IMPROVEMENT OF BITUMINOUS CONCRETE USING CEMENT AS FILLER MATERIAL

A thesis by

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DECLARATION

It is declared that all the work embodied in this thesis paper is the result of investigation laboratory work and field work carried out by the author.This thesis has been hereby performed by the author under the supervision of **Dr. Md. MizanurRahman, Professor, Department of Civil Engineering, BUET, Dhaka** and has not been submitted elsewhere for any Degree completion.

10.01.2022

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DEDICATION

Dedicated to all MUSLIM Scholars, those contributions are not illuminated to the Modern World.

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May ALLAH help us all to be become honest in our professional career and work for our country with greatest effort.

ABSTRACT

Current development in the design of asphalt concrete especially in the upper layers of flexiblepavements contains about acceptable proportion of mineral fillers, which contributes towards the mixcohesion, resistant to rutting and improves serviceability. It is well recognized that mineral fillers playan important role in the properties of mastics and Hot-Mix Asphalt (HMA) mixtures. Better understanding of the effects of fillers on the properties of mastics and HMA mixtures is crucial to goodmix design and high performance of HMA mixtures. This study presents a laboratory investigation intothe effects of different fillers on some properties of HMA mixtures and also the actual behavior of bituminous material on existing road network.

It has long been recognized the importance of the role of fillers in the hot mix asphalt (HMA) behavior. The filler fills the voids between the coarse and fine aggregates in the mixtures and changes the properties of asphalt binders, because it acts as an active part of the mastic. In the HMA design, the mastic influences the lubrication of the larger aggregates particles and affects the voids in mineral aggregate, the compaction characteristics and the optimum asphalt binder content. The HMA volumetric properties are necessary requirements to ensure a good performance, and these properties are directly influenced by the mixture grading, aggregates surface characteristics and compaction energy.

In this study, 1 (One) filler type (Portland cement),05 (Five) filler contents and 1 (One) loading patterns (Heavy Loads)are used toinvestigate the effect of filler / asphalt ratio on the characteristics of HMA mixtures on Laboratoryusing the Marshall Mix design method. These mixtures were prepared using Portland cement with varying the content by the total mixtureand their effects on Marshall Properties are assessed. Optimum Bitumen Content (OBC) is found as **5.60%**. Different Cement Content (**0.00%,0.50%, 1.00%, 1.50% and 2.50%**) as filler material in mixtures having OBC is 5.625% are collected from secondary source by performing Tensile Strength Ratio (TSR) test. The indirect tensile strength (using Cement Content as filler of **0.50%**) determined by TSR test is **85.03%** which represents the mixture having good ability against moisture damage under a variety of temperatures and stress levels that simulate the conditions of a pavement that is subjected to moving wheel loadsconstruct a road strip so that it could explore the actual scenario of bituminous characteristics.

The Life Cycle Cost Analysis (LCCA) of the typical section of a flexible pavement having the properties obtained from this study represents the two probable results e.g., Deterministic and Probabilistic result. From the Deterministic results, the LCCA method shows that the Flexible pavement (per Km) using 60/70 Bitumen with 0.50% cement has Lowest Present Value Agency Cost. From the RHD schedule of rates, 2018, cement modified pavement needs routine maintenance having BDT 0.80 Lac where as normal pavement needs rehabilitation with cost of BDT 36.86 Lac.

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ABBREVIATIONS

ASTM	American Society for Testing and Materials
BUET	Bangladesh University of Engineering and Technology
BRRL	Bangladesh Road Research Laboratory
CA	Coarse Aggregate
ESAL	Equivalent Single Axle Load
FA	Fine Aggregate
FM	Fineness Modulus
G	Specific Gravity
HMA	Hot Mix Asphalt
HRA	Hot Rolled Asphalt
HDM	Highway Development and Management
LGI	Local Government Institutions
LGED	Local Government Engineering Department
LCCA	Life-Cycle Cost Analysis
MF	Mineral Filler
OBC	Optimum Bitumen Content
PSD	Particle Size Distribution
ROCKS	Roads Cost Knowledge System
RHD	Roads and Highways Department
SMA	Stone Mastic Asphalt
SSD	Saturated Surface Dry condition
TSR	Tensile Strength Ratio
U_C	Coefficient of uniformity
VFA	Voids Filled with Asphalt
VMA	Voids in the Mineral Aggregates
V_b	Bitumen Volume
V_a	Air Voids
WCKD	White Cement Kiln Dust
d_{25}	Density of bitumen at 25°C
ρ_{bit}	Theoretical maximum density of asphalt mix
ρ_A	Density of Asphalt mix
ρ_{min}	Density of aggregate in the blend

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CHAPTER 1

INTRODUCTION

1.1 General

Asphalt is a material comprised of three principal components; aggregate, bitumenand filler. The different ratios between these components give rise to a family of asphalt mixtures with different properties. Asphalts typically include a proportion of air, which isoften considered as the fourth component. Minimizing the quantity of air through propermixture design and adequate compaction during placement of the asphalt is essential forensuring a durable product. Additionally, too little air, as a result of overfilling the aggregatestructure, can lead to rutting.

Fillers modify the properties, increase the performance of, and provide improved durability to composites, polymers, rubbers, adhesives, coatings and construction materials (such as concrete and asphalt). Fillers are used to lower the cost of materials, increase rigidity and give special properties to a material (such as color or fire retardancy). The filler properties have considerable effects on the processing characteristics of materials such as mixing, pumping and compacting. The effects of fillers are therefore of vital importance.

Fillers are typically fine powders with a particle size distribution in the range of 0-100μm. They can be naturally occurring materials such as calcium carbonate, manufacturedfillers for example carbon black, or derived from industrial wastes such as fly ash frompower stations. Other common fillers include silica, kaolin (“China Clay”), mica, feldsparand diatomite.

Asphalt is an essential construction material. The majority of roads are constructed orsurfaced with asphalt. In 2003, close to 300 million tons of asphalt were produced inEurope and 500 million tons in the United States (EAPA, 2004). As an engineeringmaterial, asphalt is typically designed to provide stiffness and bearing capacity, and resist therepeated loading experienced by a pavement under traffic. The effect of repeated loadingmanifests itself in two ways, permanent deformation, commonly referred to as “rutting”, and “cracking” through fatigue of the asphalt.

The most frequently used filler in asphalt is limestone (calcium carbonate), which isderived from the consolidation of minute micro-organisms during the formation of the earth’scrust. Limestone is the general term for rocks where calcite, a form of calcium carbonate, is the predominant mineral. Limestone may also contain a proportion of magnesium carbonate, silica, clays, iron oxides and organic material.

Other materials commonly used as fillers in asphalt include Portland Cement and hydrated lime, which possesses well documented properties with regard to mixture durability and increased resistance to moisture damage in asphalt (Little and Epps, 2001). Additionally,recycled fillers in the form of so-called “baghouse” fines have also been frequently used in asphalt. The performance of baghouse fines was the subject of several key studies on the behavior of fillers in asphalt following changes in the Clean Air Act 1970 in the United States. (Kandhal, 1980; Anderson et al., 1982)

When bitumen is combined with mineral filler, a mastic is formed. This mastic can be viewed as the component of the asphalt mixture that binds the aggregates together and also the component of the asphalt that undergoes deformation when the pavement is stressed under traffic loading. The characteristics of the filler can significantly influence the properties of the mastic, and thus the filler properties can have significant effects on asphalt mixture performance. The importance of fillers, and the resulting mastics, has been studied for a century (Richardson, 1907). Arguably, the key advance in the understanding of asphalt fillers was made shortly after the Second World War when researchers in the UK (Rigden, 1947) proposed that the volume of fixed bitumen that a filler could accommodate in its compacted state influenced the stiffening behavior of fillers in mastics. Rigden also proposed that the volume of “fixed bitumen” that the system could accommodate scaled the effects of a given volume of filler. Since that time, several other researchers have found Rigden’s approach reasonably successful in predicting the stiffening effect of various filler types in a given bitumen type (Heukelom and Wijga, 1971; Kandhal, 1980; Anderson et al., 1992a, 1992b; Kavussi and Hicks, 1997; Kandhal et al., 1998; Cooley et al., 1998).

1.2 Road Network of Bangladesh

In November 2003, the Government has made a change to earlier road classification system anddelineated the ownership/responsibility of each category of roads for their improvement andmaintenance (Bangladesh Gazette volume-1, 2003). The new definition classifies the road systeminto six main categories. The road type/category, definition and, ownership and responsibility arelisted in **Table 1.1**.

Table 1.1: Type, Definition and Ownership of Roads in Bangladesh

Sl. No	Type	Definition	Ownership/ Responsibility
1	National Highway	Highways connecting National capital with Divisional HQs or seaports or land ports or Asian Highway.	RHD
2	Regional Highway	Highways connecting District HQs or main river or land ports or with each other not connected by National Highways.	RHD
3	Zila Road	Roads connecting District HQ/s with Upazila HQ/or connecting one Upazila HQ to another Upazila HQ by a single main connection with National/Regional Highway, through shortest distance/route.	RHD
4	Upazila Road	Roads connecting Upazila HQ/s with growth Centre/s or one Growth Centre with another Growth Centre by a single main connection or connecting Growth Centre to Higher Road System**, through shortest distance/route.	LGED*/ LGI*
5	Union Road	Roads connecting Union HQ/s with Upazila HQs, growth centers or local markets or with each other.	LGED*/ LGI*
6	Village Road	a) Roads connecting Villages with Union HQs, local markets,farms and ghats or with each other. b) Roads within a Village.	LGED*/ LGI*
-The roads under the jurisdiction belong to the Pourashavas and the City Corporations have not beenincluded in the list above. - *LGED-Local Government Engineering Department, RHD-Roads and Highways Department, LGI -Local Government Institutions. ** Higher Road System - National Highway, Regional Highways, and Zila Roads.			

Source: Published in the Bangladesh Gazette, volume 1-6, November, 2003

RHD road network comprised of National Highways (N), Regional Highways (R) & ZillaHighways (Z). At present RHD has 21,483.21 km road network. The Classification of the roadsalong with their length and number is shown in the following:

Road Network

Road Length by Classification		
National Highway	=	3,544.06 Km
Regional Highway	=	4,280.02 Km
Zilla Road	=	13,659.13 Km
Total Road Length	=	21,483.21 Km

Road Length by Surface Type according to latest survey		
Bituminous	=	16,815.61 Km
Earth	=	698.47 Km
HBB	=	660.81 Km
Cement Concrete (CC)	=	2.44 Km
Cement Blocks	=	0.37 Km
Total Paved Road Length	=	17,516.89 Km
Total Unpaved Road Length	=	660.81 Km
Total Surveyed Road Length	=	18,177.70 Km
Length of Road Not Surveyed	=	3,305.51 Km

Source: http://www.rhd.gov.bd/Databases/Databases.asp

The map of the current RHD road network is shown in **Figure 1.1**

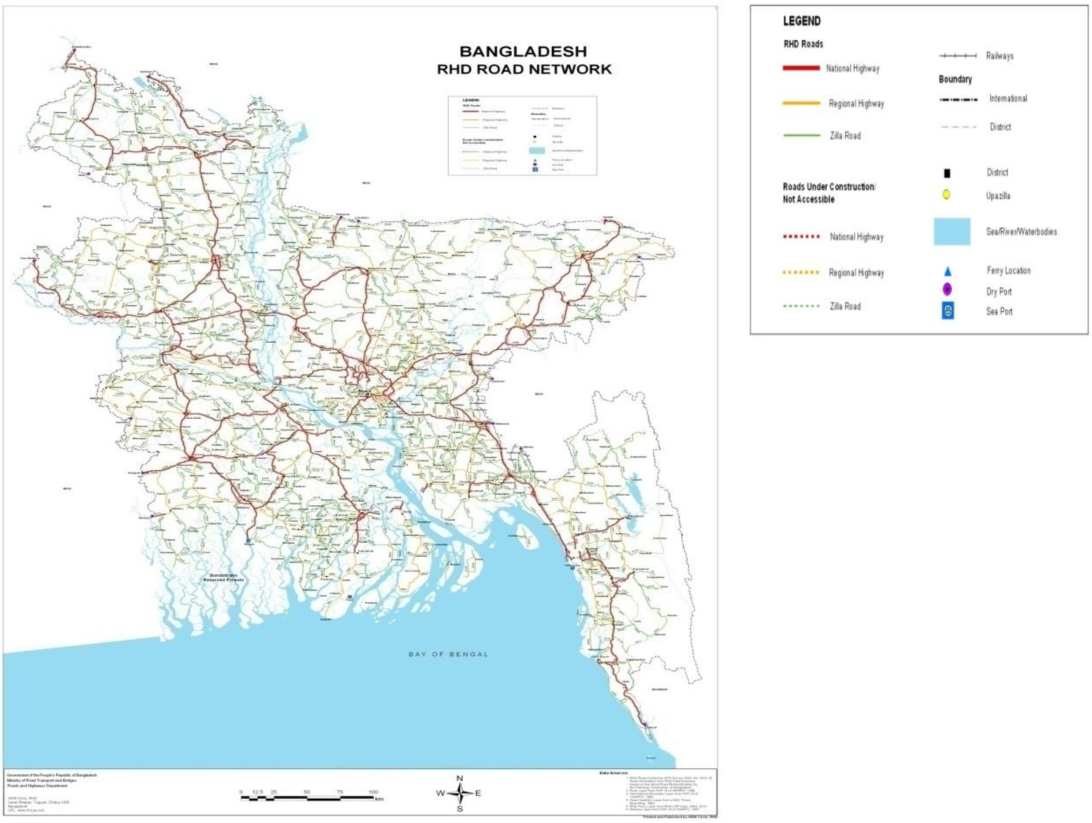


Figure 1.1:RHD Road Network

1.3 Problem Statement

Bitumen is an oil-based substance. Mostly used in road pavements and airports. Approximately, 85% of all bitumen produced is used as the binder of asphalt for roads. A road has different layers like from bottom Subgrade, Sub-Base, Base, Binder course and surface course (pavement). Every part of flexible road contains different materials like Subgrade portion is the soil part. Its thickness can be determined by CBR test of Subgrade's soil. Sub-base is the optional layer which is needed when Subgrade is in weak condition. The main load spreading part is Base layer which consists of crushed stones or over burnt brick chips in Bangladesh. The most critical and important part of the flexible pavement is bituminous surfacing. The performance of Subgrade and Base is dependent on the durability of this layer. So, the selection of type of surfacing and materials for this layer is to be done very carefully. In Bangladesh, following types of surfacing are being used:

I. Densely graded Bituminous Carpeting (BC)
II. Double Bituminous Surface Treatment (DBST)
III. Seal Coat (7mm/ 12mm)

Increase of vehicle numbers produces more traffic loads and variation of weather deteriorates the binder property which causes damage the pavement of the roads. To prevent damage, many modifier or admixture used like Ethylene Vinyl Acetate (EVA), propylene, plastic, rubber, cement etc. In many countries, bitumen is known as "asphalt cement" or "asphalt". On the other hand, "asphalt" is the term used for a mixture of small stones, sand, filler and bitumen. The asphalt mixture contains approximately 5% bitumen. Mineral filler is a mineral material, inert to the other components of the asphalt mixture, finely divided, at least 65% passing the sieve opening of 0.075 mm square mesh. The volumetric properties of asphalt mixtures are commonly used to ensure proper performance of pavements. Numerous studies have shown that the properties of mineral filler (especially the material passing No. 200 sieve) have a significant effect on the properties of the HMA mixtures.

1.4 Background of the Study

A maximum filler / asphalt ratio of 1.2 to 1.5, based on weight, is used by many agencies to limit the amount of the minus 200 material. However, the fines vary in gradation, particle shape, surface area, void content, mineral composition, and physico-chemical properties and, therefore, their influence on the properties of HMA mixtures also varies. Therefore, the maximum allowable amount should be different for different fines. (Kandhal et al, 1998).

The addition of filler to the mixture can improve adhesion and cohesion substantially (filler is a fine material, which passes a 0.063 mm sieve, derived from aggregate or other similar granular material). The bitumen-filler system (mastic), which is thicker and tougher than bitumen alone, improves the adhesive qualities and, in providing a covering film of greater thickness, also means that the aging processes can be slowed down. The effects of the addition of filler are directly related to their characteristics and the degree of concentration of the filler in the bitumen-filler system.

1.5 Objective of the Study

The objectives of the study is to investigate the effects of quality andquantity of mineral filler (Portland cement) to asphalt ratio on the design properties and performance of hot mix asphalt and give recommendation.The aim of this study is to assess the possibility of using Portland cement as filler materials in Asphalt Binder Course under the local conditions in the Bangladesh road environment.

The objectives of this study will be

I. To assess the characteristics of bituminous materials.

II. To develop an optimum percentage of bitumen and cement in bituminous mixtures.

III. To investigate the submerged strength of bituminous mixtures using optimum percentage of bitumen and cement by TSR test.

IV. To compare the maintenance cost between bituminous mixtures with cement and without cement of a typical flexible pavement.

Research Outcome:

In this study, an optimum percentages of bitumen and cement in bituminous mixtures will be developed by Marshall Mix Design method. It is expected that this study would bring the outcomes by improving binder strength of bituminous mixtures against moisture susceptibility effect especially on submerged condition. If successfully implemented, this study is quite useful in developing an appropriate technology for improving bituminous characteristics which will represent an example of the so-called **“moisture susceptible bituminous mixtures”**. Above all, this study will also cover the comparison of maintenance cost between bituminous mixtures with cement and without cement of a typical flexible pavement.

1.6 Importance of the study

i. Using Portland cement as filler in order to improve the properties of asphalt pavement.
ii. Enhancing binder strength of bituminous materials.
iii. Improve bituminous characteristics against weather susceptibility effect.

1.7 Limitations of the study

Results of this study relied on a set of limitations and criteria that were taken into account during the experimental work. These limitations include:

I. Only Portland cement was used in this study and other types of mineral filler such as lime stone dust, white cement Kiln dust (WCKD), hydrated lime and etc. are not within the concern of this research study.

1.8 Methodology of the study

To achieve the study aim and objectives, the following methodology has been considered:

I. Literature review and revision of accessible references such as books, studies, scientific papers, reports in the field of mineral filler materials and researches relative to the topic of this research.
II. Study of asphalt mix design criteria, asphalt production technology and local and international specifications.
III. Implementation of laboratory tests to identify the Optimum Bitumen Content (OBC) and Optimum Cement Content (OCC)using Marshall Mix design procedure for binder course.
IV. Collect Coarse Aggregates (Pakur Stone Chips), Fine Aggregates (Sylhet Sand), Bitumenand Mineral Filler (Portland cement) till reaches the required gradation. (Performed the all routine test for Coarse Aggregates, Fine Aggregates, Mineral Filler and Bitumen individually in the laboratory)
V. Preparation of a series of specimens composed of different percentage of Mineral Filler (Portland cement) content. (There are 02 (Two) types of sample used in this study. One type consists of bitumen,

Mineral Filler and aggregates. Another type consists of bitumen, Mineral Filler, cement and aggregates. 5 (Five) or more samples for both types with different percentages of bitumen and cement are prepared to investigate properties in various condition according toMarshall Mix Design method.)

VI. Testing the specimens which contain different percentages of Bitumen having same Mineral Filler and identify Optimum Bitumen Content (OBC)according toMarshall Mix Design method. Testing anotherspecimens with different percentages of Mineral Filler (including Portland cement) having the OBC and identify Optimum Cement Content as filler (OCC) according to Marshall Mix Design method. Comparing each result with the OBC and OCC in terms of the mechanical properties (Unit Weight, VMA (Voids filled with Mineral Aggregates), VFA (Voids Filled with Asphalt), Flow value, Stability and asphalt content etc).

VII. For Tensile Strength Ratio (TSR) testing, 5 (Five) or more samples with theoptimum percentage of cement and bitumen previously determined are used to show the weather susceptibility behavior of modified bituminous materials characteristics. The resilient modulus determined by TSR test characterizes the pavement construction materials under a variety oftemperatures and stress levels that simulate the conditions of a pavement that is subjected to moving wheel loads.

VIII. Cost Analysis of Road maintenance in Bangladesh. (The life cycle cost (including Construction and Maintenance) of a typical road (suggested by Road Note-29) is determined with examined optimum bitumen and cement as filler percentages using current schedules of rates for Road construction and maintenance of RHD roads.)

IX. Analysis & discussion of testing results.

X. Drawing conclusions and recommendations.

1.9 Thesis structure

The thesis consists of six chapters and sevenappendices. A brief commentary of the chapters' contents is presented below:

Chapter 1: Introduction

This chapter contains a general introduction is followed by statement of problem, aims and objectives, limitations, methodology of research and finally the thesis structure.

Chapter 2: Literature review

This chapter covers a general literature review related to hot mix asphalt, bituminous materials, different types of mineral filler, cost analysis of road construction in developing countries and previous researches relevant to the topic of research.

Chapter 3: Materials and experimental program

This chapter highlights two topics; the first one is the experimental evaluation of used materials properties such as aggregates, and bitumen and waste crushed glass. The second is the explanation of experimental work which has been done to achieve the aim of the study.

Chapter 4: Data analysis and Results

This chapter contains the accomplished results of the laboratory tests. Briefly tests are conducted to obtain the asphalt binder course gradation curve, bitumen properties, optimum bitumen content (OBC) and the effect of adding different percentages of Portland cement on asphalt mix properties. More over determine the best glass content percentage.

Chapter 5: Cost analysis of road maintenance inBangladesh

The cost analysis of road maintenance in Bangladeshis discussed in this chapter.The Life Cycle Cost Analysis (LCCA) of a typical road (suggested by Road Note-29) using Real Cost 2.5 software with different bitumen grades is determined with examined optimum bitumen (OBC) and cement percentages (OCC) using currentschedules of rates for Road construction and maintenance of RHD roads.

Chapter 6: Conclusion and recommendations

The conclusions and recommendations of this study are presented in this chapter.

CHAPTER 2

LITERATURE REVIEW

2.1 General

Asphalt is an essential construction material. The majority of roads are constructed or surfaced with asphalt. In 2003, close to 300 million tonnes of asphalt were produced in Europe and 500 million tonnes in the United States (EAPA, 2004). As an engineering material, asphalt is typically designed to provide stiffness and bearing capacity, and resist repeated loading as experienced by a road under traffic. The effect of repeated loading manifests itself in two ways; permanent deformation, commonly referred to as “rutting”, and “cracking”, as a result of fatigue.

Additionally, asphalt mixtures are designed to minimize the effects of water on the system by either making the mixture dense and relatively impenetrable to moisture, for example in the case of asphaltic concrete, or by adding sufficient bitumen to the mixture to provide a thick coating of bitumen on the aggregates (referred to as “bitumen film thickness”) as in the case of porous asphalt and thin wearing course mixtures.

Road pavements are typically constructed in layers, with each layer of the pavement fulfilling a slightly different function. The surface layers of the road are subjected to the highest stresses in the pavement as they are in direct contact with vehicle tires, and additionally the surface is exposed to the elements which results in the surface of the pavement reaching the highest temperatures and are subject to the highest stresses.

Typically surfacing layers are made from the highest quality components and require excellent resistance to permanent deformation. This uppermost layer of the pavement provides the running surface for the vehicles, hence, ride quality and skid resistance are key properties of this layer.

Below the surfacing layer is the main structural layer(s) of the road. This layer, referred to simply as the “base”, is designed to spread the vehicle loading to a level that can be withstood by the platform of the road pavement, which is typically constructed of unbound aggregates or consists of the naturally occurring ground conditions.

Asphalt is a material comprised of three principal components; aggregate, bitumen and filler. Air is also present in asphalt mixtures to varying extents. The different ratio between these components gives rise to a family of asphalt mixtures with different properties.

Very, broadly speaking there are four types of asphalt mixtures used in road construction:

"Asphaltic Concrete", referred to in the UK as "Macadam", has a maximum aggregate particle size of around 30mm and a continuous particle size distribution (a wide range of particle sizes, well distributed). Asphaltic Concrete is typically used for the structural base layers of a road, although they are also used for the surfacing layers.

"Stone Mastic Asphalt" (SMA) or "Thin Wearing Courses" (TWC), have very high quantities of aggregate larger than 2mm in size (circa 75% by mass). These types of material are typically used for the surfacing of the pavement.

"Hot Rolled Asphalt" (HRA) was used extensively in the UK for several decades but usage has reduced dramatically in recent years due to introduction of "Stone Mastic Asphalt" type materials referred to above. SMA type materials provide superior resistance to permanent deformation and lower noise under traffic due to the high coarse aggregate content. HRA is typically used for the surfacing of a road but can also be used for the base layers. The coarse aggregate content of HRA can vary according to the application, but for surfacing layers, around 30% of the aggregate is greater than 2mm in size. This makes HRA the inverse, in terms of particle size distribution, of SMA type mixtures.

"Porous Asphalt" is a surfacing material designed to achieve a high air void content to allow water to pass through the material and reduce spray under trafficking. The high void content is achieved by limiting the amount of fine aggregate in the mixture.
Typically, the air void content of porous asphalt is around 20%.

A summary of the typical components of the four main groups of asphalt is given in **Table 2.1** below.

Table 2.1: The main types of asphalt mixtures used in road pavements and a summary of their components.

Mixture Type	Coarse Aggregate PSD type/ %>2mm particle size	Bitumen Content by mass (%)	Filler Content by mass (%)	Air content (%)
Asphaltic Concrete	Continuous 65%	4-5	2-10	3-6
Stone Mastic Asphalt	Discontinuous 75%	5-6	6-8	2-6
Hot Rolled Asphalt	Discontinuous (35-50)%	7-8	10-12	2-6
Porous Asphalt	Single size 90%	4-5	2-4	18-25

2.2Principal components in Asphalt Mixture

As stated previously asphalt mixture is composed of three principle constituents; aggregates, bitumen and filler. Each component will be discussed in turn.

2.2.1 Aggregates

Aggregate is the term used to describe mineral materials such as gravel, sand and crushed rock. Simplistically, aggregates can be considered as the solid particles in an asphalt mixture. Aggregates provide a structural skeleton to asphalt mixtures and it is this structure that provides mechanical strength to the asphalt. Additionally, because the aggregate constitutes the solid surface of the asphalt mixture, and this surface largely governs the durability of the mixture in the presence of water, aggregate type is very important when considering the durability of asphalt mixtures (Cheng, 2002; Hefer 2004; Choi, 2005; Bhasin, 2006)

An aggregate for use in asphalt needs to possess several fundamental properties:

I. Hardness, toughness
II. Resistance to abrasion
III. Resistance to polishing to provide skidding resistance (for surface aggregates)
IV. Durability with regard to frost action and de-icing salts used on road surfaces
V. Good affinity to bitumen

The requirements for aggregates and the test methods used to characterize aggregates used in asphalt have been standardized in Europe (BS EN 13043, 2002). Aggregates commonly used in asphalt include limestone, granite, amphibolite, diorites, basalt and gneiss. Additionally, recycled aggregates such as crushed glass and secondary aggregates, such as slag from iron or steel production, are also commonly used.

2.2.2 Bitumen

Bitumen is defined in the Oxford English Dictionary as “a tar-like mixture of hydrocarbons derived from petroleum naturally or by distillation”. Bitumen acts as a binder in asphalt and binds, or “cements”, the aggregate particles together. There are several types of bitumen and the rheological properties of the bitumen reflect:

I. The source of the bitumen (the type of crude from which the bitumen is manufactured)
II. The manufacturing process of the bitumen
III. The addition of polymers or additives

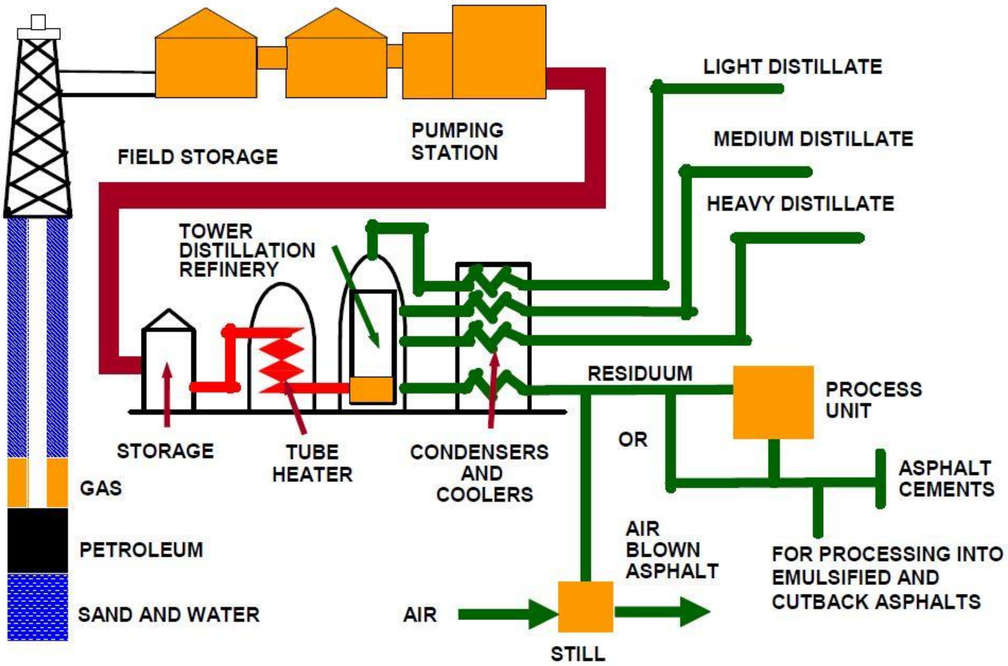
Bitumen is a complex mixture of components and as a result, bitumen is considered a material with a complex response to stress. The response of a bitumen to stress is dependent on both loading time (frequency) and temperature, and it is this behavior which characterizes the mechanical behavior of asphalt mixtures (Read and Whiteoak, 2003).

