curves and predicted pavement performance was investigated using the M-E PDG version 1.0 software.

<u>*Impact of A and VTS parameters on E* and on pavement performance*</u>: In this analysis, two binder types were identified based on the DSR test results. The neat binder #7 had the largest VTS parameter (-4.5) while the modified binder #11 possessed the lowest (-3.2). The results for the same binder types were considered for other test methods (i.e., CT and RV). The E^* master curves were generated by matching the A and VTS parameters using the M-E PDG by changing binder PGs for Level 3 inputs. A typical flexible pavement cross-section having 8-inch thick HMA and base and subbase layers each having resilient modulus (MR) of 40,000 psi under laid by an A-6 subgrade soil with MR of 14,000 psi were assumed. A wet-freeze climate for Michigan and 10 million ESALs for a 20 years design life were used in the analysis. Figure 5-5 presents the E^* master curves for the two binders types within each test method. Basically, the figure shows the sensitivity of the master curves to the A and VTS parameters. Higher absolute values of the VTS parameter cause the binder to be more temperature susceptible and consequently the master curve shifts to the left on the reduced time scale (lower time or higher frequency) or vice versa. This also means the E^* of the binder will be significantly reduced at low frequencies or at higher temperatures. As expected, the impact of the A and VTS parameters on E^* is lower at low temperatures or high frequencies. The same six E^* masters curves were used to analyze a flexible pavement with the M-E PDG software having 8-in thick HMA and base layers. The objective of the analysis was to evaluate the impact of variations in the master curves on the predicted flexible pavement performance. Figure 5-6 illustrates the predicted pavement performance after 20 years of service life. It should be noted that all variables in the analysis were kept at the same levels except the E^* master curves.

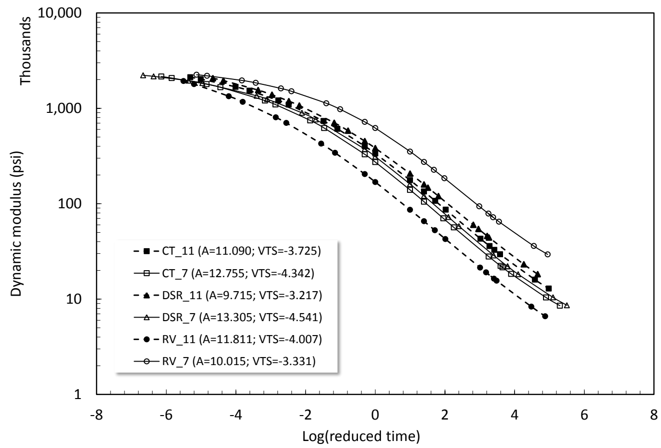
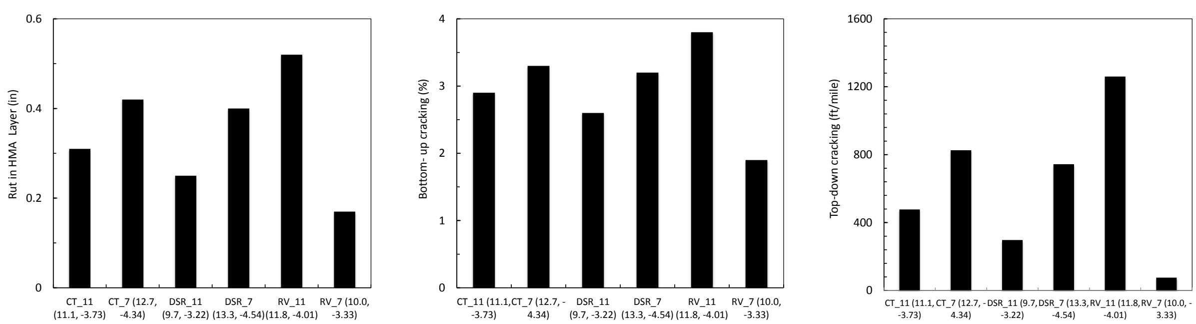


Figure 5-5 Effect of VTS parameters on the HMA master curve

The results of the performance prediction show that A-VTS parameters (or the variations in the E^* master curve) have an important effect on pavement performance, especially within each test method. The HMA layers with higher absolute VTS value are expected to have, in the long run, more HMA rutting, fatigue and longitudinal cracking. When comparing performance trends among test methods, the results show that relative to the DSR results (as reference) if the A-VTS are determined from CT, the pavement performance will be slightly over predicted. However, if the A-VTS are based on the RV test, the performance prediction will be significantly different from the results based on the other two test methods.



(a) HMA rutting (b) fatigue cracking (c) Longitudinal cracking

Figure 5-6 Effect of VTS parameters on pavement performance

5.2.1.3. Discussion of results

The binder viscosity temperature susceptibility (VTS) characteristics play a major role in determining the HMA master curve, especially when design input Level 3 is used in the M-E PDG. The Level 3 inputs for A-VTS parameters are based on default values depending on the binder PG, viscosity grading, and penetration grading systems. Because the E^* master curve determination is sensitive to A-VTS parameters, the default A-VTS parameters according to different grading systems (i.e., penetration, viscosity and PG grading) may not be able to capture the unique binder behavior and thus the expected pavement performance. It will be useful if the A-VTS parameters could be directly used as an input in the M-E PDG to circumvent this concern. As mentioned before, the range of temperature used to characterize a binder is an important factor considering its Newtonian and non-Newtonian behavior. The high temperature range used during the RV binder testing clearly shows that A-VTS parameters are significantly different from other test types considered in this study. Therefore, the most logical choice of temperature range to be used in binder testing should correspond to actual surface temperatures that pavement experience at a particular location. The DSR test temperature range corresponds closely to the field temperatures of all the test methods considered in the study.

5.2.1.4. Conclusions

In this study, the A-VTS parameters for twenty two (22) neat and polymer modified binders were obtained from three test methods and compared. The results show that A-VTS parameters could be significantly different depending on the test method. Furthermore, the impact of the VTS parameters on the E^* master curve shows that it is significantly affected by the temperature susceptibility of the binder. As a consequence, the pavement performance can be affected. The results also show that among various test methods, CT, RV, and DSR can be used to determine A-VTS parameters for the pavement design purposes; however, the designer should be aware of the consequence of using an input A-VTS that may not truly represent the field conditions (i.e., the temperature range used in testing).

5.2.1.5. Summary

The Superpave asphalt binder specifications are intended to improve pavement performance by limiting the potentials for plastic deformation and cracking. Hot-mix asphalt (HMA) behavior depends on the binder's—temperature susceptibility, and the rate of loading. The binder viscosity temperature susceptibility (VTS) characterization plays a major role in determining the HMA dynamic modulus (E^*) master curve, especially when input Level 3 is used in the M-E PDG. For input Levels 2 and 3, E^* could be predicted by the Witczak equation, whereas, three test methods are specified to determine the asphalt binder VTS. These tests are (i) dynamic shear rheometer (DSR) for determining shear modulus (G_b^*) and phase angle (δ); (ii) conventional tests such as penetration, kinematic and absolute viscosities, and ring and ball softening point; and (iii) rotational viscosity. All test methods can be utilized to determine the ASTM "A-VTS" parameters. In this study, the A-VTS parameters for twenty two (22) neat and polymer modified binders were obtained from the three test methods and compared. The results show that A-VTS parameters from the three tests could be significantly different. Further, the impact of the VTS parameters' variations on the E^* master curve and on the predicted flexible pavement performance are discussed.

5.2.2. Temperature susceptibility using activation energy of flow

Generally, asphalt binders are viscoelastic liquids; their short-time loading behavior is like that of elastic solid, long-time behavior resembles that of viscous liquids and at intermediate-time loading they behave as viscoelastic liquids. A fixed loading time at low temperatures precipitates a short-time loading behavior while high temperatures cause long-time loading behavior (*19*). Rheological properties of viscoelastic materials reflect the relationship between load and deformation. These properties are fundamental in understanding the behavior of these materials. The rheological properties of these materials play important roles in the evaluation and selection of paving materials, and in the analysis and design of asphalt pavements (*20*). Due to ease of testing and complex nature of the material, asphalt binder properties are characterized into two broad groups: physical and rheological properties (*12-14*). The need for both types of properties is further emphasized in the literature because of significant difference in the rheological behavior of binders even though they may possess similar physical properties or performance grade (PG) (*21-23*). Traditionally, the physical characterization of binders is accomplished by conducting conventional tests (CT) such as penetration, softening point, viscosity (absolute and kinematic), ductility, etc. The rheological test protocols are used by most highway agencies to better characterize the binder's viscoelastic behavior under the expected conditions (*15*). These protocols consist of Superpave tests such as dynamic shear rheometer (DSR) and bending beam rheometer (BBR) tests after preparing the binder samples in a rolling thin film oven (RTFO) and pressure aging vessel (PAV) to simulate short- and long-term binder aging, respectively. Further, rotational viscometer (RV) is used to characterize the binder's temperature-viscosity behavior and to determine the mixing and compaction temperatures of the binder (*15*).

The asphalt binder viscosity is a measure of its shear resistance to flow, which is a function of the binder's temperature. As the temperature increases, the thermal energy of the molecules increases and the resistance to flow decreases; hence the viscosity decreases. When a liquid flows, layers of liquid molecules slide over each other while intermolecular forces resist the motion and cause resistance to flow. This phenomenon results in an activation energy (AE) barrier for viscous flow which must be exceeded for flow to occur (*24, 25*). Equation (5.5) is known as "Arrhenius" equation and is used to model the viscosity-temperature dependency of asphalt binders.

$$\eta = Ae^{\frac{E_f}{RT}} \tag{5.5}$$

where;

η = viscosity, Pa.s
A= regression constant
E_f= activation energy for flow, kJ/mol
T= temperature, degree Kelvin
R= universal gas constant=8.314 J/mol/k

Equation (5.5) can be linearized by taking natural logarithm as follows:

$$\ln \eta = E_f / RT + \ln A \tag{5.6}$$

Equation (5.6) can be used to determine the AE for flow. An AE value for modified binder is reported around 67 kJ/mol (*26*). In another study, the average AE values for unmodified and modified mixtures were around 205 and 202 kJ/mol, respectively (*27*).

Salomon and Zahi (*24*) used rotational viscometer to determine the viscosity-temperature curves for various binder grades (3 Penetration grades and 6 PGs). The measured AE for the various binders ranged from 55 to 90 kJ/mol. Binders having the same PGs exhibited significantly different AEs. Binders with low AE were found to be less susceptible to temperature changes. Modified asphalt binders showed non-Newtonian behavior within the temperature range of 110 to 160 ^{o}C. It was also found that binders having high asphaltene contents showed higher AE. Aging of the binder due to oxidation increases the intermolecular forces, which results in a higher resistance to flow and AE. Increase in the polymer content in asphalt binder decreases the AE of the flow. Dongre et al. (*14*) reported AE between 192 to 243 kJ/mol for six binders (PG 62-30 to PG 78-30). The AE was found to be highly correlated to the Useful Temperature Range (UTR) of the asphalt binders. The UTR was defined as the algebraic sum of high and low PG of asphalt binders. It was also concluded that AE can be used to estimate E^* without the need of binders testing at various frequencies and temperatures.

Pavements in Pakistan are subjected to wide ranges of conditions. For example, the surface temperature may vary from -10 to more than 80 ^{o}C in cold and warm regions. Another example is that heavily or overloaded trucks may apply the loads very slowly on uphill slope where trucks move at creep speeds or quickly on downhill sections. Thus it is essential to study the properties of the asphalt binders over a wide range of temperatures and loading times to safeguard premature rutting of flexible pavements. Binder properties at long loading times are conveniently determined by viscometers while the properties for short loading times are determined using more suitable methods such as mechanical dynamic tests (*28*). Three test methods were used in this study to characterize the viscosity temperature susceptibility of the 22 original, modified, or blended binders'. These three test methods are (1) dynamic shear rheometer (DSR) for determining complex shear modulus (G_b^*) and phase angle (δ); (2) conventional tests (CT) such as penetration, kinematic and absolute viscosities, and ring and ball softening point; and (3) rotational viscosity (RV). Results of the tests were used to accomplish the overall goal of the study, to facilitate the implementation of Superpave specification in Pakistan. Further, the results will be used to make the proper and cost-effective recommendations regarding the selection of PG binders for the various regions in Pakistan according to the local climatic (high and low temperatures). Recall that the main objectives of this study are to (a) compare binders AE obtained from three test methods, and (b) evaluate the temperature susceptibility of binders using the AE concept.

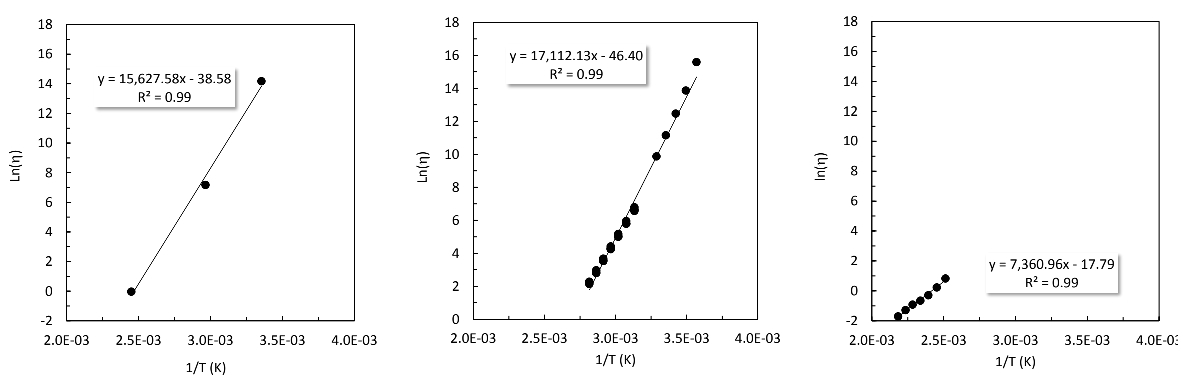
5.2.2.1. Laboratory test results

To evaluate locally available paving materials in Pakistan, samples of the asphalt binders produced by the two oil refineries were obtained. Both refineries produce limited grades of asphalt binders. Typical binder grades produced and obtained for the study from these refineries include: Attock Refinery [Pen. Grades 80/100, 60/70 and 60/70 polymer modified binder], and Karachi Refinery [Pen. Grades 80/100, 60/70, and 40/50]. It should be noted that the properties of the asphalt binders are function of several variables including the source of the crude oil, the refining process, the storage facilities, and the mixing procedures used in the refineries.

The Superpave asphalt binder specifications are intended to improve the performance of asphalt pavements by decreasing the potentials for permanent deformation and low temperature and fatigue cracking. Evaluation of local binders is needed to determine the role of each Superpave test parameter in the observed pavement performance and to select the asphalt binder grade for a specific project. Hence, results of the evaluation will determine whether or not the local binders meet the Superpave performance grading (PG) specifications for the given local climate.

Considering the prevailing environment and traffic conditions in Pakistan, blending of different asphalt binders may be needed to produce grades that meet the Superpave requirements. In addition, under certain climatic and loading conditions, polymer modification may be required. In order to optimize the use of polymers in a cost-effective manner, polymer with varying percentage with neat asphalt were examined in this study. Only one polymer type was used in the testing. The codes and descriptions for all binders tested and analyzed are presented in Table 5-5. The 22 binders (neat and modified) were individually characterized using conventional (penetration, softening point and viscosity) and Superpave (DSR, BBR and RV) tests. For each binder type three replicates were tested and the average measurement of each triplicate was used in the analyses.

<u>Activation energy of flow by different test methods</u>: The dependent variable [ln (η)] and independent variable [1/T in Kelvin] of Equation **Error! Reference source not found.** were plotted for each binder type and test method. The intercept and slope obtained from the linear regression represent the E_f/R and ln(A) parameters of the equation**Error! Reference source not found.**. The slope parameter was multiplied by the Universal Gas Constant (UGC) to estimate the AE. Figure 5-7 shows an example of the linear relationship of binder #1 for the three test methods. Please note that the test temperature range is dependent in the test method. Temperature rangs from 25 to 135 ^{o}C, 100 to 185 ^{o}C, and 7 to 82 ^{o}C for conventional, RV, and DSR tests, respectively. Since the AE was estimated by multiplying each slope in Figure 5.7 by UGC, variations in the slopes reflect variations in the AE estimate. This implies that the AE of the binder is test type dependent. This is related to the different temperature range used in each test.



(d) Conventional (e) DSR (f) RV

Figure 5-7 Calculation of AE for binders #1

Table 5-5 Binder coding for 22 binders used in the study

No.	Binder Code	Binder Description	PG Grade
1	APMB	Attock Polymer Modified	76-6
2	A6/7	Attock Pen 60/70	58-12
3	A8/10	Attock Pen 80/100	58-12
4	K4/5	Karachi Pen 40/50	64-6
5	K6/7	Karachi Pen 60/70	64-12
6	K8/10	Karachi Pen 80/100	58-12
7	BA6/7(20)8/10(80)	Blended Attock: Pen 60/70 (20%) and Pen 80/100 (80%)	-
8	BA6/7(50)8/10(50)	Blended Attock: Pen 60/70 (50%) and Pen 80/100 (50%)	-
9	PA8/10(1.35)	Polymer Modified Attock 80/100 with 1.35% Elvaloy	64-12
10	PA8/10(1.70)	Polymer Modified Attock 80/100 with 1.70% Elvaloy	70-12
11	PA8/10(2.00)	Polymer Modified Attock 80/100 with 2.00% Elvaloy	70-12
12	PA6/7(1.35)	Polymer Modified Attock 60/70 with 1.35% Elvaloy	64-12
13	PA6/7(1.70)	Polymer Modified Attock 60/70 with 1.70% Elvaloy	70-12
14	PA6/7(2.00)	Polymer Modified Attock 60/70 with 2.00% Elvaloy	70-12
15	BK6/7(20)8/10(80)	Blended Karachi Pen 60/70 (20%) and Pen 80/100 (80%)	-
16	BK6/7(50)8/10(50)	Blended Karachi Pen 60/70 (50%) and Pen 80/100 (50%)	-
17	BK8/10(20)4/5(80)	Blended Karachi Pen 80/100 (50%) and Pen 40/50 (80%)	-
18	BK8/10(50)4/5(50)	Blended Karachi Pen 80/100 (50%) and Pen 40/50 (50%)	-
19	BK6/7(50)4/5(50)	Blended Karachi Pen 60/70 (50%) and Pen 40/50 (50%)	-
20	PK8/10(2.5)	Polymer Modified Karachi 80/100 with 2.5% AC	64-12
21	PK8/10(3.5)	Polymer Modified Karachi 80/100 with 3.5% AC	64-12
22	PK8/10(4.5)	Polymer Modified Karachi 80/100 with 4.5% AC	64-12

Figure 5-8 shows the estimated AE for all binders (blend, neat and modified) by three test methods. The AE values calculated based on RV testing is the lowest among all test methods. The AE values from DSR testing are always higher than the other test methods. The relative ranking of the binders depends on the test method. For example, the RV test yielded the highest AE value for binder #11, the DSR test for binder #19 and the conventional tests for binder #7. As stated earlier, higher AE values imply higher binder sensitivity to temperature changes. Conversely, lower AE indicates that the asphalt binder is less sensitive to temperature changes. Table 5-6 shows the descriptive statistics for AE for all 22 binders and for various test methods. From the results, it can be observed that RV test showed the highest coefficient of variation (CoV) (about 15%) while the AE values estimated from conventional tests has the lowest CoV (about 5%). This also implies that the RV test results yielded higher range of AE as shown in the figure. Figure 5-9 depicts comparisons of the AE values obtained by the different tests. The 95% confidence intervals are also shown in the figure. The AE Means between the test methods are not significantly different from each other if the 95% confidence intervals overlap. On the other hand, non-overlapping intervals show a statistically significant difference between Means.

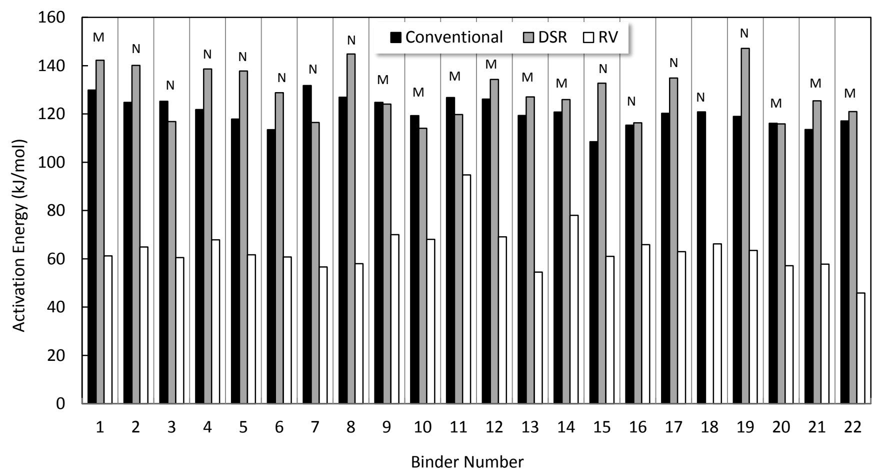


Figure 5-8 AE for all binders for different test methods

Table 5-6 Descriptive statistics for AE

Descriptive Statistics	Test type		
	CT	DSR	RV
Avg.	120.9	128.8	63.9
Std.	5.76	10.45	9.48
CoV	4.76%	8.12%	14.82%
N	22	21	22
Min.	108.50	114.07	45.84
Max.	131.79	147.19	94.73

The one way analysis of variance (ANOVA) analysis also showed that the AE Means among different test types are significantly different from each other at 5% significance level.

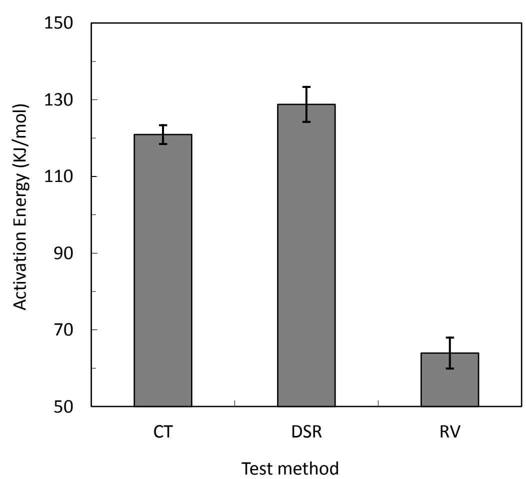
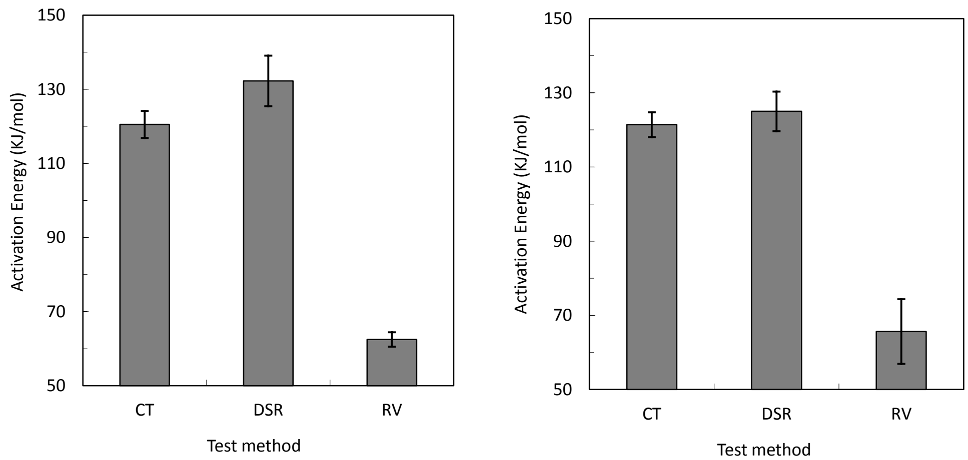


Figure 5-9 Comparison of AE by different test methods

In order to further investigate the reasons for the differences in the AE Mean between different test methods, the binders were stratified according to type i.e., neat vs. modified. The AE mean comparisons were made again within each binder type. Figure 5-

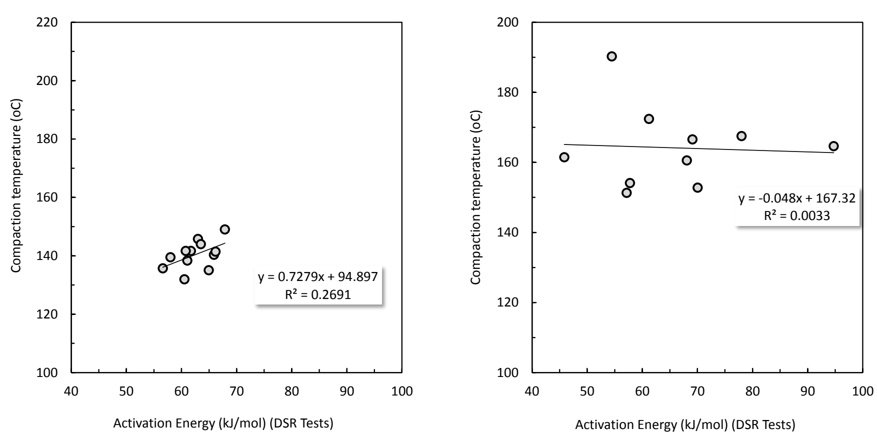
10 shows the results of those comparisons. Figure 5-10a shows that for neat binders, the AE Means are significantly different among all test methods while the difference only exists between RV and the other two test types for the modified binders.



(c) Neat binders (d) Modified binders

Figure 5-10 Comparison of AE by binder and test type

Because the AE values obtained from RV test methods showed the maximum variations for all binders and since the same test is used to determine mixing and compaction temperatures, the correlation between AE and compaction temperatures for the binders were investigated. Figure 5-11 illustrates the relationships between AE from the RV test and compaction temperature obtained through the conventional viscosity criteria. A poor correlation was found between AE and compaction temperatures for neat binders (see Figure 5-11a) while no relationship could be found for the modified binders (see Figure 5-11b). These results imply that AE may not be used to estimate mixing or compaction temperatures for the binders, especially for the modified ones.



(a) Neat binders (b) Modified binders

Figure 5-11 Relationship between AE and compaction temperatures

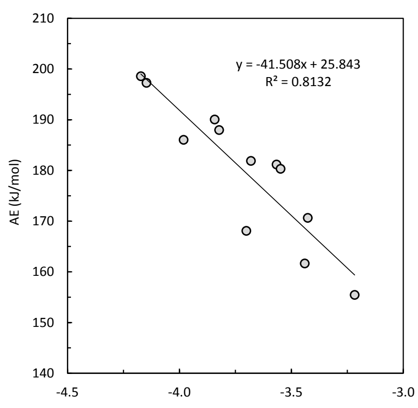
<u>*Relation between activation energy of flow and performance grade*:</u> In order to investigate the relationship between AE and PG grades, the M-E PDG software (version 1.0) was used. Note that a temperature range between 4.5 and 54.5 ^{o}C is considered in the

M-E PDG analysis for generating temperature-viscosity relationship. Nevertheless, a typical flexible pavement structure was analyzed by only changing the PG grade at Level 3. From the A-VTS information generated by the M-E PDG, the AE was calculated using Equation (2). The pavement performance at the end of the 20 years analysis period in terms of distresses was predicted. Table 5-7 lists details of PG grades, A-VTS parameters, predicted pavement performance, and UTR.

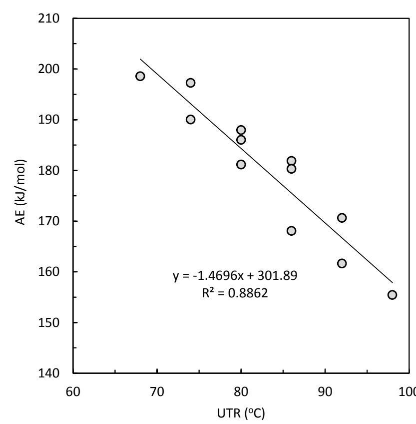
Table 5-7 The M-E PDG results for different binder performance grades

No	PG	A	VTS	AE (kJ/mol)	IRI (in/mile)	HMA rut (in)	Total rut (in)	Long. crack. (ft/mile)	Fatigue crack. (%)	UTR
1	58-10	12.32	-4.17	199	123.20	0.30	0.65	449.00	2.80	68
2	58-16	12.25	-4.15	197	123.20	0.30	0.64	443.00	2.80	74
3	58-22	11.78	-3.98	186	124.10	0.32	0.67	519.00	2.90	80
4	58-28	11.02	-3.70	168	126.00	0.36	0.71	669.00	3.20	86
5	64-10	11.44	-3.84	190	120.40	0.25	0.58	252.00	2.40	74
6	64-16	11.38	-3.82	188	120.70	0.25	0.59	272.00	2.50	80
7	64-22	10.98	-3.68	182	121.50	0.27	0.60	326.00	2.60	86
8	64-28	10.32	-3.44	162	122.80	0.29	0.63	435.00	2.80	92
9	70-10	10.70	-3.57	181	118.40	0.21	0.53	147.00	2.20	80
10	70-16	10.65	-3.55	180	118.40	0.20	0.53	145.00	2.20	86
11	70-22	10.30	-3.43	171	119.40	0.22	0.55	197.00	2.30	92
12	70-28	9.72	-3.22	155	120.90	0.25	0.59	297.00	2.60	98

Figure 5.12 shows the relationships of VTS vs. AE and UTR vs. AE based on the M-E PDG results. Figure 5.12a indicates that higher AE values correspond to higher temperature susceptibility or higher VTS. Whereas the data in Figure 5.12b indicate that lower AE values correspond to harder binders or higher UTR. This result contradicts with the previous findings in the published literature. In order to investigate the impact of aging, the AE values were determined from the DSR test results in which RTFO and PAV aged binders were used. Figure 5-13 shows the relative comparison among all binders. In general, higher aging caused higher AE values, which indicate that aged binders require higher energy levels for flow, which implies lower rut potential.



(a) Temperature susceptibility



(b) Temperature sensitivity

Figure 5-12 Relationship between AE and VTS using the M-E PDG

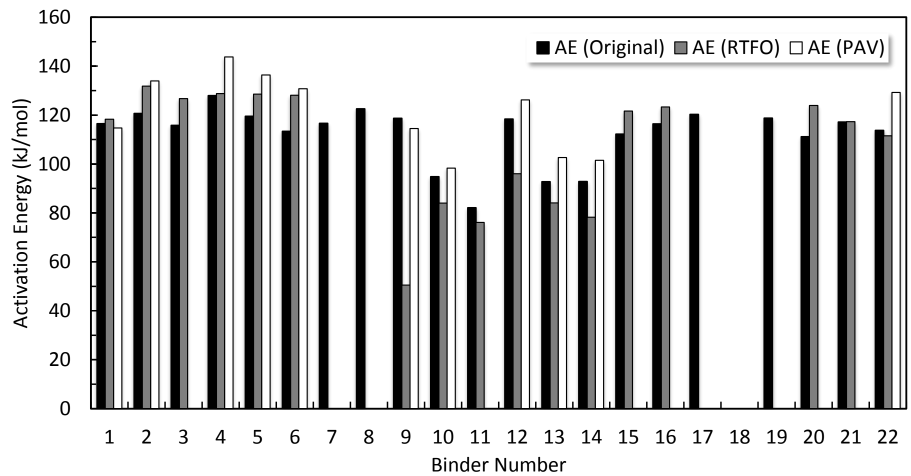
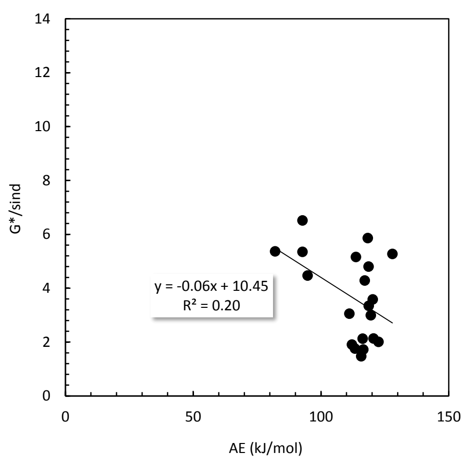
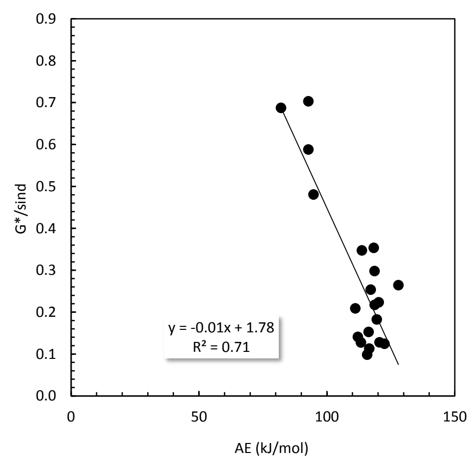


Figure 5-13 AE comparison for all binders for different aging conditions (DSR)

The temperature sweep data from DSR tests for all binders were utilized to investigate correlations between the AE and rheological parameters like $G^*/\sin\delta$ & $G^*\sin\delta$.. Figure 5-14 shows examples of such relationships between AE and $G^*/\sin\delta$ for two different temperatures and for all binders. it can be seen that at the intermediate temperature of 58°C, the AE showed a poor association with $G^*/\sin\delta$ and very reasonable correlation at 82°C. The latter observation indicates that binders having higher AE are more susceptible to rutting. Similar trends were also observed for the $G^*\sin\delta$ parameter. The results apply to all original, RTFO and PAV aged binders.



(a) 58 °C



(b) 82 °C

Figure 5-14 AE relationship with DSR parameters

5.2.2.2. Discussion of results

Asphalt binders behavior is significantly affected by temperature and rate of loading. The binder reacts to a load (shear or axial) in an elastics manner at low temperatures or high loading rate. The response becomes viscoelastic at intermediate temperatures while at very high temperatures; the binders behave as a viscous material. Although the binder grade (stiff vs. soft) impacts the range of temperature or loading rate, the general behavior of the materials remains the same. As stated earlier, three temperature ranges were used in this study depending on the test method: 25 to 135 ^oC, 100 to 185 ^oC, and 7 to 82 ^oC for conventional, RV, and DSR tests, respectively. It is expected that binders behave as a viscoelastic material within the temperature ranges used for conventional and DSR testing and as viscous material for the RV testing. The AE concept was utilized in this study to characterize the temperature susceptibility of the binders; however, contradicting results were found, especially if the DSR temperature sweep and the M-E PDG are used. Setting aside the difference in various test methods, temperature under which the tests are performed was found to be one of the most significant factors defining the binder behavior. The AE determined from the M-E PDG and DSR data show similar trends for binders (i.e., lower AE for harder binders). This could be a consequence of the viscoelastic binder behavior within the test temperature range. In order to capture the viscous behavior of the binders, RV test data could be used to quantify the temperature susceptibility of binders.

5.2.2.3. Conclusions

The temperature-viscosity relationships for twenty two neat and modified asphalt binders were evaluated using variety of binder tests including penetration, kinematic and absolute viscosities, ring and ball softening point, rotational viscometer (RV), and dynamic shear rheometer (DSR). Binders having similar penetration, viscosity and performance grade (PG) could show dissimilar rheological behaviors. Hence, there is a need to fully characterize the binder behavior to capture their rheology over a wide range of temperatures and loading frequencies, especially if the binders are modified. Results of the binder temperature susceptibility data show that the AE of the binders depends on the test methods, which employ different temperatures used to characterize the binders. Correlations between AE and $G^*/\sin\delta$ and $G^*\sin\delta$ parameters show that the binders with higher AE will be more viscous at higher temperatures. A negative correlation between AE and UTR were shown by the M-EP DG. It is recommended that RV data could be used to characterize binders using the AE concept. The AE concept did not show a good correlation with mixing and compaction temperatures, especially for the modified binders.

5.2.2.4. Summary

The temperature-viscosity relationships for twenty two neat and modified asphalt binders were evaluated using conventional tests (CT) such as penetration, kinematic and absolute viscosities, and ring and ball softening point; rotational viscometer (RV); and dynamic shear rheometer (DSR). The temperature ranged from 25 to 135 ^oC, 100 to 185 ^oC, and 7 to 82 ^oC for conventional, RV, and DSR tests, respectively. The test results revealed that binders having similar penetration, viscosity and performance grade (PG) could show dissimilar rheological behaviors. Hence, there is a need to fully characterize asphalt binders to capture their rheology over a wide range of temperatures and loading frequencies. The activation energy (AE) concept was utilized in this study to characterize the temperature susceptibility of the asphalt binders. Correlations between AE and binder rheological properties (e.g., $G^*/\sin\delta$ and $G^*\sin\delta$) were evaluated. In addition, a relationship between AE and UTR determined from the performance grades (PG) was developed. The advantages of characterizing the relative temperature susceptibility of the binders using AE are discussed. Finally, discussion on mixing and compaction temperatures, especially for polymer modified binders, by using AE is also included in this study.

5.2.3. Required Binder Grades for the Local Climate

From October 1987 through March 1993, the Strategic Highway Research Program (SHRP) sponsored and funded a research study to develop new ways to specify, test, and design asphalt materials (*29-33*). The end result was the development of Superior Performing Asphalt Pavements, which I known as Superpave. One of the key aspects of Superpave is the development of the performance based binder specifications termed as Performance Grade (PG) asphalt. The major objective of the SHRP program was to relate mechanical properties of asphalt binders to the expected field performance. Therefore, new test procedures and equipment, and specification were developed to characterize asphalt binders at a broad range of temperature, loading and aging conditions. The three aging conditions specified are original, short term and long term aging. *Original* refers to virgin asphalt from the production plant; *short term aging* refers to properties at the time of production and placement of asphalt mixtures while *long term aging* refers to properties of asphalt binder during the service life of asphalt pavements. In addition to aging conditions, Superpave characterizes the asphalt binders at the expected pavement temperatures in the field. A brief summary of the binder grading systems for the purpose of asphalt mix design is given in the following sections.

5.2.3.1. Binder specifications

The development of binder testing goes back to 1888, when H. C. Bowen invented the Bowen Penetration Machine (*34-39*). In 1910, the penetration equipment, after several modifications, became standard for establishing the consistency of the asphalt binders at 25°C. In 1918 the Bureau of Public Road (USA) introduced *penetration grading* system and by 1931, the American Association of State Highway and Transportation Officials (AASHTO) published the standard specification for penetration graded asphalt binders. The next major change in asphalt grading specification came with the introduction of *viscosity grading* system in the early 1960s. Both ASTM and AASHTO adopted the viscosity grading system and provided grading specifications based on measuring the viscosity at 60 °C (*40, 41*).

The penetration grading system is empirical in nature and provides the relative consistency of binders at specific temperatures, which can be used as an indicator of susceptibility of the asphalt binders to rutting or cracking. The system has performed quite satisfactorily for many decades in identifying the permanent deformation and cracking (fatigue and thermal) potentials of asphalt binders. ASTM D946 specified the five binder grades based on penetration at 25 °C. For a constant temperature, higher penetration values indicate softer asphalt binders. Binders viscosity at 60°C (close to maximum pavement temperature) as well as a minimum viscosity at 135°C are specified in the viscosity grading system,. ASTM D3381 specifies six binder grades based upon the viscosity measured at 60 °C. Table 5-8 provides the standard penetration and viscosity grades. The top row values in the table represent relatively harder asphalt binders while the lower rows represent the softer ones.

Table 5-8 Penetration and viscosity grading system

Penetration Grading		Viscosity Grading	
Grade	Penetration in 0.1 mm	Grade	Viscosity @ 60°C, Poise
Pen 40/50	40-50	AC-40	4000±800
Pen 60/70	60-70	AC-30	3000±600
Pen 85/100	85-100	AC-20	2000±400
Pen 120/150	120-150	AC-10	1000±200
Pen 200/300	200-300	AC-5	500±100
		AC-2.5	250±50

Generally, softer binder grades are used in cold climates to resist cracking potential and harder binders for warmer climates to safeguard against rutting potential. In Pakistan, a standard penetration grade of 60/70 is used for construction of flexible pavements. The refineries in Pakistan produce three penetration graded binders 80/100, 60/70, and 40/50. Viscosity grading is not yet established in Pakistan.

Viscosity grading system based on the fundamental property is considered a step forward in specifying the binder as compared to penetration grading. It requires binder to be tested at 60 °C and 135 °C, which corresponds to typical maximum pavement temperature and temperature at the time of asphalt mixture production and placement in the field, respectively. The viscosity specification at 60 °C is established to minimize rutting potential, whereas, viscosity at 135°C to minimize the potential for tender mixtures during the paving operation. With all these added benefits, the system fails to characterize the binder at low temperatures to minimize the potential of thermal cracking and pavement performance prediction. Figure 5-15 shows the criteria used for penetration and viscosity grading systems. It can be seen that two asphalt binders A and B meeting the penetration and viscosity specifications may behave very differently at other temperatures.

The Superpave system is unique in a sense that asphalt binders are specified based on the expected maximum and minimum pavement temperatures in the field. Mechanical properties requirement remain the same, but the temperature at which the asphalt binders achieve the physical properties corresponds to the pavement minimum and maximum temperatures. For example, high temperature requires binder to have $G^*/\sin\delta$ to be at least 1.0 kPa for un-aged condition (G^* is the shear modulus in kPa and δ is the phase angle). The value of 1.0 kPa remains constant but the temperature at which this value has to be achieved depends upon the maximum pavement temperature. Another important feature of Superpave is that the mechanical properties of the asphalt binders are measured at three conditions: un-aged, short-term aged and long-term aged. The short and long-term aging is simulated in the laboratory using Rolling Thin Film Oven (RTFO) and Pressure Aging Vessel (PAV), respectively. The required mechanical properties at the three aging conditions both for high and low temperatures are specified in the Superpave specifications [MP1: Specification for Performance-Graded Asphalt Binder (*15*)].

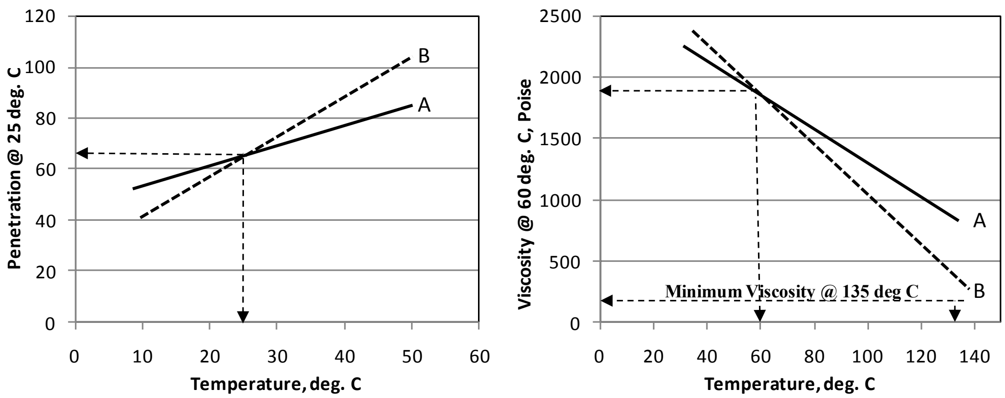


Figure 5-15 Comparison of two binders meeting penetration and viscosity grading

5.2.3.2. Temperature grading

As mentioned earlier, asphalt binders are specified based on the localized temperature regime in which flexible pavement will be constructed. The required physical properties such as stiffness and shear modulus are constant among all binder grades. What differentiates the various binder grades is the temperature at which the requirements must be met. For example a binder classified as PG 64-22, means that the asphalt binder must meet the required properties at a maximum pavement temperature of 64 ^{o}C and at a minimum pavement temperature of -22 ^{o}C. Table 5-9 presents the typical PG binder grades as specified in the SUPERPAVE systems. To illustrate, for a high temperature of 46^{o}C and low temperatures of -34, -40, and -46^{o}C; the PG grades are PG 46-34, PG 46-40 and PG 64-46, respectively. It is important to understand that PG grades are not limited to the ones given in Table 5-9. It is very possible to have PG 58-10, which is not included in the table.

Table 5-9 Binder grades as specified in Superpave specifications (*15, 16*)

High temperature grade, ^{o}C	Low temperature grade, ^{o}C
PG 46	34, 40, 46
PG 52	10, 16, 22, 28, 34, 40, 46
PG 58	16, 22, 28, 34, 40
PG 64	10, 16, 22, 28, 34, 40
PG 70	10, 16, 22, 28, 34, 40
PG 76	10, 16, 22, 28, 34
PG 82	10, 16, 22, 28, 34, 40

The temperatures given in Table 5-9 correspond to the pavement temperature and can be estimated from the air temperature data collected over the years. Superpave defines the high and low temperatures by 7-day average maximum air and l-day minimum air temperatures. <u>The 7-day average maximum temperature is defined as the average highest air temperature for a period of 7 consecutive days within a given year. The 1-day minimum temperature is defined as the lowest air temperature recorded in a given year</u>. The data are collected over multiple years and the design high and low pavement temperatures are then estimated using the average and standard deviations of the data collected for a desired reliability level.

5.2.3.3. Pavement temperature

Several research efforts have been undertaken to relate the air temperature to pavement temperature. Regression equations along with mathematical heat flow theories have been used to develop such correlation. One of these models is that developed as a part of the SHRP. Later, the SHRP Long Term Pavement Performance (LTPP) program was established to support a broad range of pavement performance analysis leading to improved engineering tools to design, construct, and manage pavements. In this regards, the Seasonal Monitoring Program (SMP), a task of LTPP, evaluated the effects of temperature variations on pavement performance and validated the available temperature correlation models (*15, 42-44*). This resulted in a new set of pavement temperature prediction models for the high and low temperature grade. These models are presented below.

High Temperature Models

The SHRP high temperature model was developed from the results of theoretical heat transfer modeling (*45-47*). Based on the data collected from several sites throughout the US, regression model was then developed for predicting the high pavement temperature as a function of depth. Superpave defines the high pavement design temperature at a depth of 20 mm below the pavement surface. Equation (5.7) represents a model developed under the SHRP program, whereas Equation (5.8) is the LTPP model.

$$T_{pav,h} = (T_{air} - 0.00618\ Lat^2 + 0.2289\ Lat + 42.4)(0.9545) - 17.78 + z.\sigma_{air} \quad (5.7)$$

$$T_{pav,h} = 54.32 + 0.78 T_{air} - 0.0025 Lat^2 - 15.14 \log_{10}(d + 25) + z(9 + 0.61\,\sigma_{air}^2)^{1/2} \quad (5.8)$$

where

$T_{pav,h}$	= High AC pavement temperature at 20 mm from surface, °C.
$T_{pav,h,d}$	= High AC pavement temperature at depth d from surface, °C.
T_{air}	= High 7-day mean air temperature, °C.
Lat	= Latitude of the section, degrees.
d	= Pavement depth in mm,
σ_{air}	= Standard deviation of the 7-day maximum air temperature, Deg. C
z	= Standard normal dist. table, z = 2.055 for 98% reliability, and z = 0.0 for 50% reliability

Low Temperature Models

The SHRP protocol considers the 1-day low air temperature as the design low pavement temperature (*46-49*). This can be mathematically represented by the following relationship.

$$T_{pav,l} = T_{air} + 0.051 d - 0.000063 d^2 - z\sigma_{air} \quad (5.9)$$

The LTPP low pavement surface temperature, on the other hand, is presented in Equation (10) below.

$$T_{pav,l} = -1.56 + 0.72T_{air} - 0.004Lat^2 + 6.26\log_{10}(d+25) - z(4.4 + 0.52\sigma_{air}^2)^{1/2} \quad (5.10)$$

where $T_{pav,l}$ = low AC pavement temperature in deg. C, all other parameters are the same as in Equations 5.7 and 5.8

In Equations 5.7 to 5.10, the factors "*z* and σ_{air}" are included to introduce reliability in the selection of binder grade. For example a high pavement temperature grade of 64 at 98% reliability level means that 98% of times the pavement temperature will not exceed 64 ^{o}C.

5.2.3.4. Temperature database

In order to establish the Performance Grade (PG) for temperature conditions in Pakistan, high and low pavement temperature zones were established first. Using the above models, the pavement surface temperatures were estimated using the air temperature data. The data were collected from 64 weather stations located across Pakistan and the following four sources (*50-54*):

1. Master Degree Thesis of Aurangzeb Qazi (National University of Science and Technology, 2008)
2. Pakistan Metrological Department
3. Weather Underground Web Site
4. Surface Water Hydrology Project, WAPDA Pakistan

Weather information from 21 weather stations for the years 1986 to 2006 were obtained from the work done by Qazi (*50, 52*). The database for the 21 stations was expanded for additional three years (2007 to 2009) using the "Weather Underground Web Site". The latter is a commercial weather service that provides weather information including temperature data via the Internet. The website has information on major cities around the world on its web site (*53*). Temperature data from another 43 weather stations were added using the information obtained from Surface Water Hydrology Project, WAPDA in Pakistan (*54*) resulting in a total of 64 weather stations.

Table 5-10 summaries the weather stations used for the development of pavement temperature zoning. The table provides information on the weather station, latitude, and number of years of data used for the analysis. The data collected from each station were used to estimate the yearly high and low air temperatures. The high temperature was based on 7 consecutive days maximum average and the minimum on the lowest one day temperature. The yearly high and low temperature data were then used for the computation of the average and standard deviation values required for computing pavement temperatures. The pavement high and low temperatures were estimated using both the SHRP and the LTPP models (Equations 7 to 10). Table 5-11 provides a summary of pavement temperature data used for estimating the PG requirements at 50% and 98% levels of reliability. In addition to the PG grade information, the table summarizes the average and standard deviation for each weather station. The data indicate that the temperature in Pakistan varies significantly from one end of the country to another, with the northern part much cooler compared to the southern part. The lowest air temperature observed is -14.6 ^{o}C and the high 7-day average air temperature is 48 ^{o}C. Using the SHRP model and 98% reliability level, the range in the pavement temperature is higher than the range in air temperature.

5.2.3.5. Temperature data analysis

The predicted high and low pavement temperatures using the SHRP and the LTPP models for 98% reliability level and foe all 64 weather station sites are listed in Table 5-12 and shown in Figure 5.16. Figure 5-17 presents the same data in the form of a histogram. A paired student t-test was carried out to compare the results of the SHRP and the LTPP models for both 7-days high average and 1-day low pavement temperatures. The results of the paired t-test are summarized in Table 5-13. Values of *t-statistics*, *t-*critical and the probability of null hypothesis being true (*p-value*) are summarized for four cases. In all situations, the absolute value of *t-statistics* is greater than *t-critical*, i.e. because the probability of null hypothesis being true is very small (close to zero) suggest that significant difference exists between the predictions of pavement temperature from the two models. Therefore, the null-hypothesis that there is no statistical difference is rejected.