Bituminous Materials are used for highway construction because of excellent binding & cementing power, water-proofing properties, relatively low cost. Bitumen is a Black or dark colored solid or viscousCementous substances composed of high molecular weight hydrocarbons. Bitumen is soluble in carbon disulfide (CS_2).

Refinery Operation

Asphalts are the residue, byproducts of the refinery ofpetroleum oils.Depending on the sources & characteristics of thecrude oils & on properties of asphalt required morethan one processing method may be employed.Consistency can be controlled by the mount of heavygas oil removed.Consistency can be further modified by air blowing.Air blowing is used to increase viscosity of asphaltresidue. The Refinery process of crude petroleum is shown below:

Figure 2.1: Refinery Operation of Crude Petroleum (https://www.e-education.psu.edu/eme801/node/470)

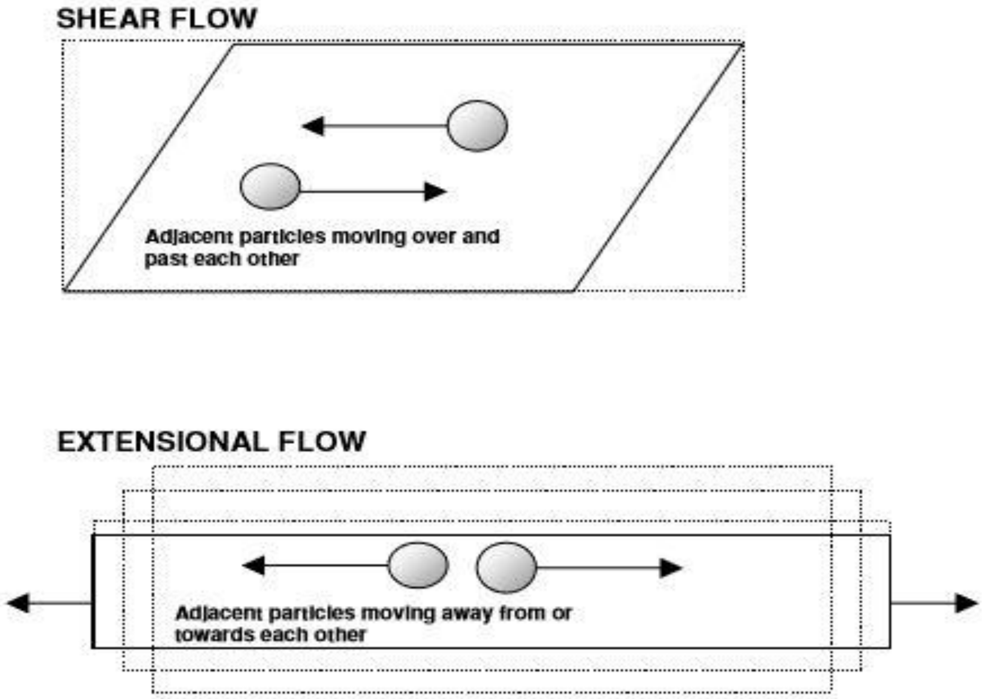


The asphaltic materials obtained from thedistillation of petroleum are:

1. **Asphalt Cement (AC):** HMA in pavement base and surface in highways, air ports. Parking etc.
2. **Slow-Curing (SC):** cold laid and mix in place.
3. **Medium-Curing (MC):** Mixed in place and surface treatment.
4. **Rapid-Curing (RC):** Mixed in place and surface treatment.
5. **Blown Asphalt:** Relatively stiff and not used as paving materials. Suitable as roofing material, automobile undercoating, and as joint filler for concrete pavements.
6. **Asphalt Emulsions:** Mixed in place and surface treatment.

In order to discuss the rheological behavior of bitumen it is necessary to first briefly discuss the rheology of liquids. Rheology describes the interrelation between force, deformation (flow) and time. Flow is typically described in two forms; shear flow and extensional flow (see **Figure 2.2**). In shear flow the molecules within a liquid can be considered as flowing over and past each other, whereas in extensional flow the molecules can be thought of as flowing apart (Barnes, 2000).

Figure 2.2: Shear Flow and Extensional Flow (After Barnes, 2000)



Viscosity is the term used to describe the resistance of a liquid to flow. To make a liquid flow a force must be applied. For shear flows, the rate at which this force is applied to the liquid is termed the shear rate and the magnitude of the shear referred to as the shear stress. Liquids are considered "Newtonian" when the viscosity remains constant over changes in shear stress. This means that the viscosity remains constant, regardless of the applied shear stress. Whereas, liquids that exhibit a change in viscosity with a change in the shear rate are known as "Non-Newtonian" fluids. These terms, frequently used in rheology, are given in **Table 2.2**.

Table 2.2: Frequently used terms in rheology.

Quantity	Symbol	Units
Shear strain	γ	-
Shear rate ($d\gamma/dt$)	γ	s^{-1}
Shear stress (Force/Area)	σ	Pa
Shear viscosity (σ/γ)	η	Pa.s

At high temperatures, (>80°C) bitumen behaves as a liquid and is typically characterized by viscosity and the viscosity governs the acceptable temperatures for manufacture and placing of asphalt mixtures. At low temperatures (<-5°C), bitumen behaves essentially as a solid material. Tests for low temperature behavior typically assess cracking or ductility. At intermediate temperatures (-5°C to 80°C), which covers the range in which asphalt pavements is under traffic, bitumen behaves as a "visco-elastic" solid and has properties between that of an ideal solid and a liquid.

Several methods have been developed to measure the intermediate temperature behavior of bitumen and these range from simple index tests, carried out at single temperatures, to complex detailed analysis of the bitumen behavior over a range of loading frequencies and temperatures.

Historically, two empirical tests dominate the simple characterization of bitumen, Needle Penetration (BS EN 1426:2000) and Ring and Ball Softening Point (BS EN 1427:2000). Since the 1990's, the Dynamic Shear Rheometer (DSR) has rose to prominence in the fundamental rheological characterization of bituminous bitumen. (Kennedy et al., 1990).

The penetration test is probably the most commonly used quality control test for bitumen in the world. This test involves the penetration of bitumen by a needle of known load and dimensions, at a fixed temperature (25°C) and loading time (5 seconds). The result is reported in units of tenths of a millimeter (dmm). Soft bitumen has high values of penetration, whereas hard bitumenhas low values. Road paving bitumen can be thought of as intermediate in terms of penetration value. This bitumen typically has a penetration between 50 and 200dmm, although both harder and softer grades are used according to the climate. For example, in France and the United Kingdom, hard 10/20dmm penetration bitumen are used (Sanders and Nunn, 2005), whereas in cold countries such as Canada much softer grades are commonly employed to resist the effects of the very cold climate.

The softening point test consists of the placing of a steel ball of mass 3.5g onto a brass ring filled with bitumen. The test specimens are then suspended in water, or glycerol, and heated at a rate of 5°C per minute. At the moment the ball drops through the ring and hits a plate 25mm below the ring, the temperature is noted and reported as the softening point of the bitumen.

As previously stated, bitumen is a complex material with a complex response to loading time and temperature, and at normal pavement temperatures the bitumen has properties that are in the visco-elastic region. There has been a considerable increase in more complex assessment of bitumen with the aim of better characterizing the rheological behavior of bitumen and the resulting behavior of the asphalt mixture. Much of this change was bought about by the climate and loading condition based “Performance Grade” classification of bitumen resulting from the US Strategic Highway Research Program (SHRP) of the 1990’s (Kennedy et al., 1990).

SHRP introduced the Dynamic Shear Rheometer (DSR) as a key tool in the Dynamic Mechanical Analysis (DMA) of bitumen. Various testing geometries, such as cone and plate, parallel plates and cup and plate, can be used in dynamic mechanical testing. Forman materials cone and plate geometry is preferred as shear stress and shear rate are constant over the entire area of the plate, thereby simplifying calculations and giving accurate fundamental rheological properties. However, for bitumen testing parallel plate geometry is almost invariably used to avoid the very small gap present at the center of the cone and plate geometry.

In general, two testing (plate) geometries are commonly used with the DSR, namely an 8 mm diameter spindle with a 1- or 2-mm testing gap and a 25 mm diameter spindle with 1mm testing gap. The selection of the testing geometry is based on the operational conditions with the 8 mm plate geometry generally being used at lower temperatures (5°C to 20°C) and the 25 mm geometry at intermediate to high temperatures (20°C to 80°C). However, it is possible to use the same testing geometry over a wide temperature range, although the precision of the results may be limited as a result of compliance errors and the reduction in precision with which the torque can be measured at low stress levels.

Small-strain oscillatory measurements and creep compliance tests can both be carried out using a DSR but assess different characteristics of the bitumen. Oscillatory tests are typically carried out in the linear visco-elastic region of the bitumen and represent the bitumen behavior when the bitumen is subjected to small strains. A test for measuring the behavior of bitumen in the linear visco-elastic region over a range of temperatures and frequency has been standardized in Europe (EN 14770, 2005).

This method produces a master curve, using the time-temperature superposition principle, of the bitumen's response to small strains as measured in the linear visco-elastic region. A DSR can also perform creep tests and the rheological behaviour of bitumen can also be defined in terms of its creep compliance. At the surface of the road, especially at high ambient temperatures, higher strains are experienced and creep compliance tests may be better representative of the conditions under which the bitumen is loaded than small strain oscillatory tests (Delgadillo et al. 2006).

To determine creep properties, material is subjected to prolonged loading and creep compliance is obtained by applying a constant stress and measuring the resulting time dependent strain.

Compliance is defined as:

$$\mathbf{J(t) = \gamma(t)/\sigma 0}$$

Where

J(t) = creep compliance, a function of time and temperature

γ(t) = time dependent strain

σ0 = constant stress

There has been an increase in interest in the United States and Europe in the measurement of creep and recovery characteristics of bitumen, and its relationship to permanent deformation characteristics of asphalt mixtures, and standard procedures to measure such properties have been developed. (NCHRP Publication 459, 2002; Collop et al., 2002; Taherkani and Collop, 2006; Delgadillo et al. 2006).

In summary, at high temperatures, (>80°C) bitumen behaves as a liquid and typically is characterized by viscosity whereas at low temperatures (<-5°C), bitumen behaves essentially as a solid material. The intermediate temperatures (-5°C to 80°C), represent the temperatures at which the bitumen in asphalt pavements is under traffic, and at these temperatures bitumen behaves as a "visco-elastic" solid and has properties between that of an ideal solid and a liquid.

Several methods have been developed to measure the intermediate temperature behavior of bitumen. These range from simple index tests carried out at single temperatures up to complex detailed analysis of the bitumen behavior over a range of frequencies and temperatures.

2.2.3 Asphalt fillers

Fillers modify the properties, increase the performance of, and provide improved durability to composites, polymers, rubbers, adhesives, coatings and construction materials. Fillers are used to lower the cost of materials, change processing characteristics and increase rigidity. Additionally, fillers can be used to give special properties to a material such as fire retardancy, electrical or magnetic properties. Fillers are used for cost reduction, density, color, surface properties (such as controlling stickiness) and thermal properties (for example conductivity).

In general terms, fillers are typically fine powders with a particle size distribution in the range of 0 - 100μm. They can be naturally occurring materials such as calcium carbonate (Limestone), manufactured fillers, for example carbon black, or derived from industrial wastes such as fly ash from power stations. Other common fillers include silica, kaolin ("China Clay"), mica, feldspar and diatomite.

Fillers in asphalt can be defined as "finely divided mineral matter such as rock dust, slag dust, hydrated lime, hydraulic cement, fly ash or other suitable matter" and typically this definition refers to the size fraction smaller than 75μm or 63μm. Fillers in asphalt are used to obtain increased stiffness or rigidity, reducing creep (permanent deformation), increase density and lower the cost of asphalt mixtures. Too much filler in asphalt mixtures can lead to cracking or fatigue problems as the stiffness is increased. Too little can lead to "bleeding" of bitumen from the mixture (Kandhal, 1980; Anderson et al. 1982).

The most frequently used filler in asphalt is limestone (calcium carbonate), which is derived from the consolidation of minute micro-organisms during the formation of the earth's crust. Limestone is the general term for rocks where calcite, a form of calcium carbonate, is the predominant mineral. Limestone may also contain a proportion of magnesium carbonate, dolomite, silica, clays, iron oxides and organic material.

Other materials commonly used as fillers in asphalt include Portland Cement and hydrated lime, which possesses well documented properties with regard to mixture durability and reduced potential for moisture damage in asphalt (Little and Epps, 2001). Additionally, recycled fillers in the form of so-called "baghouse" fines are frequently used. The performance of baghouse fines was the subject of several key studies (Kandhal, 1980; Anderson et al., 1982) on the behavior of filler in asphalt.

Kavussi and Hicks (1997) proposed that in order to provide satisfactory properties in the finished asphalt, filler should:

- Not have adverse chemical reactions with bitumen
- Not possess hydrophilic surfaces to ensure good adhesion
- Not possess high porous particles which may lead to excessive stiffening through selective adsorption
- Contain a dense (well graded) Particle Size Distribution

With regard to the specification of fillers, each European country has its own specifications and National Guidance Documents that list the requirements, test methods and acceptable limits. In the UK, the asphalt mixture specifications BS 4987 and BS 594, together with the National Guidance Document, PD 6682-2, Aggregates for asphalt and chippings, and contract specifications, such as the Highways Agency, Specification for Highway Works, are used to specify properties for mineral fillers.

Geometrical properties

1. Particle size distribution (Air-jet sieving)
2. Requirement for harmful fines - Methylene blue value

Physical and Mechanical

1. Water content
2. Particle density
3. Stiffening properties - Voids of dry compacted filler (Rigden voids)
4. Stiffening properties - “Delta Ring and Ball”
5. “Bitumen number”
6. Loose bulk density in kerosene

Chemical

1. Water solubility
2. Water susceptibility
3. Calcium carbonate content of limestone filler
4. Calcium hydroxide (hydrated lime) content of mixed filler
5. Loss on ignition of coal fly ash
6. Loss on ignition of blast-furnace slags

Surface area - fineness

1. Blaine test (Blain specific surface)

Although several test methods are specified, the performance level for each test is a National concern and EU member states have National Guidance Documents that outline the requirements for the asphalt filler. In the UK, the specification for fillers is not onerous and consists of minimal requirements, namely particle size distribution and filler type is specified, for example Limestone or Portland cementis permitted.

Physical Properties of Filler

Each type of filler has different characteristics that influence the end properties of the finished products in which they are used. Aside from their chemical composition, fillers are traditionally characterized by their particle size distribution, shape, particle packing, surface area and surface activity (Wypych, 1999). Some of the key properties of fillers and their means of measurement are discussed in turn in the following section.

Particle Density

The particle density of fillers is typically measured in the classical way by measuring the volume of particles by displacement of water, or other liquids, using Archimedes' principle. The particle density is then calculated by:

$$\mathbf{p}_{\mathbf{particle=}} \mathbf{:m}_{\mathbf{particles}}/\ \mathbf{v}_{\mathbf{particles}}(\mathrm{g/m^3})$$

Where

$\mathbf{m}_{\mathbf{particles}}$ = the mass of particles (g)

$\mathbf{v}_{\mathbf{particles}}$= the volume of particles (m^3)

Particle density of fillers in all applications can cover a significant range, from hollow glass or ceramic beads with densities of around 0.1 - 0.2 g/cm3 up to above 9 g/cm3 for metal powder based fillers (Wypych, 1999). For asphalt fillers, the range is relatively narrow as most filler are derived from natural aggregates. Typically, asphalt fillers have particle densities in the range 2.65 - 2.75 g/cm3. Fillers used in asphalt that are not derived from natural aggregates have a wider range of particle density, for example Portland Cement 3.15 g/cm3 and Hydrated Lime, 2.30 g/cm3.

Particle Size Distribution

Particle size distribution (PSD) describes the range of particles found within a substance. Particle size distribution is typically expressed as a percentage passing a particular size. PSD is normally depicted graphically, either by percentage mass or volume of particles within a particular size range (See **Figures 2.3 and 2.4**).

Figure 2.3: A typical particle size distribution curve, represented as % by mass passing a particular particle size.

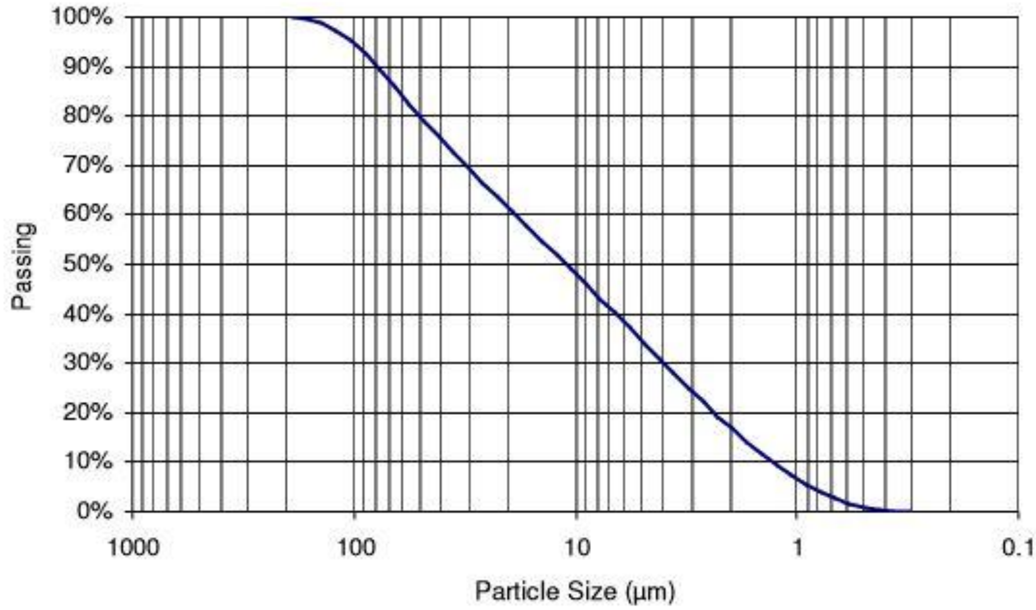
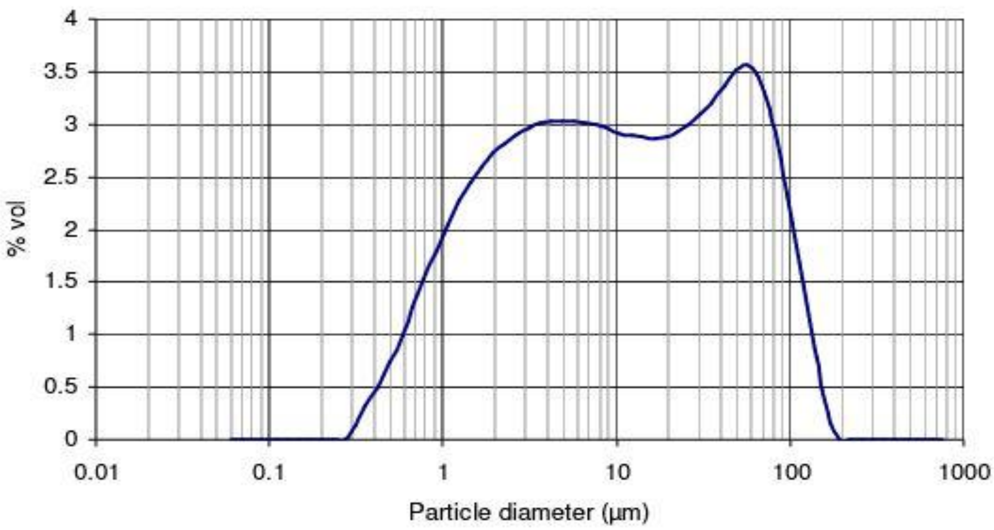


Figure 2.4: Identical data as that in **Figure 2.3**, presented as volumetric particle size distribution.



Volumetric particle size distribution and particle size distribution by mass can be inter-converted by making assumptions of the density and shape of the particles. Typically in such calculations the particles are taken to be spherical.

In order to allow more convenient comparisons between two or more sets of particle size distribution data, mathematical descriptors of PSD curves such as Fineness Modulus and Coefficients of Uniformity are sometimes used. The Coefficient of Uniformity gives an indication of the range of sizes within a particle size distribution. A low coefficient of uniformity indicates a PSD with a small range of particles. It is typically calculated as follows:

$$\mathbf{U_C = D_{60}/D_{10}}$$

Where

$\mathbf{U_C}$= Coefficient of uniformity

$\mathbf{D_{60}}$ = the size (μm) at which 60% of particles are smaller than (by mass)

$\mathbf{D_{10}}$ = the size (μm) at which 10% of particles are smaller than (by mass)

Fineness modulus is an empirical mathematical description of the fineness of a material and is typically derived by taking the sum of the percentage of material passing different sizes. The higher the value of fineness modulus, the finer the material. This approach has been adopted for asphalt fillers (Kandhal et al., 1998; Harris and Stuart 1998) and the Fineness Modulus of asphalt fillers has been calculated using the following formula:

$$\mathbf{FM= (P_{75}+P_{50}+ P_{30}+ P_{20}+ P_{10}+P_{3}+P_{1})/100}$$

Where

FM = Fineness Modulus

$\mathbf{P_x}$ = the percentage passing diameter size x by mass, where x = μm

There are several methods that can be employed to produce particle size distribution information for fillers. It is important to note that different techniques lead to different results as a result of the test method employed. Additionally, the conditions of the test method may differ from the situation in which the filler is going to be used making the link between particle size distribution and filler performance difficult to make.

For example, in laser diffraction measurements, the fillers are typically dispersed in a liquid. The conditions of the filler in the liquid may be very different to the conditions in the product formed when the filler is mixed with the liquid phase of the product. Additionally, mechanical (such as ultrasound) and chemical means of dispersing the filler during testing may lead to better (or worse) dispersion than in real life situations, another factor which makes linking particle size distribution to product performance problematic.

For many materials, sieving is a simple technique for producing PSD data. However, for very fine materials, such as fillers, this method has limitations. When particles become very small the electrostatic interactions between the particles causes agglomerations. Additionally, the particles become attracted to the sieves causing bridging across the apertures known as "blinding" which prevents further particles from passing through the sieve.

Air-jet sieving has been developed to overcome the effects of blinding. This technique can be used to measure the PSD of particles larger than 20μm. The method has been standardized in Europe (BS EN 933-10, 2002). The particles finer than 20μm are a major factor governing the behavior of fillers as they possess the highest surface area, thus air-jet sieving has a significant limitation in measuring the coarser filler particles distribution only.

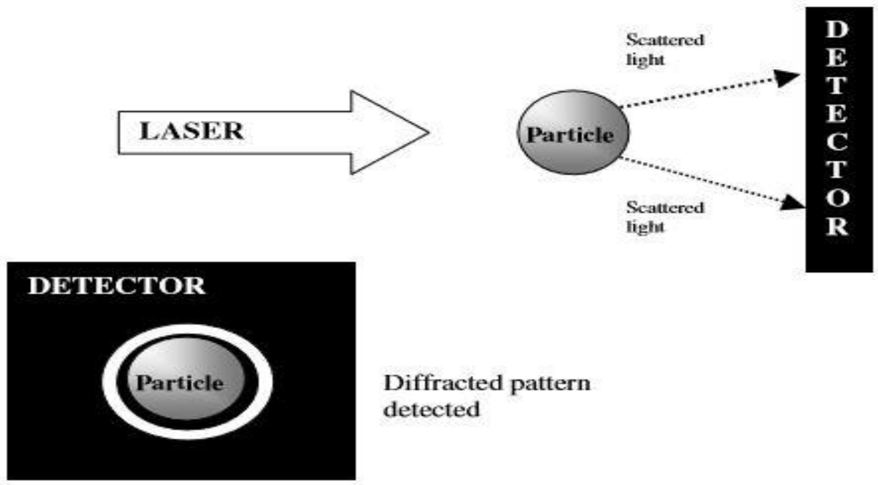
A particle size distribution can also be obtained by microscopy. This technique has the additional advantage of allowing a measurement of the particle shape and (at a sufficient magnification) texture. It is an accurate technique for obtaining the particle size but it is a time consuming process and only a relatively small amount of particles can be measured practicably. In a material such as a filler, the number of particles required to obtain an accurate particle size distribution is extremely large and using microscopy to estimate the PSD of fillers can lead to errors.

By assuming the particle density and the shape of the particles (usually as spheres), a particle size distribution expressed as percentage by mass can be derived. Sedimentation can be an inexpensive technique but requires relatively large sample sizes and is a slow test to perform. Additionally, it is not suitable for very fine particles (smaller than 2μm) as these particles tend to float in the sedimentation fluid. Laser diffraction is commonly used to measure the particle size distribution of fine materials.

This technique represents a fast, accurate means of obtaining a particle size distribution. Additionally, the technique can measure very small particle sizes, as low as 0.05µm. The laser diffraction technique is based on the phenomenon that particles scatter light in all directions with an intensity pattern that is dependent on particle size. The diffracted patterns are detected and analyzed to produce a particle size distribution (See **Figure 2.5**). The principles of laser diffraction are set out in BS ISO13320-1:1999.

Laser diffraction is a volume-based technique i.e. it reports the volume of particles which have a given particle size. This makes the technique extremely sensitive to the presence of large particles. Although they may be present in a powder in small numbers, they contain a large volume of material compared to finer particles for example; a 1mm diameter particle has a volume of 0.52ml, which is equivalent to one million particles of 1µm diameter. Hence, the presence of larger particles in the sample can have a significant effect on the volumetric distribution.

Figure 2.5: Schematic of the principles of particle size distribution by laser diffraction technique.



Studies relating to the behavior of fillers in asphalt mixtures typically include a particle size distribution as a classification test for the fillers. In earlier studies, (Kandhal, 1980; Anderson et al., 1982) the particle size distribution tended to be measured by microscopy or sedimentation techniques, whereas later studies (Harris and Stuart, 1998; Kandhal et al., 1998; Cooley et al., 1998) made use of laser diffraction techniques, as this technique became more widespread.

However, there has been little success in predicting the behavior of fillers in asphalt based on their particle size distribution. The general observation has been made that finer fillers tend to lead to a higher relative viscosity when mixed with bitumen. Fillers have been separated into individual size fractions and rheological measurements using a sliding plate viscometer taken to study the stiffening effect on bitumen. It was proposed that not only was particle size important, with smaller particle size fillers stiffening the bitumen the most, but the mineralogy of the surface also played a role in modifying the rheology of the mastic (Anderson and Goetz, 1973). In fact, this study could be viewed as producing fillers with different surface areas, as producing fillers of progressively smaller particle size increases the surface area massively.

Particle Shape

Particle shape of fillers is typically examined using microscopy, due to the very small particle sizes. Particle shape is important as the shape of the filler particles directly affects the maximum packing fraction (discussed in the following section). In asphalt filler studies Scanning Electron Microscopy has been use to examine the particle shapes of fillers (Kandhal, 1980; Anderson et al., 1973). Typically asphalt fillers derived from natural aggregates tend to be described as "grains".

Aspect ratio describes the relationship between the longest to the shortest dimension of a particle, and the aspect ratio of solid particles in suspension has a significant effect on the rheology of suspensions (Barnes, 2000). For asphalt fillers derived from natural aggregates, aspect ratios fall in a relatively small range (See Chapter III). Particle shape does have a significant effect on the packing characteristics of filler and the value of maximum packing fraction. As the effects of the filler on viscosity of a suspension are scaled by the maximum packing fraction, and particle shape is clearly an important factor.

Maximum packing fraction

The space filler occupies in its compacted state is close to a key term in describing the rheology of suspensions, the maximum packing fraction, φ_{max}. The maximum packing fraction is defined as the solid volume content of a suspension at which the viscosity of the suspension becomes infinite. The combined effects of particle size distribution, particle size, shape and density are encompassed in this key property of fillers.

The packing of filler can be estimated from the particle size distribution (Barnes, 2000; Wypych, 1999). However, because the relationship between particle size distribution, shape, texture and packing is complicated, it is more convenient to measure bulk density directly than to calculate or correlate from other properties.

"Tap Density" or "Compacted Density" refers to the maximum density filler can attain through repeated tapping, tamping, vibration or other means of compaction. Filler is added to a container of known volume and tapped a prescribed number of times, or alternatively, vibrated for a set time. The filled container is weighed and the bulk density calculated by dividing the mass of filler by the volume of the container.