The data indicate that significant differences in the low and high temperatures were found between the two models. Further, higher variability was found in the predicted low temperatures for both models. Finally, for the same air temperature data, the predicted low pavement temperatures using the SHRP model are generally lower than the ones predicted using the LTPP model. This implies that the SHRP model yields better requirement on the PG binders to resist low temperature cracking.

Table 5-10 Summary of weather stations and data availability

No.	Station	Latitude (degrees)	Data Availability (years) (Low/High)[1]	No.	Station	Latitude (degrees)	Data Availability (years) (Low/High)[1]
1	Bagh	33.83	1986/1998-2007	33	Massan	33.00	1996-2007
2	Bahawal pur	29.03	2004-2009	34	Miani Forest	25.48	1996-2007
3	Bannu	33.00	1996-2007	35	Multan	30.20	1986/1996-2009
4	Besham Qila	34.93	1996-2007	36	Munda Dam	25.56	2000-2007
5	Chillya	24.83	1996-2007	37	Murree	33.92	1997/1996-2009
6	Chitral	35.85	1986/1996-2009	38	Nabisir	25.52	1996-2007
7	Daggar	34.50	1996-2007	39	Naran	34.90	1996-2007
8	Dainyor	35.92	1996-2007	40	Nokkundi	28.82	1986/1996-2009
9	Dalbandin	28.88	1997/1996-2009	41	Oghi	34.50	1996-2007
10	Domel	34.38	2005-2007	42	Panjgur	26.97	1997/1996-2009
11	Doyian	35.55	1996-2007	43	Parachinar	33.87	1986/1996-2009
12	Faisalabad	31.41	2004-2009	44	Pasni	25.25	2004-2009
13	Fort Lock	33.55	1996-2007	45	Peshawar	34.02	1986/1996-2009
14	Gilgit	35.92	1986/1996-2005	46	Phulra	34.33	1996-2007
15	Gujar Khan	33.25	1996-2007	47	Plandri	33.72	1996-2007
16	Gungi	34.34	2002-2007	48	Quetta	30.25	1986/1996-2005
17	Hub Dam	25.25	1996-2007	49	Rehman Bridge	33.48	1996-2007
18	Hyderabad	25.38	1986/1996-2005	50	Rohri	27.70	1997/1996-2009
19	Islamabad	33.62	1986/1996-2009	51	Sakrand	26.12	1996-2007
20	Jacobabad	28.28	2004-2009	52	Sargodha	32.08	2004-2009
21	Kachura	35.45	1996-2007	53	Sehwan	26.42	1996-2007
22	Kakul	34.18	1986/1996-2009	54	Shinkiari	34.47	1996-2007
23	Kala Bagh	32.95	1996-2007	55	Sialkot	32.52	2004-2009
24	Kalam	35.53	1996-2007	56	Sibbi	29.55	1986/1996-2005
25	Kallar	33.42	1996-2007	57	Skardu	35.30	1986/1996-2005
26	Kandia	33.87	2005-2007	58	Sukkur	27.69	2005-2009
27	Karachi	24.90	1986/1996-2009	59	Tank	32.22	2003-2006
28	Karim Abad	36.30	1996-2007	60	Tarbela	34.07	2002-2007
29	Khuzdar	27.83	1997/1996-2009	61	Thana Bula	25.37	1996-2007
30	Lahore	31.55	1986/1996-2008	62	Yugo	35.18	1996-2007
31	Mangla	33.13	1996-2007	63	Zhob	31.35	1986/1996-2009
32	Mardan	34.20	1996-2007	64	Zulam Br	26.97	2000-2006

Note: [1] 1986/1996-2009: low temperature data from 1986 to 2009 and high temperature data from 1996 to 2009.

5.2.3.6. Temperature zoning and PG system

Based on the information collected from 64 weather stations, 7-days average high and low pavement temperatures were estimated for 50% and 98% reliability levels using the SHRP and the LTPP models (Equations 7 to 10). Table 5-11 presents the PG grading for each weather station for SHRP and LTPP models for 50% and 98% reliability levels.

Table 5-11 High and low air and pavement temperatures with PG grading

No.	Location	High Air Temp. °C		Low Air Temp. °C		SHRP Models		LTPP Models		PG Grade @ 50% Reliability		PG Grade @ 98% Reliability	
		Avg.	Std. Dev	Avg.	Std. Dev	High Pavement Temp. °C	Low Pavement Temp. °C	High Pavement Temp. °C	Low Pavement Temp. °C	SHRP Model	LTPP Model	SHRP Model	LTPP Model
1	Bagh	35.0	3.1	-1.8	3.6	63.1	-9.2	61.6	-5.5	PG 58-10	PG 54-10	PG 64-10	PG 64-10
2	Bahawalpur	43.2	1.4	3.5	1.9	68.1	-0.4	67.4	1.2	PG 70-10	PG 64-10	PG 70-10	PG 70-10
3	Bannu	43.1	1.0	-1.7	2.0	66.6	-5.9	66.6	-3.7	PG 70-10	PG 64-10	PG 70-10	PG 70-10
4	Besham Qila	42.5	1.0	2.3	1.5	65.7	-0.7	65.7	-0.8	PG 64-10	PG 64-10	PG 70-10	PG 70-10
5	Chillya	39.7	1.5	8.1	2.9	65.5	2.2	65.3	4.5	PG 64-10	PG 64-10	PG 70-10	PG 70-10
6	Chitral	39.2	1.6	-5.4	2.0	63.6	-9.5	63.3	-7.0	PG 64-10	PG 58-10	PG 64-10	PG 64-10
7	Daggar	42.2	1.4	-3.8	1.7	66.5	-7.2	65.8	-5.3	PG 64-10	PG 64-10	PG 70-10	PG 70-10
8	Dainyor	41.1	1.6	-5.6	0.8	65.4	-7.2	64.8	-6.5	PG 64-10	PG 64-10	PG 70-10	PG 70-10
9	Dalbandin	46.2	1.0	-5.1	2.4	70.4	-10.1	69.7	-5.4	PG 70-10	PG 64-10	PG 76-10	PG 70-10
10	Domel	39.1	2.8	0.0	0.6	66.3	-1.2	64.5	-2.0	PG 64-10	PG 58-10	PG 70-10	PG 70-10
11	Doyian	40.1	1.4	-1.4	1.1	64.2	-3.6	64.0	-3.5	PG 64-10	PG 58-10	PG 70-10	PG 64-10
12	Faisalabad	41.5	2.3	2.8	1.3	68.2	0.2	66.5	0.5	PG 64-10	PG 64-10	PG 70-10	PG 70-10
13	Fort Lock	29.4	1.7	-4.8	1.5	54.8	-8.0	56.1	-5.7	PG 52-10	PG 52-10	PG 58-10	PG 58-10
14	Gilgit	41.2	1.5	-7.7	1.2	65.4	-10.2	64.8	-8.2	PG 64-10	PG 64-10	PG 70-10	PG 70-10
15	Gujar Khan	41.5	2.1	-1.8	1.2	67.3	-4.3	65.9	-3.2	PG 64-10	PG 64-10	PG 70-10	PG 70-10
16	Gungi	35.3	1.6	-3.3	2.3	60.1	-7.9	60.5	-5.3	PG 58-10	PG 58-10	PG 64-10	PG 64-10
17	HubDam	40.5	2.0	8.7	1.6	67.2	5.5	66.2	6.0	PG 64-10	PG 64-10	PG 70-10	PG 70-10
18	Hyderabad	41.4	1.2	6.5	1.4	66.5	3.5	66.5	4.5	PG 64-10	PG 64-10	PG 70-10	PG 70-10
19	Islamabad	41.0	2.5	-0.2	1.0	67.7	-2.2	65.8	-2.0	PG 64-10	PG 64-10	PG 70-10	PG 70-10
20	Jacobabad	46.2	2.3	5.3	3.2	72.8	-1.3	70.4	1.4	PG 70-10	PG 64-10	PG 76-10	PG 76-10
21	Kachur	35.4	1.9	-10.2	2.1	60.7	-14.5	60.7	-10.5	PG 58-10	PG 58-10	PG 70-16	PG 64-10
22	Kakul	35.4	2.5	-2.8	2.0	62.2	-6.9	61.3	-4.7	PG 58-10	PG 58-10	PG 64-10	PG 64-10
23	Kalabagh	44.7	1.0	0.0	0.9	68.4	-1.8	67.9	-1.7	PG 70-10	PG 64-10	PG 70-10	PG 70-10
24	Kalam	30.4	2.3	-11.7	1.8	56.8	-15.3	57.0	-11.3	PG 54-16	PG 54-10	PG 58-16	PG 58-16
25	Kallar	40.9	1.0	2.9	1.2	64.4	0.4	64.7	0.2	PG 64-10	PG 58-10	PG 70-10	PG 64-10
26	Kandia	39.1	3.2	-3.2	3.1	67.2	-9.6	64.9	-6.0	PG 64-10	PG 58-10	PG 70-10	PG 70-10
27	Karachi	37.2	1.2	5.5	3.1	62.5	-0.9	63.2	2.3	PG 64-10	PG 58-10	PG 64-10	PG 64-10
28	Karimabad	34.7	1.7	-6.0	1.5	59.5	-9.2	59.8	-7.3	PG 58-10	PG 58-10	PG 64-10	PG 64-10
29	Khuzdar	40.3	1.2	-1.0	2.4	65.0	-5.8	65.2	-2.1	PG 64-10	PG 64-10	PG 70-10	PG 70-10
30	Lahore	41.4	2.0	3.2	2.4	67.3	-1.8	66.1	-0.1	PG 64-10	PG 64-10	PG 70-10	PG 70-10
31	Mangla	42.5	1.3	1.7	0.9	66.8	-0.2	66.2	-0.5	PG 64-10	PG 64-10	PG 70-10	PG 70-10
32	Mardan	42.9	0.7	-2.2	1.0	65.7	-4.3	66.1	-3.6	PG 70-10	PG 64-10	PG 70-10	PG 70-10
33	Massan	45.0	1.6	2.1	1.6	69.8	-1.2	68.3	-0.6	PG 70-10	PG 64-10	PG 70-10	PG 70-10
34	Miani Forest	44.1	3.4	5.5	3.9	73.5	-2.5	70.3	1.4	PG 70-10	PG 64-10	PG 76-10	PG 76-10
35	Multan	43.8	1.5	2.0	1.1	68.7	-0.3	67.8	0.4	PG 70-10	PG 64-10	PG 70-10	PG 70-10
36	Munda Dam	42.4	1.1	4.2	0.7	67.2	2.8	67.2	3.2	PG 70-10	PG 64-10	PG 70-10	PG 70-10
37	Murree	27.9	2.2	-6.8	3.0	54.4	-12.9	55.2	-8.5	PG 52-10	PG 52-10	PG 58-16	PG 58-10
38	Nabi Sar	43.4	1.7	4.4	1.1	69.3	2.2	68.2	3.2	PG 70-10	PG 64-10	PG 70-10	PG 70-10
39	Naran	28.2	3.4	-14.6	1.5	57.1	-17.7	56.5	-13.0	PG 52-16	PG 52-10	PG 58-22	PG 58-16
40	Nokkundi	46.0	1.1	-3.1	1.6	70.4	-6.3	69.5	-3.3	PG 70-10	PG 64-10	PG 76-10	PG 70-10
41	Oghi	36.3	2.5	-1.5	1.1	63.0	-3.7	62.0	-3.2	PG 58-10	PG 58-10	PG 64-10	PG 64-10
42	Panjgur	42.3	1.0	-2.2	2.1	66.7	-6.5	66.8	-2.6	PG 70-10	PG 64-10	PG 70-10	PG 70-10
43	Parahinar	33.8	1.5	-9.7	3.3	58.6	-16.5	59.4	-10.9	PG 58-10	PG 58-10	PG 58-16	PG 64-16
44	Pasni	37.1	1.8	9.0	2.7	63.6	3.4	63.5	5.2	PG 64-10	PG 58-10	PG 64-10	PG 64-10
45	Peshawar	42.3	2.1	1.0	1.4	68.0	-1.9	66.4	-1.5	PG 64-10	PG 64-10	PG 70-10	PG 70-10
46	Phulra	41.2	1.4	-3.4	1.0	65.5	-5.5	65.0	-4.6	PG 64-10	PG 64-10	PG 70-10	PG 70-10
47	Plandri	35.4	1.2	-0.8	0.8	59.6	-2.3	60.5	-2.4	PG 58-10	PG 58-10	PG 64-10	PG 64-10
48	Quetta	39.5	1.2	-9.3	1.8	64.1	-12.9	64.3	-8.2	PG 64-10	PG 58-10	PG 70-16	PG 70-10
49	Rehmab Br.	41.0	1.2	1.3	1.2	64.9	-1.1	64.9	-1.1	PG 64-10	PG 58-10	PG 70-10	PG 70-10
50	Rohri	45.4	1.5	4.1	1.4	70.5	1.2	69.3	2.3	PG 70-10	PG 64-10	PG 76-10	PG 70-10
51	Sakrand	44.4	3.0	7.0	3.1	73.0	0.7	70.1	3.2	PG 70-10	PG 64-10	PG 76-10	PG 76-10
52	Sargodha	42.5	3.1	4.0	3.5	70.5	-3.1	67.8	-0.7	PG 70-10	PG 64-10	PG 76-10	PG 70-10
53	Sehwan	48.2	1.0	5.8	1.4	72.3	2.9	71.5	3.8	PG 76-10	PG 70-10	PG 76-10	PG 76-10
54	Shinkiari	37.9	1.1	-1.0	0.7	61.6	-2.5	62.3	-2.7	PG 64-10	PG 58-10	PG 64-10	PG 64-10
55	Sialkot	41.4	2.6	1.8	1.0	68.3	-0.2	66.3	-0.3	PG 64-10	PG 64-10	PG 70-10	PG 70-10
56	Sibbi	47.6	1.9	2.3	1.3	73.3	-0.4	71.1	0.6	PG 70-10	PG 70-10	PG 76-10	PG 76-10
57	Skardu	36.5	2.5	-14.3	4.5	63.0	-23.5	62.0	-16.0	PG 58-16	PG 58-10	PG 64-28	PG 64-16
58	Sukkar	41.6	2.6	2.0	2.6	69.3	-3.4	67.3	-0.3	PG 64-10	PG 64-10	PG 70-10	PG 70-10
59	Tank	47.0	2.7	1.7	2.9	73.9	-4.3	70.9	-1.8	PG 70-10	PG 64-10	PG 76-10	PG 76-10
60	Tarbela	44.3	1.3	3.2	1.3	68.1	0.6	67.4	0.1	PG 70-10	PG 64-10	PG 70-10	PG 70-10
61	Thana Bulla	44.5	0.9	3.5	1.7	68.8	-0.1	68.7	2.1	PG 70-10	PG 64-10	PG 70-10	PG 70-10
62	Yugo	33.4	1.8	-9.9	1.2	58.6	-12.3	59.1	-9.6	PG 58-10	PG 58-10	PG 64-16	PG 64-10
63	Zhob	39.6	1.0	-1.0	2.4	63.7	-5.8	64.1	-3.0	PG 64-10	PG 58-10	PG 64-10	PG 70-10
64	Zulam Br.	39.6	1.7	-1.4	3.0	65.7	-7.6	65.1	-2.9	PG 64-10	PG 64-10	PG 70-10	PG 70-10

Table 5-12 Comparison of SHRP and LTPP pavement surface temperature models

Statistical Parameter	SHRP Models		LTPP Models	
	High Temperature	Low Temperature	High Temperature	Low Temperature
Minimum Temp. deg. C	54.4	-23.5	55.2	-16.0
Maximum Temp. deg. C	73.9	5.5	71.5	6.0
Average, deg. C	65.7	-4.5	65.1	-2.5
Standard Deviation, deg. C	4.5	5.8	3.7	4.6
Coefficient of Variation (%)	6.8	-128.4	5.8	-181.0

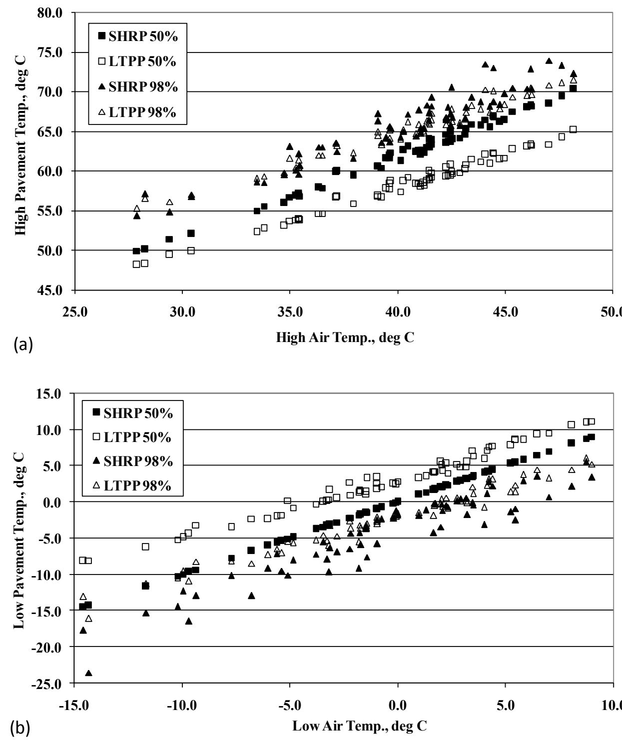


Figure 5-16 High and low pavement temperatures as a function of air temperature

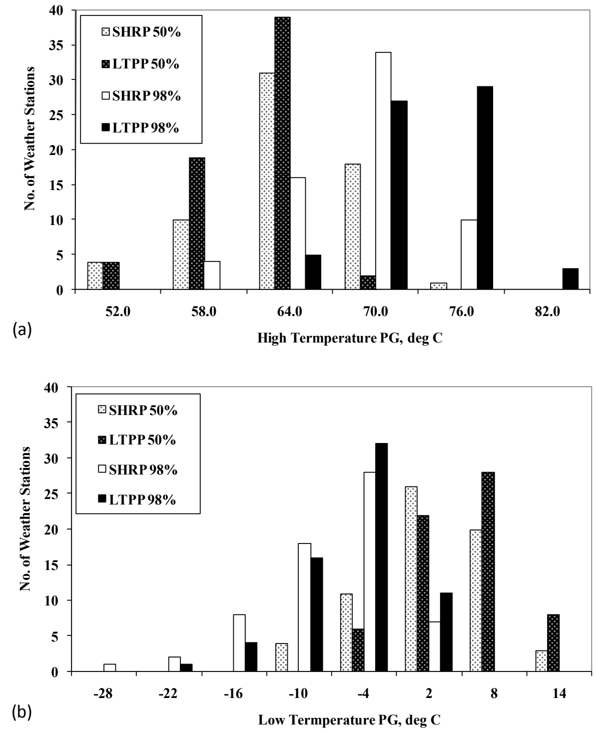


Figure 5-17 Histogram of high and low pavement temperatures for 64 stations

Table 5-13 Paired *t*-test comparison of SHRP and LTPP models

Pavement Temp. Condition	Reliability Level (%)	Average Pavement Temperature, deg. C		t Stat	t Critical two-tail	P(T<=t) two-tail
		SHRP	LTPP			
High	50	61.90	58.20	37.55	2.00	0.00
	98	65.55	69.72	35.46	2.00	0.00
Low	50	-0.76	2.64	23.81	2.00	0.00
	98	-4.60	-2.60	8.99	2.00	0.00

As mentioned earlier, significant differences in the prediction of pavement temperatures were observed, with SHRP models being more severe compared to LTPP models. When considering the conditions in Pakistan, most of Pakistan experiences moderate to high temperatures. The most common pavement distress is rutting, which is mainly due to high temperatures coupled with uncontrolled traffic loads. Low temperature cracking is not a common flexible pavement distress because temperatures below -10 ^{o}C are observed only on a limited scale. Because of the high temperatures and uncontrolled traffic loading, it was decided to use the SHRP models to generate temperature zoning. In general, the SHRP models require higher Performance Grade (PG) asphalt binder and thus provide extra protection against high temperatures. Since most the national highways experience uncontrolled axle loading, it is important that the final required PGs are based on 98% level of reliability.

Figure 5-18 shows the PG temperature zoning across Pakistan based on SHRP models for 98% level of reliability. It can be observed that most of Pakistan falls in PG 70-10 category with PG 64-10 in the second place. PG 58-22 is only required for limited areas in parts of northern Pakistan.

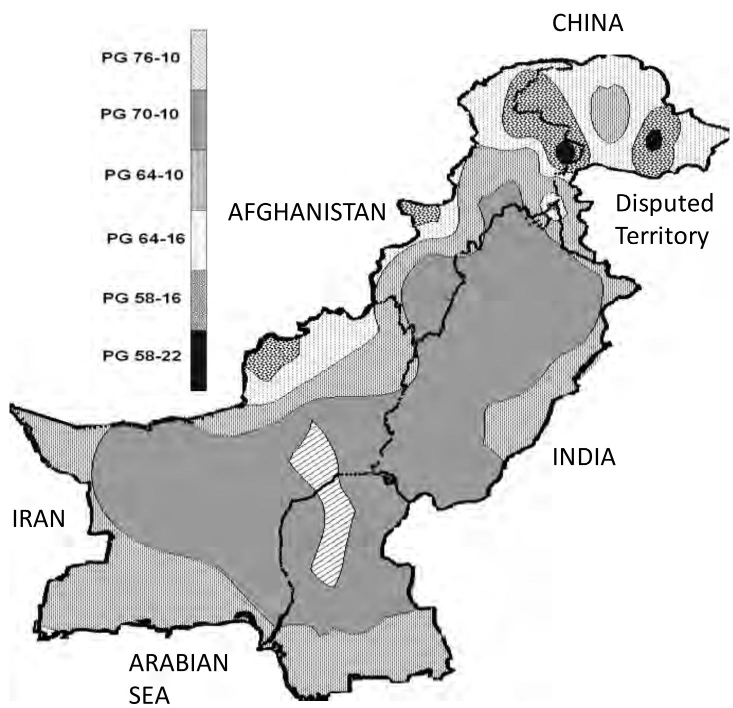


Figure 5-18 Temperature zoning for PG system in Pakistan

The Attock and Karachi are the two major refineries in Pakistan, where each refinery produces three asphalt binder grades based on the penetration grading system. As a part of Pak-US Cooperative Research Program (10), available asphalt binders were graded based on the penetration, viscosity, and PG system and are summarized in Table 5-14.

As shown in Figure 5-18, the Pakistan is divided among six PGs depending upon the temperature zoning. A summary of these grades is provided in Table 5-15 as required PG grade (column 1). The second column lists the availability of the grades, whereas the third column lists the combination of grades that fulfill the requirements specified in column 1. That is PG 64-16, satisfies the requirements of PG 64-10 according to AASHTO MP1. Under the remarks column, all binders are considered acceptable except for PG 70-10. That is PG 76-16 satisfies the requirement of PG 70-16, but may result in harder binder which may not be desirable and prone to cracking. In addition, PG 76-16 (A-PMB) is polymer modified asphalt, which is relatively more expensive compared to neat (unmodified) asphalt.

Table 5-14 A summary of asphalt binder viscosity grading in Pakistan

Sr. No.	Refinery	Binder	Penetration Grade	Viscosity Grade	PG Grades of Available Asphalts in Pakistan
1	Attock	A-PMB	Not Available	AC-40	PG 76-16
2	Attock	A-60/70	Pen 60/70	AC-20	PG 58-22
3	Attock	A-80/100	Pen 60/70	AC-10	PG 58-22
4	Karachi	K-40/50	Pen 40/50	AC-40	PG 64-16
5	Karachi	K-60/70	Pen 60/70	AC-20	PG 64-22
6	Karachi	K-80/100	Pen 85/100	AC-10	PG 58-22

Note: “A” for Attock Refinery and “K” for Karachi Refinery

PG 70-10 which is the requirement for more than 70 percent of Pakistan is not produced by either of the two refineries. This is considered to be one of the most important requirements.

Table 5-15 A summary of asphalt binder penetration grading in Pakistan

Required PG Grade	Available Exact Grade	Possible Grade Availability	Equivalent Penetration Grade	Remarks
PG 70-10	No	PG 76-16	A–PMB	PMB, Polymer Modified may be too hard for PG 70-10 and may result in cracking
PG 64-10	No	PG 64-16	K–40/50	Acceptable
PG 64-16	Yes	PG 64-16 PG 64-22	K–40/50 K–60/70	Acceptable
PG 58-16	No	PG 58-22	K–80/100 A–80/100 A–60/70	Acceptable
PG 76-10	No	PG 76-16	A–PMB	Acceptable
PG 58-22	Yes	PG 58-22	K–80/100 A–80/100 A–60/70	Acceptable

5.2.3.7. Observations and conclusions

The main objective of the climatic data analysis is to divide Pakistan into temperature zones according to the PG requirements developed under SHRP. Several observations and conclusions are briefly presented below.

1. The two model forms, SHRP and LTPP for the prediction of pavement temperatures resulted into significantly different predictions both for high and low pavement temperatures. However, relatively greater differences were observed for low temperature compared to high temperature predictions.
2. SHRP prediction models for high and low temperatures were selected for the development of PG zones for Pakistan, since SHRP requires higher grade requirement for the high temperature grade, which is critical for uncontrolled axle loading.
3. The use of 98% level of reliability provides additional margin of safety against high traffic levels and uncontrolled loadings. No additional bumping of PG is

needed, as is recommended by AASHTO MP1 specifications, since it will result in excessively stiff binder.

4. Pakistan is divided among six temperature zones requiring PG 70-10 as the most common binder that covers more than 70 percent of the land area.
5. PG 70-10 is not produced by either of the two refineries in Pakistan. The closest grade that fulfills the requirements of PG 70-10 is PG 76-16 (A-PMB). A-PMB is relatively harder and may be prone to cracking. Use of PG 64-16 is relatively softer than the required grade and may be prone to rutting.
6. At present, commonly used asphalt binder grade in Pakistan is A-60/70 and K-60/70. The corresponding PGs are PG 58-22 and PG 64-22 which is likely to experience rutting in areas requiring PG 70-10. This may be one of the major reasons of premature failures, especially rutting, in most of Pakistan's pavements.

5.2.3.8. Summary

The current asphalt binder specifications in Pakistan are based on the penetration grade: the test is performed at 25 ^{o}C. Penetration is an empirical measure of the consistency that is used as an indicator of the rutting and fatigue susceptibility of asphalt binders, and is not related to pavement performance. The new asphalt mixture design methodology developed under the Strategic Highway Research Program (SHRP), called the Superpave, is a performance-based approach. The first step in the implementation of Superpave methodology is to establish high and low pavement temperatures for a location. The temperatures define the required asphalt binder Performance Grade (PG). This study documents the initial ground work towards implementation of Superpave mixture design for establishing high and low geographical temperature zones. The temperature zoning of Pakistan was carried out by using temperature data obtained from 64 weather stations. The SHRP and LTPP prediction models were utilized for predicting pavement temperatures. A significant difference was observed between the predicted pavement temperatures from the models. The SHRP model yields higher, high temperature PG grade providing additional protection against rutting. Since rutting is the most common distress on flexible pavements in Pakistan, the SHRP model, at 98% level of reliability, is recommended. PG 70-10 binder seems to be the most common grade that encompasses more than 70% of the lane area of Pakistan. However, neither of the two local refineries produce PG 70-10 binder. The polymer modified asphalt binder produced by Attock refinery (A-PMB) fulfills the requirements of the harder PG 76-16 while A-60/70 (PG 58-22) or K-60/70 (PG 64-22) produced at Attock and National refineries respectively are softer compared to the required PG 70-10. Harder asphalt binder is more prone to cracking, whereas softer binders are more prone to rutting. Consequently, the current asphalt mixtures, which utilize A-60/70 or K-60/70, may be prone to excessive rutting.

5.3 ASPHALT MIXTURES

The asphalt binders were characterized using conventional and Superpave tests and the results were presented in section 5.2. Asphalt binders play a significant role in the overall rheological behavior of HMA mixtures. However, when combined with aggregates, the characterization of asphalt binder in the mixture (i.e., HMA) requires special evaluation. This evaluation is essential to assess the HMA's expected performance under ambient temperatures and axle loadings. Several HMA mixtures were designed in this study considering various aggregate gradations, binder, and compaction types. The following sections present the HMA mixture evaluations using conventional and Superpave tests. Later, the HMA performance evaluation is described to identify the best methodology which could improve the field pavement performance in Pakistan.

5.3.1. Conventional and Superpave HMA Characterization

As mentioned before, HMA is a mixture of mineral aggregates and asphalt binder. These two ingredients are combined according to certain mixture design to perform under prevailing climate and traffic load at a specific location. Asphalt binders control the rheological behavior of the HMA mixture; thus should be selected to meet the climatic conditions of a location. The details regarding the asphalt binder testing conducted in the study were mentioned in Chapter 4, while the results were discussed in section 5.2. In addition, aggregate testing details such as gradations, source, and consensus properties, were presented in Chapter 4. The conventional HMA testing results were also included in Chapter 4 for the HMA mixtures shown in Table 5-16.

Table 5-16 Coding schemes for mixtures

Mixture Type	Binder Code	Aggregate Gradation	Mix Code
Wearing Course	A6/7	NHA	A6/7NHAW
		SPF	A6/7SPFW
		SPC	A6/7SPCW
	K6/7	NHA	K6/7NHAW
		SPF	K6/7SPFW
	PA6/7(1.35)	SPF	PA6/7(135)SPFW
	PA6/7(1.70)	SPF	PA6/7(170)SPFW
Base Course	A6/7	NHA	A6/7NHAB
		SPF	A6/7SPFB
		SPC	A6/7SPCB

In this section, the state-of-the-art HMA testing results are presented and discussed. These performance tests include resilient modulus, fatigue, and permanent deformation characterization of HMA mixtures. It should be noted that the performance testing was only carried out for wearing course HMA mixtures because of the limitation of specimen size (i.e., only Marshall specimens could be used for smaller aggregate sizes and for the wearing course).

5.3.1.1. Resilient modulus testing

Resilient modulus testing was conducted on Marshall sized specimens. Typical diameter and thickness for the test specimens were 102 mm and 63 mm, respectively. Resilient modulus is one of the most important parameters estimated during the fatigue testing. Resilient modulus values were used as an indication of the damage with load repetitions. A typical plot of resilient modulus with load repetitions is shown in Figure 5-19. The figure shows a change in the resilient modulus value as a function of load repetitions. The reduction in the resilient modulus is the result of the damage induced because of the loading cycles. The initial resilient modulus represents a fully intact specimen with no damage and is taken as the average of the first 100 cycles. The cycle number at 50% resilient modulus reduction is one of the criteria used to define the fatigue failure.

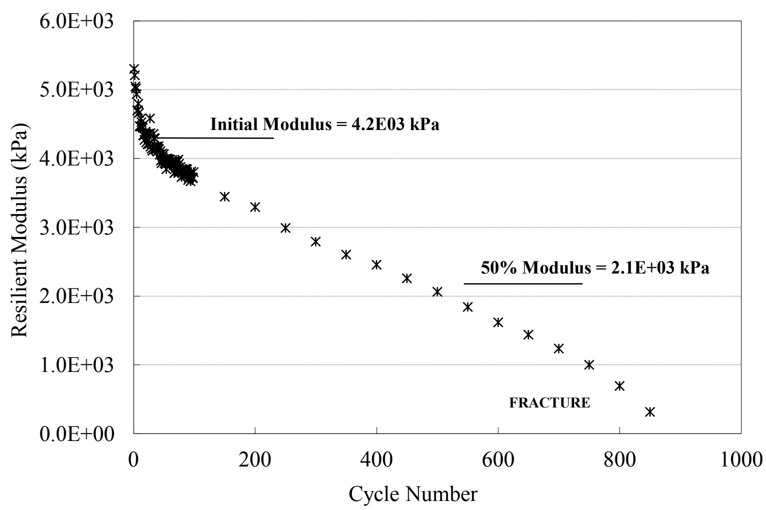


Figure 5-19 Estimation of resilient modulus

No vertical measurements were made during the resilient and fatigue testing because of potential damage to the vertical LVDTs close to the location of failure of the specimen. Estimation of the resilient modulus requires Poisson's ratio, which can only be obtained if both vertical and horizontal LVDT's are used. Since only horizontal measurements were made during the fatigue testing, resilient modulus was estimated by using a Poisson's ratio estimated by using Equation (4.13) in Chapter 4. A summary of results for the resilient modulus is tabulated in Table 5-17. The resilient modulus tests were conducted at a single temperature of 25 ^{o}C; therefore, the resilient modulus only shows the stiffness of the material at the tested temperature. Consequently, the resilient modulus value cannot be used to compare the expected performance of the material in the field as pavement surface temperatures vary during a day and among different seasons. Therefore, fatigue testing of the HMA mixtures was conducted in this study to compare their expected performance.

5.3.1.2. Fatigue analysis

The fatigue resistance of an asphalt mixture is its ability to withstand repeated loading without fracture. Fatigue cracking is a common form of distress in asphalt concrete (flexible) pavements due to repeated traffic loading. For pavement design, two factors are considered to control cracking within the pavement system. The first arethe material properties that are related to the phenomena that cause cracking in the pavement system. The second important factor is the establishment of the failure limits. That is, how many repetitions are needed for a specific stress or stain condition to cause failure in the pavement system?

The stiffness of the HMA mixture is one of the most important material properties that is used by the researcher to establish fatigue criteria. Horizontal tensile strain under the HMA layer(s) due to axle loading is generally used to establish failure limits. The material property and the failure limits within a pavement system are dependent upon the climatic conditions and the rate of loading, because asphalt concrete (i.e., HMA) is a thermoplastic/viscoelastic material. HMA at high temperatures has stiffness close to an unbound material, whereas; at cold temperatures it is close to Portland cement concrete (PCC). Similarly, higher rate of loading (frequency) results is higher modulus values compared to lower rates of loading.

Fatigue cracking distress is defined as a series of interconnecting cracks caused by the fatigue failure of HMA layer under repeated traffic loading in the wheel-path. Generally, the cracking initiates at the bottom of the asphalt layer where tensile stress or strain is highest under the wheel load. The cracks propagate to the pavement surface initially as one or more longitudinal parallel cracks. However, after repeated traffic loading, the cracks connect and form many sided, sharp-angled pieces that develop a pattern resembling chicken wire or the skin of an alligator. Fatigue cracking occurs only in areas that are subjected to repeated traffic loading. It would not occur over an entire area unless the entire area was subjected to traffic loading.

Researchers and engineers have used different geometrically shaped laboratory specimens and loading conditions for characterizing the fatigue behavior of HMA mixtures. Among these, the most common are the simple beam with third point or center point loading, cantilever beam with rotating bending and indirect tensile test. In this study, indirect tensile testing mode is evaluated for characterizing the fatigue behavior of HMA mixtures. The advantage of the indirect tensile mode is that it is one of the most economical and practical modes. The test can be conducted on cores obtained from the field or prepared in the laboratory.

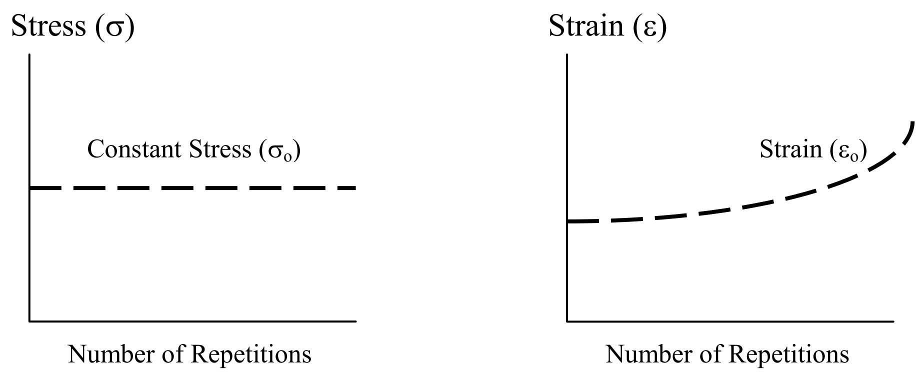
As with the other modes of testing, indirect testing mode can be used either by controlling the stress (stress-control) or controlling the strain (stain-control). These two modes of testing are schematically shown in Figure 5-20. In constant stress test (Figure 5-20a), the stress remains constant (applied load is constant) but the strain increases with the number of load repetitions. The increase in strain with load repetitions is due to the damage accumulation with repetitive loading within the specimen. Higher stress level will result in lower number of load repetitions before the failure strain limit is reached

Table 5-17 A summary of resilient modulus results

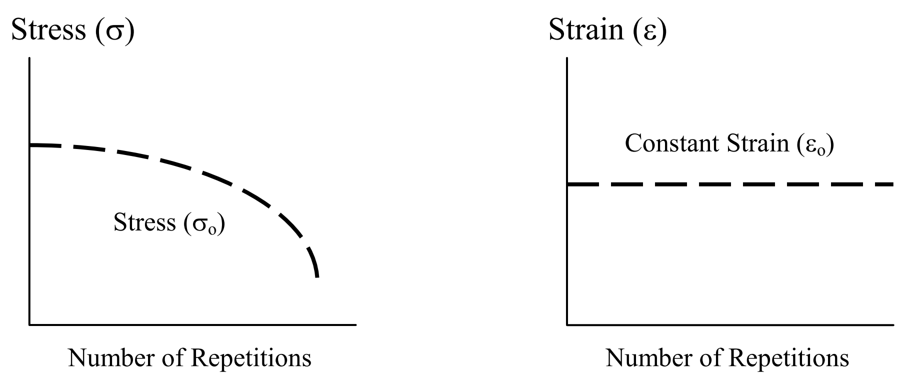
Mix Code	Stress Level		Modulus (N/mm^2)			Average Mr (psi)	Standard Deviation (psi)	CV (%)
	N/mm^2	kPa	N/mm^2	kPa	psi			
A6/7SPFW	0.425	425.4	4980	4.98E+06	722,145	728,738	57,563	7.9%
	0.405	405.0	5007	5.01E+06	725,971			
	0.376	375.9	5612	5.61E+06	813,712			
	0.326	325.6	4492	4.49E+06	651,412			
	0.348	348.3	5038	5.04E+06	730,451			
K6/7SPFW	0.552	552.5	6212	6.21E+06	900,687	847,006	87,391	10.3%
	0.402	401.9	4953	4.95E+06	718,126			
	0.376	375.7	6045	6.05E+06	876,582			
	0.351	350.8	5210	5.21E+06	755,384			
	0.450	450.5	6302	6.30E+06	913,810			
	0.500	499.9	6327	6.33E+06	917,447			
A6/7NHAW	0.451	450.9	4450	4.45E+06	645,214	830,926	192,962	23.2%
	0.401	401.1	4844	4.84E+06	702,365			
	0.376	376.5	7333	7.33E+06	1,063,239			
	0.427	426.8	6296	6.30E+06	912,885			
K6/7NHAW	0.301	300.9	7231	7.23E+06	1,048,468	850,125	151,755	17.9%
	0.351	350.7	6234	6.23E+06	903,895			
	0.376	376.1	5900	5.90E+06	855,517			
	0.401	400.8	4345	4.34E+06	629,986			
	0.451	450.6	5605	5.61E+06	812,758			
A6/7(135)SPFW	0.401	401.2	5792	5.79E+06	839,800	816,952	76,780	9.4%
	0.452	451.9	5910	5.91E+06	856,938			
	0.501	501.3	6639	6.64E+06	962,611			
	0.526	526.5	4986	4.99E+06	722,920			
	0.551	551.1	5288	5.29E+06	766,723			
	0.399	399.0	5883	5.88E+06	853,016			
	0.499	498.6	5363	5.36E+06	777,607			
	0.597	596.8	5214	5.21E+06	755,998			
A6/7(170)SPFW	0.551	551.4	3286	3.29E+06	476,498	534,041	108,540	20.3%
	0.451	451.4	2738	2.74E+06	396,984			
	0.451	450.9	3504	3.50E+06	508,048			
	0.401	401.0	4421	4.42E+06	641,041			
	0.351	350.7	4466	4.47E+06	647,632			
A6/7(200)SPFW	0.476	476.1	3318	3.32E+06	481,045	602,012	139,124	23.1%
	0.501	501.0	3823	3.82E+06	554,360			
	0.427	426.7	3306	3.31E+06	479,309			
	0.401	400.9	5425	5.42E+06	786,580			
	0.451	451.0	4888	4.89E+06	708,767			

and vice versa. Varying the stress level will result in different number of load repetitions to failure for specific failure criteria.

On the other hand, in the constant strain test, the strain is kept constant and the load or stress decreases with the number of load repetitions to keep the stain constant as the damage develops in the sample. Both of the testing modes have advantages and disadvantages. In the case of indirect tensile testing, simplicity of the constant stress overrides the other advantages provided by the constant strain testing. Controlling the stress is much easier and simpler compared to running the test in control strain mode. For the purpose of this study, testing was carried out in a constant stress mode.



(a) Constant stress test



(b) Constant strain test

Figure 5-20 Schematics of stress and strain control mode

Irrespective of the testing mode, constant stress or constant strain, the fatigue characteristics of asphalt mixtures are usually expressed as relationships between the initial stress or strain and the number of load repetitions to failure. Several relationship forms are available in the literature; however, the simplest one is of the following form:

$$N_f = a(\sigma)^b \text{ or } N_f = c(\varepsilon)^d \tag{5.11}$$

where: N_f = number of load applications to crack initiation,
ε, σ = tensile strain and stress, respectively, and
a, b, c, d = experimentally determined coefficients dependent on test temperature or rate of loading.

The above relationships can be graphically represented by Figure 5-21.

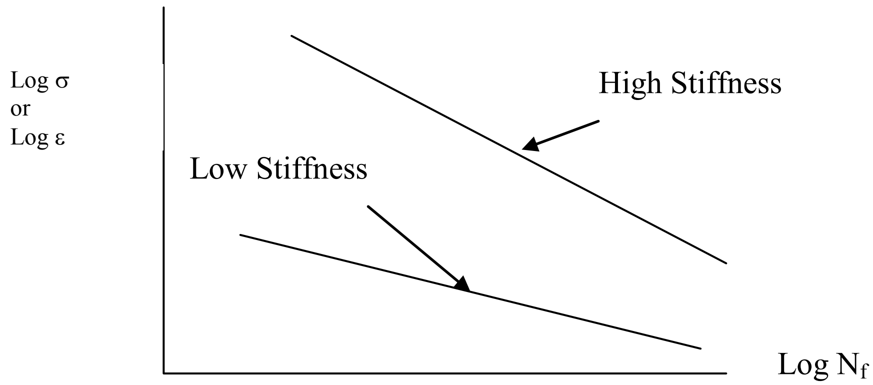


Figure 5-21 Relationship between stress/strain and number of repetitions to failure

Test Setup

In this study, fatigue testing was carried out in diametral mode and the setup is shown in Chapter 4. The load was applied on the specimen through the top-loading strip along the diametral axes of the specimen. Specimens having a diameter of 102 mm and thickness of 65 mm were used for fatigue characterization. A servo-hydraulic testing machine capable of applying a haversine waveform for fatigue testing was utilized. The frequency of the sinusoidal waveform was determined from the tensile strength results. Failure horizontal strain for the tensile strength test and the strain under the fatigue testing were measured using LVDTs (Linear Variable Differential Transformer) on both faces of the specimen. Testing was carried out until failure, as defined by 50% reduction in resilient modulus as is shown in Figure 5-19.

Range of stress levels were selected to obtain the fatigue relationship described by Equation 11. Normally, 5 to 8 test specimens are required to establish the fatigue relationship for a given temperature (25 ^{o}C) and the rate of loading (0.1 second and 0.4 rest period). All testing was carried out at target air voids of 4.0 percent. The air voids were all within the tolerable range of ±1.0% of the target value.

Fatigue Curves

Using the above mentioned failure criteria, fatigue curves were developed for initial stress and number of repetitions to failure. Figure 5-22 shows the typical fatigue curve and the parameters of the fatigue equation (Equation 5.11). The equation for constant stress is given below:

$$N_f = a(\sigma)^b \tag{5.12}$$

Where σ is the initial stress and “a” and “b” are the fatigue parameters. A tabular summary of these parameters is given in Table 5-18.

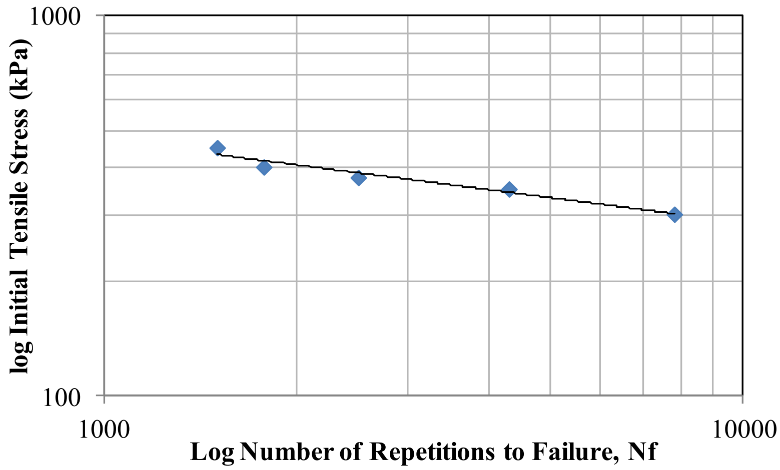


Figure 5-22 Typical Fatigue Curve using IDT Setup

Table 5-18 A summary of fatigue parameters

Mix Code	Avg. Mr (kPa)	Avg. Mr (psi)	Fatigue Parameters		
			a	b	R^2
A6/7SPFW	5.03E+06	728,738	6.70E+10	-2.739	0.90
K6/7SPFW	5.84E+06	847,006	2.45E+14	-4.114	0.87
A6/7NHAW	5.73E+06	830,926	4.65E+24	-8.168	0.99
K6/7NHAW	5.86E+06	850,125	5.27E+14	-4.376	0.95
A6/7(135)SPFW	5.63E+06	816,952	2.51E+26	-8.426	0.94
A6/7(170)SPFW	3.72E+06	540,097	6.20E+20	-6.578	0.91
A6/7(200)SPFW	4.15E+06	602,012	3.78E+15	-4.514	0.84

The R^2 values for all of the fatigue curves were close to 0.9, showing a linear relationship between the initial tensile stress and the number of load repetitions to failure. The parameters, “a” and “b” are the estimated fatigue parameters of Equation 5.12. The fatigue curves for the seven wearing course HMA mixtures are shown in Figure 5-23. From these results, the following conclusion can be made:

- The HMA mixtures with NHA gradation and 60/70 penetration binders (i.e., Attock and Karachi refineries) showed the lowest fatigue lives, especially at higher stress levels. However, the mixtures with Attock refinery binder exhibited better fatigue life at lower stress levels.
- The HMA mixtures with Superpave gradations and 60/70 penetration binders (i.e., Attock and Karachi refineries) showed better fatigue lives than those with NHA gradations, especially at higher stress levels. However, the mixtures with

Superpave gradations and conventional binders did not show significant improvement in fatigue lives at lower stress levels.

- The HMA mixtures with Superpave gradations and polymer modified binders consistently showed better fatigue lives than those with NHA gradations, especially at intermediate and high stress levels. However, the same mixtures did not show significant improvement in fatigue lives at low stress levels.

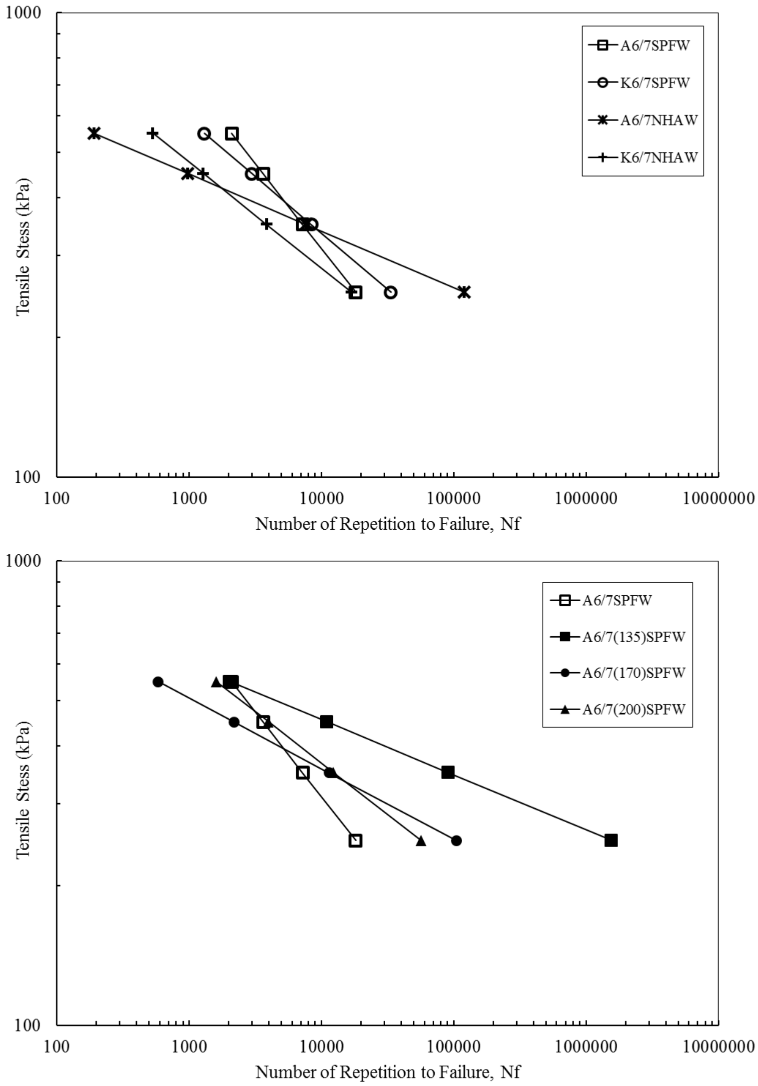


Figure 5-23 Fatigue curves for the selected mixtures

5.3.1.3. Permanent deformation analysis

As a standard practice, indirect mode of testing using IDT equipment was used for estimation of fatigue parameters. However, in this study, the horizontal deformation was used to estimate accumulated permanent horizontal strains. These permanent strain values were used to estimate the permanent deformation parameters and the flow number. A brief discussion of the analysis is given below.