The percentage voids in the filler ($\%V_{FILLER}$) can then be calculated by:

$$\%V_{FILLER} = 100\{1 - (\rho_{tap}/\rho_{particle})\}$$

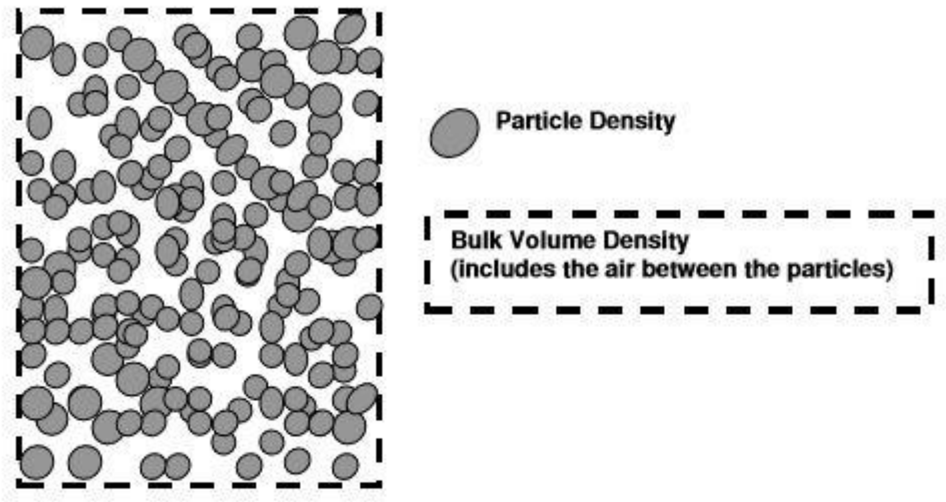
Where

ρ_{tap}= the "tap density" (bulk volume density)

$\rho_{particle}$= the particle density

The tap density is the bulk volume of filler and includes the air voids between the particles (See **Figure 2.6**).

Figure 2.6: Bulk volume density and particle density.



The maximum packing fraction can also be estimated using "Oil-Drop" of "Water drop" tests. These tests involve adding a liquid of known density drop-wise to a quantity of filler until a coherent mass is formed during mixing.

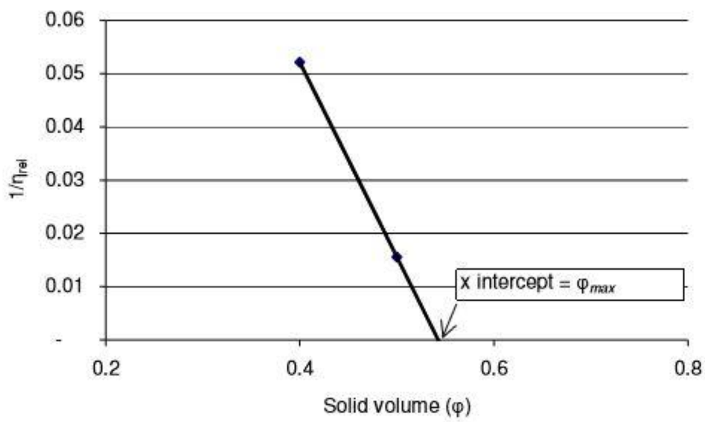
The quantity of liquid, divided by its Particle Density Bulk Volume Density (includes the air between the particles) density, determines the level of voids in the filler and hence the solid bulk volume can be estimated. The end point of such drop tests is difficult to judge and is subject to operator error, but this method has the advantage of including the interaction between the filler and a liquid.

Similarly, bulk density of filler under settlement can be used to estimate the maximum packing fraction of filler Typically kerosene is used as the test liquid, but other liquids, such as benzene and toluene have been used (Mitchell and Lee, 1939). Measuring the different levels of bulk density in liquids has been used to provide a measure of surface activity (Craus et al., 1978, 1981).

The maximum packing fraction can also be determined experimentally using rheological tests. As stated previously, in suspensions, the maximum solid packing fraction is the point at which the viscosity of the suspension becomes infinite. Increasing solid volume content produces an exponentially increasing relative viscosity. Carrying out several measurements at different solid volume contents allows the vertical asymptote of the exponential curve to be defined, which gives the maximum packing fraction.

An alternative method using minimal experimentation has been developed to derive the maximum packing fraction of powders in ceramic pastes (Hurysz and Cochrane, 2004). By carrying out measurements of viscosity at two or more concentrations close to the estimated maximum packing fraction and plotting the reciprocal of viscosity against φ (solid volume fraction) extrapolating to zero viscosity obtains a value of φ_{max}(See **Figure 2.7**).

Figure 2.7: Rheological determination of φmax using a two-point projection technique– example data (TaylorRPhD thesis - University of Nottingham).



In this section we have seen how the combined effects of particle density, particle shape and particle size distribution are captured in a single property of fillers, the maximum packing fraction. This property can be estimated in several ways, including examining the packing characteristics of the filler in air. In asphalt technology, this approach is expressed as "Rigden Voids" which is the air void content of the filler obtained under standard test conditions (BS EN 1097-4, 1999) and which relates approximately to 1 – the maximum packing fraction.

2.2.4 The Effect of Mineral Fillers on HMA

Mineral filler is a mineral material, inert to the other components of the asphalt mixture, finely divided, at least 65% passing the sieve opening of 0.075 mm square mesh. However, as a result of the small size of the particles and their surface characteristics, the filler acts as an active material, manifested in the interface filler / asphalt binder properties. Two mechanisms describe the role played by the filler in asphalt mixture: the filler provides additional points of contact between the larger aggregates and can be considered as a continuation of the fraction of asphalt aggregate mixture, and the filler increases the stability of the mixture by increasing the viscosity of the asphalt binder and changing their properties. It is evident that all the fillers have two functions in the asphalt mixture, but depending on the characteristics of the aggregate, the asphalt binder and fillers, a feature predominate.

In the mixture design, the mastic influences coarse aggregate lubrication and voids in mineral aggregate, compaction characteristics and the optimum asphalt binder content. The mastic stiffness affects the resistance of HMA to permanent deformation at high temperatures, fatigue strength at intermediate temperatures and resistance to cracking at low temperatures.

The volumetric properties of asphalt mixtures are commonly used to ensure proper performance of pavements. In 1915, noted the importance of volumetric proportions of the components of asphalt mixtures with respect to performance of pavements. In the 1940s, Marshall proposed the incorporation of conceptual voids volume and degree of saturation of the voids of the mixtures by asphalt (voids filled with asphalt) for the design of asphalt mixtures. By the 1950s, spread the concept of voids in mineral aggregate, highlighting the importance of its use to ensure pavement durability.

The volumetric properties of mixtures are the basis for the development of projects and have an important influence on asphalt mixtures performance. The main factors that control and alter these volumetric properties are: grain size, the volume of aggregate in the mix, the degree of compaction, the asphalt content and the type and amount of fillers in the mixture.

Numerous studies have shown that the properties of mineral filler (especially the material passing No. 200 sieve) have a significant effect on the properties of the HMA mixtures. The introduction of environmental regulations and the subsequent adoption of dust collection system have encouraged the return of most of the fines to the HMA mixture.

A maximum filler / asphalt ratio of 1.2 to 1.5, based on weight, is used by many agencies to limit the amount of the minus 200 material. However, the fines vary in gradation, particle shape, surface area, void content, mineral composition, and physico-chemical properties and, therefore, their influence on the properties of HMA mixtures also varies. Therefore, the maximum allowable amount should be different for different fines. (Kandhal et al, 1998). The addition of filler to the mixture can improve adhesion and cohesion substantially (filler is a fine material, which passes a 0.063 mm sieve, derived from aggregate or other similar granular material). The bitumen-filler system (mastic), which is thicker and tougher than bitumen alone, improves the adhesive qualities and, in providing a covering film of greater thickness, also means that the aging processescan be slowed down. The effects of the addition of filler are directly related to their characteristics and the degree of concentration of the filler in the bitumen-filler system.The advantages that filler offers for the durability of the bituminous mixtures in the case of water action are due, in principle, to its physical characteristics, reducing the porosity of the granular structure and thereby making the access of water and air difficult. Moreover, the chemical nature of filler may mean greater affinity with the asphalt binder, improving the resistance to the displacement that the water causes the bitumen. Using immersion tests (Craus, Ishai and Sides, 1978)assessed the influence that the type of filler had on the durability of the bituminous mixtures.

The researchers reviewed the usual criteria of mixture design, with analyses that simulate short periods of exposure to the environment (for example, for the case under study, the residual Marshall stability and the resistance of immersion-compression), noting that mixtures that pass these tests, usually fail completely in service. With the obtained results they were able to modify the existing criteria for the classification of fillers, which had only been based on basic properties without considering the durability factor. From this works, the authors have continued studying the effect of the characteristics of fillers on the durability of the mixtures (Miro et al, 2004).

In asphalt concretes (ACs), as in cement concretes, fillers are the finest particles among the aggregates. Fillers are powdery materials of various types, most of them pass the 0.063 mm sieve (EN 13043), and their inclusion in bituminous and non-bituminous binders and in aggregate mixtures confers special characteristics to these mixtures. Fillers play a major role in the production of asphalt, in terms of the composition of the mixtures and their physical and mechanical properties. Despite being widely utilized in the production of asphalt, it is still difficult to propose a general classification describing all the functions carried out by fillers used in mixtures. Fillers are the finest part in asphalt concrete mixtures, completing the granulometry, thereby helping to reduce the voids in the mixture. Various studies and experimental applications have shown that fillers can also perform other important functions, diminishing the asphalt concretes' thermal susceptibility and regulating the thickness and mechanical properties of the film of mastic covering the stone-based aggregates. Fillers must have certain physical and chemical properties that encourage and strengthen binding between aggregates and bituminous mastic, while also ensuring that the rheological behavior of the latter is optimal at the various operating temperatures. These are generally properties of commonly used fillers, such as slaked lime, Portland cement and calcium carbonate powder.

Mineral fillers are added to asphalt paving mixtures to fill voids in the aggregate and reduce the voids in the mixture. However, addition of mineral fillers has dual purpose when added to asphalt mixtures. A portion of the mineral filler that is finer than the asphalt film thickness mixed with asphalt binder forms a mortar or mastic and contributes to improved stiffening of mix. This modification to the binder that may take place due to addition of mineral fillers could affect asphalt mixture properties such as rutting and cracking.

The other portion of fillers larger than the asphalt film thickness behave as a mineral aggregate and serves to fill the voids between aggregate particles, thereby increasing the density and strength of the compacted mixture. In general, filler have various purposes among which, they fill voids and hence reduce optimum asphalt content and increase stability, meet specifications for aggregate gradation, and improve bond between asphalt cement and aggregate [Bouchard, 1992]

A research (asphalt institute,1993) was conducted on the effects of mineral fillers on rutting potential of bituminous mixtures. The mineral aggregate used in the research was crushed limestone aggregate in combination with different materials passing 0.075mm sieve, such as limestone dust, hydrated lime, and Portland cement.

Within the current new awareness about ensuring better use of natural resources and recycling waste materials, a number of new experimentations have been carried out, over the past 20 years, to look at the possibility of replacing some of the natural components in the materials used in road construction with industrial by-products and waste materials from recycling processes. Most recycled fillers currently used come from Construction and Demolition (C & D) products, which form the largest volume of waste products from the building sector. Given the need to manage this vast volume of waste products, in recent years, there have been many studies concerning their reuse in civil engineering. As of today, a number of studies and tests have used the most diverse materials, and not all of them of civil engineering provenance. In several well-known cases, researchers have experimented with glass powder, silicon carbide, coal ash, solid urban waste, polyvalent powder from fire extinguishers and even biomass powder. These are only some of the many studies with a positive outcome, underlining the growing scientific interest in using alternative, used and waste materials. This work presents the results of several laboratory tests carried out to determine some of the physical and chemical properties of three different waste materials for their application as filler in ACs. The first (Ud filler) is a digested spent bentonite clay derived from successive industrial processes and currently sent to landfill. Following the positive results of a previous study on the use of spent bentonite clays as filler in ACs, it emerged that further, more detailed work was required to analyzethephysicalandchemicalcharacteristicsofthisfiller.
Adriedmudwaste(MWfiller),whichis produced during the tungsten extraction in Panasqueira mine (Portugal), was also studied.

The third filler (Gl filler) is a powder from ground waste glass disposed to landfill. It has been produced by milling waste bottles, without any restriction given by glass color. The testing must be framed in the context of protecting the environment and sustainable development, since it proposes a functional use for wastes otherwise sent to landfill, while at the same time limiting the use of natural raw materials.

Hydrated lime and Portland cement as Mineral Filler

The research was carried out using limestone dust (control mix) and replacing by hydrated lime and Portland cement in different proportions. From various tests conducted, the authors arrive at following conclusions.

I. Greater raise in softening point of asphalt mastics was achieved when replacing limestone dust with hydrated lime than Portland cement.

II. Mixtures prepared by replacing limestone dust with hydrated lime at higher filler content, acquire higher optimum asphalt content, higher air voids, and lower unit weights than those containing Portland cement. This is attributed to the higher specificsurface area and asphalt absorption of the hydrated lime particles than Portland cement.

III. Increasing filler content in the mixture enhances the Marshal and Hveem stability, as expected. This is because increasing filler content from 3% to 5.5% fills the voids among aggregate particles thus producing dense mixes, hence increasing stability, whereas increasing filler content beyond 5.5% reduce the contact among coarse aggregate particles, hence reducing the stability.

IV. While replacing limestone dust by either hydrated lime or Portland cement in the mixes, there was a decrease in resilient modulus values.

V. Replacing part of limestone dust by hydrated lime or Portland cement aggravates the resistance of mixes to rutting. The rut depth increases as the percentage replaced increases; where higher rut depth was observed when replacing limestone mixes with hydrated lime than Portland cement.

White Cement Kiln Dust as Mineral Filler

Cement Kiln Dust (CKD) is a byproduct material that generated during the production of Portland cement. Raw material are heated in the kiln, dust particles are produced and then get out with the exhaust gases at the upper end of the kiln. These gases are gradually cooled and the accompanying dust particles are captured by efficient dust collection systems. The composition of CKD is quite variable from source to another due to raw materials and process variations.

It is primarily made up of a variable amount of fine calcined and uncalcined feed materials, fine cement clinker, fuel combustion by-products, and condensed alkali compounds. Some of CKD is recycled back again with the clinker but the amounts are limited by alkalinity requirements for Portland cement and kiln operation issues. However, most of the material is disposed of on-site without any further reusing or reclamation. Waste material recycling into useful products has become a main solution to waste disposal problems. Many highway agencies are conducting wide variety of studies and research projects concerning the feasibility, environmental suitability, and performance of using recycled products in highway construction. Many researches referred to its uses in asphalt concrete mixtures and its uses in soil stabilization. But a few investigations had been focused on its importance in asphalt concrete pavement in Iraq [6]. The aim of this study is to investigate the possibility of using CKD as mineral filler (partially or fully) in producing Hot Mix Asphalt (HMA).

Waste Glass powder as filler in Glasphalt

Various studies have been conducted to study the properties of mineral filler and to evaluate its effect on the performance of asphalt paving mixtures in terms of mechanical properties while the use of waste glass as filler in hot mix asphalt is still not widely experimented (Jony H. et al., 2011). Pereira et al. (2010) studied the use of waste flat glass as a filler in asphalt mixtures and it concluded that the effect of, waste glass on the asphalt mixture does not differ from those made with conventional materials and may be used effectively in asphalt paving. Jony et al. (2011) compared the effect of using different fillers with different contents, glass powder is proposed as an alternative to traditional lime stone powder and ordinary Portland cement fillers in hot asphalt mixtures. The results indicate that there is a satisfactory stability, where using glass powder filler improve the Marshall Stability values for all mixtures comparing to Portland cement or Limestone powder fillers.

2.2.5 Importance of Volumetric Properties in Asphalt Mixtures

Currently, the volumetric properties of asphalt mixtures are subdivided and classified as primary and secondary volumetric parameters. The primary volumetric parameters are directly related to the relative volumes of the individual components of the mixtures: air voids (V_v); aggregates volume (V_s), and asphalt binder volume (V_b). It is important to consider that the aggregate cavities porous (pore space) and the asphalt portion absorbed share the same space. It means that the sum of the volumes ($V_b + V_s$) is larger than their combined volumes (V_{b+s}).

This phenomenon leads to a subdivision of the primary volumetric parameters:

I. **Effective binder volume (V_{be}):** volume of asphalt not absorbed by the aggregate;

II. **Absorbed binder volume (V_{ba}):** volume of asphalt absorbed into the external pore structure of the aggregates;

III. **Effective volume of aggregate (V_{se}):** aggregate volume including the volume of pores permeable to water and the volume of pores permeable to the asphalt;

IV. **Bulk volume of aggregate (V_{sb}):** aggregate volume that includes volume permeable porous to water but not to the asphalt;

V. **Apparent volume of aggregate (V_{sa})** only the solid volume of the aggregate excluding the volume of permeable pores to water or asphalt.

Secondary volumetric parameters (or volumetric properties of mixtures) are Void Volume (V_v), Voids in Mineral Aggregates (VMA) and Voids Filled with Asphalt (VFA), that are determined based on the primary volumetric parameters. Conceptually, these parameters can be defined as:

I. **Void volume (V_v):** is the air volume (V_{ar}) between the aggregate particles surrounded by the film of asphalt, expressed as a percentage of the total volume of the compacted mixture;

II. **Voids in Mineral Aggregates (VMA):** is the sum of the void volume (V_v) and volume effective asphalt (VEAC), expressed as a percentage of the total volume of the compacted mixture;

III. **Voids Filled with Asphalt (VFA):** is the degree of VMA filled by asphalt, expressed in percentage.

The methods commonly used in asphalt mixtures design incorporate volumetric criteria, which is calculated from the volumetric proportions of the constituent materials of the mixtures. The Marshall and Super pave methods [5], determine the optimum asphalt binder using HMA volumetric properties (V_v, VMA and VFA). The Super pave method also evaluates the filler content in the mixture and the percentages of initial and maximum compaction as a function of the number of gyrations in the Super pave Gyratory Compactor (SGC).

The asphalt mixtures are expected to be stable enough to prevent permanent deformations, flexible enough to delay fatigue cracks development and durable to resist traffic action, weather and time. To achieve optimum performance properties, it must be established a balance between the skeletal structure formed by aggregates and asphalt binder amount added to the mix. The mixture should be formed by aggregates sizes, shapes, angularity and surface textures that allow enough space for the addition of the adequate amount of asphalt to ensure durability and flexibility of the mixture.

The Super pave method suggests the volumetric parameters of V_v, VMA and VFA to design of asphalt mixtures. It is established a V_v of 4% as the main parameter to select the optimum asphalt binder content. Excessive V_v or VFA and inadequate VMA suggest potential durability problems. Also, insufficient $V_v$or excessive VFA indicate potential rutting. Super pave Method establishes minimum values of VMA **Table 2.3**, based on the mixture Nominal Maximum Size (NMS) and minimum and maximum values of the VFA, and based on traffic volume **Table 2.4**.

Table 2.3: Minimum VMA recommended for Super pave Method

Mixture Nominal Maximum Size (NMS) (mm)	Minimum VMA (%)
9.5	15
12.5	14
19.0	13
25.0	12
37.5	11

Table 2.4: VFA criteria for Super pave Method (https://pavementinteractive.org/reference-desk/design/mix-design/superpave-mix-design/)

Traffic (ESALs)	Design VFA (%)
$<3 \times 10^5$	70 - 80
$>3 \times 10^5$	65 - 78
$< 1 \times 10^8$	65 - 75
$> 1 \times 10^8$	65 - 75

Marshall Mix Design

The mix design (wetmix) determines the optimum bitumen content. This is preceded by the dry mix design discussed in the previous chapter. There are many methods available for mix design which varies in the size of the test specimen, compaction, and other test specifications. Marshall Method of mix design is the most popular one. The Marshall stability and flow test provides the performance prediction measure 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. Load is applied to the specimen till failure, and the maximum load is designated as stability. During the loading, an attached dial gauge measures the specimen's plastic flow (deformation) due to the loading. The flow value is recorded in 0.25 mm (0.01 inch) increments at the same time when the maximum load is recorded.

Specimen preparation

Approximately 1200gm of aggregates and filler is heated to a temperature of 175−190°C. Bitumen is heated to a temperature of 121−125°C with the first trial percentage of bitumen (say 3.5 or 4% by weight of the mineral aggregates). The heated aggregates and bitumen are thoroughly mixed at a temperature of 154−160°C. The mix is placed in a preheated mould and compacted by a rammer with 50 blows on either side at temperature of 138°C to 149°C. The weight of mixed aggregates taken for the preparation of the specimen may be suitably altered to obtain a compacted thickness of 63.5+/-3 mm. Vary the bitumen content in the next trial by +0.5% and repeat the above procedure. Number of trials are predetermined. The prepared mould is loaded in the Marshall test setup as shown in **Figure 2.8**.

Properties of the mix

The properties that are of interest include the theoretical specific gravity G_t, the bulk specific gravity of the mix G_m, percent air voids V_v, percent volume of bitumen V_b, percent void in mixed aggregate VMA and percent voids filled with bitumen VFB. These calculations are discussed next. To understand these calculations a phase diagram is given in **Figure 2.9**.

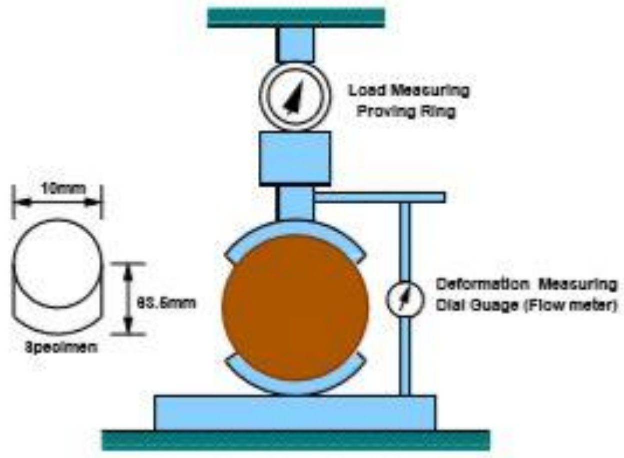


Figure 2.8: Marshall Test setup
(https://www.civil.iitb.ac.in/tvm/nptel/407_lnTse/web/web.html)

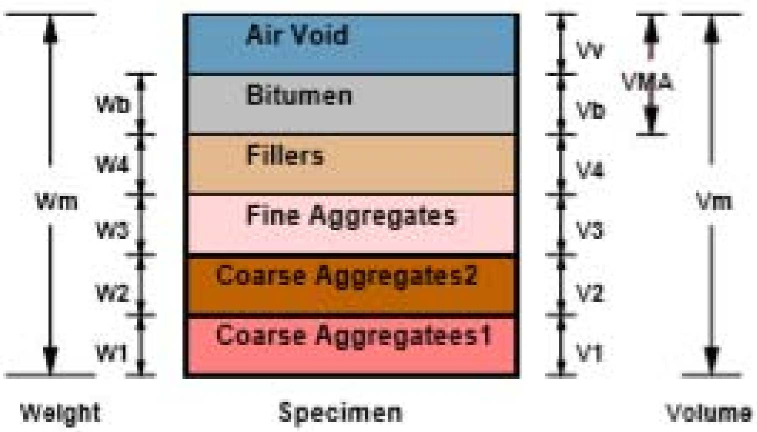


Figure 2.9: Phase diagram of a bituminous mix
(https://www.civil.iitb.ac.in/tvm/nptel/407_lnTse/web/web.html)

Theoretical specific gravity of the mix G_t

Theoretical specific gravity $\mathbf{G_t}$ is the specific gravity without considering air voids, and is given by:

$$G_t = (W_1 + W_2 + W_3 + W_b)/(W_1/G_1 + W_2/G_2 + W_3/G_3 + W_b/G_b) \quad \text{----------(2.1)}$$

Where, $\mathbf{W_1}$ is the weight of coarse aggregate in the total mix, $\mathbf{W_2}$ is the weight of fine aggregate in the total mix, $\mathbf{W_3}$ is the weight of filler in the total mix, $\mathbf{W_b}$ is the weight of bitumen in the total mix, $\mathbf{G_1}$ is the apparent specific gravity of coarse aggregate, $\mathbf{G_2}$ is the apparent specific gravity of fine aggregate, $\mathbf{G_3}$ is the apparent specific gravity of filler and $\mathbf{G_b}$ is the apparent specific gravity of bitumen.

Bulk specific gravity of mix G_m

The bulk specific gravity or the actual specific gravity of the mix $\mathbf{G_m}$ is the specific gravity considering air voids and is found out by:

$$G_m = W_m/(W_m - W_w) \text{----------(2.2)}$$

Where, $\mathbf{W_m}$ is the weight of mix in air, $\mathbf{W_w}$ is the weight of mix in water, Note that $\mathbf{(W_m - W_w)}$ gives the volume of the mix. Sometimes to get accurate bulk specific gravity, the specimen is coated with thin film of paraffin wax, when weight is taken in the water. This however requires considering the weight and volume of wax in the calculations.

Air voids percent V_v

Air voids $\mathbf{V_v}$ is the percent of air voids by volume in the specimen and is given by:

$$V_v = \{(G_t - G_m)\ 100\}/G_t \quad \text{----------(2.3)}$$

Where**Gt** is the theoretical specific gravity of the mix, given by equation **(2.1)**.and**Gm** is the bulk or actual specific gravity of the mix given by equation **(2.2)**.

Percent volume of bitumen V_b

The volume of bitumen $\mathbf{V_b}$ is the percent of volume of bitumen to the total volume and given by:

$$V_b = (W_b/G_b)/\{(W_1 + W_2 + W_3 + W_b)/G_m\} \text{----------(2.4)}$$

Where, $\mathbf{W_1}$ is the weight of coarse aggregate in the total mix, $\mathbf{W_2}$ is the weight of fine aggregate in the total mix, $\mathbf{W_3}$ is the weight of filler in the total mix, $\mathbf{W_b}$is the weight of bitumen in the total mix, $\mathbf{G_b}$ is the apparent specific gravity of bitumen, and $\mathbf{G_m}$ is the bulk specific gravity of mix given by equation **(2.2)**.

Voids in mineral aggregate, VMA

Voids in mineral aggregate,**VMA** is the volume of voids in the aggregates, and is the sum of air voids and volume of bitumen, and is calculated from

$$VMA = V_v + V_b \text{----------(2.5)}$$

Where, $\mathbf{V_v}$ is the percent air voids in the mix, given by equation **(2.3)**. and$\mathbf{V_b}$ is percent bitumen content in the mix, given by equation **(2.4)**.

Voids filled with Asphalt, VFA

Voids filled with Asphalt, **VFA** is the voids in the mineral aggregate frame work filled with the bitumen, and is calculated as:

$$\mathbf{VFA = (Vb \times 100)/VMA\text{----------}(2.6)}$$

Where, $\mathbf{V_b}$ is percent bitumen content in the mix, given by equation **(2.4)**. and**VMA** is the percent voids in the mineral aggregate, given by equation **(2.5)**.

Determination of Marshall Stability and flow

Marshall Stability of a test specimen is the maximum load required to produce failure when the specimen is preheated to a prescribed temperature placed in a special test head and the load is applied at a constant strain (5 cm per minute). While the stability test is in progress dial gauge is used to measure the vertical deformation of the specimen. The deformation at the failure point expressed in units of 0.25 mm is called the Marshall flow value of the specimen.

Application of stability correction

It is possible while making the specimen the thickness slightly vary from the standard specification of 63.5 mm. Therefore, measured stability values need to be corrected to those which would have been obtained if the specimens had been exactly 63.5 mm. This is done by multiplying each measured stability value by an appropriated correlation factors as given in **Table 2.5** below.

Table 2.5: Correction factors for Marshall Stability values (https://www.civil.iitb.ac.in/tvm/1100_LnTse/407_lnTse/plain/plain.html)

Volume of specimen, (cm^3)	Thickness of specimen, (mm)	Correction Factor
457 - 470	57.1	1.19
471 - 482	68.7	1.14
483 - 495	60.3	1.09
496 - 508	61.9	1.04
509 - 522	63.5	1.00
523 - 535	65.1	0.96
536 - 546	66.7	0.93
547 - 559	68.3	0.89
560 - 573	69.9	0.86

Prepare graphical plots

The average values of the above properties are determined for each mix with diff erent bitumen content and the following graphical plots are prepared:

1. Binder content versus corrected Marshall Stability

2. Binder content versus Marshall flow

3. Binder content versus percentage of void (V_v) in the total mix

4. Binder content versus voids filled with Asphalt (VFA)

5. Binder content versus unit weight or bulk specific gravity (G_m)

Determine optimum bitumen content

Determine the optimum binder content for the mix design by taking average value of the following three bitumen contents found form the graphs obtained in the previous step.

1. Binder content corresponding to maximum stability
2. Binder content corresponding to maximum bulk specific gravity (G_m)
3. Binder content corresponding to the median of designed limits of percent air voids (V_v) in the total mix.

The stability value, flow value, and VFA are checked with Marshall Mix design specification chart given in Table below. Mixes with very high stability value and low flow value are not desirable as the pavements constructed with such mixes are likely to develop cracks due to heavy moving loads.