Test Background

Repeated load permanent deformation tests were conducted using the IDT setup at multiple stress levels. All testing was carried out at 25 °C. Haversine pulse loads were applied at 2 load cycles per second with load time of 0.1 second and dwell time of 0.4 seconds. Each test was carried out until failure under each stress level.

Test Interpretation

Repeated load permanent deformation tests were used to characterize the rutting properties of all paving materials. Figure 5-24 illustrates a typical relationship between total cumulative strain, plastic strain, and load cycles.

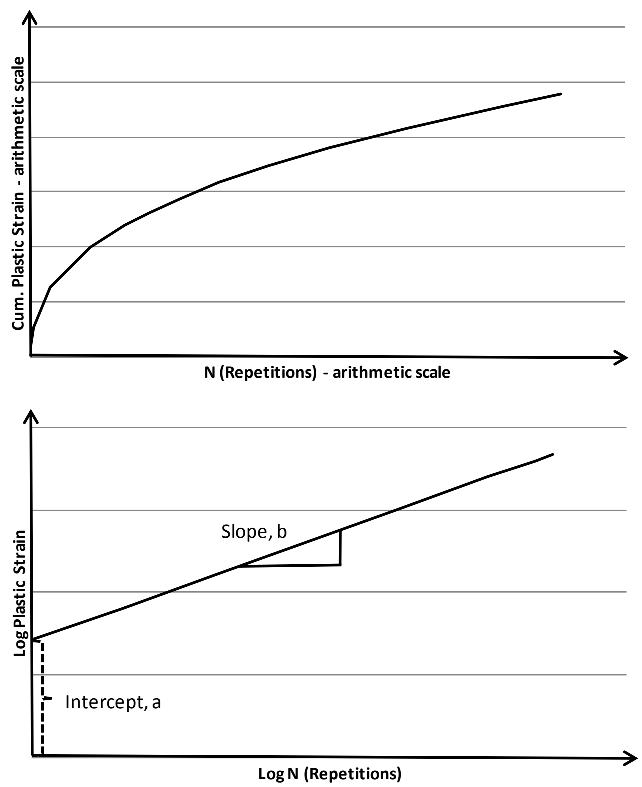


Figure 5-24 Typical repeated load permanent deformation response

When plotted on a log-log scale, the relationship is generally linear and can be expressed by:

$$\text{Plastic Strain} = \varepsilon_p = aN^b \tag{5.13}$$

With *a* and *b* being the regression constants dependent upon the material test conditions. An alternative form of the mathematical model used to characterize the plastic strains per load repetition (ε_{pn}) relationship is:

$$\frac{\partial \varepsilon_p}{\partial N} = \varepsilon_{pn} = \frac{\partial \left(aN^b \right)}{\partial N} \tag{5.14}$$

$$\varepsilon_{pn} = abN^{(b-1)} \tag{5.15}$$

Because the resilient strain (ε_r) may generally be assumed to be independent of the number of load repetitions (N). The ratio of the plastic and the resilient strain components of the material in question can be defined by:

$$\frac{\varepsilon_{pn}}{\varepsilon_r} = \left(\frac{ab}{\varepsilon_r} \right) N^{(b-1)} \tag{5.16}$$

The value on the left side of the equation is plotted against the load repetitions as shown in Figure 5-25. The number of load repetitions corresponding to the minimum value is termed the flow number. Table 5-19 in addition to the rutting parameters presents the flow numbers for each mix at the corresponding stress levels.

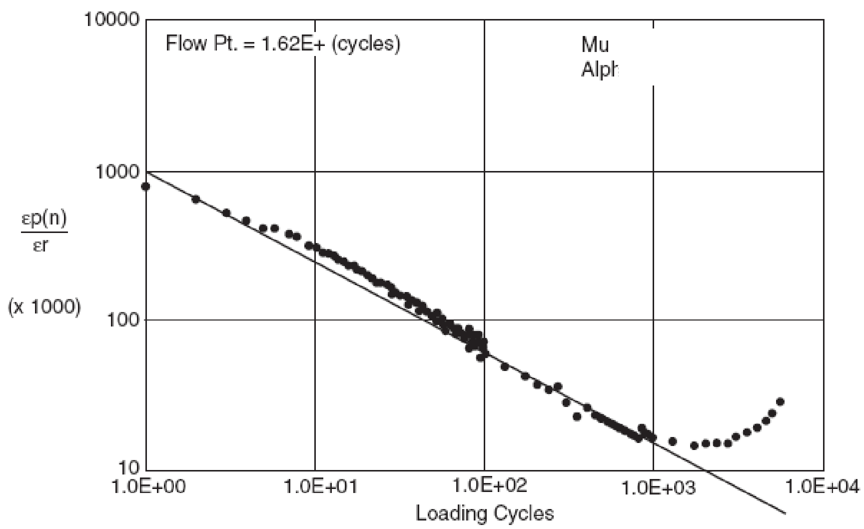


Figure 5-25 Modeling of flow number

Table 5-19 Rutting parameters and flow numbers using IDT testing

Mix Code	Stress Level		Modulus (N/mm^2)		a	b	Recoverable Strain (εr)	Flow Number (FN)
	N/mm^2	kPa	N/mm^2	kPa				
A6/7SPFW	0.326	325.6	4492	4.49E+06	5.27E-05	5.44E-01	1.20E-04	745
	0.348	348.3	5038	5.04E+06	4.68E-05	5.39E-01	1.15E-04	709
	0.376	375.9	5612	5.61E+06	2.47E-05	6.14E-01	1.09E-04	720
	0.405	405.0	5007	5.01E+06	3.07E-05	6.36E-01	1.35E-04	854
	0.425	425.4	4980	4.98E+06	3.23E-05	6.52E-01	1.44E-04	933
K6/7SPFW	0.552	552.5	6212	6.21E+06	1.12E-05	9.05E-01	1.47E-04	499
	0.402	401.9	4953	4.95E+06	1.50E-05	7.84E-01	1.34E-04	640
	0.376	375.7	6045	6.05E+06	9.71E-06	7.86E-01	1.03E-04	577
	0.450	450.5	6302	6.30E+06	1.40E-05	7.85E-01	1.18E-04	746
	0.500	499.9	6327	6.33E+06	1.46E-05	7.91E-01	1.29E-04	671
A6/7NHAW	0.451	450.9	4450	4.45E+06	4.65E-06	1.07E+00	1.69E-04	247
	0.401	401.1	4844	4.84E+06	8.36E-06	8.25E-01	1.69E-04	655
	0.376	376.5	7333	7.33E+06	1.97E-07	1.18E+00	1.69E-04	516
	0.427	426.8	6296	6.30E+06	6.17E-05	4.39E-01	1.69E-04	734
K6/7NHAW	0.301	300.9	7231	7.23E+06	1.31E-05	6.62E-01	6.82E-05	641
	0.351	350.7	6234	6.23E+06	1.14E-05	7.30E-01	9.35E-05	921
	0.376	376.1	5900	5.90E+06	2.12E-05	7.53E-01	1.06E-04	655
	0.401	400.8	4345	4.34E+06	2.34E-05	8.17E-01	1.53E-04	664
	0.451	450.6	5605	5.61E+06	4.28E-05	7.31E-01	1.33E-04	563
A6/7(135)SPFW	0.401	401.2	5792	5.79E+06	2.33E-05	4.24E-01	1.15E-04	1642
	0.452	451.9	5910	5.91E+06	3.32E-05	4.70E-01	1.27E-04	683
	0.501	501.3	6639	6.64E+06	1.51E-05	5.66E-01	1.22E-04	1144
	0.526	526.5	4986	4.99E+06	1.14E-05	7.34E-01	1.22E-04	662
	0.551	551.1	5288	5.29E+06	1.60E-05	7.08E-01	1.74E-04	646
	0.399	399.0	5883	5.88E+06	5.50E-05	3.22E-01	1.14E-04	741
	0.499	498.6	5363	5.36E+06	1.66E-05	6.33E-01	1.54E-04	310
	0.597	596.8	5214	5.21E+06	1.53E-05	6.68E-01	1.86E-04	302
A6/7(170)SPFW	0.551	551.4	3286	3.29E+06	1.59E-06	1.21E+00	2.78E-04	314
	0.451	451.4	2738	2.74E+06	3.95E-05	7.06E-01	2.79E-04	635
	0.451	450.9	3504	3.50E+06	3.42E-05	6.12E-01	2.15E-04	737
	0.401	401.0	4421	4.42E+06	3.19E-05	5.44E-01	2.26E-04	678
	0.351	350.7	4466	4.47E+06	8.74E-06	6.27E-01	1.29E-04	756
A6/7(200)SPFW	0.476	476.1	3318	3.32E+06	2.89E-05	6.50E-01	2.19E-04	744
	0.501	501.0	3823	3.82E+06	3.20E-05	5.91E-01	2.19E-04	707
	0.427	426.7	3306	3.31E+06	4.10E-05	4.39E-01	2.19E-04	904
	0.401	400.9	5425	5.42E+06	4.63E-06	6.84E-01	2.19E-04	587
	0.451	451.0	4888	4.89E+06	4.66E-05	4.43E-01	2.19E-04	1097

A comparison of permanent deformation behavior at different stress levels is provided in Figures 5-26 to 5-28. The plots were made for permanent deformation rather than strains as is given by Equation 5.10.

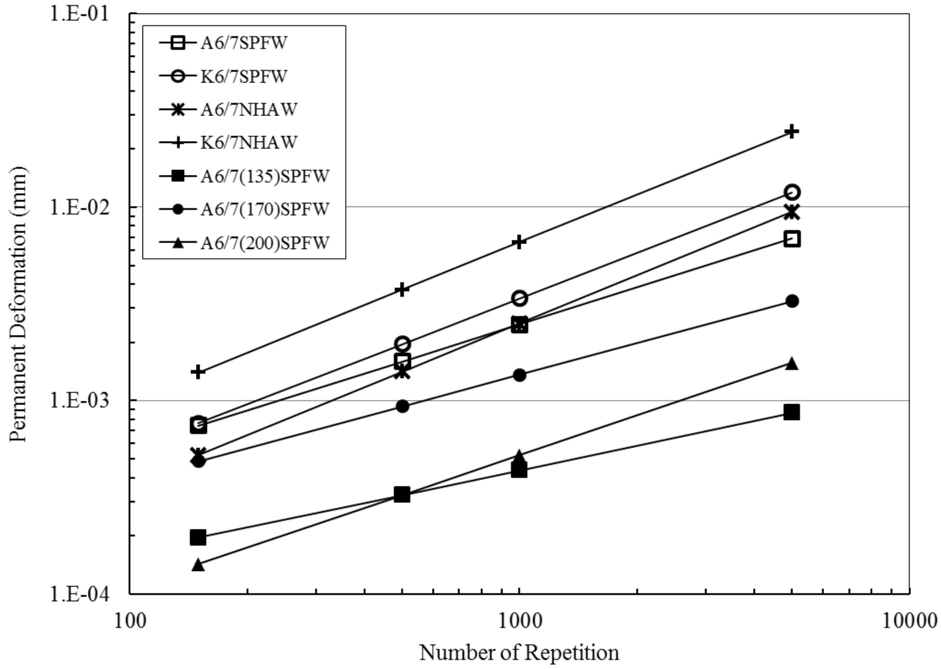


Figure 5-26 Comparison of permanent deformation behavior at stress level = 400 kPa

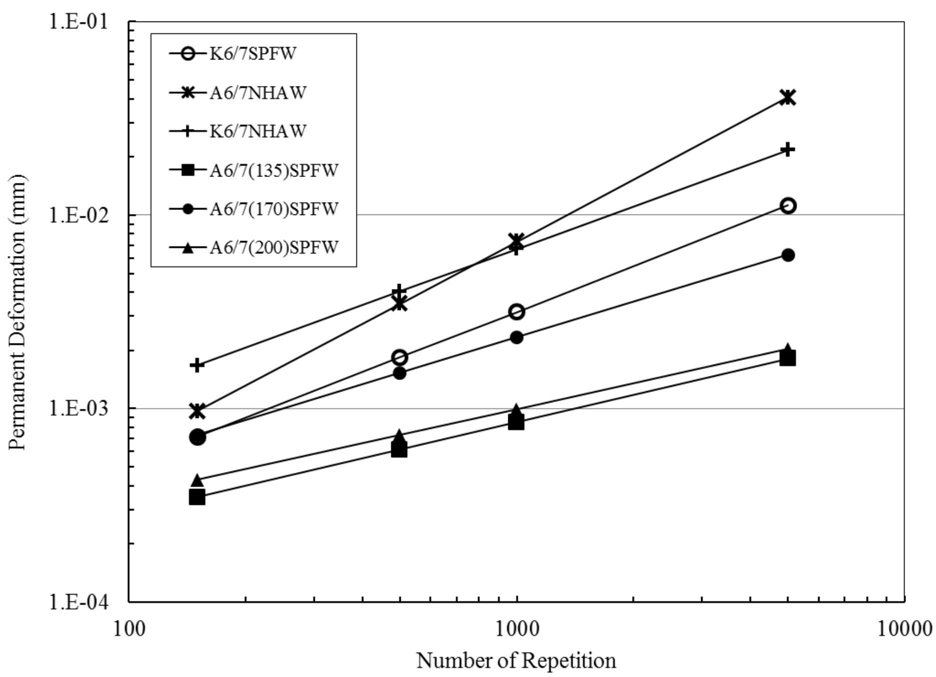


Figure 5-27 Comparison of permanent deformation behavior at stress level = 450 kPa

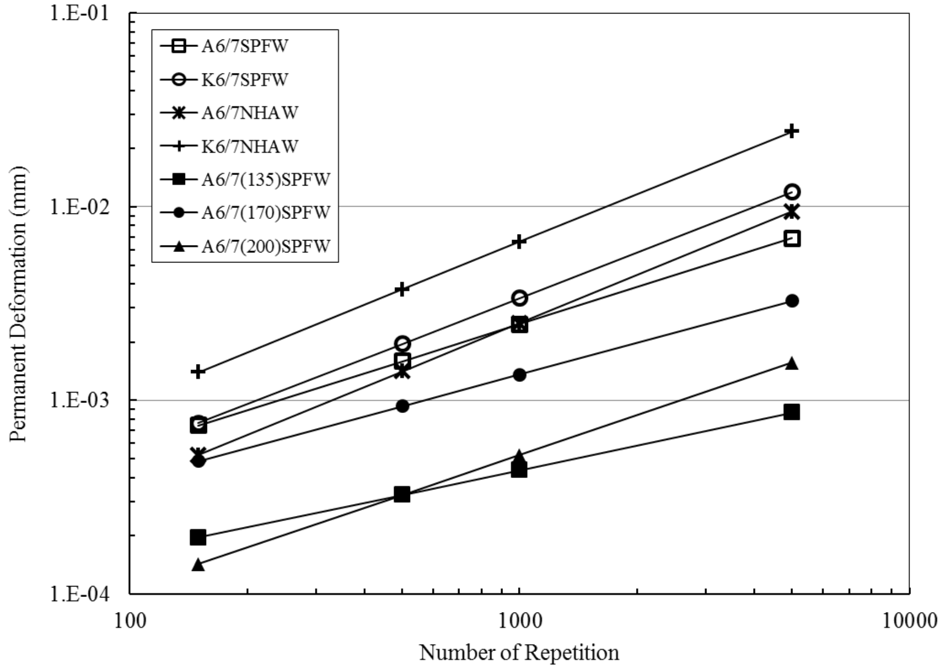


Figure 5-28 Comparison of permanent deformation behavior at stress level = 500 kPa

The following conclusions can be made from the results of permanent deformation curves for all seven HMA mixtures:

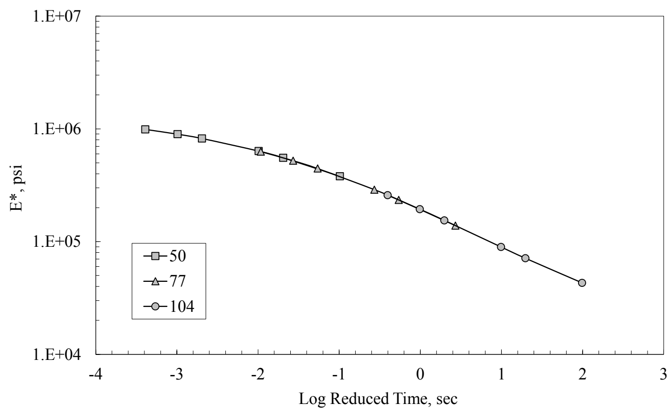
- The HMA mixtures with NHA gradation and 60/70 penetration binders (i.e., Attock and Karachi refineries) showed higher rutting susceptibility. The mixtures with asphalt binder from the Karachi refinery exhibited higher potential for rutting irrespective of the aggregate gradation.
- The HMA mixtures with Superpave gradations and polymer modified binders consistently showed better rutting performance than those with NHA gradations at all stress levels. The best rutting performance was observed for Superpave HMA mixtures with lower percentages of polymers in the asphalt binders.

In this research, limited samples could be prepared with the gyratory compactor because of equipment operational issues. These samples were prepared in another university in Pakistan. The following three asphalt mixtures were used for the gyratory specimens:

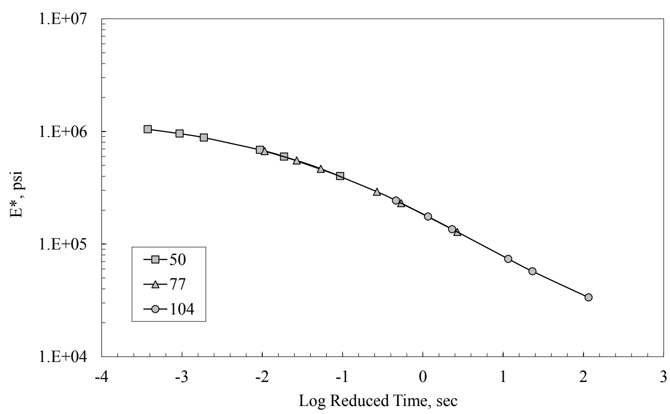
1. NHA wearing course with Karachi 60/70 binder
2. Superpave wearing course with Karachi 60/70 binder
3. NHA wearing course with Karachi polymer modified binder

These specimens were prepared according to the Superpave requirements following the AASHTO PP 60-90 (*55*). The dynamic modulus and flow number performance tests were performed on these laboratory prepared specimens. The

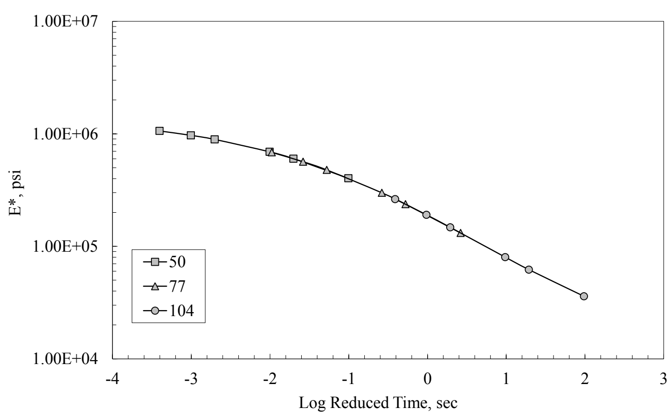
AASHTO standards were followed for performing these tests (*56, 57*). Master curves were prepared based on the results of dynamic modulus test at three different temperatures and six frequencies by following the AASHTO PP 61-10 and 62-11 (*58, 59*). It should be noted that for each asphalt mixtures, two specimens were tested. Figure 5-29 shows master curves based on average values for all three asphalt mixtures.



(a) K67NHAW



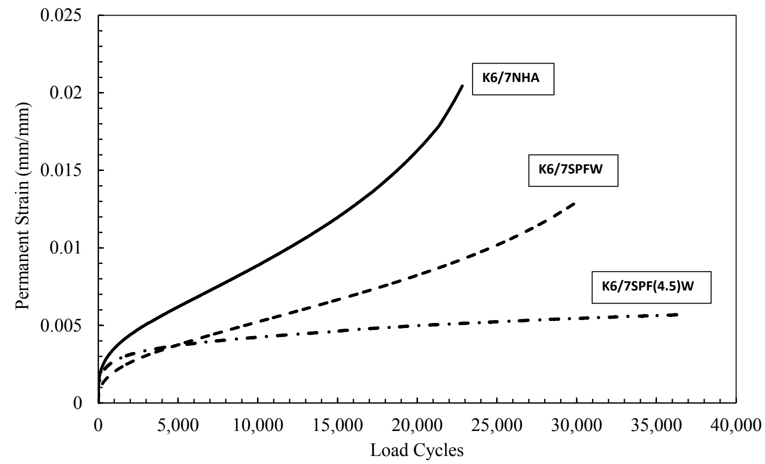
K67SPFW



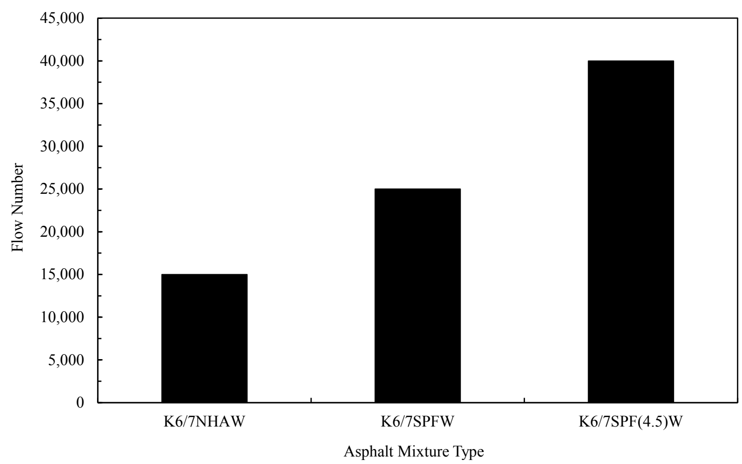
K67(4.5)SPW

Figure 5-29 Master curves for three asphalt mixtures from E^* tests

Figure 5-30 presents the results of flow number (FN) test for the same three asphalt mixtures. The results of the performance tests show that NHA asphalt mixture has the highest potential for rutting as the FN is the lowest in this case (see Figure 5-30b). The Superpave mixtures exhibited better rutting performance, especially with polymer modified asphalt binder.



(a) Permanent deformation with load cycles



(b) Flow number

Figure 5-30 Rutting potential for asphalt mixtures

The proceeding sections presented the comparison of the HMA mixtures expected performance based on the state-of-the-art laboratory tests. However, the laboratory test results can only be used for relative comparison among the various HMA mixtures considered in this study. In order to quantify the expected performance of HMA mixtures, and expand the impact of HMA mixture design methodology on pavement performance, another comparison among different mixtures was conducted using the mechanistic-empirical pavement design guide (MEPDG). This analysis is based on material properties obtained from the laboratory tests and is presented next.

5.3.2. Impact of HMA Design Methodology on Pavement Performance

The dynamic modulus |E*|, is one of the fundamental properties necessary to determine the stiffness characteristics of HMA mixtures. |E*| is an important property to characterize HMA because it: (a) is required as an input in the MEPDG which predicts pavement performance, (b) can be predicted from the Superpave simple performance tests, (c) is a linear viscoelastic material property which is used in advanced HMA and pavement models based on viscoelasticity.

In order to predict the dynamic modulus |E*| master curves, asphalt binder test data were utilized. In this study dynamic shear rheometer (DSR) and rotational viscometer (RV) tests were conducted on 22 binders (see section 5.2). Based on the binder test data, the following parameters were obtained:

- Viscosity temperature susceptibility parameters—A-VTS, determined from DSR and RV tests.
- Shear modulus and phase angle—*G** and *δ*, obtained from the DSR tests.

These above mentioned asphalt binder properties were utilized to predict the |E*| master curves for all 22 binders using artificial neural networks for asphalt concrete dynamic modulus prediction (ANNACAP) software (*60*).

5.3.2.1. |E| Master curve prediction using ANNACAP software*

The Artificial Neural Networks for Asphalt Concrete Dynamic Modulus Prediction (ANNACAP) program uses laboratory data to predict the HMA |E*| master curves. The ANNACAP program uses several different data types to predict the dynamic modulus. These three artificial neural network (ANN) modules are based on the following different input types:

- Resilient modulus (M_R) of HMA mixtures
- |G*| of asphalt binders
- Viscosity of asphalt binders

The resilient modulus (M_R) based ANN for HMA is the first input type. The M_R values for a HMA mixture are needed for three specific temperatures (i.e., 5 °C, 25 °C, and 40 °C) and have the unit of Giga Pascal (GPa) with the following input ranges:

- 5°C - 34.1 to 4.8 GPa
- 25°C- 15.4 to 1.1 GPa
- 40°C- 6.9 to 0.38 GPa

The |G*| based ANN predicts the dynamic modulus by using |G*| data. There are three different levels of inputs for the |G*| based model. |G*| values obtained from laboratory tests along with HMA, VMA, and VFA are required. Three input levels are offered and the following details are necessary for the different levels:

- Level 1: Users must have access to |G*| values at different temperatures and frequencies.
- Level 2: Users must have access to |G*| values and possibly BBR stiffness values at multiple temperatures. There should be 2 temperatures above 46°C and 2 below 46°C and at a fixed frequency of 10 rad/s and load time of 60 seconds. The measured moduli values should be at the same aging level. The |G*| values for Level 2 inputs have the units of kilo Pascal (kPa). The values for S(t) have the units of Mega Pascal (MPa).
- Level 3: Users must have |G*| data values and possibly BBR stiffness values similar to Level 2. The aging conditions are RTFO and PAV. Units remain the same as Level 2 inputs. Users can also choose the high temperature Superpave PG. Only RTFO aging can be used for Level 3 analysis.