Table 2.6: Marshall Criteria for Asphalt Mix Design

Name of the Test	**Marshall Design Criteria** Mix Design Methods for Asphalt Concrete and Other Hot-Mix Types. Manual Series No. 2 (MS-2) Sixth Edition, 1997. Asphalt Institute. Lexington, KY.					
	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)	35		50		75	
Stability (N)	3336	--	5338	--	8006	--
Flow Index (0.01 in)	8	18	8	16	8	14
Air voids (%)	3	5	3	5	3	5
Voids Filled with Bitumen (VFA) (%)	70	80	65	78	65	75
Voids in Mineral Aggregates (VMA) (%)	See the **Table 2.7**					

Table 2.7: Marshall Minimum Voids in mineral aggregates (VMA)Mix Design Methods for Asphalt Concrete and Other Hot-Mix Types. Manual Series No. 2 (MS-2). Sixth Edition, 1997, Asphalt Institute. Lexington, KY.

Nominal Maximum Size		Minimum VMA (%)		
		Air Voids (%)		
(mm)	Sieve Designation	3.0	4.0	5.0
63.0	2.5 inch.	9.0	10.0	11
50	2.0 inch.	9.5	10.5	11.5
37.5	1.5 inch.	10.0	11.0	12.0
25.0	1.0 inch.	11.0	12.0	13.0
19.0	0.75 inch.	12.0	13.0	14.0
12.5	0.50 inch.	13.0	14.0	15.0
9.5	0.375 inch.	14.0	15.0	16.0
4.75	#4	16.0	17.0	18.0
2.36	#8	19.0	20.0	21.0
1.18	#16	21.5	22.5	23.5

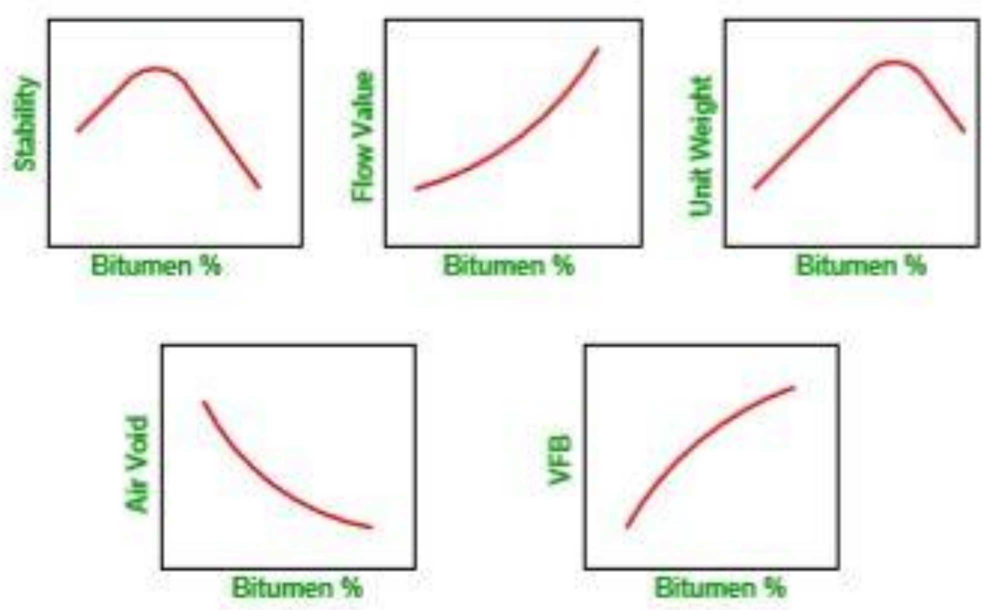


Figure 2.10: Marshal graphical plots ((https://www.civil.iitb.ac.in/tvm/1100_LnTse/407_lnTse/plain/plain.html))

2.3Asphalt Concrete Pavements

Roads are built up in several layers, consisting of sub-grade, sub-base, base and surface layer; these layers together constitute the pavement. Because asphalt concrete is much more flexible than Portland cement concrete, asphalt concrete pavements are sometimes called flexible pavements. Asphalt concrete is composed primarily of aggregate and asphalt binder. Aggregate typically makes up about 95% of a Hot Mix Asphalt (HMA) mixture by weight, whereas asphalt binder makes up the remaining 5%. By volume, a typical HMA mixture is about 85% aggregate, 10% asphalt binder, and 5% air voids. Asphalt binder glues the aggregate together and that means without asphalt binder HMA would simply be crushed stone or gravel. Small amounts of additives and admixtures areadded to many HMA mixtures to enhance their performance or workability (Transportation research board committee, 2011)

2.3.1 Flexible Pavement Layers

Asphalt concrete pavements are not a thin covering of asphalt concrete over soil, they are engineered structures composed of several different layers. **Figure 2.11** illustrates a vertical section of flexible pavement structure.

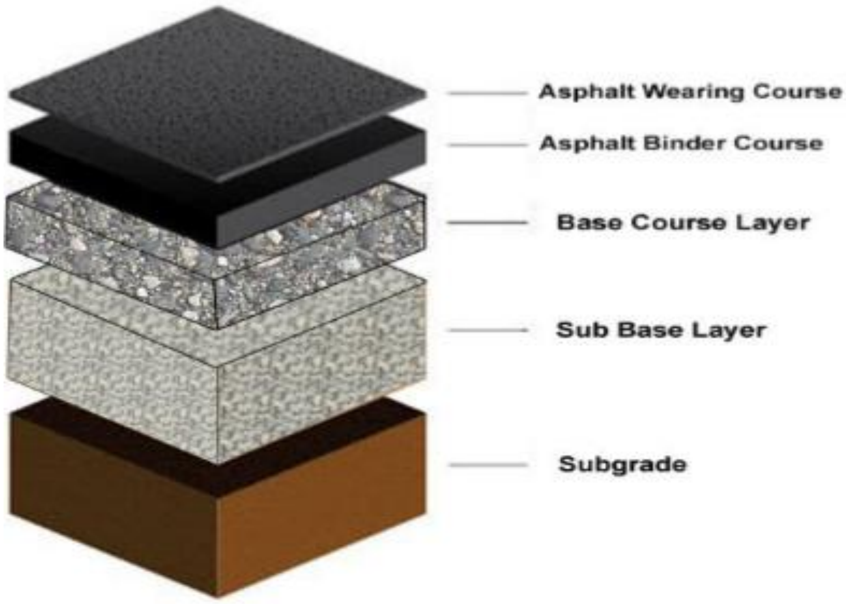


Figure 2.11: Flexible Pavement Layers (https://theconstructor.org/transportation/flexible-pavement-composition-structure/5499/)

Subgrade

The natural soil surface is the boundary between the base soil (Sub grade) and the upper layers of pavement, and it's called the formation (Jendia, 2000). The in-place soils, called the sub grade, serve as the foundation that supports the road. After removal of topsoil and other organic materials, the subgrade may be stabilized by compaction alone, or by compaction after mixing in asphalt emulsion, foamed asphalt, Portland cement, lime, or other proprietary stabilizing materials. The properties and characteristics of the sub grade soil determine the pavement thickness needed to carry the expected traffic loads (Blades, C. et al, 2004).

Sub-base layer

The sub-base course is the layer of material beneath the sub grade and the base course. It provides structural support, improve drainage and reduce the intrusion of fines from the subgrade in the pavement structure. Moreover if the base course is open graded the subbase course with more fines can serve as filler between subgrade and the base course (Blades, C. et al, 2004). Sometimes the subbase course is omitted from a pavement and a relatively thick base course is placed directly on the subgrade soil (Transportation research board committee, 2011).

Base course layer

The base course is the layer of a specified material of designed thickness placed immediately beneath the surface (wearing) or binder course. It provides additional load distribution. It may be composed of crushed stone, crushed slag, and other untreated or stabilized materials (Mathew and Rao, 2007).

Asphalt binder course

Binder course is a hot mix asphalt (HMA) course between the wearing course and either a granular base course or stabilized base course, an existing pavement, or another HMA binder course (Ontario Provincial Standard Specification, 2002) Its purpose is to distribute traffic loads so that stresses transmitted to the pavement foundation will not result in permanent deformation of that layer. Additionally, it facilitates the construction of the surface layer (Garcia, J., and Hansen, k., 2001).

Binder course gradation

Grading of aggregates complies with American Society for Testing and Materials (ASTM D 3515-01) that indicates international gradation limits for the asphalt binder course. **Table 2.8** and **Figure 2.12** indicate international gradation limits for the dense graded asphalt binder course (ASTM D 3515-01).

Table 2.8: Gradation limits of dense graded Asphalt Binder Course (ASTM D 3515)

Sieve size (mm)	Percentage by Weight Passing	
	Min	Max
25.00	100	100
19.00	90	100
12.50	67	85
9.50	56	80
4.75	35	65
2.36	23	49
0.30	5	19
0.15	3	14
0.075	2	8

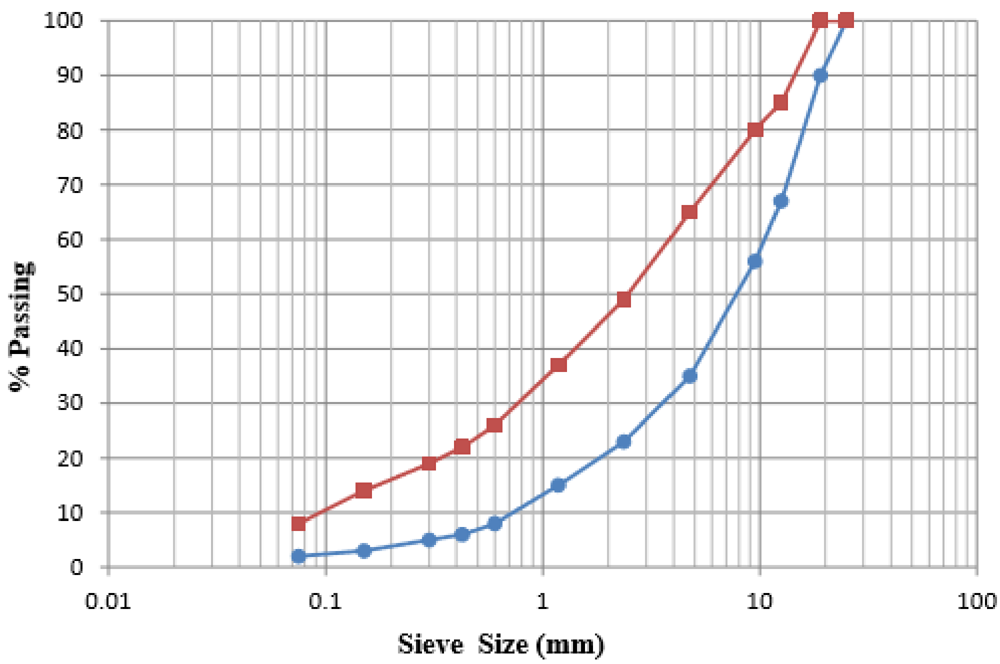


Figure 2.12: Gradation limits of dense graded Asphalt Binder Course (ASTM D 3515)

Asphalt wearing course

It is the top layer of the pavement and it is directly exposed to traffic and environmental forces (Transportation research board committee, 2011). Wearing course provides characteristics such as friction, smoothness, noise control, rut and shoving resistance, and drainage. In addition, it serves to prevent the entrance of excessive quantities of surface water into the underlying HMA layers, bases, and subgrade (Garcia, J., & Hansen, k., 2001).

2.4 The cost of road infrastructure in developing countries

Roads are archetypal of public economic infrastructure. While telecoms, powerand railways are often privately financed, the practical scope for private financing of roads in developingcountries has proved to be extremely limited. Yet over recent decades donors have shiftedtheir support from such infrastructure, which was the initial rationale for aid, to social priorities,as exemplified by the Millennium Development Goals. In low-income counties this may have contributedto the deterioration in provision: for example, there is evidence that since the 1980s theAfrican road stock has actually contracted (Teravaninthorn and Raballand 2009).

Table 2.9: Complete ROCKS Database for Low and Middle Income Countries

	N	Percent
Low income	780	23.48
Lower middle income	1,352	40.70
Upper middle income	1,190	35.82
Total	3,322	100

Notes: Income classification based on World Development Indicators 2012.
Source: ROCKS data and World Development Indicators 2012.

If poor countries must self-finance much of their road networks, their costs of constructionand maintenance become more important. Where costs are unusually high, it is usefulto discover why. If the cause of high costs is readily remediable, then it can become anobjective of policy. But even if high costs are attributable to factors that are beyond influence,there are important implications. Connectivity is essential for economic development.It enables trade, which in turn enables people to harness the productivity gains that comefrom specialization and scale. However, the density of a national road network necessary toachieve a given level of connectivity depends upon population dispersion.

Connectivity canpotentially be increased either by building more roads for a given dispersion, or by encouragingpeople to relocate into larger settlements. A country in which roads are unalterablyvery expensive should give greater priority to reducing dispersion. Hence, in studying variationin the unit cost of roads, it is useful to discover both the extent of variation, and thelikely reasons for that variation.

Table 2.10: Unit Costs per km of Asphalt Overlays 40 mm to59 mm

Country	Cost per km in $1000	Number	Year	Country	Cost per km in $1000	Number	Year
	Work activities undertaken between 1996–1998						
Dominican Republic	33.5	1	1997	Argentina	69.7	1	1997
Ghana	42.9	5	1998	Brazil	74.4	1	1998
Lithuania	44.4	1	1996	Argentina	74.9	1	1996
Indonesia	48.5	1	1996	Cameroon	76.8	4	1997
Lithuania	49.7	1	1998	Bangladesh	79.1	26	1998
Mexico	50.7	1	1997	Vietnam	79.6	2	1998
Ghana	52.7	1	1996	Bangladesh	83.6	1	1997
Costa Rica	57.9	1	1996	Panama	84.1	1	1997
Armenia	60.7	1	1997	Nigeria	95.1	1	1997
Brazil	62.5	2	1996	El Salvador	102.2	1	1998
Bolivia	67.4	1	1997	Pakistan	105.0	1	1997
India	68.1	3	1997	Tanzania	111.7	1	1996
	Work activities undertaken between 2005–2007						
Paraguay	31.2	1	2005	Botswana	68.0	1	2006
India	35.9	2	2006	Nigeria	73.0	1	2007
Bulgaria	40.7	1	2006	Argentina	76.2	3	2006
Ecuador	41.6	1	2005	Georgia	82.6	1	2006
India	45.6	1	2005	Brazil	82.9	2	2005
Burkina Faso	48.0	1	2007	Georgia	84.9	1	2005
Brazil	55.2	3	2006	Vietnam	85.4	1	2005
Brazil	58.2	1	2007	Macedonia	85.7	1	2007
Thailand	59.5	1	2005	Rwanda	90.6	1	2006
Philippines	60.8	1	2006	Philippines	94.8	1	2005
Bosnia and Herzegovina	61.9	2	2006	Chile	98.9	1	2006
Nepal	63.1	1	2006				

Notes: Costs per km of asphalt overlays 40 to 59 mm; all costs are in 2000 US$; number denotes the number of work activities in a given country over which a simple average is taken.
Source: ROCKS data.

The descriptions given to the road works on **World Bank reports** are very general (for example:rehabilitation, strengthening, periodic maintenance, reconstruction, improvement, construction, etc.).Most of the time no detailed information was found,such as road width, terrain, traffic, overlay thickness,regravelling thickness, rehabilitation surface,improvement type, etc. It was only possible to estimate average costs andcosts statistics for a series of road work classesbased on the general descriptions.

Road Works Classes

Paved Roads

I. Seals (reseals, surface dressings)
II. Functional Overlays (thickness <= 5.0 cm)
III. Structural Overlays (thickness > 5.0 cm)
IV. Rehabilitation (strengthening, reconstruction)
V. Construction (widening, new construction)

Unpaved Roads

I. Re-gravelling
II. Rehabilitation
III. Improvement
IV. Paving

Average Works Costs per Km

Table 2.11: Average Works Costs per Km for different road class

Road Class	Average Costs
Paved Roads	
I. Seals (reseals, surface dressings)	20,000 $/km
I. Functional Overlays (thickness <= 5.0 cm)	56,000 $/km
I. Structural Overlays (thickness > 5.0 cm)	146,000 $/km
. Rehabilitation (strengthening, reconstruction)	214,000 $/km
. Construction (widening, new construction)	866,000 $/km
Unpaved Roads	
. Re-gravelling	11,000 $/km
I. Rehabilitation	31,000 $/km
I. Improvement	72,000 $/km
I. Paving	254,000 $/km

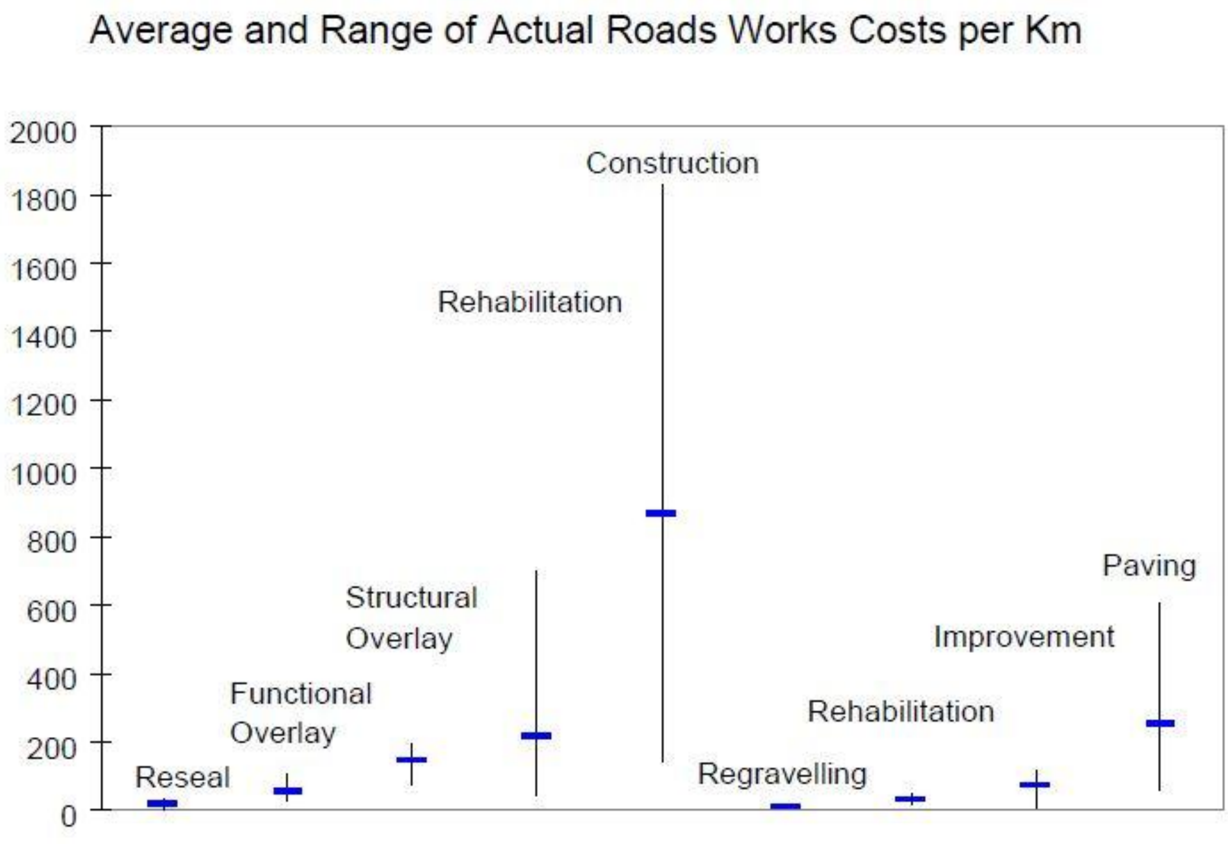


Figure 2.13: Average and Range of Roads Works Costs per km

(Information from World Bank completedhighways projects, from 1995 to 1999)

2.4.1 Conflict and Corruption and Unit Costs of road work activities

Low income or developingstates lag behind on measures like poverty reduction and other developmental outcomes (**World Bank 2011b**). If these finance constrained states face high roadconstruction costs, and roads construction and a better network reduce conflict by raising the opportunitycost of joining rebel groups through employment, as well as improved economic outcomesthrough better connectivity, then they might be trapped in an equilibrium with high costs of transportinfrastructure and instability. Further, public work contracts, including roads, are subject tosubstantial levels of corruption. According to Transparency International's Bribe Payers Survey ofover 3,000 business executives' worldwide, public works contracts and construction is the sectorwith the highest propensity of paying bribes to officials and other firms (Transparency International2011). As this paper attempts to establish a first set of facts on differences in costs, a focus on thelink between corruption and costs is a natural priority.

A review by the World Bank's Transport Research Support Program on the roads sector in Low income or developing countries like Bangladesh states that "...projects that take place in conflict settings would almost always be morecostly than in other settings because of challenges such as insecurity and low government capacity"(Rebosio and Wam 2011). Higher costs can be due to the costs of monitoring of the security situationof an area, potentially undergoing substantial risks to visit the construction site, and theassociated limited planning possible. In addition to protection of the staff working on the particularroads project, firms also risk that supplies are cut off due to disruptions of transport networks. Ifconflict takes place along ethnic lines, road construction firms might need to ensure to employ anethnically balanced workforce, in order not to further fuel the conflict or becoming targets of violencethemselves. Consultations with communities, while helpful, are also significantly adding tothe cost of construction. Not only the construction but also the procurement process can be riskierin conflict countries. Rebosio and Wam (2011) and Benamghar and Iimi (2011) give evidence forthese effects on risks and costs from Nepal: a government employed road engineer was killed inthe Terai regions; road construction teams were constantly monitoring the security situation andadjusting their operations accordingly; in certain regions violence and intimidation were employedduring the bidding process to prevent firms from submitting a bid for profitable project.

2.4.2 Road Construction cost Scenario in Bangladesh

According to the **Asian Development Bank (ADB)** Report, the road construction cost in Bangladesh is increasing is the highest in the world, quality roads are hardly made for lack of proper monitoring by the authorities concerned and the accountability of construction firms.

The cost of overlay and widening of the Chandina, Comilla and Feni bypasses (51.8 km); upgrading and widening of the Feni– Chittagong section (47.9 km); construction of the CPAR (13.6 km) as a pilot for Public–Private Partnership (PPP) **(Source: ADB. 2009. Project Completion Report)**is shown in**Table 2.12**.

Table 2.12: Corridor Improvement Actual Costs

Serial No.	Road Section	Length (Km)	Amount (in USD Millon)
01	Overlay and widening of Chandina, Comilla, and Feni bypasses	**51.8**	**19.15**
02	Upgrading and widening of Feni–Chittagong section 1	25.4	30.85
03	Upgrading and widening of Feni–Chittagong section 2	22.5	--
04	Construction of Chittagong Port access road (new)	13.6	18.81

Officials at the Road Transport and Highways Division and Local Government Engineering Department (LGED) said over 2,000 km roads are needed to repair every year only because of overloaded vehicles. They said there are 2.85 lakh km roads under the LGED while some 21.03 km highways and district roads under the Road Transport and Highways Division.In a report of the **World Bank** presented a list of infrastructure cost, especially in road construction. It shows the cost of per kilometer road construction is **$2.5 million to $11.9 million** in Bangladesh, which is the highest in the world.

2.4.3 Routine Maintenance Costs

The costs of routine maintenance for the RHD paved road network should be around Tk. 60Crore per year (**Table 2.13**). This is a small figure in comparison to periodic maintenancerequirements. It should have priority in resource allocation, simply on the basis of economicreturn. Acomparison of the intervention costs required revealed that the value of routine maintenance is asaving in other forms of intervention of around Taka 300 Crore per year.

Table 2.13: Routine Maintenance Costs of the existing paved network

Road Class	Paved length (km)	Annual Routine MaintenanceCost (Crore Taka)
National	3,485	17.4
Regional	4,117	12.4
Zila	9,832	29.5
Total	17,434	59.3

Routine maintenance costs will rise as the paved road network is increased in length. The totalcosts of routine maintenance over the 20-year period are estimated to be Taka 1,392.4 Crore (Road Master plan-2009, RHD, Volume-I).

2.4.4 Periodic MaintenanceCosts

It is vital that sufficient resources are devoted to periodic maintenance on an annual basis. Theactual needs will be determined by HDM each year, but the average expected needs are set outin **Table 2.14**. The HDM Circle will need to fully sustained over the master plan period (Road Master plan-2009, RHD, Volume-I).

Table 2.14: Periodic Maintenance Costs

Road Class	Annual Requirement (Crore Taka)		Requirement over 20 year plan period
	Rising from	Rising to	
National	250	850	11421
Regional	150	250	4050
Zila	125	560	7620
Total	500	1760	23091

2.5 Moisture Susceptibilityof Hot-Mix Asphalt Pavements

Moisture susceptibility is a primary cause of distress in HMA pavements. HMA should not degrade substantially from moisture penetration into the mix. HMA mixtures may be considered susceptible to moisture if the internal asphalt binder-to-aggregate bond weakens in the presence of water. This weakening, if severe enough, can result in stripping **(Figure 2.14)**.

To measure the potential for moisture damage to HMA mixtures, moisture susceptibility testing can be performed. Results from the moisture susceptibility test may be used to predict the potential for long-term stripping and to evaluate anti-stripping additives, which are added to the asphalt binder, aggregate, or HMA mixture to help prevent stripping.

Figure 2.14: Fatigue cracking caused by stripping.

2.5.1 Background

Moisture damage is the result of moisture interaction with the asphalt binder-aggregate adhesion within a HMA mixture. This interaction can cause a reduction of adhesion between the asphalt binder and aggregate (**Figure 2.15** and **Figure 2.16**), called stripping, which can lead to various forms of HMA pavement distress including rutting and fatigue cracking.

Over the years, many different tests have been used to evaluate a particular HMA mixture's susceptibility to moisture damage. These tests range from simple (e.g., the boiling test) to the more complex (e.g., Hamburg wheel tracking test). The moisture susceptibility test specified by Superpave mix design is typically called the modified Lottman test. This test is described in the Test Description section. This section, taken largely from Hicks (1991) describes the actual moisture damage mechanism, factors influencing moisture damage, preventative measures and alternative tests.

Figure 2.15: HMA samples with no moisture damage (left) and moisture damage (right). Notice the amount of uncoated aggregate on the damaged sample.

Figure 2.16: HMA samples with no moisture damage (left) and moisture damage (right). A more subtle example than **Figure 2.15**, but still with noticeable uncoated aggregate.

2.5.2 Asphalt Binder and Aggregate Adhesion

Moisture damage is the reduction in adhesion between the asphalt binder and aggregate surface in an HMA mixture. In order to understand its causes and preventive measures, a brief discussion of adhesion mechanisms is presented here. There are 4 principal means of asphalt binder-aggregate adhesion:

IX. **Mechanical.** Asphalt binder gets into the surface irregularities and pores of the aggregate and hardens causing a mechanical lock. Moisture on the aggregate can interfere with asphalt binder penetration into the aggregate and decrease the mechanical lock, thus increasing susceptibility to stripping.

X. **Chemical.** A chemical reaction between the asphalt binder and aggregate surface occurs causing chemical adhesion. In general, aggregates with acidic surfaces do not react as strongly with asphalt binders. This weaker reaction may not be strong enough to counter other moisture damage factors.

XI. **Adhesion tension**. The tension between the asphalt binder and aggregate at the wetting line (as a drop spreads over a surface, the edge of the drop is the “wetting line”) is generally less than the tension between water and aggregate. Therefore, if all three are in contact, water will tend to displace asphalt binder. This can result in poor wetting of the aggregate surface by the asphalt binder and lead to stripping. This interfacial tension between asphalt binder and aggregate varies with asphalt binder type, aggregate type and aggregate surface roughness.

XII. **Molecular orientation**. When in contact with aggregate, asphalt molecules tend to orient themselves in relation to the ions on the aggregate surface essentially creating a weak attraction between the asphalt binder and aggregate surface. If water molecules, which are dipolar, are more polar than asphalt binder molecules, they may preferentially satisfy the energy demands of the aggregate surface. The resulting weak asphalt binder-aggregate bond can result in stripping.

2.5.3 Factors Influencing Moisture Damage

Moisture susceptibility is a complex phenomenon dependent upon the previously discussed mechanisms. The nature of these mechanisms and their interaction makes it difficult to predict with certainty whether a particular characteristic will be the overriding factor in determining moisture susceptibility. In general, moisture susceptibility is increased by any factor that increases moisture content in the HMA, decreases the adhesion of asphalt binder to the aggregate surface or physically scours the asphalt binder. Each of the factors listed below influence moisture susceptibility to some degree but no single one is a foolproof benchmark for predicting moisture susceptibility.

XIII. **Asphalt binder characteristics.** Viscosity is important because it may indicate higher concentrations of asphaltenes (large polar molecules). Polar molecules can create greater adhesion tension and molecular orientation adhesion. Therefore, lower viscosities, which may represent lower concentrations of asphaltenes, are generally more susceptible to stripping. Individual components in asphalt binder such as sulfoxides, carboxylic acids, phenols and nitrogen bases can also affect stripping potential.

XIV. **Aggregate characteristics.** In general, aggregates that are hydrophilic (attract water) are more likely to strip than aggregates that are hydrophobic (repulse water). To address this, either stripping-susceptible aggregates can be avoided or an anti-stripping asphalt binder modifier can be used. The key aggregate properties that determine this hydrophilic/hydrophobic characteristic are:

XV. **Surface chemistry.** Surfaces that can more readily form bonds with the asphalt binder are less likely to cause stripping. In general, a more acidic aggregate surface is more susceptible to stripping. Iron, magnesium, calcium and perhaps aluminum are considered beneficial, while sodium and potassium are considered detrimental (Hicks, 1991).

XVI. **Porosity and pore size.**Pore size is the critical factor. If pores are large enough to allow asphalt binder entry, they may be a contributor to moisture susceptibility. High porosity results in high absorption, which means that more asphalt binder must be used to achieve the desired effective asphalt binder content. Conversely, if high porosity is not considered, for a given amount of asphalt binder, more will be absorbed and less will be available to create the asphalt binder film around aggregate particles causing faster aging and possibly stripping.

XVII. **Air voids.** The extent to which pores in the aggregate absorb asphalt binder affects the volume of air voids in the HMA mixture. When HMA air voids exceed about 8 percent by volume, they may become interconnected and allow water to easily penetrate the HMA and cause moisture damage through pore pressure or ice expansion. To address this, HMA mix design adjusts asphalt binder content and aggregate gradation to produce design air voids of about 4 percent. Excessive air voids can be either a mix design or a construction problem and this section only addresses the mix design problem.

XVIII. **Construction weather.** Cool weather construction can lead to insufficient compaction, resulting in high air voids and a relatively permeable HMA pavement. This increases the likelihood of water in the pavement structure and thus, moisture damage. Wet weather can also increase the moisture content in the constructed HMA.

XIX. **Climate.** Wetter climates, freeze-thaw cycles and temperature fluctuations can all allow more moisture into the HMA structure thus increasing the likelihood of moisture damage.

XX. **Traffic.** If water is present in the HMA structure, increased traffic loading can accelerate moisture damage for 2 reasons:

XXI. **Pore pressure buildup.** If water is in the aggregate pores and cannot escape, traffic loading will tend to compress these pores and cause a pressure buildup, which could push asphalt binder away from the aggregate surface.

XXII. **Hydraulic scouring.** Wheel passes over a HMA pavement tend to move water in the pavement. This movement causes a scouring action that could remove asphalt binder from the aggregate surface.

I. The interactions of these variables and the different level of interaction at whichlaboratory test methods can measure relevant properties or simulated performance are shown in**Figure 2.17**.

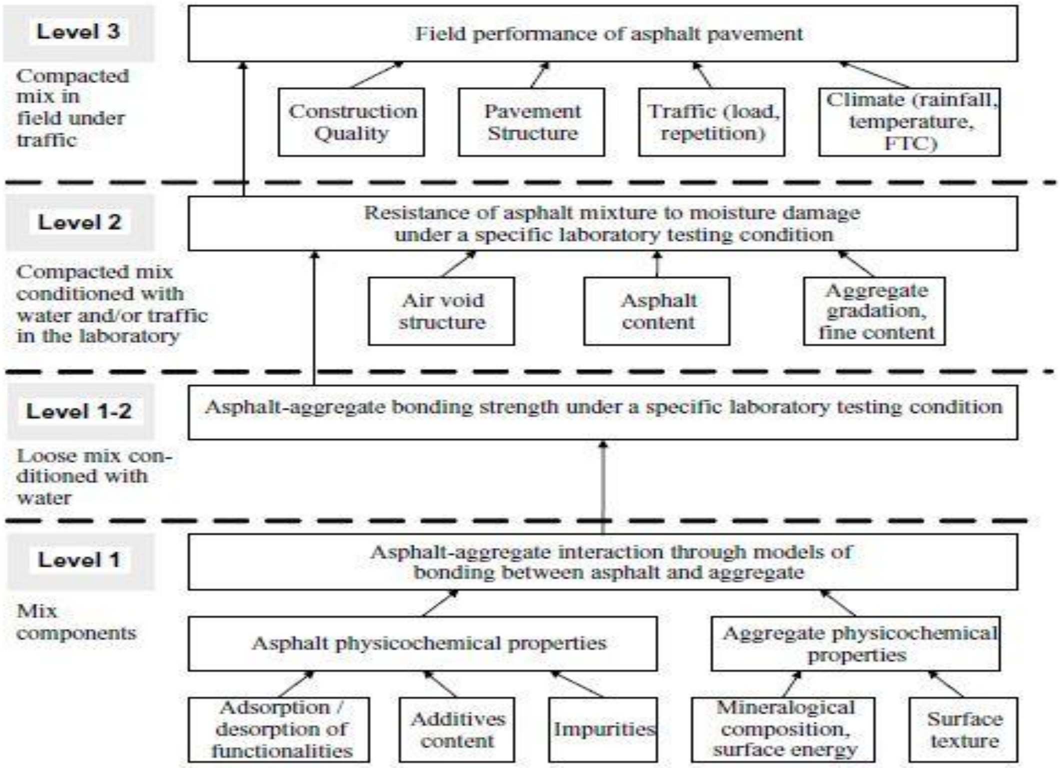


Figure 2.17: Factors influencing moisture damage of asphalt pavements

II. Traffic, which applies stresses to the mix while it is in a weakened condition frommoisture and has been shown in several studies to determine whether moisture damage andstripping occur by comparison of cores from the wheelpath with those from outside theWheel path.

There are three primary difficulties in performing a comprehensive field calibration

III. Obtaining comprehensive data for the independent variables listed above;

IV. Quantifying the dependent variable, performance; and

V. Relating results from laboratory- and field-compacted test specimens.

2.5.4 Preventive Measures

Various measures can be taken to prevent, or at least minimize, moisture damage. These measures range from material selection, to construction practice, pavement design and HMA additives

VI. **Aggregate selection.** Choose low porosity aggregate with rough, clean surfaces.

VII. **Prevent moisture penetration into the HMA pavement.** Reduce the permeability of the pavement structure by manipulating air void content, lift thickness and gradation (Figure 5 and Figure 6). Additionally, surface treatments such as fog seals, slurry seals or bituminous surface treatments (BSTs) can essentially waterproof the HMA surface.

VIII. **Pretreat aggregate.** Modify aggregate surface properties to replace ions that are likely to contribute to poor asphalt binder-aggregate adhesion.

IX. **Anti-strip additives.**Add chemicals or lime to the asphalt binder or HMA to prevent moisture damage (**Figure 2.18**).

X. **Chemicals.** Generally work to reduce surface tension in the asphalt binder, which promotes better wetting, as well as impart an electrical charge to the asphalt binder that is opposite that of the aggregate surface charge. Most chemical additives contain amines and are added at about 0.1 to 1.0 percent by weight of asphalt binder. Chemical additives are generally added to asphalt binder prior to mixing with aggregate but this can cause some waste as not all the additive is guaranteed to reach the critical asphalt binder-aggregate interface. Some additives can be added to the aggregate before mixing with asphalt binder so that all the additive is on the aggregate surface.

XI. **Lime (Figure 2.19).** Works by replacing negative ions on an aggregate surface with positive calcium ions, resulting in better asphalt binder-aggregate adhesion. Also reacts with molecules in both the asphalt binder (carboxylic acid) and aggregate (acidic OH groups) that results in molecules that are more readily absorbed on the aggregate surface or molecules that are less likely to be dissociate and associate with water molecules. Lime is usually added at about 1.0 to 1.5 percent by total aggregate weight. Moisture is needed to activate the lime, so lime is usually added as a slurry or added to slightly moist aggregate.

Figure 2.18: Aggregate on left has severe stripping. Right has 0.5 percent by weight asphalt binder antistripping modifier.

Figure 2.19: Lime in small containers for addition during mix design sample preparation.

2.5.5 Moisture Susceptibility Tests

In general, moisture susceptibility tests do not measure individual factors but rather attempt to quantify a HMA mixture's ability to resist moisture damage, no matter what the source. They are typically capable of providing gross results or comparative results and are not able to predict the degree of moisture damage. A brief description of the major tests for moisture susceptibility follows:

XII. **Boiling test (ASTM D 3625).** Add loose HMA to boiling water and measure the percentage of total visible area of aggregate surface that retains its asphalt binder coating. The test is simple but is subjective, does not involve any strength determination and examining the fine aggregate is difficult.

XIII. **Static-immersion test (AASHTO T 182).** HMA sample is immersed in water for 16 to 18 hours and then observed through the water to measure the percentage of total visible area of aggregate surface that retains its asphalt binder coating. This test is also simple but subjective and does not involve any strength determination.

XIV. **Lottman test.** Tests 3 sets of compacted samples. Group 1, the control group, is not conditioned. Group 2, representing field performance at 4 years, is subjected to vacuum saturation with water. Group 3, representing field performance at 4 to 12 years, is subjected to vacuum saturation and a freeze-thaw cycle. A split tensile test is performed on each sample and the ratio of the indirect tensile strength of the conditioned samples is compared to the control group as a ratio. A minimum tensile strength ratio (TSR) of 0.70 to 0.80 is often used as a standard.

XV. **Tunnicliff and Root conditioning.** Similar to the Lottman test, this test uses only 2 groups and eliminates the freeze-thaw group.

XVI. **Modified Lottman (AASHTO T 283).** A combination of the Lottman and Tunnicliff and Root tests. It compares the split tensile strength of unconditioned samples to samples partially saturated with water. The test subjects the conditioned group to partial vacuum saturation and an optional freeze-thaw cycle. Although it is expected that the water conditioned samples will have a lower tensile strength, excessively low values indicate the potential for moisture damage.

XVII. **Immersion-compression (AASHTO T 165).** Similar to a modified Lottman test, but the conditioned samples are only placed in water (not vacuum saturated) and an unconfined compressive strength test is used instead of the split tensile test. Precision is not good and samples showing obvious signs of stripping can give a strength ratio of near 1.0.

XVIII. **Hamburg wheel-tracking device.** Compacted HMA samples are tested underwater. Results give a relative indication of moisture susceptibility.

All of these tests have weaknesses that result in an ongoing search for a better moisture susceptibility test. These weaknesses, in addition to the ones discussed above, tend to be issues with repeatability and reproducibility of test results and questionable predictive ability. Also, small variations in key HMA parameters such as air voids (V_a), can substantially affect test results.

2.5.6 Tensile Strength Ratio (TSR) Test

The following description is a brief summary of the test. It is not a complete procedure and should not be used to perform the test. The complete test procedure can be found in:

XIX. **AASHTO T 283:** Resistance of Compacted Bituminous Mixture to Moisture-Induced Damage.

XX. **ASTM D 4867:** Effect of Moisture on Asphalt Concrete Paving Mixtures.

Two sets of HMA samples are subjected to a split tensile test (often called an indirect tensile test). One set is conditioned by partial vacuum saturation with water, soaking in water for 24 hours and an optional freeze-thaw cycle. The other set is used as a control. The ratio of the average split tensile strength of the conditioned samples over the average split tensile strength of the unconditioned (control) samples is reported as the **Tensile Strength Ratio (TSR)**. **Figure 2.20** shows the split tensile test setup.

Approximate Test Time

The total test time can be up to 6 days. Major components are:

XXI. Up to 4 days of sample preparation

XXII. 16 hours for the freeze cycle

XXIII. 24 hours for the thaw cycle

XXIV. 2 hours for getting samples to test temperature

XXV. 30 minutes to run conditioned and unconditioned sample sets through the indirect tensile test

Figure 2.20: Split tensile test setup.

Basic Procedure

XXVI. Prepare 6 HMA samples. Samples are usually 6 inches (150 mm) in diameter and 4 inches (100 mm) thick. After mixing has occurred, allow the HMA to cool to room temperature for 2 hours.Samples of other sizes may be used. If aggregate larger than 1 inch (25 mm) is present in the HMA, a larger sample size should be used.

XXVII. Cure the HMA in an oven at 140°F (60°C) for 16 hours.

XXVIII. After curing, place HMA in an oven at 275°F (135°C) for two hours before compaction.

XXIX. Compact mix to 7 percent air voids, or a void level expected in the field, using the SGC, California kneading compactor or Marshall hammer.

XXX. Store the compacted samples at room temperature for 72 to 96 hours.

XXXI. Determine the theoretical maximum specific gravity (Gmm), bulk specific gravity (Gmb), height, volume and air void content (Va) of each sample. (S**tep 6)**

XXXII. Divide the six samples into two subsets of three. The average air void content (Va) for each subset should be similar. One subset will be "unconditioned" (tested in a dry state) and the other will be "conditioned" (tested in a saturated state).

XXXIII. Unconditioned samples. While the conditioned samples are being conditioned, the unconditioned samples sit at room temperature.

XXXIV. Wrap samples in plastic or put them in a heavy duty leak proof bag.

XXXV. Store samples at room temperature until testing.

XXXVI. Conditioned samples. These samples are saturated with water to between 55 and 80 percent using the following procedure:

XXXVII. Place each sample in a vacuum container supported above the container bottom by a spacer and fill the container with water until the sample is covered by 1 inch (25 mm) of water.

XXXVIII. Apply a vacuum of 10 – 26 inches Hg partial pressure (13 – 67 KPa absolute pressure) for 5 to 10 minutes (**Figure 2.21**).

Figure 2.21: Vacuum saturation of a sample (https://pavementinteractive.org/reference-desk/testing/asphalt-tests/moisture-susceptibility/)

XXXIX. Remove the vacuum and let the sample sit under water for another 5 to 10 minutes.

XL. Calculate bulk specific gravity (Gmb) and compare the SSD mass with the SSD mass obtained in **Step 6** to determine the volume of absorbed water.

XLI. Determine degree of saturation by comparing volume of absorbed water with volume of air voids (Va) obtained in **Step 6**. f the calculated saturation of a sample is below 55 percent, repeat the saturation procedure. If the calculated saturation of a sample is above 80 percent, the sample is considered damaged and must be discarded. If freeze-thaw conditioning is desired, wrap each sample in plastic and place it in a plastic bag containing 0.6 in3 (10 mL) of water. Seal the bag and place it in a freezer at 0°F (-18°C) for at least 16 hours.

XLII. Moisture conditions the samples by placing them in a bath of distilled water at 140°F (60°C) for 24 hours (**Figure 2.22**). If the samples were freeze-thaw conditioned, remove the plastic from the samples as soon as possible after placement in the bath.

Figure 2.22: Moisture conditioning bath.

I. Place samples in a 77 °F (25 °C) water bath for a minimum of 2 hours (**Figure 2.23**).

Figure 2.23: Filling the 2-hour room temperature water bath.

II. Run an indirect tension test on each sample by placing the sample between the two bearing plates (**Figure 2.24**) in the testing machine and applying the load at a constant rate of 2 inches/minute (50 mm/minute). Make sure the load is applied along the diameter of the sample.

Figure 2.24: Sample placed between the bearing plates before testing.

III. Record the tensile strength values and calculate and report the tensile strength values.

Results

Parameters Measured

The ultimate parameter to be measured is the tensile strength ratio (TSR). However, in order to get this measurement the following other parameters need to be measured:

IV. Theoretical maximum specific gravity (Gmm) of each sample

V. Bulk specific gravity (Gmb) of each sample

VI. Air void content (Va) of each sample

VII. Percent saturation of the conditioned samples

Calculations (Interactive Equation)

Calculate the tensile strength as follows:

$$S_t = \frac{2P}{\pi t D}$$

Where:

VIII. St = tensile strength (psi)

IX. P = maximum load (lbs)

X. t = sample thickness (inches)

XI. D = sample diameter (inches)

Express the resistance to moisture damage as a ratio of the unconditioned sample tensile strength that is retained after the conditioning.

Calculate the TSR as follows:

$$TSR = \frac{S_2}{S_1}$$

Where:

XII. TSR = tensile strength ratio

XIII. S1 = average tensile strength of unconditioned samples

XIV. S2 = average tensile strength of conditioned samples

Specifications

Table 2.15: Asphalt Mix Design Moisture Susceptibility Specification.

Material	Value	Specification	HMA Distress of Concern
HMA	Tensile Strength Ratio (TSR)	≥ 0.80	Moisture damage, stripping

Typical Values

Typical **TSR** values range from 0.70 to 0.90. Depending on the type of HMA mixture, it is not uncommon to see values below 0.70 or above 0.90.

2.6Life-cycle cost analysis (LCCA)

Life-cycle cost analysis (LCCA) is a method for assessing the total cost of facility ownership. It takes into account all costs of acquiring, owning, and disposing of a building or building system. LCCA is especially useful when project alternatives that fulfill the same performance requirements, but differ with respect to initial costs and operating costs, have to be compared in order to select the one that maximizes net savings. For example, LCCA will help determine whether the incorporation of a high-performance HVAC or glazing system, which may increase initial cost but result in dramatically reduced operating and maintenance costs, is cost-effective or not. LCCA is not useful for budget allocation.

FHWA has pursued a policy of promoting LCCA for transportation investment decisions since the Intermodal Surface Transportation Equity Act of 1991. Throughout the 1990s FHWA investigated LCCA. In fall 1996, FHWA initiated a technolog6 LIFE-CYCLE COST ANALYSIS PRIMERtransfer effort under Demonstration Project 115, "Life-Cycle Cost Analysis in PavementDesign." This project resulted in an LCCA instructional workshop that has since beendelivered to more than 40 State transportation agencies. In 1998, FHWA issued an InterimTechnical Bulletin on LCCA, Life-Cycle Cost Analysis in Pavement Design. FHWA is currently developing instructional software and will continue to provide technical assistanceand training to assist individual transportation agencies as they explore the use of LCCAfor pavement design decisions.

2.6.1 Purpose of LCCA

LCCA is an analysis technique that builds on the well-founded principles of economic analysisto evaluate the over-all-long-term economic efficiency between competing alternativeinvestmentoptions. It does not address equity issues. It incorporates initial and discounted future agency,user, and other relevant costs over the life of alternative investments. It attempts to identify thebest value (the lowest long-term cost that satisfies the performance objective being sought) forinvestment expenditures.LCCA Requirements The National Highway System (NHS) Designation Act of 1995 specifically required States to conduct life-cycle cost analysis on NHS projects costing $25 million or more. Implementing guidance was provided in Federal Highway Administration (FHWA) Executive Director Anthony Kane's April 19, 1996, Memorandum to FHWA Regional administrators. The implementing guidance did not recommend specific LCCA procedures, but rather itspecified the use of good practice.The FHWA position on LCCA is further defined in its Final Policy Statement on LCCApublished in the September 18, 1996, Federal Register. FHWA Policy on LCCA is that it is adecision support tool, and the results of LCCA are not decisions in and of themselves. Thelogical analytical evaluation framework that life-cycle cost analyses fosters is as important as theLCCA results themselves. As a result, although LCCA was only officially mandated in a verylimited number of situations, FHWA has always encouraged the use of LCCA in analyzing allmajor investment decisions where such analyses are likely to increase the efficiency andeffectiveness of investment decisions whether or not they meet specific LCCA-mandatedrequirements.The 1998 Transportation Equity Act for the 21st Century (TEA-21) has since removed therequirement for SHA's to conduct LCCA on high-cost NHS useable project segments.However, the congressional interest in LCCA is continued in the new requirement that theSecretary of Transportation develop recommended LCCA procedures for NHS projects.

The purpose of an LCCA is to estimate the overall costs of project alternatives and to select the design that ensures the facility will provide the lowest overall cost of ownership consistent with its quality and function. The LCCA should be performed early in the design process while there is still a chance to refine the design to ensure a reduction in life-cycle costs (LCC).

2.6.2 Life-Cycle Cost Analysis (LCCA) Method

Transportation assets are constructed to provideservice for generations. Competing design alternativesmay each have a different service life,which is the time period that the asset will remainopen for public use. Life-cycle cost analysis(LCCA), however, uses a common periodof time to assess cost differences between thesealternatives so that the results can be fairly compared.This time period is termed the "analysisperiod." Allowing analysis periods to vary amongdesign alternatives would result in the comparisonof alternatives with different total benefitlevels, which is not appropriate under LCCA.The analysis period should demonstrate thetotal cost differences between the alternatives.Accordingly, the analysis period should be longenough to include the initial construction ormajor rehabilitation action and at least one subsequentrehabilitation action for each alternative.However, each alternative does not need to havethe same number of maintenance or rehabilitationactivities during the analysis period.

The LCCA process steps are listed below. The stepsare ordered so that the analysis builds upon informationgathered in prior steps.

I. Establish design alternatives
II. Determine activity timing
III. Estimate costs (agency and user)
IV. Compute life-cycle costs
V. Analyze the results

The LCCA approaches and techniques outlined in thissection are consistent with FHWA's LCCA Interim TechnicalBulletin, Life-Cycle Cost Analysis in Pavement Design,which was published in 1998. The Interim TechnicalBulletin provides a more detailed discussion of this methodologyand its components, particularly with regard touser cost calculations and the treatment of uncertainty in an analysis.

Step One: Establish Design Alternatives

The LCCA process is initiated after an asset has been selectedfor improvement and a range of possible alternativeshas been identified for accomplishing that improvement. Atleast two mutually exclusive options must be considered,and the economic difference between alternatives is assumedto be attributable to the total cost of each.

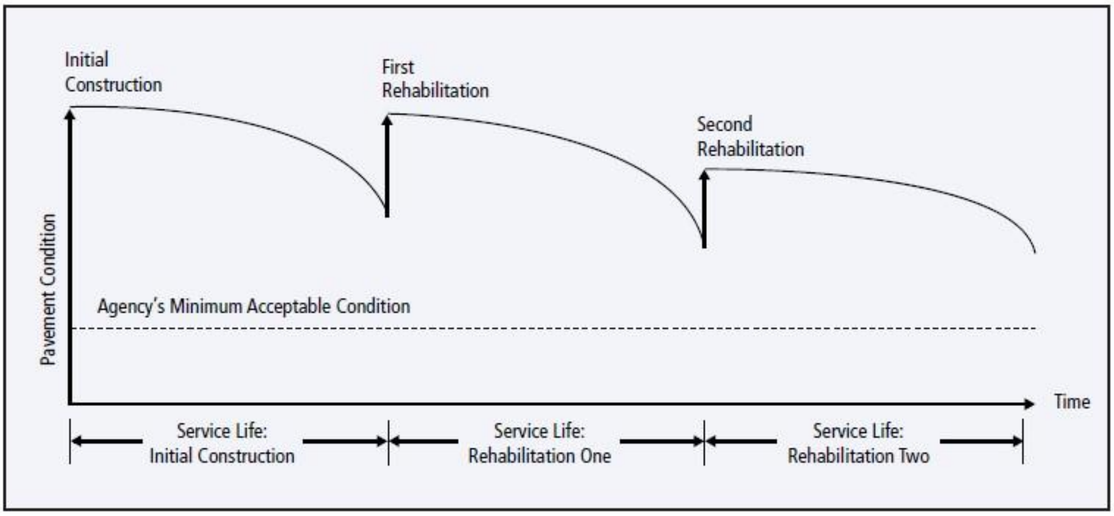


Figure 2.25: Example Lifetime of One Design Alternative (LCCA Handbook, 2005)

In the first LCCA step, component activities for eachalternative are detailed and the analysis period is defined.Each alternative is defined by the agency activities thatcreate and maintain it. Initial construction or a major rehabilitationof an asset is only the first of these activities;periodic maintenance and subsequent rehabilitation arerequired for the alternative to provide a specified level ofperformance throughout its life. Different project alternativeswill likely require different maintenance and rehabilitationactivities. Typically, the identification of maintenanceand rehabilitation activities is based on historicalpractice, research, and agency policies.Important in this first step is defining the analysis period,the common timeframe for which initial and futurecosts will be evaluated for all alternatives being considered.In general, the analysis period should be longenough to include at least one major rehabilitation activityfor each alternative being considered.

Step Two: Determine Activity Timing

After the component activities for each competing projectalternative have been identified, each alternative's maintenanceand rehabilitation plan is developed. Effectively,this plan results in a schedule of when the future maintenanceand rehabilitation activities will occur, when agencyfunds will be expended, and when and for how long theagency will establish work zones.When first constructed or substantially rehabilitated,transportation assets are in good condition and provideservice as originally intended. Use, age, weather, and otherfactors cause assets to deteriorate, and deterioration causesthe level of performance provided by the asset to fall. Periodicmaintenance and rehabilitation activities will arrestdeterioration and improve the asset's condition so asto maintain sufficient levels of condition, performance,and safety.

Each agency decides when to perform theseactivities, usually based on the desired level of serviceability.**Figure 2.25** demonstrates the cycle of construction, deterioration,and rehabilitation that a typical transportationasset undergoes. As the asset's condition nears the agency'sminimum acceptable condition, rehabilitation activitiesare conducted. The rate of deterioration, as influencedby pavement preservation practices, dictates the timingof future activities. Initial activities occur in the beginningof the analysis period, and future activities are shownin the years they are anticipated.LCCA requires that the series of maintenance and rehabilitationactivities forecasted for each improvementstrategy be as accurate as possible because the expensesassociated with these activities can account for a sizeableportion of a project's total LCC. The timing of rehabilitationactivities should be based on existing performancerecords such as those available from an agency's pavementor bridge management system. This informationmay be supplemented with findings from outside researchsuch as the national long-term pavement performanceeffort. Other data are available from local, regional, andnational sources. When actual data are unavailable or notapplicable, the judgment of experienced engineers maybe particularly useful.

Step Three: Estimate Costs

Costs considered in LCCA include those accruing tohighway agencies and to users of the highway system as aresult of agency construction and maintenance activities.LCCA does not require that all costs associated witheach alternative be calculated. Only costs that demonstratethe differences between alternatives need be explored.This is an important distinction because it maysimplify the analytical and data requirements considerably.In the case of agency costs, this means, for example,that rehabilitation activities should be included, but expensescommon to all the alternatives (e.g., land costs)may be removed from the analysis. Although user costsmay differ among alternatives over the entire analysisperiod, significant differences of importance to the LCCAprocess are usually associated with agency actions thatrequire work zone activity.When estimating future costs for an LCCA, it is appropriateto develop those costs in constant dollars.For example, the same material and laborcosts used to price an activity in the base year of the analysisshould generally be used to value them in any futureyear of the analysis.

Agency Costs

Critical to an insightful LCCA are good estimates of thevarious agency cost items associated with initial constructionand periodic maintenance and rehabilitation activities.Construction costs pertain to putting the asset intoinitial service. Data on construction costs are obtainedfrom historical records, current bids, and engineeringjudgment (particularly when new materials and techniquesare employed).Similarly, costs must be attached to the maintenanceand rehabilitation activities identified in the previous stepsto maintain the asset above some predetermined condition,performance, and safety levels. These costs includethose for preventive activities that are planned to extendthe life of the asset, day-to-day routine maintenance intendedto address safety and operational concerns, andrehabilitation or restoration activities.Another consideration affecting total agency costs isthe value of the alternative at the end of the analysis period.One type of terminal value is called "salvage value,"usually the net value from the recycling of materials atthe end of a project's life. A second type of terminal valueis the "remaining service life" (RSL) value of an alternative(the residual value of an improvement when its servicelife extends beyond the end of the analysis period). TheRSL value may vary significantly among different alternatives,and should be included in the LCCA.

User Costs

Best-practice LCCA calls for including both the costsaccruing to the transportation agency, described above,and costs incurred by the traveling public. In LCCA, usercosts of primary interest include vehicle operating costs,travel time costs, and crash costs. Such user costs typicallyarise from the timing, duration, scope, and numberof construction and rehabilitation work zones characterizingeach project alternative. Because work zones typicallyrestrict the normal capacity of the facility and reducetraffic flow, work zone user costs are caused by speedchanges, stops, delays, detours, and incidents. While usercosts do result during normal operations, these costs areoften similar between alternatives and may be removedfrom most analyses.Incorporating user costs into LCCA enhances the validityof the results, but at the same time is a challengingtask. Some of these challenges are discussed later in thisPrimer.

Step Four: Compute Life-Cycle Costs

In previous steps, the alternatives were defined with respectto agency costs, user costs, and the time when theseevents will occur. At this point, the objective is to calculatethe total LCCs for each alternative so that they maybe directly compared. However, because dollars spent atdifferent times have different present values, the projectedactivity costs for an alternative cannot simply be addedtogether to calculate total LCC for that alternative. Economicmethods are available to convert anticipated futurecosts to present dollar values so that the lifetime costsof different alternatives can be directly compared.