The viscosity based ANN uses viscosity based inputs to predict |E*|. As with the |G*| based model, there are three different levels of input. For each level, HMA, VMA, and VFA data are needed. The following are details of different levels:

- Level 1: Users enter A-VTS data directly.
- Level 2: Users choose the type of viscosity measures available. The available measures are: ring and ball temperature, penetration, absolute viscosity, and kinematic viscosity. At least two measures of viscosity are needed as inputs in order to calculate A-VTS.
- Level 3: Users choose the asphalt binder grade from a drop down menu. The asphalt binders can be Superpave based, viscosity based, or penetration based.

Based on all the different input types, the software predicts the |E*| values for a range of frequencies and temperatures. Based on the asphalt binder test data obtained through the DSR and RV tests, it was determined that the viscosity based model will be used to predict the |E*| master curves. The master curves were predicted for all 22 binders using both the DSR and the RV data. The VMA and VFA values were held constant at values of 15% and 75%, respectively. Table 5-20 shows an example of predicted |E*| values for the asphalt binder 1.

Table 5-20 |E*| prediction for asphalt binder 1

Temperature	Frequency					
	0.1	0.5	1	5	10	25
14	1912135	2515010	2697107	3106188	3233054	3336781
40	976069	1395763	1555126	2002045	2165258	2376110
70	143431	276223	339332	559950	670814	842523
100	32502	53113	64487	114753	142591	185677
130	27070	42649	51366	90741	112576	145932

The values for |E*| (from ANNACAP prediction), |G*|, and δ (from DSR test data) were used to plot the master curves using the MEPDG. Figure 5-31 illustrates an example of the master curve for asphalt binders 1 and 2. Figures 5-32 and 5-33 show the screen shots of the MEPDG for HMA mixture and binder data needed to predict the |E*| master curves.

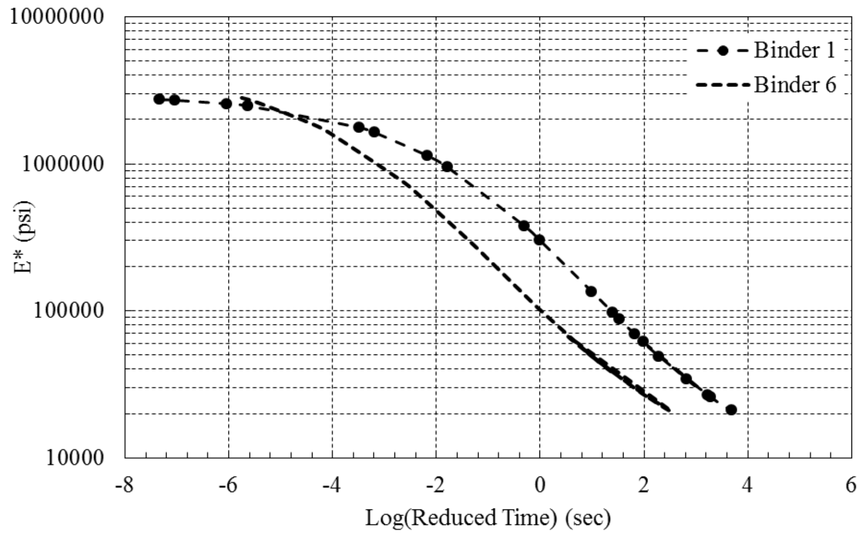


Figure 5-31 Example master curves for asphalt binders 1 and 6

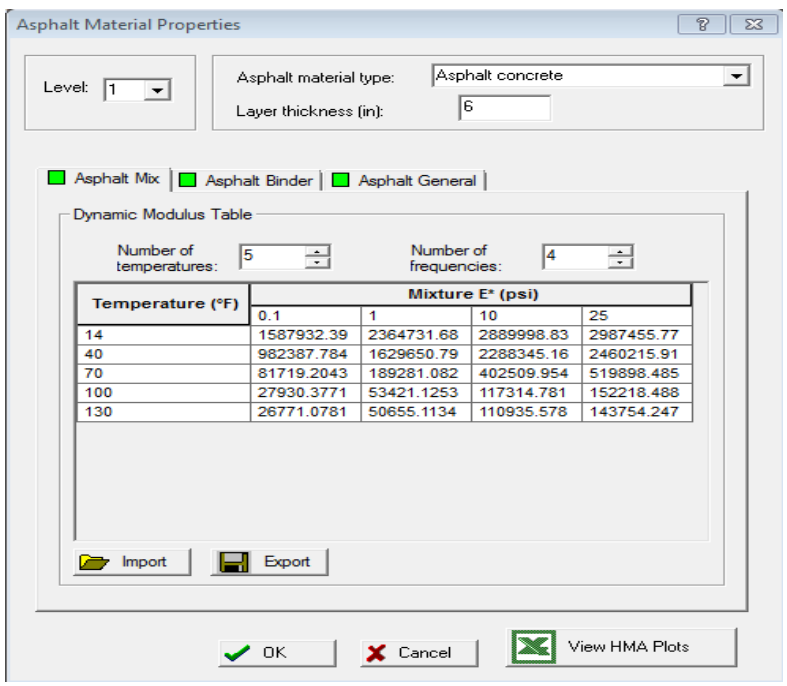


Figure 5-32 Asphalt mixture inputs for the MEPDG

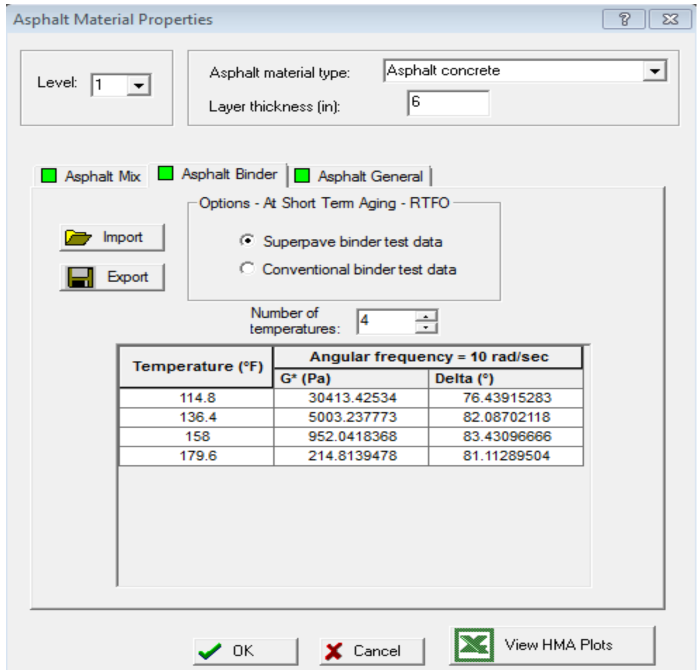


Figure 5-33 Asphalt binder inputs for the MEPDG

All 22 asphalt binders |E*| master curve results from the ANNACAP program will be discussed in the following section.

5.3.2.2. Performance analysis using the MEPDG

Once the master curves were predicted using the ANNACAP, they were directly used in the MEPDG. The MEPDG uses the asphalt binder and HMA mixture data to predict the performance of a standard pavement over a 20 year design life. In these analyses, other than asphalt binder and HMA mixture data, all other inputs were kept at default values. Table 5-21 lists the different inputs for design life, traffic, and the standard pavement structure.

Table 5-21 The MEPDG inputs used for pavement performance prediction

Design life	20 years	
Initial two way AADTT	1500	
Pavement structure	Thickness (inch)	Modulus (psi)
Asphalt	6	|E*| master curves based on binder type
Granular Base	8	40000
Granular Base	10	15000
Subgrade	-	7500

Some of the |E*| values for certain asphalt binders caused the MEPDG to crash; therefore, for those binders, Level 2 or Level 3 analysis was used to obtain the performance summary. The MEPDG analysis results present time series performance data for the 20 year design life. The performance measures predicted by the software include:

- Longitudinal cracking
- Alligator cracking
- Subtotal AC rutting
- Total surface rutting
- IRI

A sample output of the pavement performance summary can be seen in Table 5-22. The sample output only shows distress prediction for a few months.

Table 5-22 MEPDG distress prediction output for asphalt binder 1

Pavement age		Month	Longitudinal Cracking (ft/mi)	Alligator Cracking (%)	Subtotal AC Rutting (in)	Total Rutting (in)	IRI (in/mi)
mo	yr						
1	0.08	October	0.96	0.0294	0.017	0.239	72.6
2	0.17	November	1.49	0.0511	0.019	0.267	73.8
3	0.25	December	1.76	0.0676	0.019	0.284	74.5
4	0.33	January	1.91	0.0797	0.02	0.296	75
5	0.42	February	2.08	0.0946	0.02	0.307	75.5
6	0.5	March	2.96	0.127	0.024	0.325	76.2
7	0.58	April	4.35	0.166	0.028	0.344	77
8	0.67	May	7.09	0.221	0.039	0.37	78.2

Based on the pavement performance prediction data, the following charts were made to illustrate the temporal performance for 20 year design life.

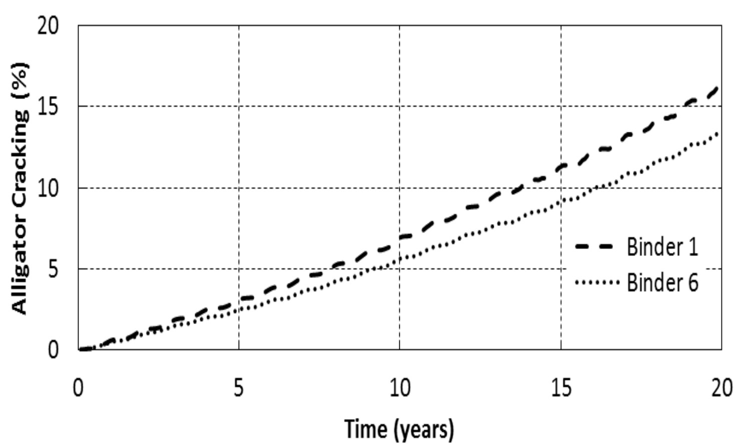


Figure 5-34 Example of pavement fatigue cracking performance prediction

5.3.2.3. Performance comparison based on asphalt binder properties

As mentioned before, two types of asphalt binder data were utilized to predict HMA mixture E^* master curves by using ANNACAP software. These data types include:

1. Viscosity-temperature data obtained through RV testing for all 22 asphalt binders.
2. G^*-temperature data collected using DSR testing for all 22 asphalt binders.

The results are presented below based on the RV and DSR data, respectively. The 22 asphalt binders used in this analysis are presented again in Table 5-23 for the convenience of the readers.

Table 5-23 Asphalt binder coding for the 22 binders used in the study

No.	Binder Code	Binder Description	PG Grade
1	APMB	Attock Polymer Modified	76-6
2	A6/7	Attock Pen 60/70	58-12
3	A8/10	Attock Pen 80/100	58-12
4	K4/5	Karachi Pen 40/50	64-6
5	K6/7	Karachi Pen 60/70	64-12
6	K8/10	Karachi Pen 80/100	58-12
7	BA6/7(20)8/10(80)	Blended Attock: Pen 60/70 (20%) and Pen 80/100 (80%)	-
8	BA6/7(50)8/10(50)	Blended Attock: Pen 60/70 (50%) and Pen 80/100 (50%)	-
9	PA8/10(1.35)	Polymer Modified Attock 80/100 with 1.35% Elvaloy	64-12
10	PA8/10(1.70)	Polymer Modified Attock 80/100 with 1.70% Elvaloy	70-12
11	PA8/10(2.00)	Polymer Modified Attock 80/100 with 2.00% Elvaloy	70-12
12	PA6/7(1.35)	Polymer Modified Attock 60/70 with 1.35% Elvaloy	64-12
13	PA6/7(1.70)	Polymer Modified Attock 60/70 with 1.70% Elvaloy	70-12
14	PA6/7(2.00)	Polymer Modified Attock 60/70 with 2.00% Elvaloy	70-12
15	BK6/7(20)8/10(80)	Blended Karachi Pen 60/70 (20%) and Pen 80/100 (80%)	-
16	BK6/7(50)8/10(50)	Blended Karachi Pen 60/70 (50%) and Pen 80/100 (50%)	-
17	BK8/10(20)4/5(80)	Blended Karachi Pen 80/100 (50%) and Pen 40/50 (80%)	-
18	BK8/10(50)4/5(50)	Blended Karachi Pen 80/100 (50%) and Pen 40/50 (50%)	-
19	BK6/7(50)4/5(50)	Blended Karachi Pen 60/70 (50%) and Pen 40/50 (50%)	-
20	PK8/10(2.5)	Polymer Modified Karachi 80/100 with 2.5% AC	64-12
21	PK8/10(3.5)	Polymer Modified Karachi 80/100 with 3.5% AC	64-12
22	PK8/10(4.5)	Polymer Modified Karachi 80/100 with 4.5% AC	64-12

The asphalt binders are further grouped into three main types: neat, polymer modified, and blended. Figures 5-35 to 5-37 present the predict E^* master curves based on the RV data for neat , polymer modified, and blended asphalt binders, respectively. The master curves show the E^* values for a range of frequencies (or time). Asphalt binder behavior at higher time (lower frequency) also represents its behavior at a higher temperature and vice versa. The master curves show that E^* values approach the maximum value at low temperature (lower time or high frequency) while E^* values are at the lowest value for high temperature (high time or low frequency).

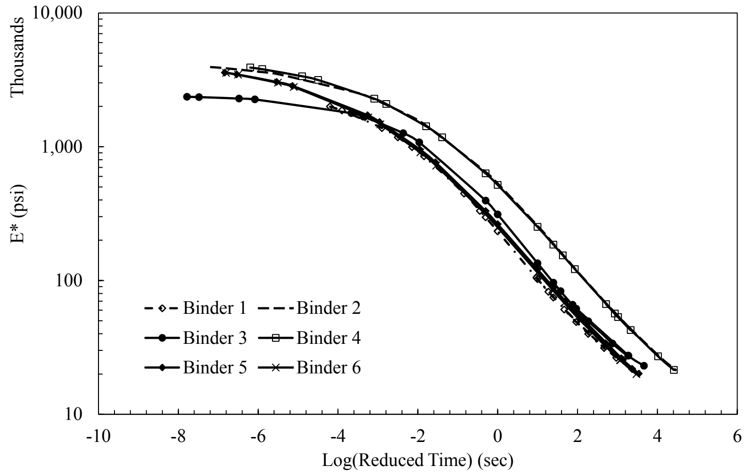


Figure 5-35 Neat asphalt binder E* master curves – RV

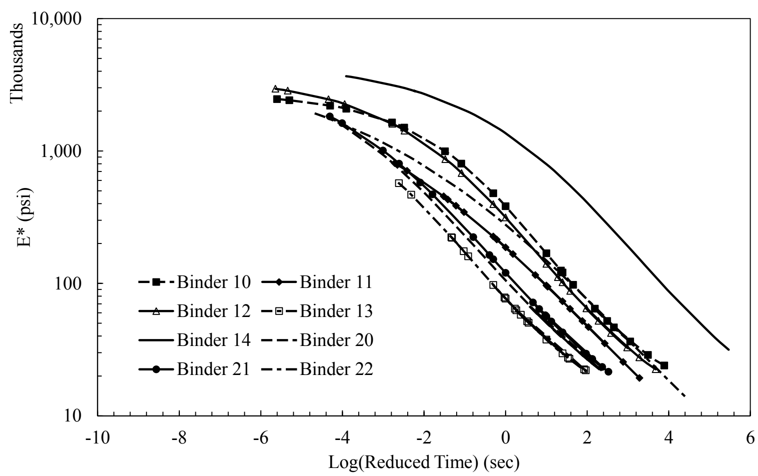


Figure 5-36 Polymer modified asphalt binder E* master curves – RV

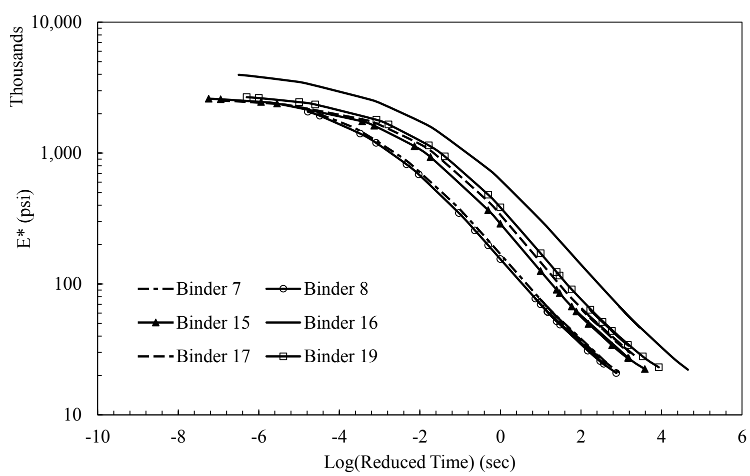


Figure 5-37 Blended asphalt binder E* master curves – RV

Generally, a shift left to right of the E^* mater curve shows a HMA mixture which will have higher E^* values at higher temperature and thus will be suitable to resist pavement rutting. Figures 5-38 to 5-40 show HMA E^* master curves using the DSR asphalt binder data.

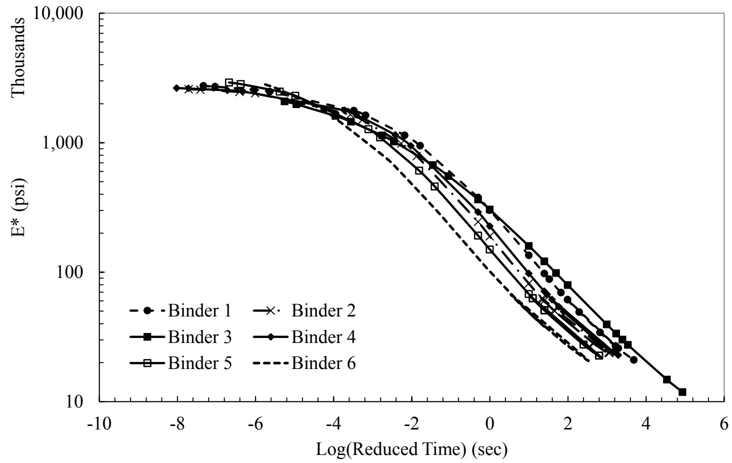


Figure 5-38 Neat asphalt binder E* master curves – DSR

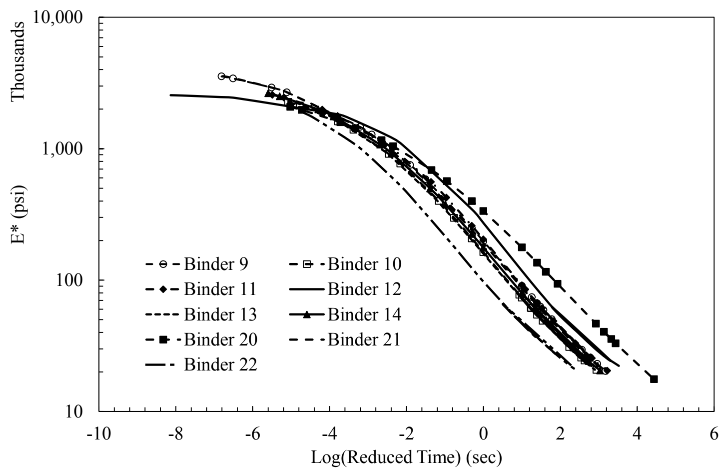


Figure 5-39 Polymer modified asphalt binder E* master curves – DSR

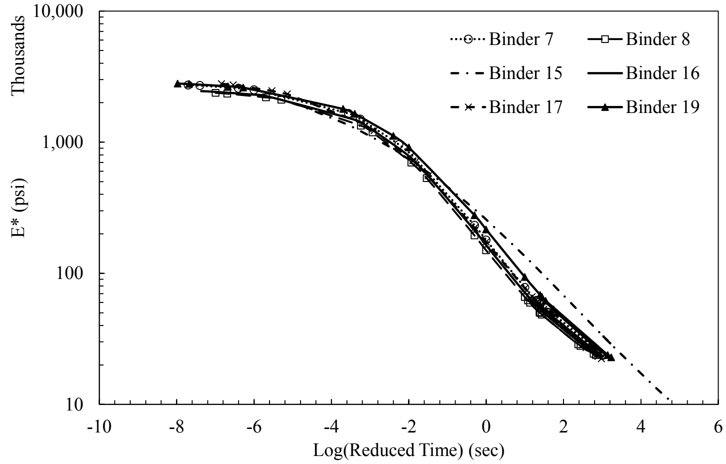
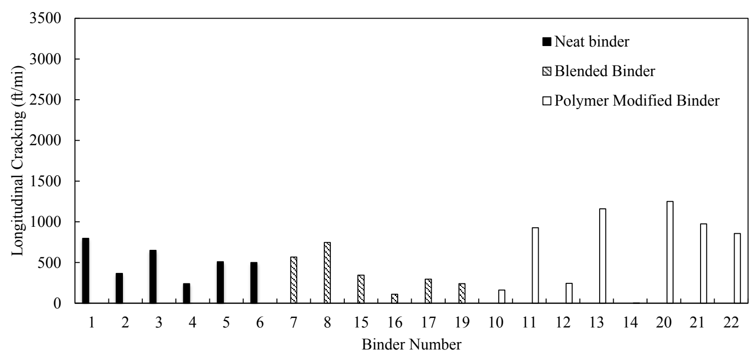
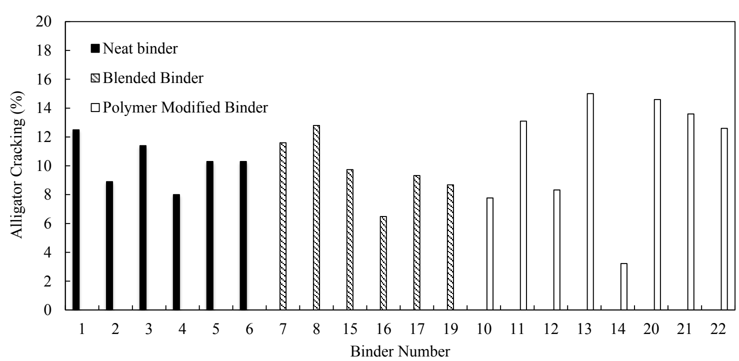


Figure 5-40 Blended asphalt binder E* master curves – DSR

In order to assess the performance of HMA mixtures having different E^* master curve, the MEPDG software was utilized for pavement performance prediction. In this analysis, a typical flexible pavement cross-section was assumed as mentioned in Table 5-21. All other input variables were kept constant so that the impact of different HMA mixtures (i.e., 22 master curves) on predicted performance may be investigated. Figure 5-41 presented the cracking performance of all HMA mixtures when RV data were utilized. No clear difference was observed among different groups for both cracking types.



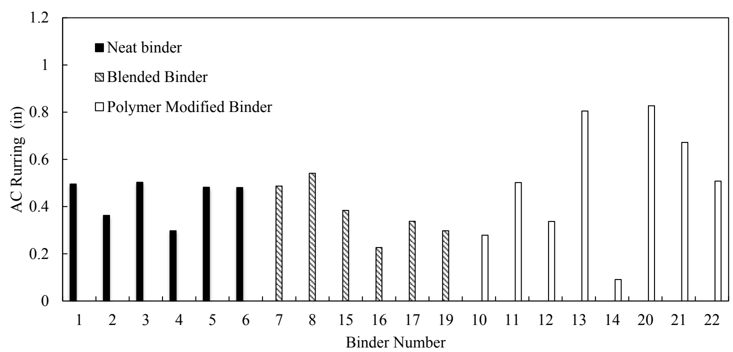
(a) Longitudinal cracking



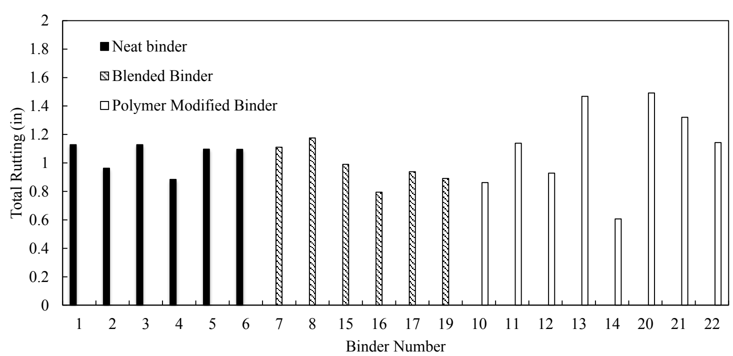
(b) Fatigue cracking

Figure 5-41 Predicted pavement cracking performance for all asphalt binders – RV

Figure 5-42 shows the comparison of pavement rutting performance among all groups. Generally, HMA mixtures with polymer modified and blended asphalt binders showed better pavement rutting performance than the neat binders after 20 years. Figure 5-43 presents the ride quality in terms of international roughness index (IRI). The IRI is strongly impacted by cracking and rutting distresses; therefore, similar trends were observed among different groups. As mentioned above, the RV testing was conducted between 100 to 185 ^{o}C to determine viscosity-temperature characterization for each asphalt binder. This temperature range does not correspond to the actual field pavement temperatures. However, such data are required to investigate the asphalt binder mixing during production and compaction temperatures requirements. Therefore, predicted E^* for HMA mixtures from RV data may not depict the actual HMA characteristics and thus the predicted pavement performance will be significantly impacted. In order to address this concern DSR data were utilized and the results are presented next.