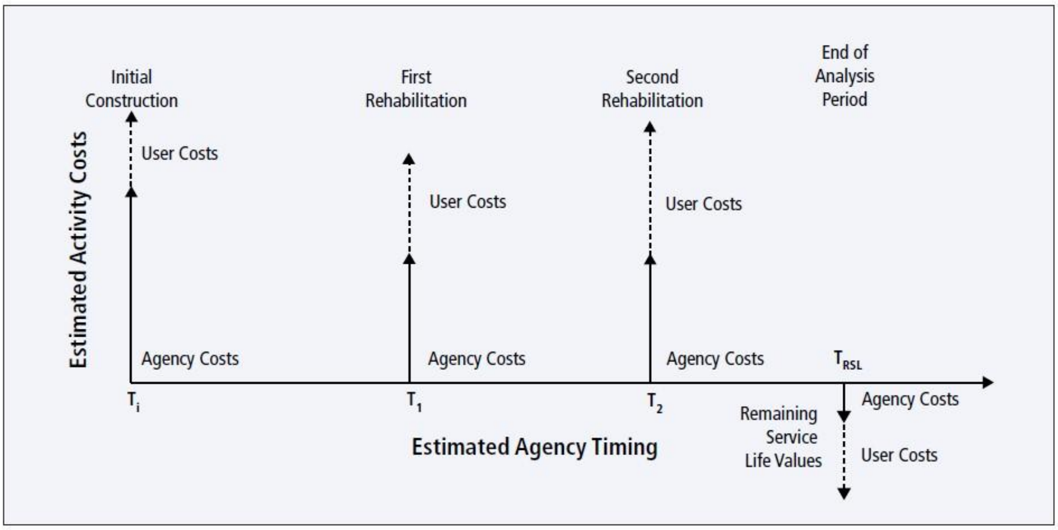


Figure 2.26: Expenditure Stream Diagram, Showing Activities, Costs, and Timing

Expenditure Stream Diagrams

To assist the analyst in visualizing the quantity and timingof expenditures projected over the life of the analysisperiod, expenditure stream diagrams may be developed.An expenditure diagram (see **Figure 2.26**) depicts a designalternative's (1) initial and future activities; (2) agency anduser costs associated with these activities; and (3) the timingof these activities and costs. Upward arrows on thediagram are expenditures with the relative costs reflectedin the length of each arrow. The horizontal arrow segmentsshow the timing of the work zone activities andthe periods of normal operations between them. The RSLvalue (or the salvage value, if the asset is to be terminated) is represented as a downward arrow and reflects anegative cost (or cost offset) accruing at the end of theanalysis period. The value of an expenditure stream diagramis that it presents in a simple graphic all the costand timing inputs required to perform an LCCA.

Economic Analysis Technique

Important to understanding LCCA is the concept of thetime value of money. A given amount of money receivedtoday has a higher value than the same amount receivedat a later date. One way to understand this concept is thatfunds received today may be invested and immediatelybegin to earn interest. The time value of money is germaneto LCCA because costs included in the analysis areincurred at varying points in time.For LCCA, costs occasioned at different times mustbe converted to their value at a common point in time. Anumber of techniques based on the concept of discountingare available. The FHWA recommends the presentvalue (PV) approach (also known as "present worth"), butthe equivalent uniform annual cost (EUAC) approach isalso commonly used (see the box on this page). Eithermethod is suitable as a measure of LCC. The PV approachbrings initial and future dollar costs to a singlepoint in time, usually the present or the time of the firstcost outlay. The box on the next page discusses dollars,inflation, and discounting, and supplies the formula forcalculating the PV of any cost component.

Computational Approach

There are two approaches to preparing an LCCA: **Deterministic**and **Probabilistic**. The methods differ in the waythey address the variability and uncertainty associated withLCCA input parameters such as activity cost, activity timing,and discount rate.

Deterministic Approach

The deterministic approachassigns each LCCA input variable a fixed, discrete value.The analyst determines the value most likely to occur foreach input parameter. This determination is usually basedon historical evidence or professional judgment. Collectively,these input values are used to compute a singleLCC estimate. Traditionally, applications of LCCA havebeen deterministic ones. A deterministic LCC computationis straightforward and can be conducted manuallyusing a calculator or automatically with a spreadsheet.However, it fails to convey the degree of uncertainty associatedwith the PV estimate. The results of deterministic analysis can be enhancedthrough the use of a technique called sensitivity analysis.This procedure involves changing a single input parameterof interest, such as the discount rate or initial cost,over the range of its possible values while holding all otherinputs constant, and estimating a series of PVs (outputvalues). Each PV result will reflect the effect of the inputchange. In this way input variables may be ranked accordingto their impacts on the bottom-line conclusions.This information is important to decision-makers whowant to understand the variability associated with alternativechoices.

It also allows the agency to identify thoseinput factors or economic conditions that warrant specialattention in terms of their estimation procedures.Deterministic sensitivity analysis is not well suited tomeasuring the impact that a simultaneous change ofseveral inputs would have on a particular LCCA outcome.In addition, it does not give any information on thelikelihood that a selected input value will actually occur.Therefore, while a deterministic LCCA approach providesconsiderably more information about the economicreasonableness of a project than just its initial cost, it doesnot offer decision-makers a complete picture of the expectedPVs.

Probabilistic Approach

With deterministic LCCA, discretevalues are assigned to individual parameters. In contrast,probabilistic LCCA allows the value of individualanalysis inputs to be defined by a frequency (probability)distribution. For a given project alternative, the uncertaininput parameters are identified. Then, for each uncertainparameter, a sampling distribution of possible valuesis developed. Simulation programming randomlydraws values from the probabilistic description of eachinput variable and uses these values to compute a singleforecasted PV output value. This sampling process isrepeated through thousands of iterations. From this iterativeprocess, an entire probability distribution of PVsis generated for the project alternative along with themean or average PV for that alternative. The resultingPV distribution can then be compared with the projectedPVs for alternatives, and the most economical option forimplementing the project may be determined for anygiven risk level.Probabilistic LCCA accounts for uncertainty andvariation in individual input parameters. It also allowsfor the simultaneous computation of differing assumptionsfor many different variables. It conveys the likelihoodthat a particular LCC forecast will actually occur.

The formula to discount future constant value costs to present value is

Present Value = Future Value x 1/ (1+r) n

Where,**r** = real discount rate and **n** = number of years in the future when the cost will be incurred.

The term **1/ (1+r) n** is known as the discount factor and is always less than or equal to one.

Using this formula,a $1,000 cost incurred in year 30, discounted to the present (year zero) at a 4 percent real discount rate,would have a present value of $308.It should be noted that the term “net present value” (NPV) is sometimes used when referring to thepresent value of life cycle costs. However, NPV is more appropriately used in benefit-cost analysis to conveythe net difference between the present values of benefits and costs of an alternative or project.

From the perspective of most transportation agencies, theapplication of probabilistic LCCA is relatively new.Probabilistic LCCA has been made more practical dueto the dramatic increases in computer processing capabilitiesof the last two decades. Simulating and accountingfor simultaneous changes in LCCA input parametersmay now be accomplished easily and quickly.

Step Five: Analyze the Results

Step five involves analyzing and interpreting the LCCAresults. With the deterministic or probabilistic LCCscomputed, the PVs of the differential costs may be comparedacross competing alternatives. Because the deterministicapproach results in a single PV for each alternativeand the probabilistic approach yields a distributionof PV results, the procedures used to analyze the resultsare different.Although best-practice LCCA considers both agencyand user costs, in actual practice many analysts are reluctantto assign the same level of validity to user costs thatthey assign to agency costs. Thus, alternatives are oftencompared chiefly on agency costs. User costs may be comparedto see if an alternative has a disproportionately highor low impact on users compared to other alternatives. If the lowest agency-cost alternative also has a disproportionatelyhigh user-cost impact, the analyst may use thisinformation to revisit that alternative to mitigate usercosts, or may recommend that an alternative with somewhathigher agency costs but much lower user costs be pursuedin preference to the lowest-agency-cost alternative.

Analysis of Deterministic LCCA Results

The most basic analysis of a deterministic LCCA is tocompare the agency and user cost PVs among alternatives.However, this comparison does not address theuncertainty contained in those outputs.As noted above, application of sensitivity analysis canreveal where analysis results may be subject to uncertainty.Deterministic sensitivity analysis is helpful in determiningthe “most likely” scenario where the selected inputvalues are most likely to occur (based on objective dataor expert opinions). Ideally, the “best” alternative will havethe lowest PV in the most likely of “what-if” situations.

Analysis of Probabilistic LCCA Results

Probabilistic LCCA attempts to model and report on thefull range of possible PV outcomes. It also shows the estimatedlikelihood that any given outcome will actuallyoccur. The analyst is able to array this information sothat the underlying uncertainty inherent in each projectalternative is reflected in the PV output results. Thisanalysis also provides important statistical information toassist the decision-maker.

As with deterministic LCCA, probabilistic LCCA canbe enhanced by incorporating sensitivity analysis into theprocess. The sensitivity analysis will point to the variablesmost significant in influencing the LCCA results.When interpreting the probabilistic LCCA, decisionmakersmust define the level of risk with which they aremost comfortable. For example, those with a low tolerancefor risk prefer less variability in the results, whichmay affect their selection between two or more options.In this case, the decision-maker may select an alternativewith a somewhat higher PV but with much lower risk ofcost overrun.

Reevaluate Alternatives

The LCCA concludes with a review of the findings todetermine if adjustments or modifications to any of theproposed alternatives might be indicated prior to finalizingthe alternative selection. Revisions might includedesign changes, newly defined work zone criteria forthe contractors, or altered traffic plans to reduce highuser costs.

The FHWA encourages the use of LCCA for certain transportation investmentdecisions. LCCA is an important analytical tool that is applicable to a broadrange of routine decisions facing State and local transportation agencies. It isappropriately applied once a decision has been made to undertake a project or improvementbut the specific design for accomplishing the project's objectives has notbeen chosen.

The LCCA methodology provides a structured approach to evaluating design alternatives.By focusing on the project life cycle, it prompts the analyst to address not onlythe initial costs of a project, but the timing, scope, and resources required for futurerehabilitation and maintenance activities. Best-practice LCCA also directs the analystto quantify and compare the effects of different project implementation options onhighway users, who may experience significant costs due to congestion and safety issuesassociated with work zones.

2.7 Summary

As seen in the literature review, Portland cement was used in asphalt mix as filler and as a part of wearingcourse; it was used in laboratory specimens.So this research will study its use as filler in asphalt mix in the binder course to investigate the characteristics of bituminous materials and construct a road strip to investigate field condition. The comparative cost analysis of road construction in Bangladesh considering a typical road section is also taken in this research to carry outthe Road construction cost Scenario in a cost effective manner by using cement as filler material.

CHAPTER 3

MATERIALS AND EXPERIMENTAL PROGRAM

3.1 General Information

This chapter highlights the materials and the materials properties used during the laboratory testing such as bitumen, aggregates and Portland cement. Also it illustrates how experimental work has been done to achieve the objectives of the research.

3.2 Materials

The raw materials, used for this study are natural aggregates, bitumen, and cement. The main and local sources of these materials are presented in the **Table 3.1**.

Table 3.1: Sources of used materials.

Material	Source
	Local
Bitumen	Importer (Dhaka City)
Aggregates	Importer (Dhaka City)
Portland Cement	Crown Cement (Dhaka City)

3.2.1 Bitumen (Asphalt Cement)

In this research, a kind of asphalt binder with 60-70 penetration grade was used for producing all test specimens. Physical property tests for this asphalt cement were conducted in the Transportation Engineering Laboratory of Bangladesh University of Engineering and Technology (BUET). **Table 4.10** shows the physical properties of used bitumen.

3.2.2 Aggregates

The aggregates commonly used for asphalt mixes are natural fine and coarse aggregates. Used aggregates are presented in **Table 3.2**. **Figure 3.1** shows aggregate types.

Table 3.2: Used aggregates types.

Type of Aggregates	Type of aggregates *	Particle size (mm)
Coarse	Pakur Stone	19.00
Fine	Sylhet Sand	1.18

*** Local names of aggregates**

Pakur Stone **Pakur Stone** **Sylhet Sand**

Figure 3.1: Types of used aggregates

3.2.3 Aggregates properties

Laboratory tests have been conducted to evaluate the physical properties of used aggregates. Gradation tests were conducted to determine the size distribution for each aggregate type.

3.2.4 Physical properties of aggregates

Required tests were performed on the aggregate to evaluate their physical properties. The results together with the specification limits are summarized in **Table 3.3**.

Table 3.3: Physical properties of used aggregates.

Test	ASTM designation	CoarseAggregates (Pakur Stone)	FineAggregates (Sylhet Sand)
Bulk dry S.G	C127	2.95	2.65
Bulk SSD S.G		2.96	2.66
Apparent S.G		2.98	2.68
Effective S.G		2.97	2.67
Absorption (%)	C128	0.42	0.45

3.2.5 Sieve analysis of aggregates

A gradation test according to specification (ASTM C 136) is performed on a sample of used aggregate for each type of aggregate in a laboratory and the results are presented below in **Table 3.4** and **Figures 3.2 - 3.4**.

Table 3.4: Aggregates sieve analysis results

Sieve size (mm)	Sieve #	Sample passing %	
		CoarseAggregates (Pakur Stone)	FineAggregates (Sylhet Sand)
19	3/4"	76.51	100.00
12	1/2"	43.24	100.00
9.5	3/8"	26.62	100.00
4.75	#4	0.07	96.73
2.36	#8	0.07	91.78
1.18	#16	0.07	71.30
0.60	#30	0.07	39.60
0.30	#50	0.07	16.88
0.15	#100	0.07	6.15
0.075	#200	0.07	2.07
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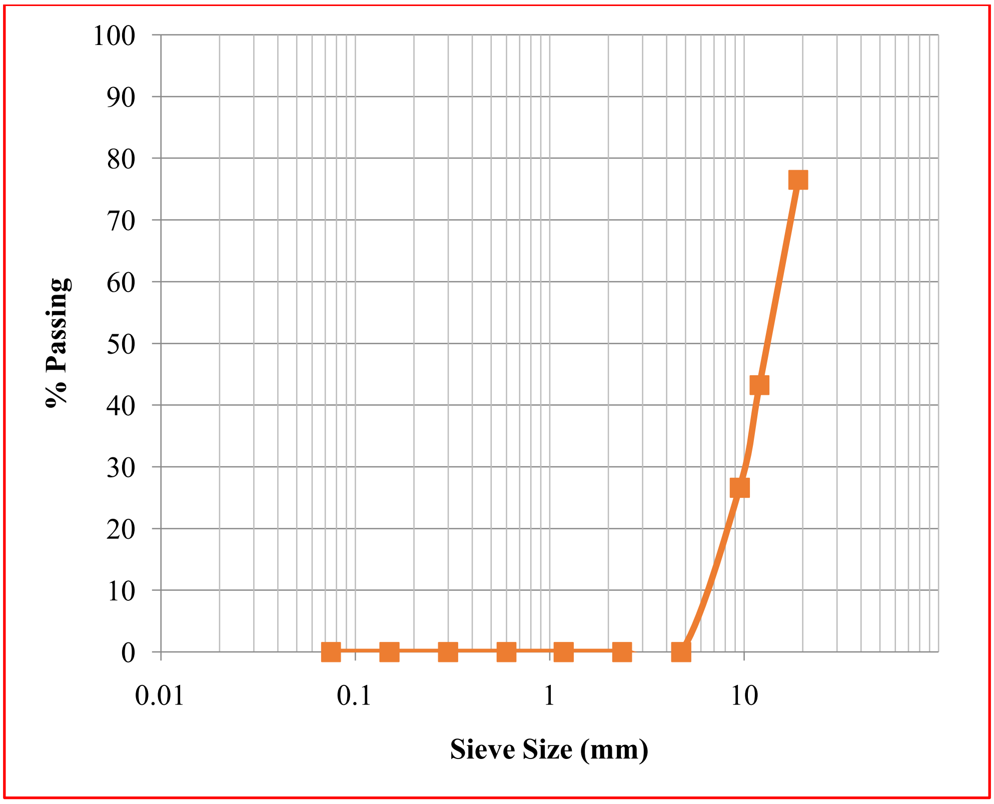


Figure 3.2: Gradation curve for used (Pakur Stone) coarse aggregate.

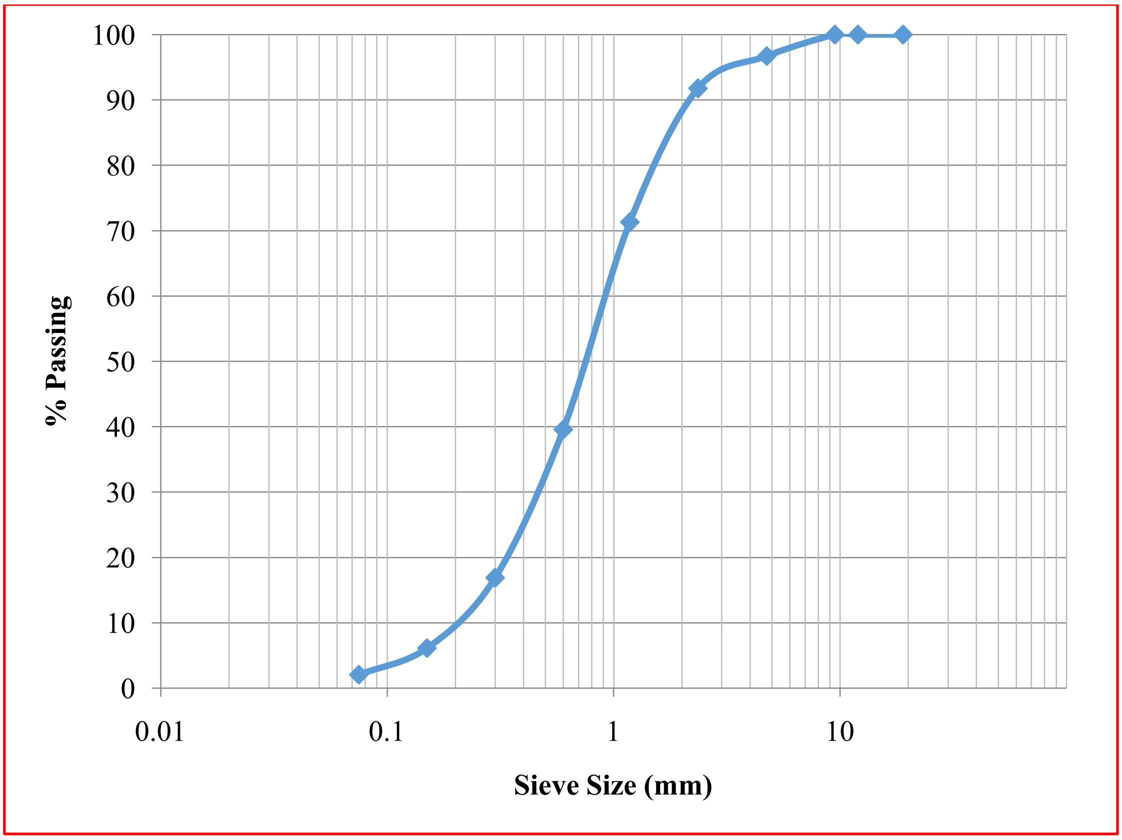


Figure 3.3: Gradation curve for used (Sylhet Sand) fine aggregate.

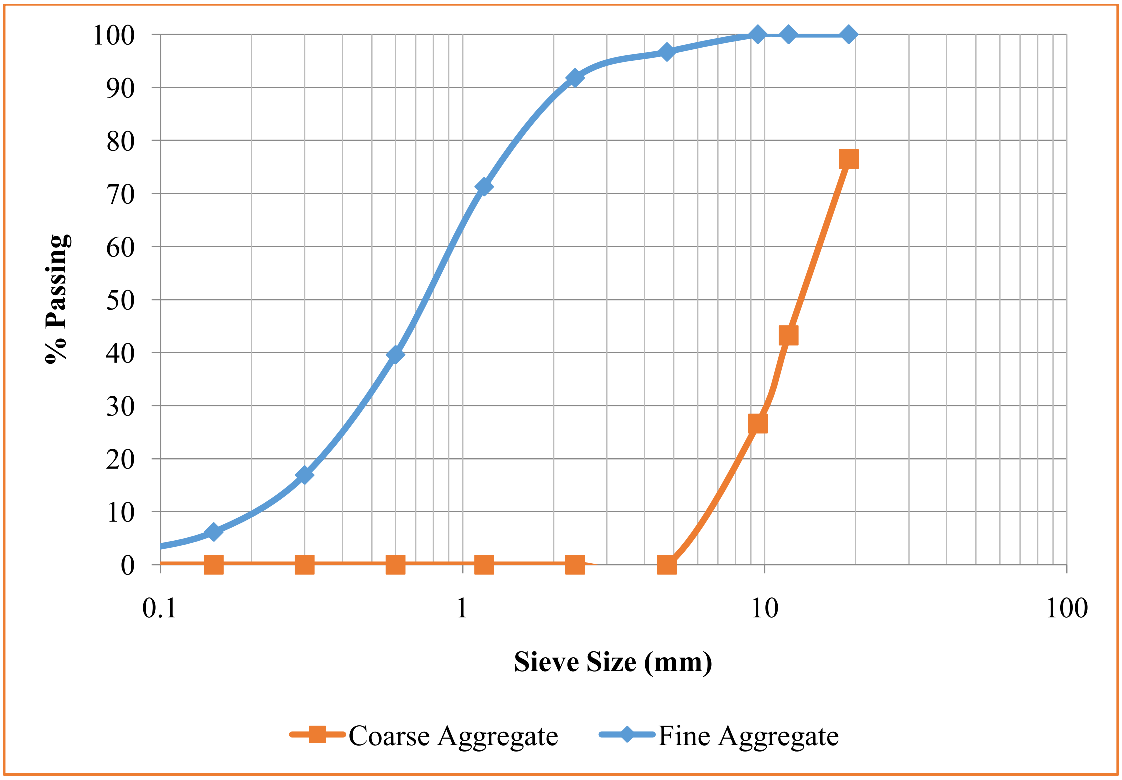


Figure 3.4: Used aggregates gradation curves.

3.2.6 Portland cement

The Portland cement, which was blended with the aggregates, was produced by heating limestone and clay minerals in a kiln to form clinker, grinding the clinker and adding 2 to 3 percent of gypsum. Portland cement is used primarily as filler in hot-mix asphalt to prevent stripping of the binder. It is also used to enhance the coating of wet aggregates with bitumen.

3.2.7Portland cement Properties

Only Portland cement from limestone clinker was used in this study and other types of Portland cement such as Portland Pozzolana Cement, Hydrophobic Cement, Portland Blast Furnace Cement, Air Entraining Cement, High Alumina Cement, Waterproof Portland Cement, Sulfate Resisting Cement etc. are not within the concern of this study. **Table 3.5** shows the properties of used Portland cement.

Table 3.5: Used Portland cement Properties

Property		Detail
Source		Crown Cement
Composition		**Clinker:** 95-100%**Gypsum:** 0-5%
Specific Gravity		3.12
Fineness (m^2/Kg)		354
Normal Consistency (%)		25.5
Initial Setting Time (min)		160
Final Setting Time(min)		309
Compressive Strength (psi)	3 Days	3680
	7 Days	4900
	28 Days	6370

Source: https://www.crowncement.com/products/quality/certificates-test-report/

Sylhet Sand **Portland Cement**

Figure 3.5: Used Portland cement and Sylhet Sand.

3.3 Experimental work

For investigating the properties of prepared sample and to find out the suitability of using Portland cement in asphalt mixtures, an extensive experimental work was conducted. After evaluating the properties of used materials as bitumen, aggregates, and Portland cement and carrying out sieve analysis for Portland cement and each aggregate type, blending of aggregate carried out to obtain the binder course gradation curve which used in the preparation of the asphalt mix. After that, with different bitumen contents asphalt mixes are prepared to obtain optimum bitumen content by Marshall Test. Then optimum bitumen content is used to prepare asphalt mixes with various percentages of Portland cement. Marshall Test was used to evaluate the properties of these prepared sample mixes. Finally, laboratory tests results are obtained and analyzed. **Figure 3.6** shows a flowchart of experimental work for this study.

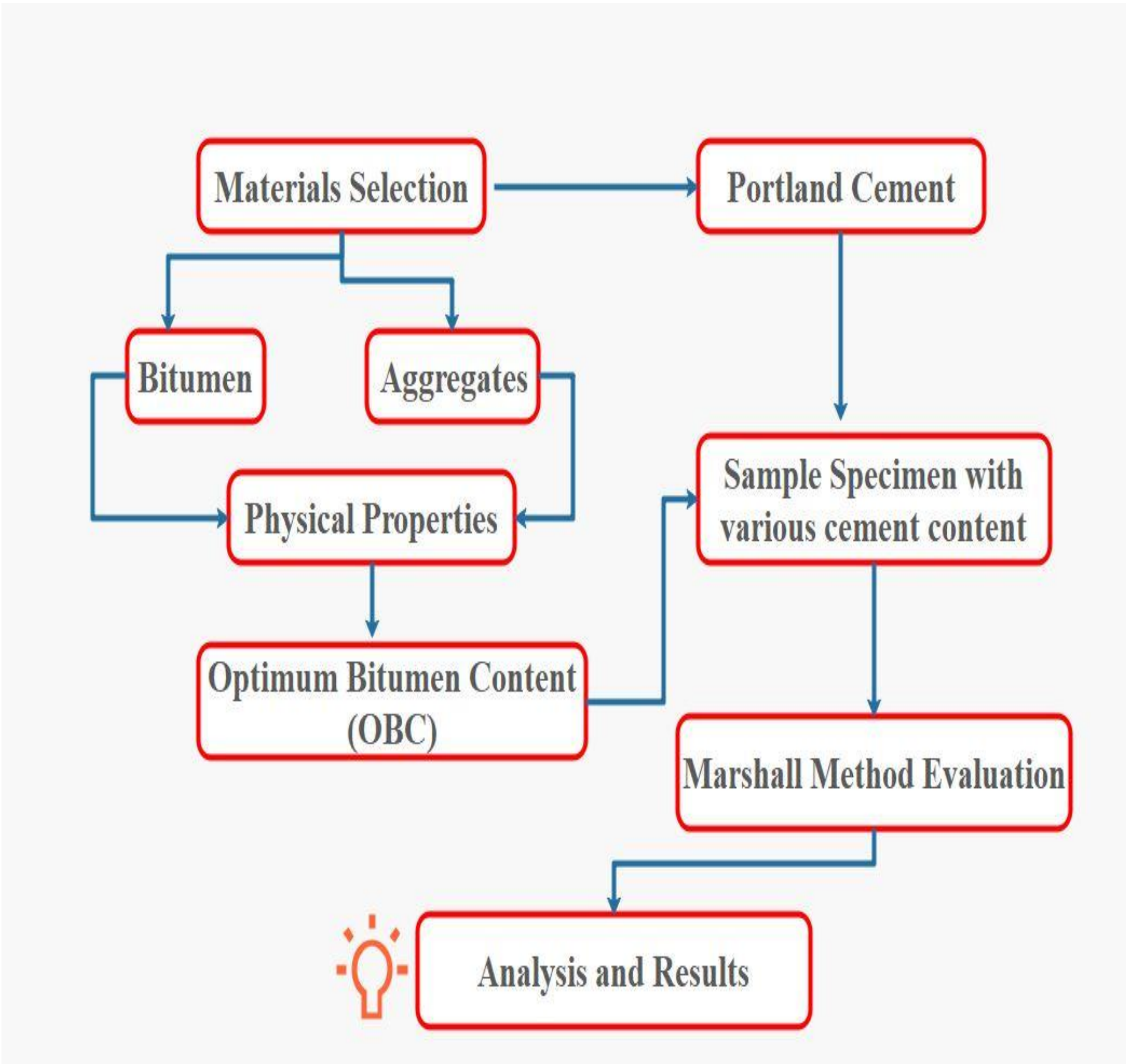


Figure 3.6: Flow Chart of experimental work

3.3.1 Preparation of Mixtures

According to ASTM specifications using mathematical trial method, aggregates are banded together in order to get a proper gradation. Mathematical trial method depends on suggesting different trial proportions for each type of aggregate. The percentage of each type of aggregates is to be computed and compared to specification limits. If the calculated percentages for, each type of aggregate, gradation is within the specifications limits, no further adjustments need to be made; if not, an adjustment in the proportions must be made till the percentage of each size of aggregate are within the specifications limits shown in **Table 3.6 (Table 19.3, Page-535, Highway Engineering, 7th Edition)**.

Table 3.6: Mineral Aggregate and Mix Composition limits

Passing Sieve Designation, mm	Retained on Sieve Designation	Percent (by weight)
19	1/2"	0-6
12	3/4"	9-40
9.5	3/8"	9-45
4.75	#4	8-27
Total Coarse Aggregate	**#10**	**50-65**
2.00	#10	6-22
0.475	#40	8-27
0.177	#80	5-17
0.075	#200	5-8
Total Fine Aggregate and Filler	**Passing No. 10**	**35-50**
Total Mineral Aggregate		**100**
Total Mix		
Total Mineral Aggregate		**92-95**
Asphalt Content		**5-8**
Total Mix		**100**

Aggregate are first dried to constant weightat 110±5 ºC. The aggregates are then heated to a temperature of 135 ºC before mixing with asphalt cement. Asphalt was heated up to 145ºC prior mixing. Pre-heated asphalt was avoided and excess heated asphalt was disposed of to avoid variability in the asphalt properties. The required amount of asphalt were then added to the heated aggregate and mixed thoroughly for at least three minutes and until a homogenous mix is obtained. Standard Marshall Molds were heated in an oven up to 130 ºC. The hot mix is placed in the mold and compacted with different compaction energy such as 75 blows for Heavytraffic loading pattern.

Table 3.7: List of Minimum Sample size

Size of Maximum size of aggregate (inch.)	Minimum Sample size (g)
$1^{1/2}$	**4000**
1	**2500**
3/4	**2000**
1/2	**1500**
¼	**1000**

3.3.2 Determining the Optimum Binder Content(Marshall Test Method)

Marshall Stability test is used in this study for both determining the optimum binder content (OBC) and evaluation the prepared specimens. Marshall Method is essentially empirical and it is useful in comparing mixtures under specific conditions. This method covers the measurement of the resistance to plastic flow of cylindrical specimens of bituminous paving mixture loaded on the lateral surface by means of the Marshall apparatus according to ASTM D 1559-89. The prepared mixture was placed in preheated mold 4inch (101.6mm) in diameter by 2.5 inch (63.5mm) in height, and compacted with 75 blows for each face of specimen. The specimens were then left to cool at room temperature for 24 hours. Marshall stability and flow tests were performed on each specimen, where the cylindrical specimen was placed in water path at 60 ºC for 30 to 40 minutes then compressed on the lateral surface at constant rate of 2 inch/min. (50.8mm/min.) until the maximum load (failure) is reached. The maximum load resistance and the corresponding flow value are recorded. Three specimens for each combination were prepared and the average results were reported. The final value of stability test were used for further steps after applying stability correlation ratios (**Table 3.8**) The bulk specific gravity, density, air voids in total mix, and voids filled with bitumen percentages are determined for each specimen.

Table 3.8: Stability Correlation Ratios

Volume of Specimen, cm^3	Approximate Thickness of Specimen, in.[B]	mm	Correlation Ratio
406 to 420	2	50.8	1.47
421 to 431	2 1/16	52.4	1.39
432 to 443	2 1/8	54.0	1.32
444 to 456	2 3/16	55.6	1.25
457 to 470	2 1/4	57.2	1.19
471 to 482	2 5/16	58.7	1.14
483 to 495	2 3/8	60.3	1.09
496 to 508	2 7/16	61.9	1.04
509 to 522	**2 ½**	**63.5**	**1**
523 to 535	2 9/16	65.1	0.96
536 to 546	2 5/8	66.7	0.93
547 to 559	2 11/16	68.3	0.89
560 to 573	2 3/4	69.8	0.86
574 to 585	2 13/16	71.4	0.83
586 to 598	2 7/8	73.0	0.81
599 to 610	2 15/16	74.6	0.78
611 to 625	3	76.2	0.76

[A]The measured stability of a specimen multiplied by the ratio for the thickness of the specimen equals the corrected stability for a 2 1/2 in. (63.5 mm) specimen.

[B]Volume-thickness relationship is based on a specimen diameter of 4 in. (101.6 mm).

3.3.3 Optimum Binder Content

Marshall Test has been used to determine the optimum binder content. Five percentages of bitumen were examined to determine the optimum percentage of bitumen for the aggregates used, which include **4.0, 4.5, 5.0, 5.5 and 6.0 %** by weight of the mix with three samples for each percentage. The optimum binder content was found equal to **5.60**% by weight of the total mix, which is calculated as the median of the percent air voids limits which is 4.00% (four) percent. All of the calculated and measured mix properties at this asphalt content is then evaluated by comparing them to the mix design criteria in **Table 2.6**.

3.3.4 Optimum Portland cementContent

A number of laboratory investigations were performed in order to determine the mixproperties of prepared sample with Portland cement using Marshall Test procedure. All mixtures are prepared with the same binder content (5.60 %). To determine the best percentage of Portland cement that could be used in prepared sample, six percentages of Portland cement were investigated which are **0.5, 1.0, 1.5, 2.0 and 2.5%** by weight of the total aggregate having compaction energy of**75 blows** with **3 samples** for each percentage and compaction energy.

The steps of preparing sample with Portland cement can be summarized as follows:

a) Portland Cement collected, cleaned and then sieved.

b) The gradation of used Portland cement was different with the grain size distribution of Sylhet Sand aggregate was used.

c) **5 (Five)** percentages of Portland cement were investigated which were **(0.5, 1.0, 1.5, 2.0 and 2.5 %)** by weight of the total aggregate with **3 samples** for each percentage and compaction energy.

d) The mixed Portland cement and aggregates are then heated to a temperature of 135 ºC before mixing with asphalt cement.

e) Asphalt was heated up to 145 ºC prior mixing with aggregates. Pre-heated asphalt was avoided and excess heated asphalt was disposed of to avoid variability in the asphalt properties.

f) The required amount of asphalt were then added to the heated aggregate and mixed thoroughly for at least three minutes until a homogenous mix is obtained.

g)Standard Marshall Molds were heated in an oven up to 130 ºC and then the hot mix isplaced in the mold and compacted with 75 blows for each face of specimen.

h) Specimens are prepared, compacted, and tested according to Marshall Method designated ASTM D 1559-89. **Figure 3.7** show Marshall Specimens of prepared sample with different percentages of Portland cement.

Figure 3.7: Marshall Specimens with cement as filler matrial

3.3.5 Tensile Strength Ratio (TSR) Test

Two sets of HMA samples are subjected to a split tensile test (often called an indirect tensile test). One set is conditioned by partial vacuum saturation with water, soaking in water for 24 hours and an optional freeze-thaw cycle. The other set is used as a control. The ratio of the average split tensile strength of the conditioned samples over the average split tensile strength of the unconditioned (control) samples is reported as the **Tensile Strength Ratio (TSR)**.

The total test time can be up to 6 days. Major components are:

1. Up to 4 days of sample preparation
2. 16 hours for the freeze cycle
3. 24 hours for the thaw cycle
4. 2 hours for getting samples to test temperature
5. 30 minutes to run conditioned and unconditioned sample sets through the indirect tensile test.

The steps of performing **TSR** test sample can be summarized as follows:

6. Prepare 6 HMA samples. Samples are usually 6 inches (150 mm) in diameter and 4 inches (100 mm) thick. After mixing has occurred, allow the HMA to cool to room temperature for 2 hours.Samples of other sizes may be used. If aggregate larger than 1 inch (25 mm) is present in the HMA, a larger sample size should be used.
7. Cure the HMA in an oven at 140°F (60°C) for 16 hours.
8. After curing, place HMA in an oven at 275°F (135°C) for two hours before compaction.
9. Compact mix to 7 percent air voids, or a void level expected in the field, using the SGC, Marshall Hammer.
10. Store the compacted samples at room temperature for 72 to 96 hours.
11. Determine the theoretical maximum specific gravity (Gmm), bulk specific gravity (Gmb), height, volume and air void content (Va) of each sample.
12. Divide the six samples into two subsets of three. The average air void content (Va) for each subset should be similar. One subset will be "unconditioned" (tested in a dry state) and the other will be "conditioned" (tested in a saturated state).
13. Unconditioned samples. While the conditioned samples are being conditioned, the unconditioned samples sit at room temperature.
14. Wrap samples in plastic or put them in a heavy duty leak proof bag.
15. Store samples at room temperature until testing.
16. Conditioned samples. These samples are saturated with water to between 55 and 80 percent using the following procedure:
17. Place each sample in a vacuum container supported above the container bottom by a spacer and fill the container with water until the sample is covered by 1 inch (25 mm) of water.

Figure 3.8: Vacuum saturation of a sample.

18. Apply a vacuum of 10 – 26 inches Hg partial pressure (13 – 67 KPa absolute pressure) for 5 to 10 minutes (**Figure 3.8)**.

19. Remove the vacuum and let the sample sit under water for another 5 to 10 minutes.

20. Calculate bulk specific gravity (Gmb) and compare the SSD mass with the SSD mass obtained in **Step 6** to determine the volume of absorbed water.

21. Determine degree of saturation by comparing volume of absorbed water with volume of air voids (Va) obtained. If the calculated saturation of a sample is below 55 percent, repeat the saturation procedure. If the calculated saturation of a sample is above 80 percent, the sample is considered damaged and must be discarded. If freeze-thaw conditioning is desired, wrap each sample in plastic and place it in a plastic bag containing 0.6 in^3 (10 mL) of water. Seal the bag and place it in a freezer at 0°F(-18°C) for at least 16 hours.

22. Moisture conditions the samples by placing them in a bath of distilled water at 140°F (60°C) for 24 hours. If the samples were freeze-thaw conditioned, remove the plastic from the samples as soon as possible after placement in the bath.

23. Place samples in a 77 °F (25 °C) water bath for a minimum of 2 hours.

24. Run an indirect tension test on each sample by placing the sample between the two bearing plates in the testing machine and applying the load at a constant rate of 2 inches/minute (50 mm/minute). Make sure the load is applied along the diameter of the sample (**Figure 3.9)**.

25. Record the tensile strength values and calculate and report the tensile strength values.

Figure 3.9: Sample placed between the bearing plates before testing

CHAPTER 4

DATA ANALYSIS AND RESULTS

4.1 Introduction

Data analysis and results of laboratory investigations that conducted to study the effect of using Portland cement in asphalt binder course specifically, the influence of cement content on the stability, flow, and air voids content of asphalt concrete will be presented in this chapter. Also this chapter will discuss in detail all results of laboratory tests that conducted on used materials such as aggregate, bitumen and Portland cement.

Marshall Method for designing hot asphalt mixtures was used to determine the optimum bitumen content to be added to specific aggregate blend. Also Marshall Method for designing hot asphalt mixtures was used to evaluate the cement mixture specimens to determine the optimum content of Portland cement.

The results of this study only apply to the specific content of Portland cement in bituminous mixes that were used. Other content of Portland cement or various type of cement may produce different results.

4.2 Aggregates Blending

Natural fine and coarse aggregates are used in this research with physical properties presented in **Table 3.3**and**Table 3.4**. To produce identical controlled gradation, aggregates were sieved and recombined in laboratory to meet the selected gradation which is satisfying ASTM specifications for asphalt binder course gradation. **Table 4.1** shows the final proportion of each used aggregate in asphalt binder course. The selected gradation of aggregates with ASTM gradation limits is presented in **Table 4.2** and**Figure 4.1**. For more details see **Appendix (B).**

Table 4.1: Final proportion of used aggregate

Aggregates type	Size (mm)	Proportion of proposed mix (%)
Pakur Stone	**19.00**	**55**
Sylhet Sand	**1.15**	**40**
Mineral Filler	**<0.075**	**05**
	Sum=	**100**

Table 4.2: ASTM D 3515 dense binder gradation limits and used aggregates gradation

Sieve #	Sieve size (mm)	% Passing Used gradation	ASTM specifications limits (%)	
			Min	Max
1"	25	100	100	100
3/4"	19	95	90	100
1/2"	12.5	76	67	85
3/8"	9.5	68	56	80
#4	4.75	50	35	65
Coarse Aggregate, % (Passing 3/4" to Retained #4)		55	--	--
#8	2.36	36	23	49
#16	1.18	26	15	37
#30	0.6	17	8	26
#40	0.425	--	6	22
#50	0.3	12	5	19
#100	0.15	8.5	3	14
#200	0.075	5	2	8
Fine Aggregate, % (Passing #8 to Retained #200)		40	--	--
Mineral Filler, % (Passing #200)		05	--	--
Total Aggregate (gm)=		1200	--	--

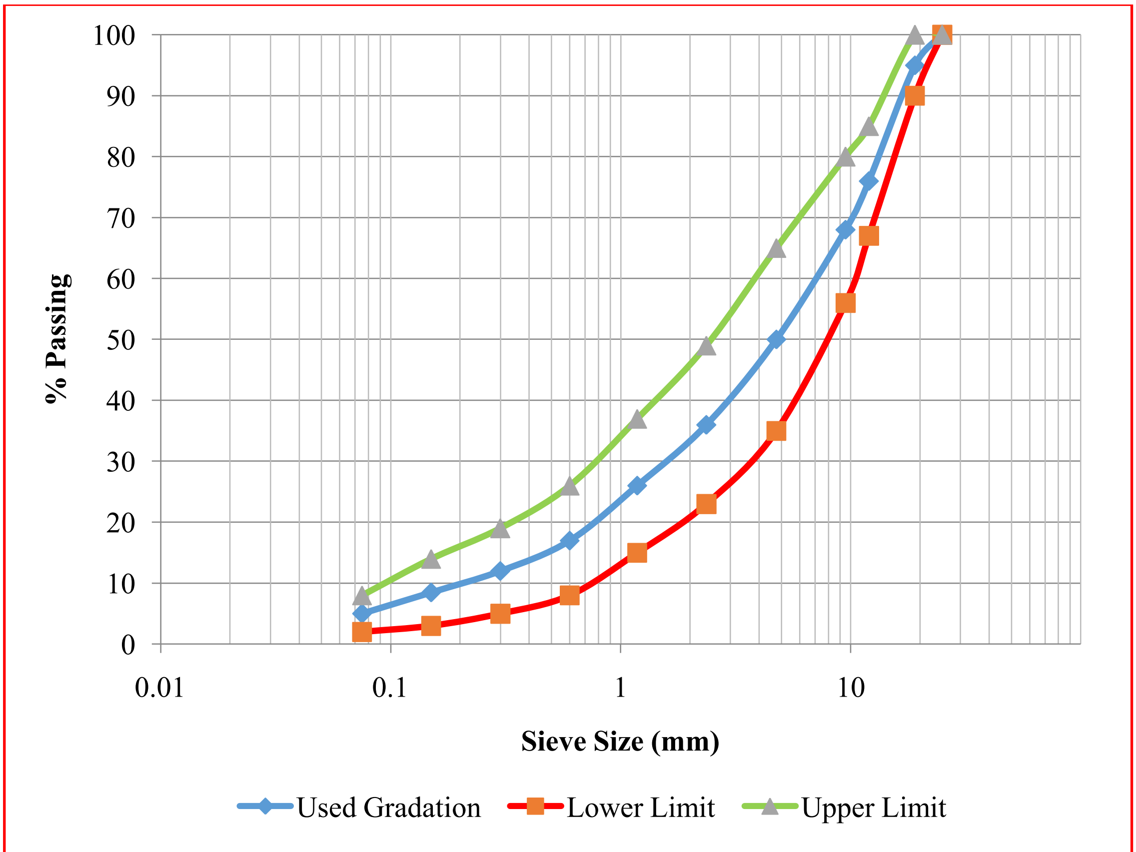


Figure 4.1:ASTM D 3515 dense binder gradation curves limits and aggregates mixture gradation curve.

4.3 Bitumen Results

Loss on Heating,Penetration, Ductility, Specific Gravity, Softening Point(Ring and Ball Method), Flash point and Fire point(Cleveland Open Cup Method), Solubility tests have been performed to measure the properties of asphalt binder.

4.3.1 Penetration test

According to ASTM D5-86 specification, penetration test for bitumen was performed and results presented in **Table 4.3** below.

Table 4.3: Bitumen penetration test results

	Sample (1)			Sample (2)		
Trial	**1**	**2**	**3**	**1**	**2**	**3**
Penetration Value (0.1 mm)	69	69	71	72	70	71
Sample Average Value=	70			71		
Average Value=	70.5					

4.3.2 Ductility test

According to ASTM D113-79 specification, ductility test for bitumen was performed and results presented in **Table 4.4** below. **Figure 4.2** show ductility test for a bitumen sample.

Table 4.4: Bitumen ductility test results

Sample	Ductility (cm)
1	100+
2	100+
3	100+
Average Value=	100+

Figure 4.2: Ductility test for a bitumen sample

4.3.3 Specific Gravity test

According to ASTM D70 specifications, specific gravity test for bitumen was performed and results presented in **Table 4.5** below.

Table 4.5: Bitumen Specific Gravity test results

Specific Gravity= **1.037** at 25/25 ºC

4.3.4 Softening point test

According to ASTM D36-89 specification, softening point test for bitumen was performed and results presented in **Table 4.6** below. **Figure 4.3** shows softening point test for bitumen samples.

Table 4.6: Bitumen softening point results

Sample	**Softening point (ºC)**
1	49.8
2	50
Average Value=	49.9

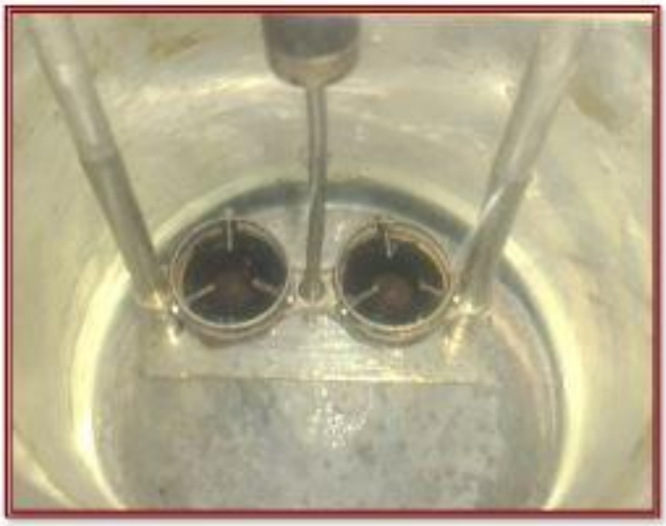


Figure 4.3: Softening point test for bitumen samples

4.3.5 Flash point test

According to ASTM D92-85 specification, flash point test for bitumen sample was performed and results presented in **Table 4.7** below.

Table 4.7: Bitumen flash point test results

Flash point (ºC)	280ºC

4.3.6 Fire point test

According to ASTM D92-85 specification, fire point test for bitumen sample was performed and results presented in **Table 4.8** below.

Table 4.8: Bitumen fire point test results

Fire point (ºC)	340ºC

4.3.7 Loss on heating test

According to ASTM D6 / D6M – 80 specifications, Loss on heating test for bitumen was performed and results presented in **Table 4.9** below.

Table 4.9: Bitumen Loss on heating results

Sample	**Loss on heating (%)**
1	0.0005
2	0.0005
Average Value=	0.0005

4.3.8 Summary of physical properties of bitumen

Table 4.10: Physical properties of used bitumen

Test	**Unit**	**Specification**	**Test result**
Penetration at 25 ºC	1/10 mm	ASTM D5-86	70.5
Specific Gravity at 25 ºC	g/cm^3	ASTM D70	1.037
Ductility at 25 ºC	cm	ASTM D113-79	100+
Softening Point	ºC	ASTM D36-89	49.9
Flash Point	ºC	ASTM D92-85	280
Fire Point	ºC	ASTM D92-85	340
Loss on heating	%	ASTM D6 / D6M – 80	0.0005

4.4 Determining the Optimum Bitumen Content (OBC)

Marshall Test was used to examine the specimens of asphalt mixture with different percentages of bitumen content which were **(4.0, 4.5, 5.0, 5.5 and 6.0%)** to obtain the optimum bitumen content.

4.4.1 Marshall test results

Marshall Test results of mixturehaving a specific gradation (See **Table 4.2**) with different binder content are shown in **Table 4.11**.The relationships between binder content and the properties of mixtures such as stability, flow, VFA, VMA, V_a, V_b and bulk density (ρ_A) are shown in **Figures 4.4 – 4.9**. A number of 15 samples each one of them weigh **1200 gram**, were prepared using five different bitumen contents (**4.0, 4.5, 5.0, 5.5 and 6.0 % by total weight**) with constant compaction energy (**75 blows**) for simulatingtraffics load patterns (**Heavy**) in order to determine the optimum bitumen content. Further details are presented in **Appendix (C).**

1. For Heavy loads (75 blows) (CA:FA:MF= 55: 40: 05)

Table 4.11: Marshall Test results for determination of **OBC** at **75 blows**

Name of the Sample	Asphalt Content (% by total weight)	Corrected Stability (KN)	Flow Index (0.01 in)	ρ_A (g/cm^3)	V_a (%)	V_b (%)	VMA (%)	VFA (%)
H-4.0-1	4.0 %	11.42	8.26	2.35	7.27	9.05	16.32	55.46
H-4.0-2		11.50	8.24	2.30	9.26	8.85	18.12	48.87
H-4.0-3		11.37	8.28	2.31	8.57	8.92	17.50	51.00
Average Value=		11.43	8.26	2.32	8.37	8.94	17.31	51.78
H-4.5-1	4.5 %	12.59	9.53	2.37	5.42	10.30	15.72	65.54
H-4.5-2		12.64	9.55	2.34	6.75	10.16	16.91	60.06
H-4.5-3		12.54	9.51	2.34	6.61	10.17	16.79	60.59
Average Value=		12.59	9.53	2.35	6.26	10.21	16.47	62.07
H-5.0-1	5.0 %	14.91	10.03	2.35	5.50	11.35	16.84	67.37
H-5.0-2		14.89	10.05	2.35	5.59	11.34	16.92	66.99
H-5.0-3		14.96	10.01	2.37	4.94	11.41	16.35	69.78
Average Value=		14.92	10.03	2.36	5.34	11.36	16.71	68.05
H-5.5-1	5.5 %	11.83	11.05	2.37	4.11	12.56	16.67	75.35
H-5.5-2		11.93	11.07	2.34	5.11	12.43	17.54	70.86
H-5.5-3		11.88	11.03	2.33	5.63	12.36	17.99	68.72
Average Value=		11.88	11.05	2.35	4.95	12.45	17.40	71.65
H-6.0-1	6.0 %	10.43	12.57	2.36	3.77	13.64	17.41	78.36
H-6.0-2		10.56	12.59	2.34	4.43	13.55	17.98	75.34
H-6.0-3		10.50	12.53	2.36	3.70	13.65	17.35	78.68
Average Value=		10.50	12.58	2.35	3.97	13.61	17.58	77.46

4.4.2 Marshall stability

The stability of the specimen is the maximum load required to produce failure of the specimen when load is applied at constant rate 50 mm / min. From figure below it is noticed that the stability of asphalt mix is **12.25 KN** at 5.60 % bitumen content. **Figure 4.4** shows the stability result for different bitumen contents.

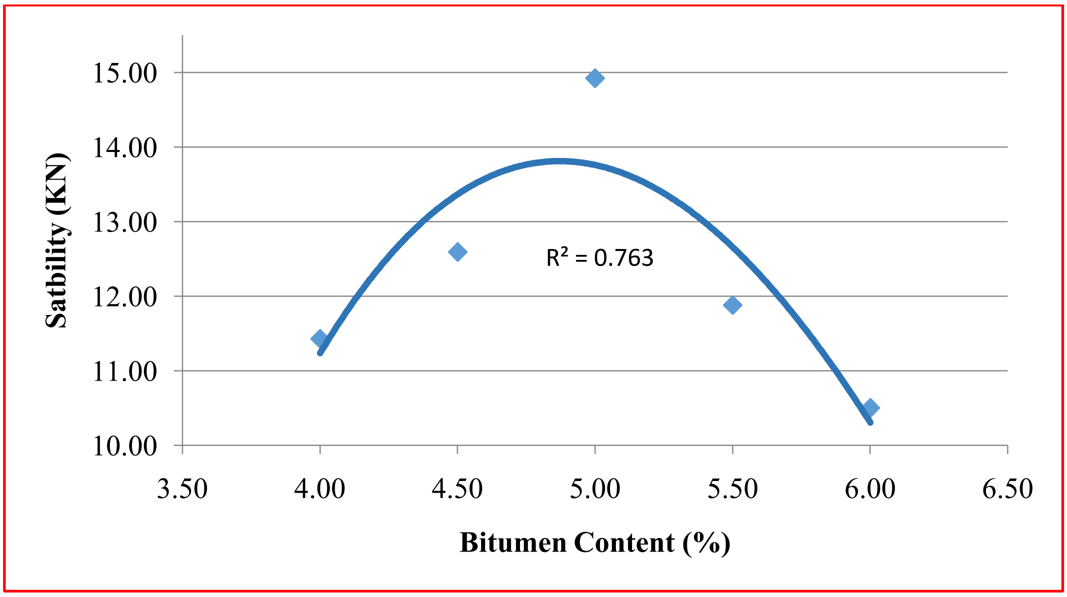


Figure 4.4: Stability vs. bitumen content

4.4.3 Flow

Flow is the total amount of deformation which occurs at maximum load. From the figure below it is noticed that the flow of asphalt mix is 10.75 (0.01 in)at **5.60 %** bitumen content. **Figure 4.5** shows bitumen flow results for different bitumen contents.

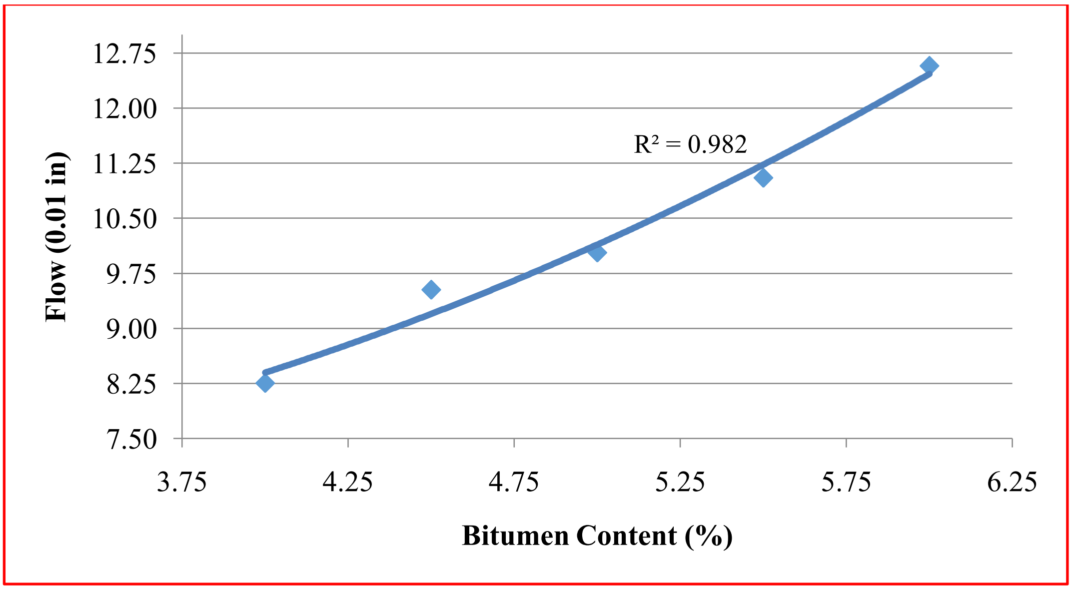


Figure 4.5: Flow vs. bitumen content

4.4.4 Bulk density

Bulk density is the actual density of the compacted mix. **Figure 4.6** represents the bulk density results for different bitumen contents. From the figure below, it is noticed that the maximum bulk density is **2.35 g/cm^3**.

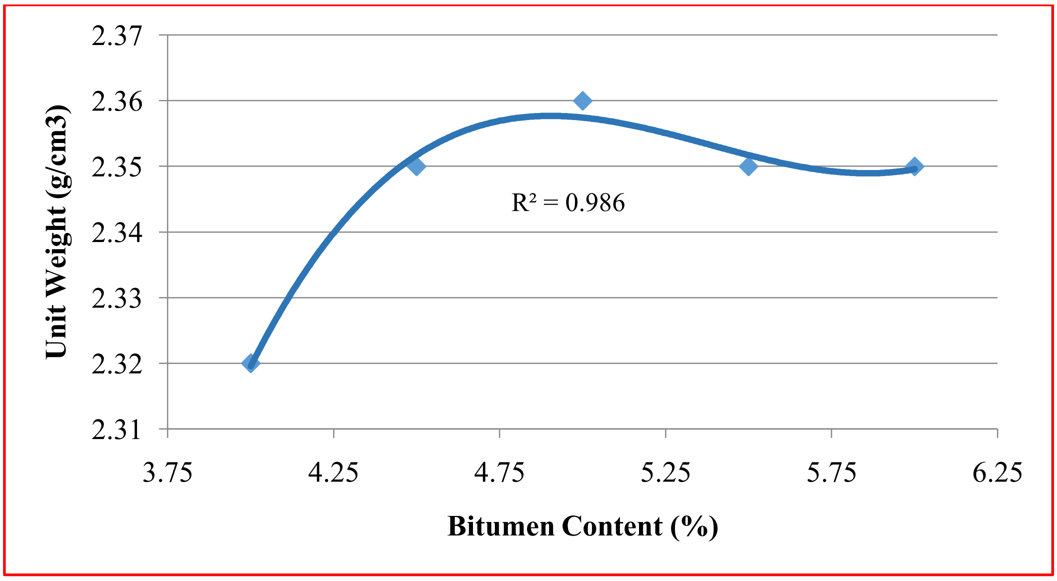


Figure 4.6: Bulk density vs. bitumen content

4.4.5 Air Voids content (V_a)

The air voids, V_a, is the total volume of the small pockets of air between the coated aggregate particles throughout a compacted paving mixture, expressed as a percent of the bulk volume of the compacted paving mixture. From **Figure 4.7**, it is noticed that the air voids content gradually decreases with increasing the bitumen content and that due to the increase of voids percentage filled with bitumen in the asphalt mix. The median of the percent air voids limits which is four percent represents the bitumen contents is 5.60%. This is the optimum bitumen content that satisfies all mix design criteria.