(a) HMA rutting



(b) Total surface rutting

Figure 5-42 Predicted pavement rutting performance for all asphalt binders – RV

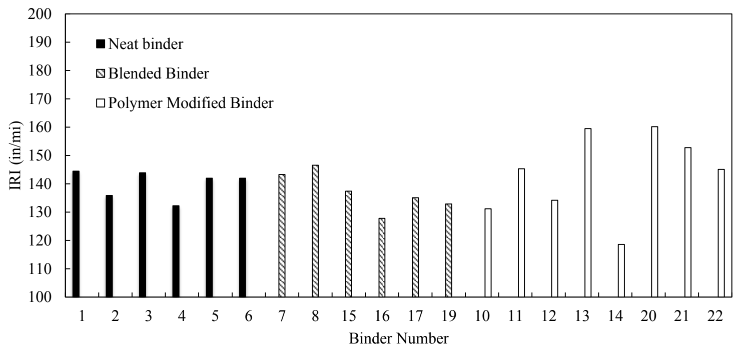
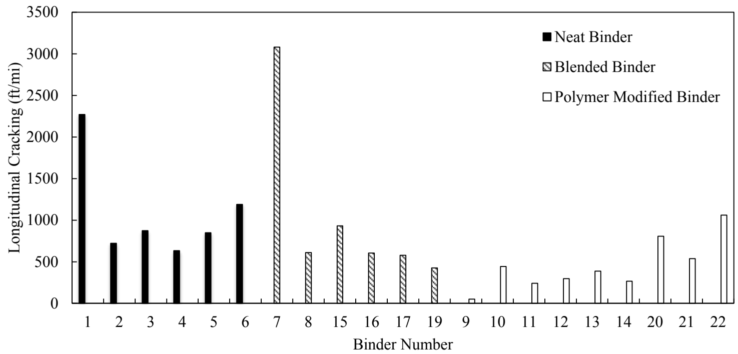
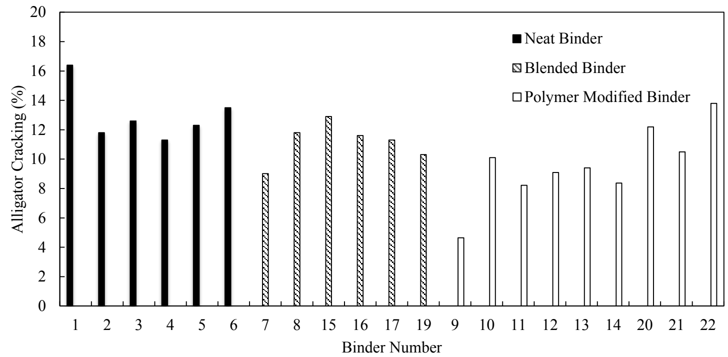


Figure 5-43 Predicted pavement IRI for all asphalt binders – RV

The DSR testing was conducted for all 22 asphalt binders between 7 to 82 ^{o}C. This temperature range is more representative of the field pavement temperatures. Again, the *E** HMA master curves were predicted using ANNACAP software. Subsequently, these *E** master curves were used in the MEPDG to predict various performance measures. Figure 5-44 shows the pavement cracking performance for all HMA mixtures having the same 22 asphalt binders as mentioned in Table 5-23. Again, all of the HMA mixtures were grouped into three categories based on the asphalt binder type: neat, polymer modified, and blended. The results show that, in general, HMA mixtures with modified asphalt binders exhibit lower cracking after 20 years of service. However, the improvement in the pavement cracking performance is not very significant among different groups.



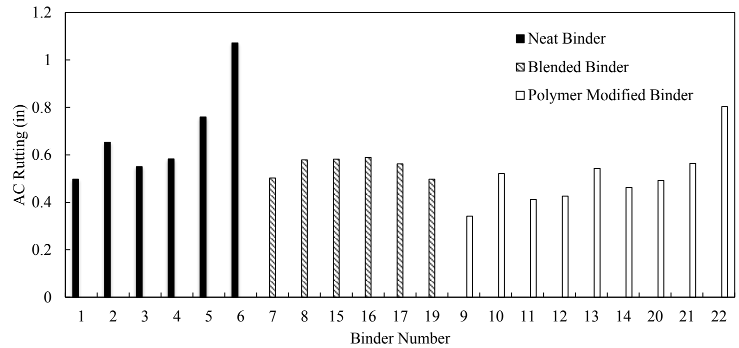
(a) Longitudinal cracking



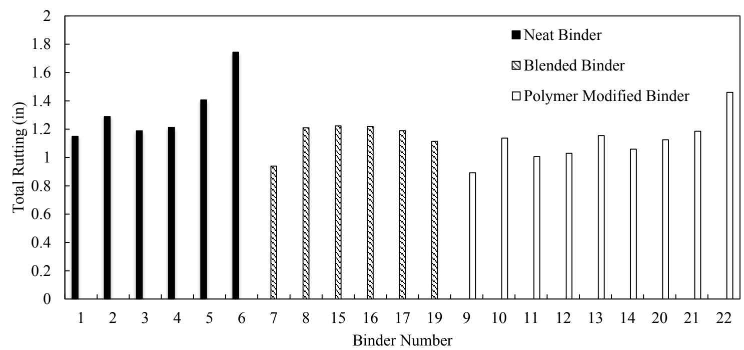
(b) Fatigue cracking

Figure 5-44 Predicted pavement cracking performance for all asphalt binders – DSR

Figure 5-45 presents the comparison of pavement rutting performance for HMA mixtures having different asphalt binders. The results show that , on average, the HMA mixtures with modified asphalt binders exhibited the lowest pavement rutting while those with neat binders (locally available) showed the maximum rutting after 20 years of service. Thus, the results clearly demonstrate that the HMA mixtures with local asphalt binders may produce much higher levels of pavement rutting at prevailing temperatures in Pakistan. Therefore, there is a need to use the appropriate asphalt binders in the HMA mixtures to improve pavement performance in Pakistan. Figure 5-46 presents the comparison of IRI among different groups. As expected, the results show that HMA mixtures with modified asphalt binders remain smoother than others. Smoother pavement over time also means higher road user benefits i.e., lower vehicle operating costs, higher time savings, and lower agency costs. Thus, HMA mixtures with appropriate and more suitable asphalt binders may assist the country in significant savings of limited resources in the long-run.



(a) HMA rutting



(b) Total surface rutting

Figure 5-45 Predicted pavement rutting performance for all asphalt binders – DSR

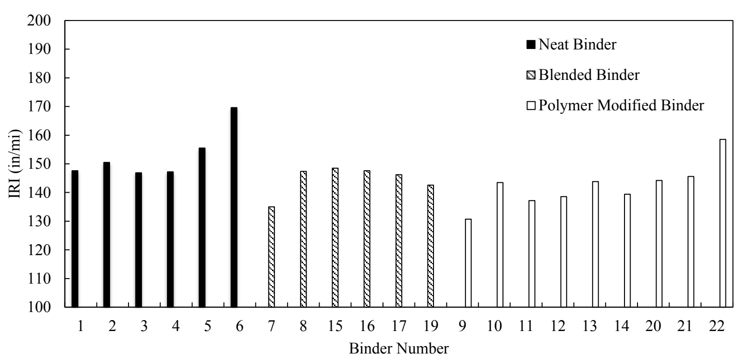
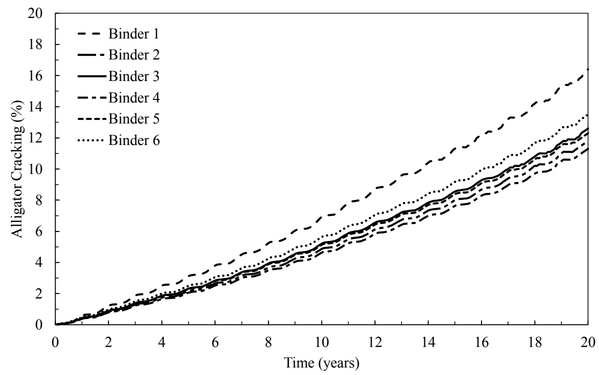
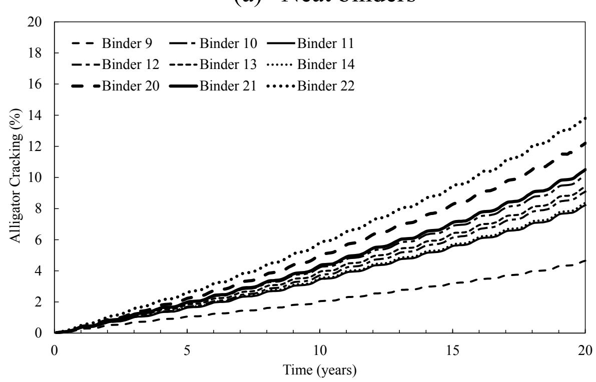


Figure 5-46 Predicted pavement IRI for all asphalt binders – DSR

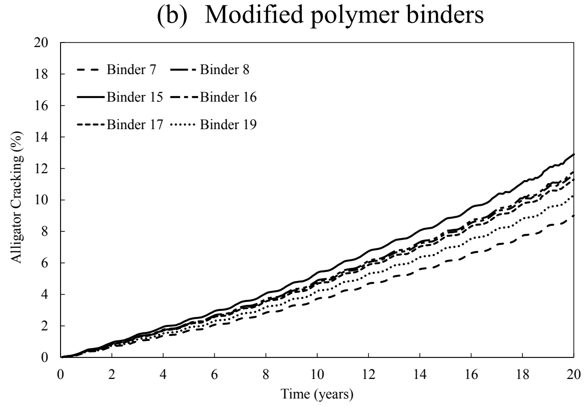
The above comparisons among various HMA mixtures (with different asphalt binders) only show the predicted pavement performance (i.e., cracking, rutting, and IRI) after 20 years of service. However, it will be more useful to compare the predicted pavement performance over time for different groups. Such comparisons could be valuable in estimating the amount of additional life extension when different asphalt binder types are utilized. Figure 5-47 presents the predicted pavement fatigue cracking performance for three groups. It can be seen from these time series curves that to reach a threshold of 10% cracking, HMA mixtures with neat asphalt binders, on an average, will take between 16 to 18 years while the HMA mixtures with modified binders will take more than 20 years. Thus, there will be a life extension of an additional 3 years to reach 10% fatigue cracking when HMA mixtures with modified asphalt binders are utilized. It should be noted that such life extension can be different for each performance measure. For example, 5-48 shows the same time series plots for the three groups for pavement rutting in the HMA layer. The results indicate that, on an average, a life extension of about 5 years can be obtained by using HMA mixtures with modified as compared to neat asphalt binders. Similar plots for the total pavement surface rutting for all three groups are shown in Figure 5-49.



(a) Neat binders

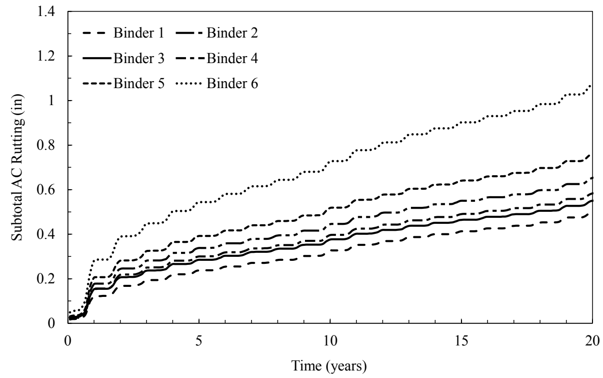


(b) Modified polymer binders

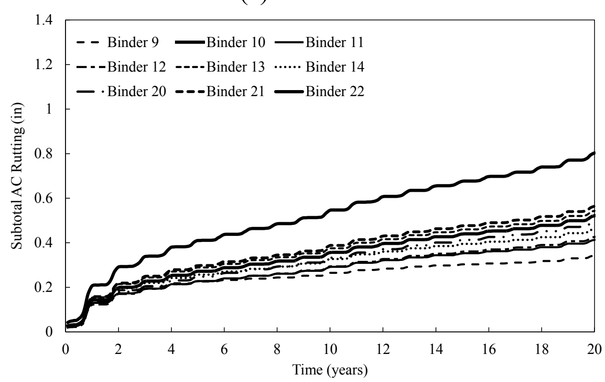


(c) Blended binders

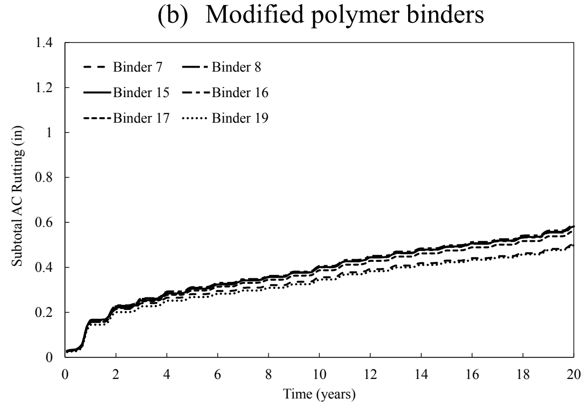
Figure 5-47 Predicted fatigue performance for all binders – DSR



(a) Neat binders

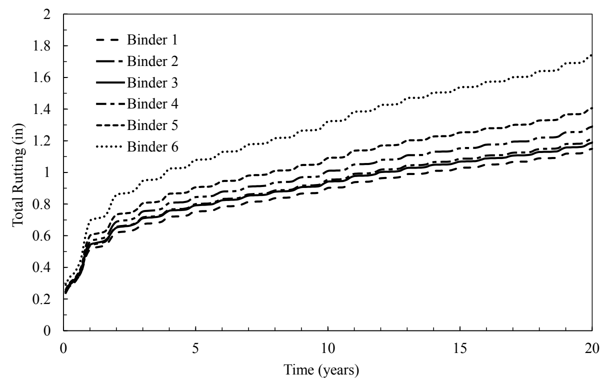


(b) Modified polymer binders

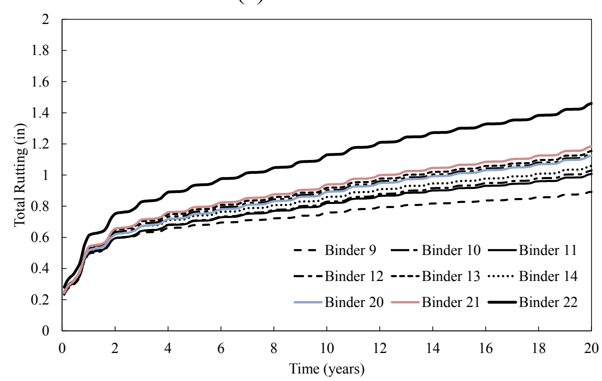


(c) Blended binders

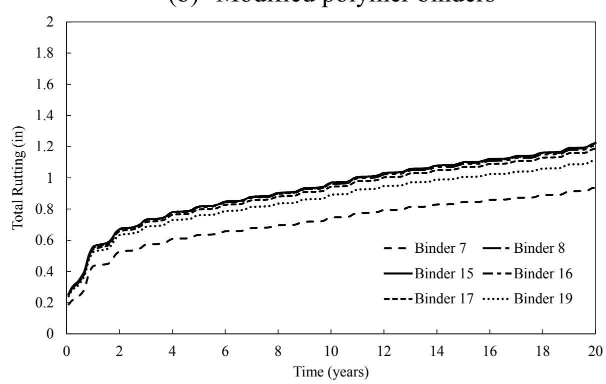
Figure 5-48 Predicted HMA rutting performance for all binders – DSR



(a) Neat binders



(b) Modified polymer binders



(c) Blended binders

Figure 5-49 Predicted total surface rutting performance for all binders – DSR

The analysis and results presented in the previous section utilized all of the 22 asphalt binders to predict HMA E^* master curves. However, the HMA properties (volumetric properties and gradations) were held constant. The objective is to assess the pavement performance for 22 HMA mixtures having different asphalt binders. In this study, 10 different HMA mixtures were also prepared by utilizing different gradations and asphalt binder types. The pavement performance analysis and results of the HMA mixtures are presented in the next section.

5.3.2.4. Performance comparison based on HMA mixture and asphalt binder properties

Table 5-16 shows the details of ten HMA mixtures used to compare performance of NHA (Marshall) and Superpave design methodologies. Locally available and polymer modified asphalt binders were used in these HMA mixtures. Figure 5-50 presents the predicted E^* master curves for all of the HMA mixtures. These master curves were predicted by utilizing the gradation and volumetric properties of each HMA mixture. In addition, the DSR data for the asphalt binders used in each HMA mixture were used to predict the E^* master curve. Therefore, each E^* master curve represents the specific mixture as designed in the laboratory and with actual material properties.

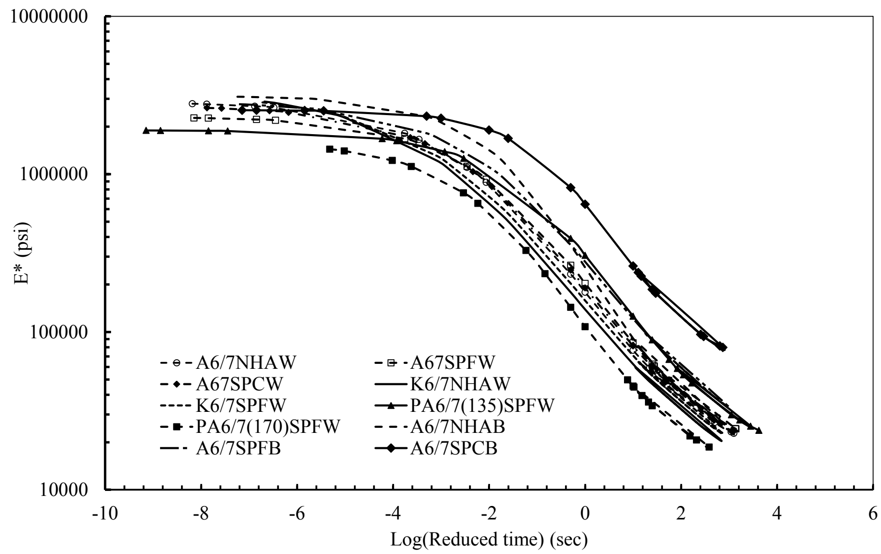
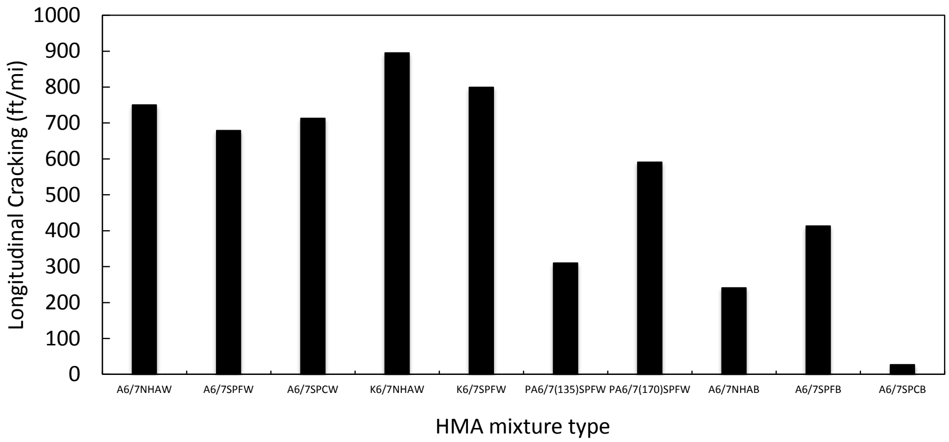


Figure 5-50 E^* master curves for different HMA mixtures

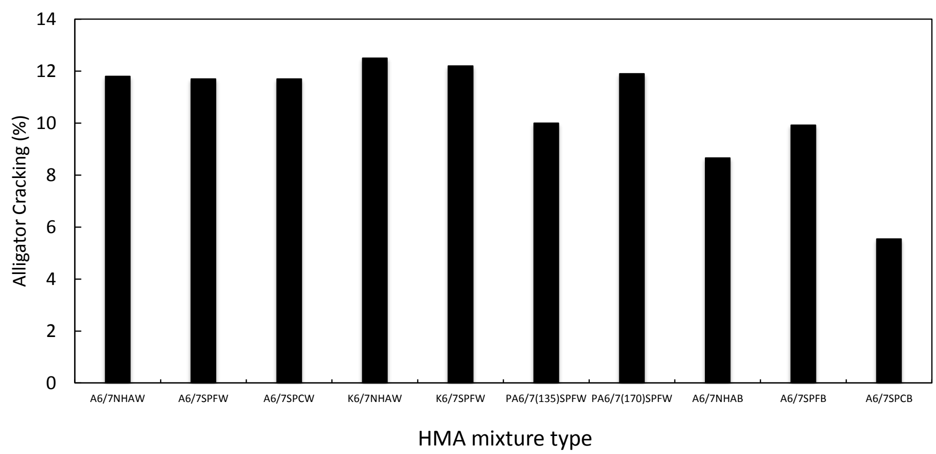
The E^* master curves shown above were used in the MEPDG software for long-term pavement performance prediction, as described in the above section. Figure 5-51 shows the comparison of the pavement cracking performance among all HMA mixtures. The results indicate that:

- NHA mixtures for the wearing course exhibit higher longitudinal and fatigue cracking than all other mixtures after 20 years of service life. NHA mixtures with asphalt binder from the Attock refinery showed slightly better pavement cracking performance than that from the Karachi refinery.
- Superpave mixtures for the wearing course show lower pavement cracking than NHA mixtures. Within Superpave mixtures, those with Attock asphalt binders showed slightly better pavement cracking performance than those with Karachi binders.
- Superpave wearing course mixtures with polymer modified asphalt binders show consistently better cracking performance than those with other binder types.

- Superpave mixtures for the base course, especially with coarse gradation, exhibit much lower potential for pavement cracking as compared to NHA base course mixtures.



(c) Longitudinal cracking



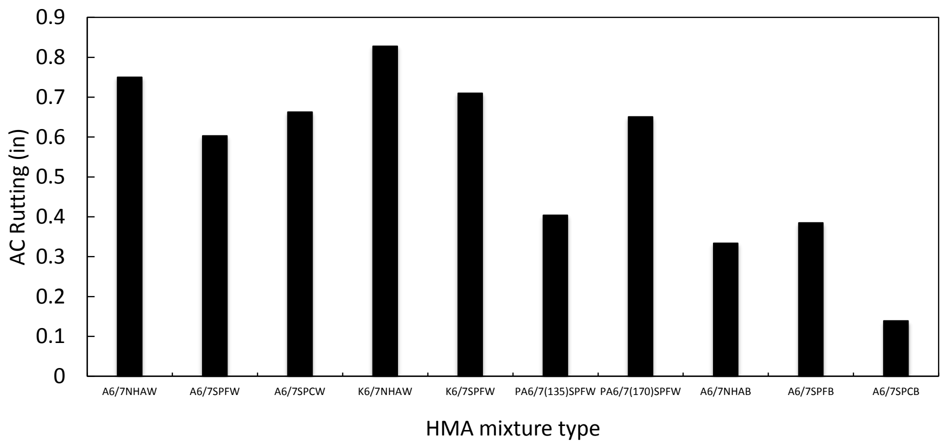
(d) Fatigue cracking

Figure 5-51 Predicted pavement cracking performance for all HMA mixtures

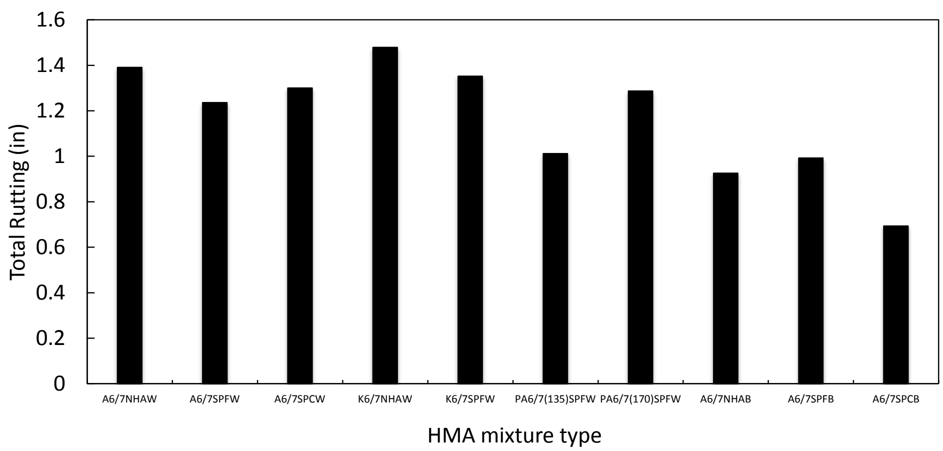
Figure 5-52 shows the pavement rutting prediction results for all HMA mixture types. The results indicate that:

- NHA mixtures for the wearing course exhibit higher pavement rutting than all other mixtures after 20 years of service life. Again, NHA mixtures with asphalt binder from the Attock refinery showed slightly better pavement rutting performance than that from the Karachi refinery.
- Superpave mixtures for the wearing course show lower pavement rutting than NHA mixtures. Within Superpave mixtures, those with Attock asphalt binders showed slightly better pavement rutting performance than those with Karachi binders.

- Superpave wearing course mixture with polymer modified asphalt binder (1.35% polymer) shows consistently better pavement rutting performance than those with other binder types.
- Superpave mixtures for the base course, especially with coarse gradation, exhibit much lower potential for pavement rutting as compared to NHA base course mixtures.



(c) HMA rutting



(d) Total surface rutting

Figure 5-52 Predicted pavement rutting performance for all HMA mixtures

Similar trends were observed for IRI among all HMA mixtures as shown in Figure 5-53.

Figures 5-54 and 5-55 show the pavement fatigue cracking and rutting for all HMA mixtures over time, respectively. It can be seen from the results that a significant improvement in pavement performance can be achieved by using Superpave mixture design methodology with polymer modified asphalt binders. For example, for NHA mixtures, only 3 to 5 years is needed to reach a rut depth of about 0.3 inches while the Superpave mixture will reach the same threshold in about 15 to 18 years of pavement service life. Thus, the pavement performance lives could be significantly enhanced by adopting Superpave methodology and by using the appropriate asphalt binders to suit the local climatic conditions in Pakistan.

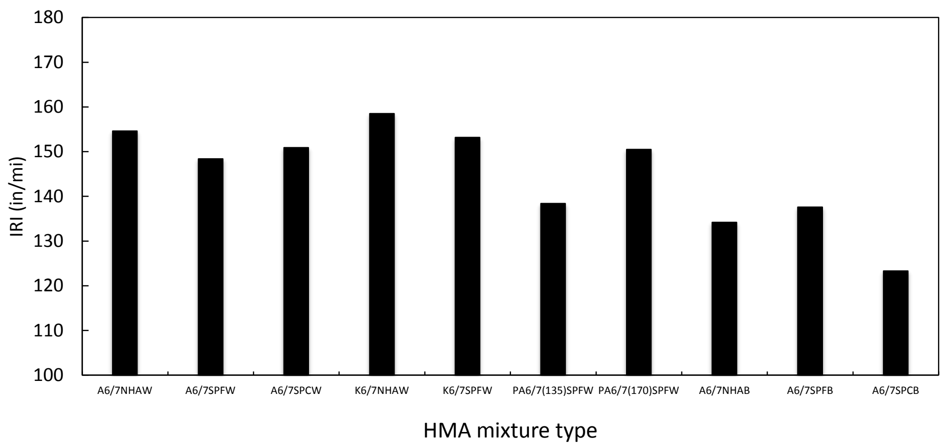


Figure 5-53 Predicted pavement IRI for all HMA mixtures

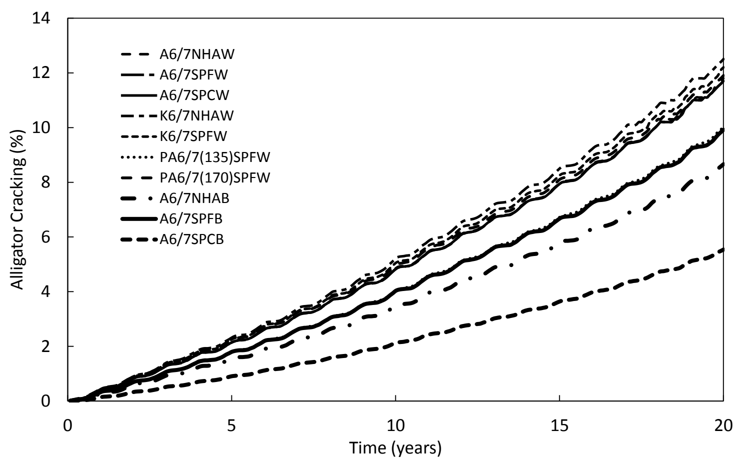


Figure 5-54 Predicted pavement fatigue performance for all HMA mixtures

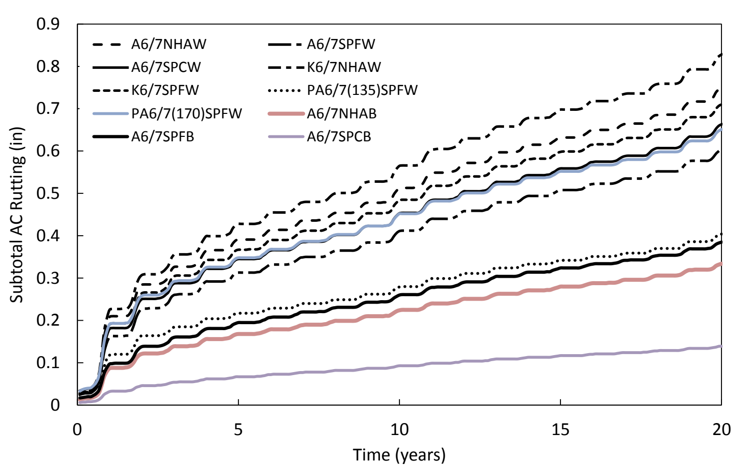


Figure 5-55 Predicted HMA rutting performance for all HMA mixtures

CHAPTER 6 - CONCLUSIONS & RECOMMENDATIONS

6.1 SUMMARY

During this study, the USA and the Pakistani research teams worked together to improve the state-of-the-practice regarding pavement material characterization to enhance flexible pavement performance in Pakistan. Their efforts yielded the following accomplishments:

1. A state-of-the-art asphalt laboratory was established at the University of Engineering and Technology (UET), Lahore in Pakistan.
2. Neat binders and aggregate samples were collected from the two oil refineries and Margalla quarry, respectively.
3. Local binders were modified using two types of polymers. Binders from the same source were blended to create different binder types. A total of 22 binders were tested in this study.
4. The state-of-the-art performance testing was performed on 10 HMA mixtures to compare current (Marshall) and Superpave methodologies.
5. Local materials currently being used in road construction including asphalt binders, aggregates, and HMA mixtures were subjected to rigorous laboratory investigations.
6. Climatic data for the last 20 years were synthesized and analyzed to determine the required suitable binder in different regions of the country.
7. The laboratory binders and HMA mixtures data were subjected to mechanistic and empirical analyses using various traditional and state-of-the-art procedures.
8. Continuing education seminars were held at UET facility in Lahore. The seminars were attended by NHA engineers and staff, the private sector and the academia.

Based on the above activities and accomplishments of the research teams, several conclusions and recommendations were drawn and are presented in the next two sections.

6.2 CONCLUSIONS

Extensive material testing was accomplished during the course of this study. Based on the analysis of the test results, the following conclusions were drawn:

1. The A-VTS parameters for twenty two (22) neat and polymer modified binders were obtained from three test methods and compared. The results show that A-VTS parameters from the three tests could be significantly different.
2. The impact of the VTS parameter on the E^* master curve shows that it is significantly affected by the temperature susceptibility of the binder. As a consequence, the pavement performance can be affected.

3. Among various test methods, CT, RV, and DSR can be used to determine A-VTS parameters for the pavement design purposes; however, the designer should be aware of the consequence of using an input A-VTS that may not truly represent the field conditions (i.e., the temperature range used in testing).
4. The temperature-viscosity relationships for twenty two (22) neat and modified asphalt binders were evaluated using conventional and other binder tests such as penetration, kinematic and absolute viscosities, and ring and ball softening point; rotational viscometer (RV); and dynamic shear rheometer (DSR). Binders having similar penetration, viscosity and performance grade (PG) showed dissimilar rheological behaviors.
5. There is a need to fully characterize the behavior of the asphalt binders to capture their rheology over a wide range of temperatures and load frequencies, especially if the binders are modified.
6. Binder temperature-susceptibility data showed that the activation energy (AE) of the binders depends on the test methods, which employ different temperatures used to characterize the binders. Correlations between AE and $G^*/\sin\delta$ and $G^*\sin\delta$ parameters show that the binders with higher AE will be more viscous at higher temperatures.
7. A negative correlation between AE and useful temperature range (UTR) of binders were shown by the MEPDG. The RV data could be used to characterize binders using activation energy concept. The AE concept did not show a good correlation with mixing and compaction temperatures of the binders, especially for the modified binders.
8. The SHRP and LTPP models for predicting pavement temperatures resulted in significantly prediction differences for both high and low temperatures. Although, higher differences were observed for low temperature compared to high temperature predictions.
9. SHRP prediction models for high and low temperatures were selected for the development of PG zones for Pakistan, since SHRP requires higher grade requirement for the high temperature grade, which is critical for uncontrolled axle loading.
10. Pakistan can be divided into six temperatures zones requiring the PG 70-10 as the most important binder that covers more than 70 percent of the area.
11. The use of 98% level of reliability provides additional safety margin against high traffic levels and uncontrolled loadings. No additional bumping of binder grade is needed as recommended by AASHTO MP1 specifications, since it will result in excessively stiff binder.
12. The PG 70-10 is not produced by any of the two refineries in Pakistan. The closest grade that fulfills the requirements of the PG 70-10 is the PG 76-16 (APMB). The APMB is relatively harder and may be prone to cracking. The PG 64-16 is relatively softer than the required grade and may be prone to rutting.
13. At present, commonly used binder grade in Pakistan is A-60/70 and K-60/70. The corresponding performance grades are PG 58-22 and PG 64-22. These softer binders, especially at high temperatures are likely to rut in areas requiring PG 70-10. This may be one of the major reasons of premature failures especially rutting in most pavements in Pakistan.

14. The HMA mixtures with NHA gradation and 60/70 pen binders (i.e., Attock and Karachi refineries) showed the lowest fatigue lives in the laboratory, especially at higher stress levels. The HMA mixtures with Attock refinery binder exhibited better fatigue life at lower stress levels.
15. The HMA mixtures with Superpave gradations and 60/70 pen binders (i.e., Attock and Karachi refineries) showed better fatigue lives in the laboratory than those with NHA gradations, especially at higher stress levels.
16. The HMA mixtures with Superpave gradations and polymer modified binders consistently showed better fatigue lives in the laboratory than those with NHA gradations, especially at intermediate and high stress levels.
17. The laboratory data for binders and HMA mixtures were used for performance prediction by using the mechanistic-empirical pavement design guide (MEPDG). The following conclusions can be drawn from the analysis results:

    a. NHA mixtures for wearing course exhibited higher longitudinal and fatigue cracking potentials than all other mixtures considered in the study after 20 years of service life. NHA mixtures with binder from Attock refinery showed slightly better cracking performance than those from Karachi refinery.
    b. Superpave mixtures for wearing course showed lower cracking potential than NHA mixtures. Within Superpave mixtures, those with Attock binders showed slightly better cracking performance than those with Karachi binders.
    c. Superpave wearing course mixtures with polymer modified binders showed consistently better cracking performance than those with other binder types.
    d. Superpave mixtures for base course, especially with coarse gradation exhibited much lower potential for cracking as compared to NHA base course mixtures.
    e. NHA mixtures for wearing course exhibited higher rutting potential than all other mixtures considered in this study after 20 years of service life. Again, NHA mixtures with binder from Attock refinery showed slightly better rutting performance than those from Karachi refinery.
    f. Superpave mixtures for wearing course show lower rutting than NHA mixtures. Within Superpave mixtures, those with Attock binders showed slightly better rutting performance than those with Karachi binders.
    g. Superpave wearing course mixtures with polymer modified binders (1.35% polymer) showed consistently better rutting performance than those with other binder types.
    h. Superpave mixtures for base course, especially with coarse gradation exhibited much lower potential for rutting as compared to NHA base course mixture.

18. The HMA mixture performance testing results verify the performance prediction outcomes in this study. Therefore, state-of-the-art performance testing in the

laboratory could be a tool to relatively rank HMA mixtures before using those mixtures.

6.3 RECOMMENDATIONS

Based on the results of this study and the observations made during the visits of the USA research team to Pakistan, the following recommendations are made:

1. The oil refineries in Pakistan should produce the binder grades which are more suitable for the local conditions (climate and traffic) in Pakistan. It is strongly recommended that PG 70-10 be produced and used for road construction in Pakistan.
2. The NHA construction specifications should be modified to facilitate adoption and implementation of Superpave methodology. The highway agencies in Pakistan should adopt the Superpave methodology to characterize asphalt binders and HMA mixtures. Based on the results obtained in this study, it is anticipated that the use of appropriate binder type and HMA mixture in the construction of new and rehabilitation of existing roads can significantly extend their useful life.
3. There is a need for more research to characterize HMA mixtures with different aggregate types, which are typically used in road construction in Pakistan.
4. The engineers, technicians and other professionals related to road maintenance and construction should be trained and certified for conducting state-of-the-art laboratory tests for the characterization of the asphalt binder and HMA mixtures.
5. It is strongly recommended that highway agencies and construction industry in Pakistan adopt the appropriate quality control and quality assurance (QC/QA) procedures during production and compaction of HMA mixtures in the field.

REFERENCES

1. Roberts, F. L., P. S. Kandhal, and E. R. Brown, *Hot Mix Asphalt Materials, Mixtures Design, and Construction*, 2nd Ed. ed. Lanham, Maryland: NAPA Education Foundation, 1996.
2. Asphalt Institute, "Performance Graded Asphalt Binder Specification and Testing, Superpave Series No. 1 (SP-1)," Asphalt Institute, Lexington, KY 2003.
3. Bari, J. and M. Witczak, "Evaluation of the Effect of Lime Modification on the Dynamic Modulus Stiffness of Hot-Mix Asphalt: Use with the New Mechanistic-Empirical Pavement Design Guide," *Transportation Research Record: Journal of the Transportation Research Board*, vol. 1929, pp. 10-19, 2005.
4. Bari, J. and M. Witczak, "New Predictive Models for Viscosity and Complex Shear Modulus of Asphalt Binders: For Use with Mechanistic-Empirical Pavement Design Guide," *Transportation Research Record: Journal of the Transportation Research Board*, vol. 2001, pp. 9-19, 2007.
5. Bari, J. and M. W. Witczak, "Development of a new revised version of the Witczak E Predictive Model for hot mix asphalt mixtures," Association of Asphalt Paving Technologist, White Bear Lake, MN 55110, United States, Savannah, GA, United States 0270-2932, 2006.
6. Mirza, M. W. and M. W. Witczak, "Development of a global aging system for short and long term aging of asphalt cements," Portland, OR, USA, 1996, pp. 393.
7. Asphalt Institute, "Superpave Mix Design, Superpave Series No. 2 (SP-2)," Asphalt Institute, Lexington, KY 2001.
8. Asphalt Institute, "Asphalt Binder Testing, Technician's Manual for Specification Testing of Asphalt Binders (MS-25)," Asphalt Institute, Lexington, KY 2007.
9. Hafez, I. H. and M. W. Witczak, "Comparison of Marshall and Superpave Level I mix design for asphalt mixes," *Transportation Research Record*, pp. 161, 1995.
10. Burlie, R., L. Uzarowski, and J. Emery, "Evaluation of superpave volumetric mix design technology for urban hot-mix asphalt," London, Ont., Canada, 2000, pp. 308.
11. Sousa, J. B., G. Way, J. T. Harvey, and M. Hines, "Comparison of mix design concepts," *Transportation Research Record*, pp. 151, 1995.
12. Bari, J. and M. Witczak, "New Predictive Models for Viscosity and Complex Shear Modulus of Asphalt Binders: For Use with Mechanistic-Empirical Pavement Design Guide," *Transportation Research Record: Journal of the Transportation Research Board*, vol. 2001, pp. 9-19, 2007.
13. Bari, J. and M. W. Witczak, "Development of a new revised version of the Witczak E Predictive Model for hot mix asphalt mixtures," Association of Asphalt Paving Technologist, White Bear Lake, MN 55110, United States, Savannah, GA, United States 0270-2932, 2006.
14. Dongre, R., L. Myers, J. D'Angelo, C. Paugh, and J. Gudimettla, "Field evaluation of Witczak and Hirsch Models for predicting dynamic modulus of hot-mix asphalt," *Jounal of Association of Asphalt Paving Technologist*, vol. 74, pp. 381-442, 2005.

15. Asphalt Institute, "Performance Graded Asphalt Binder Specification and Testing, Superpave Series No. 1 (SP-1)," Asphalt Institute, Lexington, KY 2003.
16. Asphalt Institute, "Superpave Mix Design, Superpave Series No. 2 (SP-2)," Asphalt Institute, Lexington, KY 2001.
17. NCHRP Project 1-37A, "Guide for Mechanistic-Empirical Design of New and Rehabilitated Pavement structures," National Cooperative Research Program (NCHRP), Washington D.C. Final Report, 2004.
18. Mirza, M. W. and M. W. Witczak, "Development of a global aging system for short and long term aging of asphalt cements," Portland, OR, USA, 1996, pp. 393.
19. Jongepier, R. and B. Kuilman, "Characteristics of the Rheology of Bitumens," *Journal of Association of Asphalt Paving Technologist*, vol. 38, pp. 98-122, 1969.
20. Zeng, M., H. U. Bahia, H. Zhai, M. R. Anderson, and P. Anderson, "Rheological Modeling of Modified Asphalt Binders and Mixtures," *Journal of Association of Asphalt Paving Technologists*, vol. 70, pp. 403-444, 2001.
21. Bahia, H., D. Hanson, M. Zeng, H. Zhai, M. Khatri, and R. Anderson, "NCHRP Report 459: Characterization of Modified Asphalt Binders in Superpave Mix Design," *Transportation Research Board*, 2001.
22. Nam, K., R. Delgadillo, and H. Bahia, "Development of Guidelines for PG Binder Selection for Wisconsin," WisDOT Highway Research Study 0092-01-01, 2004.
23. Haider, S. W., G. Y. Baladi, Q. Aurangzeb, and T. Akram, "Rheology of asphalt binders in Pakistan," Politecnico Di Torino, Torino, Italy 978 88 8202 029 3, 2009.
24. Salomon, D. and H. Zhai, "Ranking Asphalt Binders by Activation Energy for Flow," *Journal of Applied Asphalt Binder Technology*, vol. 2, 2002.
25. Salomon, D. and H. Zhai, "Asphalt Binder Flow Activation Energy and Its Significance for Compaction Effort," 2002, pp. 12–14.
26. Maze, M., "Viscosity of EVA Polymer-Modified Bitumens: Modeling," 1996.
27. Pellinen, T. K., M. W. Witczak, and R. F. Bonaquist, "Asphalt Mix Master Curve Construction using Sigmoidal Fitting Function with Non-linear Least Squares Optimization," American Society of Civil Engineers, Reston, VA 20191-4400, United States, New York, NY., United States 0895-0563, 2004.
28. Robson, G. R., "The Dynamic Mechanical Properties of Bitumen," *Journal of Association of Asphalt Paving Technologist*, vol. 38, pp. 123-139, 1969.
29. Moulthrop, J. S., "Asphalt cement specifications and HMA performance," Denver, CO, USA, 1990, pp. 537.
30. Skok, E. L., Jr. and R. C. Ingberg, "SHRP Long Term Pavement Performance (LTPP) program-summary," *Asphalt paving technology*, vol. 59, pp. 621, 1990.
31. Brown, S. F., J. N. Preston, and K. E. Cooper, "Application of new concepts in asphalt mix design," Seattle, WA, USA, 1991, pp. 264.
32. Jones, D. R. I. V. and T. W. Kennedy, "Asphalt chemistry and its effect on roadway surface conditions," *ASTM Standardization News*, vol. 19, pp. 38, 1991.
33. Kennedy, T. W., R. J. Cominsky, and E. T. Harrigan, "Development of performance-based specifications and AAMAS," Seattle, WA, USA, 1991, pp. 647.

34. Dongre, R., J. W. Button, R. Q. Kluttz, and D. A. Anderson, "Evaluation of Superpave binder specification with performance of polymer-modified asphalt pavements," New Orleans, LA, USA, 1997, pp. 80.
35. Anderson, J. R. and M. R. Thompson, "Characterization of emulsion aggregate mixtures," *Transportation Research Record*, pp. 108, 1995.
36. Bahia, H. U. and D. A. Anderson, "Strategic highway research program binder rheological parameters: background and comparison with conventional properties," *Transportation Research Record*, pp. 32, 1995.
37. Anderson, D. A., D. Maurer, T. Ramirez, D. W. Christensen, M. O. Marasteanu, and Y. Mehta, "Field Performance of Modified Asphalt Binders Evaluated with Superpave Test Methods: I-80 Test Project," *Transportation Research Record*, pp. 60, 1999.
38. Bahia, H. U. and D. A. Anderson, "New proposed rheological properties of asphalt binders: Why are they required and how do they compare to conventional properties," Dallas, TX, USA, 1995, pp. 1.
39. Bahia, H. U. and D. A. Anderson, "Development of the bending beam rheometer; Basics and critical evaluation of the rheometer," Dallas, TX, USA, 1995, pp. 28.
40. Roberts, F. L., "Specification as a requirement for hot mix asphalt," San Francisco, CA, USA, 1989, pp. 81.
41. Roberts, F. L., P. S. Kandhal, and E. R. Brown, *Hot Mix Asphalt Materials, Mixtures Design, and Construction*, 2nd Ed. ed. Lanham, Maryland: NAPA Education Foundation, 1996.
42. Bosscher, P. J., H. U. Bahia, S. Thomas, and J. S. Russell, "Relationship between pavement temperature and weather data. Wisconsin field study to verify Superpave algorithm," *Transportation Research Record*, pp. 1, 1998.
43. Lukanen, E., C. Han, and E. Skok, "Probabilistic method of asphalt binder selection based on pavement temperature," *Transportation Research Record: Journal of the Transportation Research Board*, vol. 1609, pp. 12-20, 1998.
44. Mohseni, A., T.-F. H. R. Center, and P. Systems, "LTPP Seasonal Asphalt Concrete (AC) Pavement Temperature Models," US Dept. of Transportation, Federal Highway Administration, Research and Development, Turner-Fairbank Highway Research Center, 1998.
45. Anderson, D. A., D. W. Christensen, H. U. Bahia, R. Dongre, M. Sharma, C. E. Antle, and J. Button, "Binder Characterization and Evaluation. Volume 3: Physical Characterization," 0309057671, 1994.
46. Kennedy, T. W., G. A. Huber, E. T. Harrigan, R. J. Cominsky, C. S. Hughes, H. Von Quintus, and J. S. Moulthrop, "SHR-A-410 Superior Performing Asphalt Pavements (Superpave)'. The Product of the SHRP Asphalt Research Program," 1994.
47. Huber, G., S. H. R. Program, C. A. National Research Council, NW, and D. U. Washington, "Weather database for the Superpave (trademark) mix design system. Revised edition," 1994.
48. Huber, G. A., "SUPERPAVE Software: A tool for mixture design," Austin, TX, USA, 1993, pp. 529.
49. Kennedy, T. W., S. M. James, and A. H. Gerald, "Development of SHRP Mixture specification and Design and Analysis System," Austin, TX, USA, 1993, pp. 509.

50. Aurangzeb, Q., "Introduction of Performance Based Specifications for Asphalt Binders in Pakistan," *Master of Science Thesis*, Department of Civil Engineering, National Institute of Transportation, National University of Sciences and Technology, Rawalpindi, 2007.
51. Aurangzeb, Q., T. Akram, S. W. Haider, and G. Y. Baladi, "Performance Grading of Asphalt Binders to Control High Temperature Susceptibility in Pakistan," presented at 7th Liverpool John Moore University (LJMU) International conference on sustainable aggregates, asphalt technology and pavement engineering, Liverpool, UK, 2008, pp.
52. Pakistan Meteorological Department, "Climatic normals of Pakistan (1971-2000)," Climatic data processing center, Karachi, Pakistan 2005.
53. Website, <http://en.wikipedia.org/wiki/Weather_Underground_(weather_service)>, 2010.
54. River and Climatic Data of Pakistan, "Precipitation Evaporation, Temperature, Relative Humidity, Solar Radiation and Wind Movement Data (1996-2007)," Surface Water Hyprology Project, Hydrology and Water Management Organization, WAPDA, Lahore, Pakistan. 2007.
55. AASHTO PP 60-09, "Preparation of Cylinderical Performance Test Specimen Using the Superpave Gyratory Compactor (SGC)," 2009.
56. AASHTO T 342-11, "Determining Dynamic Modulus of Hot Mix Asphalt (HMA)," 2011.
57. AASHTO TP 79-11, "Determining Dynamic Modulus and Flow Number for Hot Mix Asphalt (HMA) Using the Asphalt Mixture Performance Tester (AMPT)," 2011.
58. AASHTO PP 61-10, "Developing Dynamic Modulus Master Curves for Hot Mix Asphalt (HMA) Using the Asphalt Mixture Performance Tester (AMPT)," 2010.
59. AASHTO PP 62-10, "Developing Dynamic Modulus Master Curves for Hot Mix Asphalt (HMA)," 2010.
60. Sakhaeifar, M. S., B. S. Underwood, Y. R. Kim, J. Puccinelli, and N. Jackson, "Development of Artificial Neural Network Predictive Models for Populating Dynamic Moduli of Long-Term Pavement Performance Sections," *Transportation Research Record: Journal of the Transportation Research Board*, vol. 2181, pp. 88-97, 2010.