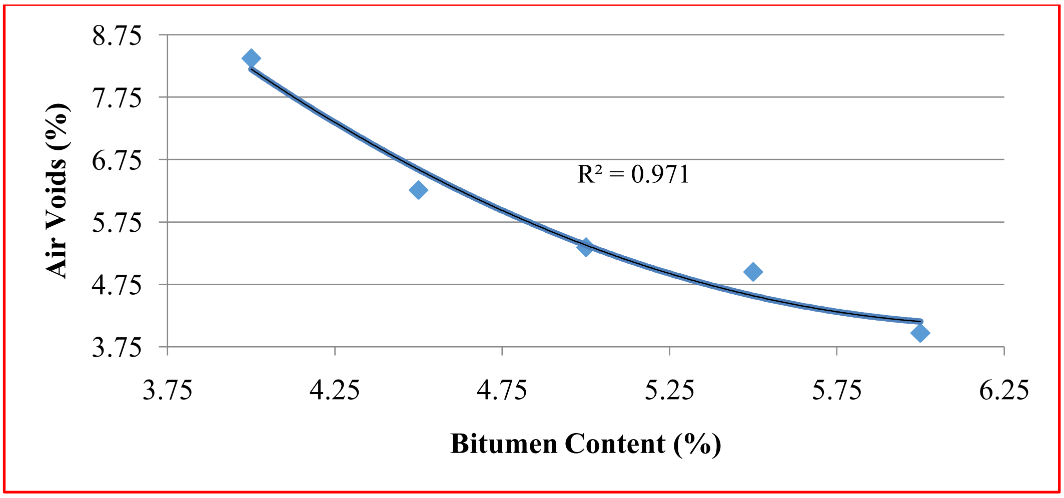


Figure 4.7: Mix air voids proportion vs. bitumen content

4.4.6 Voids in the Mineral Aggregates (VMA)

Voids in the mineral aggregate, VMA, are defined as the inter granular void space between the aggregate particles in a compacted paving mixture that includes the air voids and the effective bitumen content, expressed as a percent of the total volume. From **Figure 4.8**, it is noticed that the VMA decrease gradually as bitumen content increase within a certain limit. After crossing this limit, the VMA increase gradually as bitumen content decrease.**Figure 4.8** shows thatthe result of VMA for 5.60% bitumen contentis 17.15%.

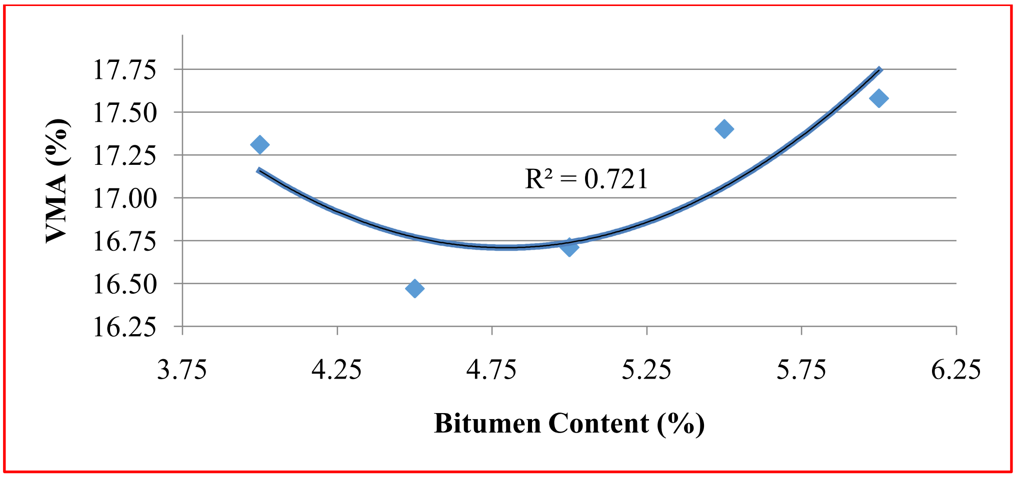


Figure 4.8: Voids of Mineral Aggregates (VMA) proportion vs. bitumen content

4.4.7 Voids Filled with Asphalt (VFA)

The voids filled with asphalt, VFA, is the percentage of inter granular void space between the aggregate particles (VMA) that are filled with bitumen. From **Figure 4.9**, it is noticed that the VFA (%) increase gradually as bitumen content increase and that due to the increase of voids percentage filled with bitumen in the asphalt mix. **Figure 4.9** shows that the result of VFA with 5.60% bitumen content is 71.10%.

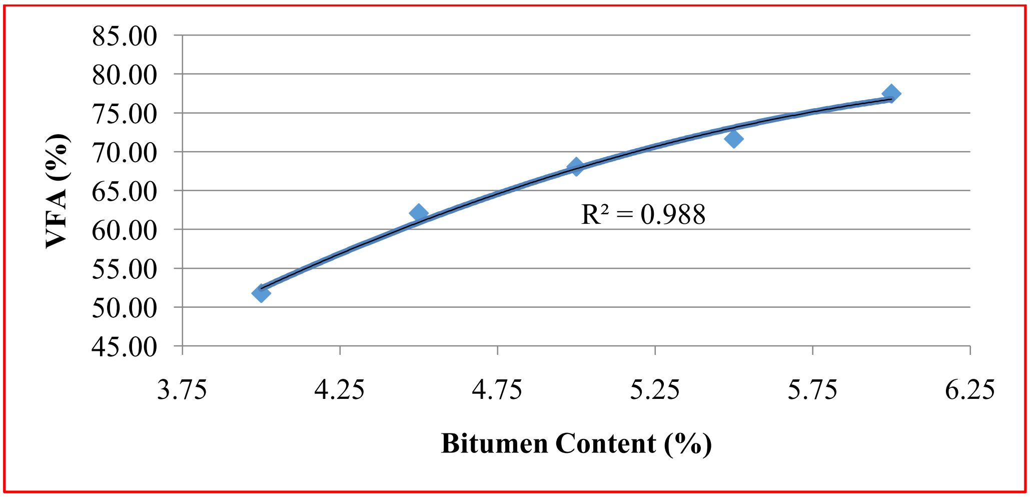


Figure 4.9: Voids Filled with Asphalt (VFA) proportion vs. bitumen content

4.4.8 Optimum bitumen content (OBC)

The optimum bitumen content was found equal to **5.60%** by weight of the total mix (having 4.00% air voids) in which all of the calculated and measured mix properties at this asphalt content is compared to the mix design criteria in **Table 2.6** and found acceptable. **Figures 4.4, 4.6 and 4.7** are utilized to find the three values.

I. At 5.60% Bitumen content, the value of Stability= 12.25 KN

II. At 5.60% Bitumen content, the value of Flow = 10.75 (0.01 in)

III. Bitumen content at the median percent (4.00%) of air voids = 5.60%

Optimum Bitumen Content (OBC) = **5.60** %

At **5.60%** bitumen content it's found from **Figures 4.4 - 4.9** that all test values consistence with the specifications limits. **Table 4.12** presents the properties of asphalt mix at **5.60 %** bitumen content and the specifications limits.

Table 4.12: Properties of the asphalt mix at **5.60 %** bitumen content

Name of the Test	**Laboratory Test Result with 5.10 % bitumen content**	**Marshall Design Criteria** Mix Design Methods for Asphalt Concrete and Other Hot-Mix Types. Manual Series No. 2 (MS-2). Sixth Edition, 1997, Asphalt Institute. Lexington, KY.					
	Traffic Loading Condition	**Light Traffic ($<10^4$ESALs)**		**Medium Traffic ($10^4 – 10^6$ESALs)**		**Heavy Traffic ($>10^6$ESALs)**	
	Heavy	**Min**	**Max**	**Min**	**Max**	**Min**	**Max**
Compaction (No. of blows)	**75**	35		50		**75**	
Stability (N)	**12250**	3336	--	5338	--	**8006**	
Flow Index (0.01 in)	**10.75**	8	18	8	16	**8**	14
Bulk density (gm/cm^3)	**2.35**	--	--	--	--	**--**	--
Air voids (%)	**4.00**	3	5	3	5	**3**	5
Voids Filled with Asphalt (VFA) (%)	**71.10%**	70	80	65	78	**65**	75
Voids in Mineral Aggregates (VMA) (%)	**17.15**	See the **Table 2.7**					

4.5 Results of prepared sample with Portland cement

As discussed in **Chapter 3**, there were 15 of Marshall sample with Portland cement each one of them weigh 1200 gm having a specific gradation (See **Table 4.2**) were prepared using five different Portland cement content (**0.5, 1.0, 1.5, 2.0** and **2.5 %**) by the weight of total aggregates and 5.10 % bitumen content (by the weight of total mix). Marshall Test also was used to evaluate the specimens with different percentages of Portland cement and the results are presented in **Table 4.13**. Further details are presented in **Appendix (D)**.

1. **For Heavy loads (75 blows) (CA:FA:MF= 55: 40: 05)**

Table 4.13: Mechanical properties of asphalt mixes with Portland cement & **5.60 %** bitumen content at 75 blows

Name of the Sample	Portland Cement (% by aggregates weight)	Asphalt Content (% by total weight)	Corrected Stability (KN)	Flow Index (0.01 in)	ρ_A (g/cm^3)	V_a (%)	V_b (%)	VMA (%)	VFA (%)
See the **Table 4.15**	0.0 %	5.10 %	13.75	10.25	2.35	4.90	11.75	16.60	68.10
HC-0.5-1	0.5 %		12.85	8.75	2.40	2.56	11.79	14.35	82.14
HC-0.5-2			12.42	8.44	2.36	4.13	12.10	16.23	74.53
HC-0.5-3			12.59	9.21	2.36	3.88	12.10	15.97	75.73
Average Value=			12.62	8.80	2.37	3.52	11.99	15.52	77.47
HC-1.0-1	1.0 %		11.17	6.51	2.40	1.61	11.81	13.41	88.02
HC-1.0-2			9.89	5.95	2.36	3.14	12.00	15.14	79.27
HC-1.0-3			10.21	6.50	2.37	2.83	12.00	14.83	80.91
Average Value=			10.42	6.32	2.38	2.53	11.94	14.46	82.73
HC-1.5-1	1.5 %		11.14	4.95	2.39	2.14	11.74	13.88	84.61
HC-1.5-2			9.03	5.10	2.36	3.39	12.00	15.39	77.95
HC-1.5-3			10.32	3.95	2.36	3.22	12.00	15.22	78.84
Average Value=			10.16	4.67	2.37	2.92	11.91	14.83	80.47
HC-2.0-1	2.0 %		12.08	3.85	2.43	0.28	11.97	12.25	97.68
HC-2.0-2			11.19	3.15	2.37	2.88	12.00	14.88	80.67
HC-2.0-3			13.59	3.45	2.38	2.42	12.00	14.42	83.22
Average Value=			11.57	3.48	2.39	1.86	11.99	13.85	87.19
HC-2.5.0-1	2.5 %		13.36	3.95	2.40	1.27	11.80	13.07	90.31
HC-2.5.0-2			10.73	2.67	2.34	3.86	11.95	15.81	75.58
HC-2.5.0-3			12.36	3.21	2.34	3.58	11.95	15.53	76.95
Average Value=			12.15	2.94	2.36	2.90	11.90	14.80	80.95

4.5.1 Marshall stability – Cement content relationship

From **Figure 4.10** it's noticed that that all values of stability with different Portland cement content achieve the**Marshall Design Criteria**for **Asphalt Institute**specification requirements, where the solid line represent the minimum required value of stability and the maximum stability at 0.50 % cement content.**Figure 4.10** shows that the stability of prepared sample with Portland cement mixes decrease as the cement content increases till it reaches the minimum stability at 1.50 % cement content then it starts to increase.

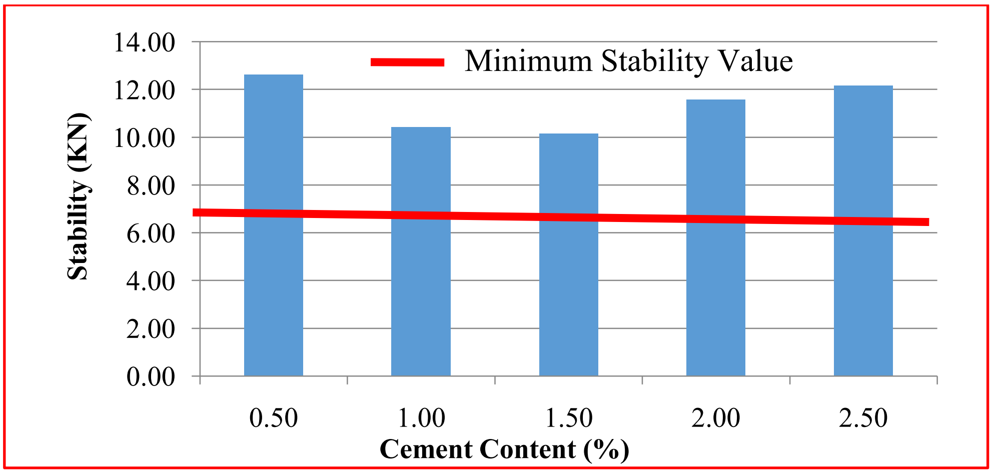


Figure 4.10: Asphalt mix Stability – Cement content relationship

4.5.2 Flow – Cement content relationship

Figure 4.11 shows that the flow of prepared sample with Portland cement mixesgradually decreasesfrom the value of **0.50 %** cementmix which is **8.80 (0.01 in)** and still in the range of **Marshall Design Criteria**for **Asphalt Institute**specification requirements. But, Flow value gradually decreases with thePortland cement mix increases.

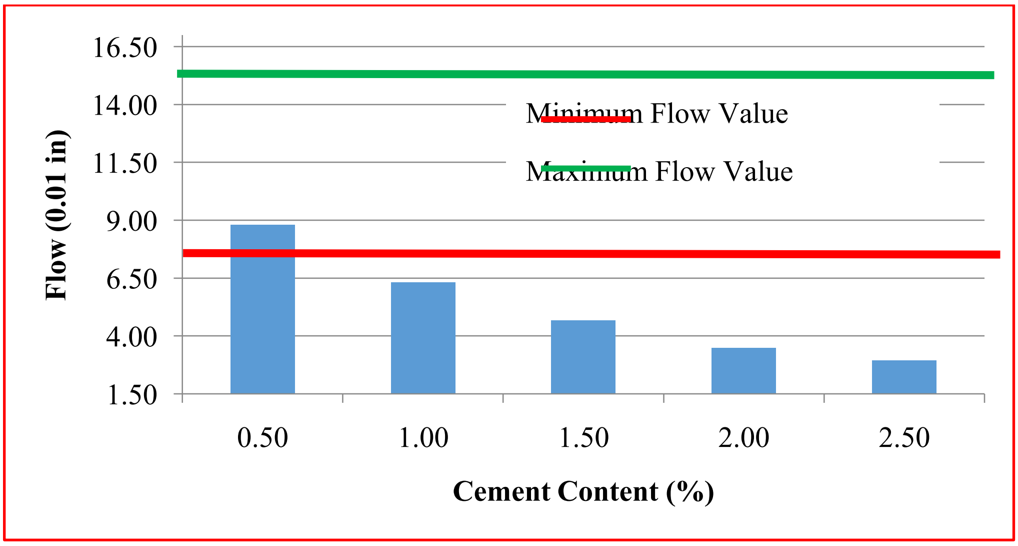


Figure 4.11: Asphalt mix flow – Cement content relationship

4.5.3 Bulk density – Cement content relationship

The bulk density prepared sample with the different percentages of cement content contains a minimum value which is 2.35 g/cm^3. The general trend shows that the bulk density increases as the cement content increases. **Figure 4.12** represents asphalt mix bulk density at different cement content.

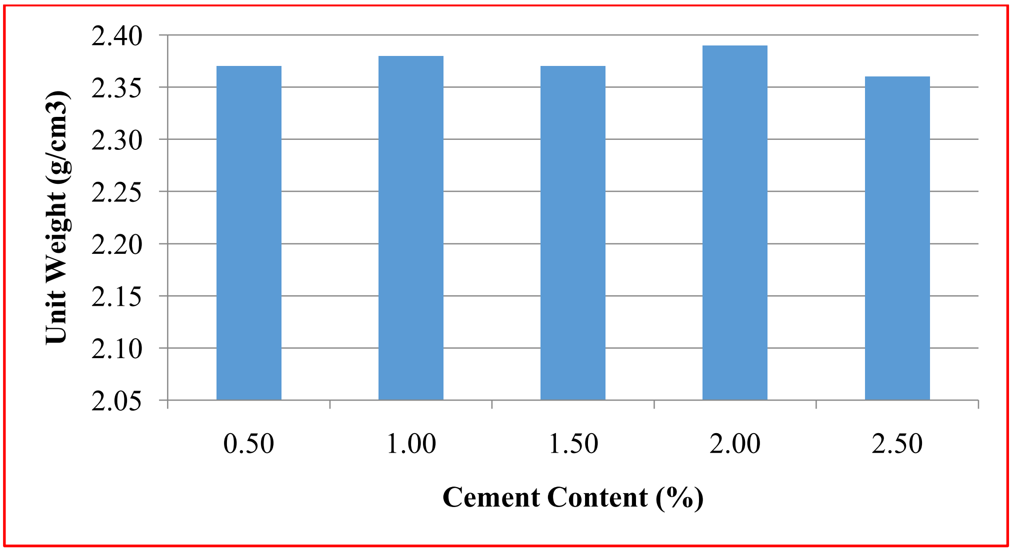


Figure 4.12: Asphalt mix bulk density – Cement content relationship

4.5.4 Air voids (V_a) – Cement content relationship

The air voids of prepared sample with Portland cement mixes decreases gradually as the cement content increase. This decline in air voids in prepared sample with Portland cement mixes return to the reduction in internal pores of cement. It's noticed from the **Figure 4.13** that at **0.50 %** cement content the air voids percentage is **3.52 %** which is the range of **Marshall Design Criteria**for **Asphalt Institute** specification. **Figure 4.13** represents the air voids of asphalt mixes at different cement content.

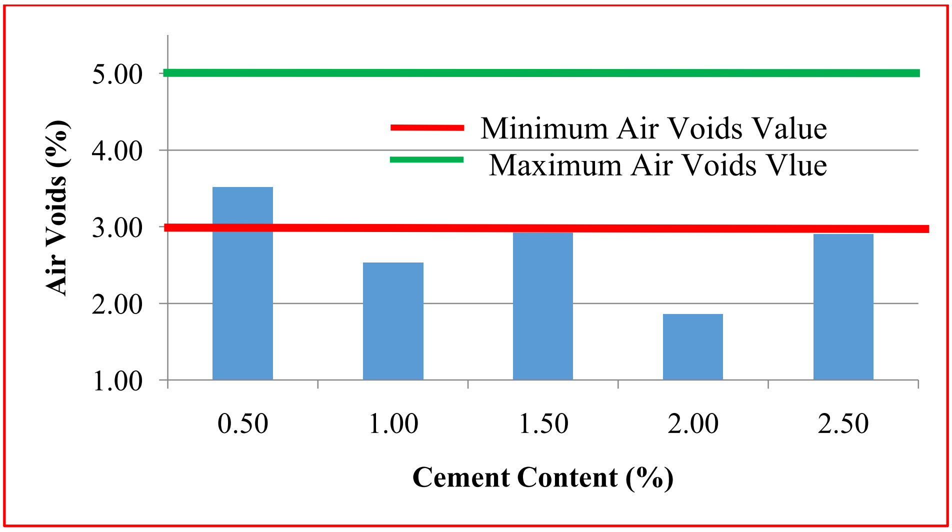


Figure 4.13: Asphalt mix air voids – Cement content relationship

4.5.5 Summary of Prepared Sample with Cement mixes properties

Table 4.14: Summarizes the properties of Prepared Sample with different cementcontent at Heavy load condition.

Property	Cement content (%)				
	0.5	1.0	1.5	2.0	2.5
	Traffic Loading Condition				
	Heavy	Heavy	Heavy	Heavy	Heavy
Corrected Stability (KN)	12.62	10.42	10.16	11.57	12.15
FlowIndex (0.01 in)	8.80	6.32	4.67	3.48	2.94
Bulk Density (gm\cm^3)	2.37	2.38	2.37	2.39	2.36
V_a (%)	3.52	2.53	2.92	1.86	2.90
V.M.A(%)	17.31	16.47	16.71	17.40	17.58
V.F.A(%)	51.78	62.07	68.05	71.65	77.46

4.6 Optimum Cement Content

From Figure 4.10 it is noticed that all values of Marshall Stability for different cement content satisfy the Marshall Design Criteria for Asphalt Institute specification requirements which are 8006 N respectively and the maximum stability corresponds 0.50 % cement content. Figure 4.13 represents the air voids percentage at different Cement content. And it’s noticed from the Figure 4.13 that at 0.50 % cement content the corresponding air voids value which is 3.52 %- is very close to the median air voids in the Asphalt Institutespecifications. From Figure 4.12 it’s noticed that all the values of bulk density at different cement content are very close to each other and all of them contain a minimum value which is 2.35 g/cm^3. Table 4.15 illustrates a comparison of the mechanical properties of prepared sample with Portland cement mixes containing 0.50 % cement content with the standard specifications of Asphalt Institute (Marshall Design Criteria).

Table 4.15: Comparison of sample mixeswith optimum cement content and specifications range.

Name of the Test	Laboratory Test Result with 0.50 % cement content having constant 5.10 % bitumen content	Marshall Design Criteria Mix Design Methods for Asphalt Concrete and Other Hot-Mix Types. Manual Series No. 2 (MS-2). Sixth Edition, 1997, Asphalt Institute. Lexington, KY.					
	Traffic Loading Condition	Light Traffic ($<10^4$ESALs)		Medium Traffic (10^4 – 10^6ESALs)		Heavy Traffic ($>10^6$ESALs)	
	Heavy	**Min**	**Max**	**Min**	**Max**	**Min**	**Max**
Compaction (No. of blows)	**75**	35		50		**75**	
Stability (N)	**12620**	3336	--	5338	--	**8006**	**--**
Flow Index (0.01 in)	**8.80**	8	18	8	16	**8**	**14**
Bulk density (gm/cm^3)	**2.37**	--	--	--	--	**--**	**--**
Air voids (%)	**3.52**	3	5	3	5	**3**	**5**

* **ESAL**= Equivalent Single Axle Load

As obviously shown in **Table 4.15** the prepared sample with optimum cement content **0.50 %** by weight of aggregates satisfies the requirements of **Asphalt Institute specifications (Marshall Design Criteria)** for all tested properties.

Asphalt paving mixes designed by the Marshall method have been failing prematurely on our roads. One of the reasons for such failures is inadequate initial compaction. Densities achieved under **75-blow** Marshall Compaction in the laboratory do not simulate the field densities of the mix after it has undergone secondary compaction due to traffic.

When **air voids** in the mix decrease to below 3 per cent during such densification and as the viscosity of asphalt in the mix decreases sharply in summer; the mix permanently deforms as a rut under the wheel loads. Three factors contribute to good performance of an asphalt mix carrying heavy axle loads in hot climates. They are adequate initial compaction so that secondary compaction under traffic is minimized, sufficient asphalt content for durability of the mix and enough air voids in the mix for its stability. All the three factors are influenced by the VMA of the mix. A high VMA would permit the incorporation of higher asphalt content while ensuring enough air voids under increased compaction.

Aggregate shape and surface texture influence the VMA to some extent but is largely influenced by the aggregate grading. Dense grading give rise to low VMA and open grading to high VMA.

There is a requirement for modification to the Marshall Design procedure we follow for the design of asphalt mixes quality for heavy traffic. The modification involves adjusting the aggregate grading to achieve higher VMA values for creating more space to incorporate higher asphalt contents and checking the mixes for " in place density" for ensuring its stability under secondary compaction due to traffic.

In essence, the reasons for the poor performance of asphalt mixes in Bangladesh could be attributed to one or more of the following characteristics of the mixes.

I. Inadequate initial compaction making the mix vulnerable to high secondary compaction under traffic.

II. Relatively high asphalt contents that permit the reduction of air voids to less than 3 per cent under secondary compaction, leading to rutting under heavy axle loads when pavement temperatures rise in summer.

III. Low asphalt contents and high air voids in the mix leading to top-down cracking, raveling and stripping making the mix less durable.

Almost all RHD national and regional highways are heavily trafficked and the mix design for the wearing course and dense bituminous show asphalt pavement layers; with 75 Marshall Blow does not simulate the traffic load on roads. The modified Marshall method for mix design shall be introduced in the design. To overcome the asphalt pavement rutting on heavily traffic road the BRRL has conducted series of trial tests with modified Marshall Blow of 125, 130, 150and 200 in place of75 Marshall Blow in mix design to prevent rutting in the asphalt layers.

In almost all national and regional highway corridors the traffic volume (EASL) is more than 10 million (ESA). Thus the Marshall mix design with 75 blows are not technically valid and improved Marshall mix design with 125, or 150, or 250 blow shall be used as it will provide good compaction of material with the same crushed stone and 60/70 grade bitumen and the mix design will simulate the present and future traffic on asphalt roads.

4.7 TheTensile Strength Ratio (TSR) Test

The fact is that moisture susceptibility is a primary cause of distress in HMA paved roadways. Moisture damage is the result of moisture interaction with the asphalt binder and thereby reducing adhesion of the binder, to the aggregate; which in turn can lead to rutting and fatigue cracking.

To test for the effects of moisture on a specific asphalt mix design, a number of asphalt core specimens, which look like large hockey pucks, are prepared. Using Tensile Strength Ratio (TSR) testing equipment, the Indirect Tensile Strength test characterizes the pavement construction materials under a variety of temperatures and stress levels that simulate the conditions of a pavement that is subjected to moving wheel loads. One test is performed by applying pressure on a specimen that has not been exposed to moisture, heat, or freezing conditions and measuring the results. Subsequent pressure tests are performed on identical specimens that have been "conditioned" through emersion in water, overnight freezing, and high heat. The difference in the ability of the unconditioned and the conditioned asphalt to respond to pressure during the tests, called the "modulus ratio," is recorded and provides engineers with key information as to how well a roadway using that specific asphalt mix design will perform in field condition (For more details see **section 3.3.5** in this thesis paper).

The ultimate parameter to be measured is the tensile strength ratio (TSR). However, in order to get this measurement the following other parameters need to be measured:

I. Theoretical maximum specific gravity (Gmm) of each sample
II. Bulk specific gravity (Gmb) of each sample
III. Air void content (Va) of each sample
IV. Percent saturation of the conditioned samples

4.7.1 Results of TSR Test

As discussed in **section 3.3.5**, the laboratory sample and core of constructed road with specification from field were collected and performed TSR test by Engineering Corp of Bangladesh Army. The TSR test was used to evaluate the Resistance of Compacted Bituminous Mixture to Moisture-Induced Damage and of Moisture on Asphalt Concrete Paving. The results are presented in **Table 4.16**. Further details are presented in **Appendix (E)**.

Table 4.16:TSR Results on different samples

Serial	Optimum Bitumen Content	Percentage of Cement	TSR %	Remarks
1.	5.625	0.00	83.53	
2.	**5.625**	**0.50**	**85.03**	**Highest**
3.	5.625	1.00	82.70	
4.	5.625	1.50	81.79	
5.	5.625	2.00	76.44	

4.7.2 Graphical representation of Results

Figure 4.14 shows that the **TSR** value for laboratoryprepared sample with Portland cement mixesis almost equal to **TSR**value for existing road strip sample. For both cases, The **TSR** value is uniform and still higher than the range of**Asphalt Mix Design Moisture Susceptibility Specification Criteria**.

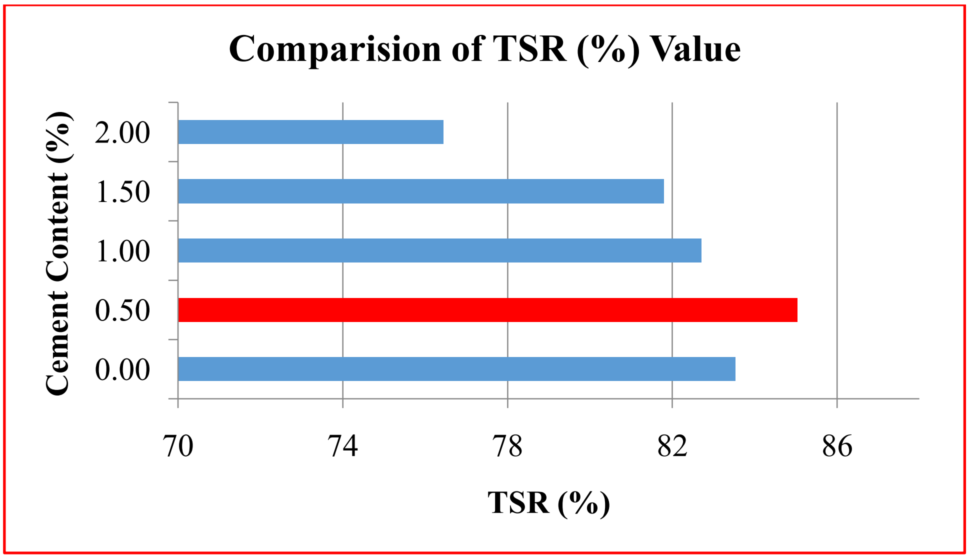


Figure 4.14: TSR (%) Value at 5.625% Bitumen Content

4.7.3 Comparison between TSR Values

Table 4.17: Comparison of TSR values on HMA sample by Engineering Corp.

Portland Cement (% by aggregates weight)	Asphalt Content (% by total weight)	Tensile Strength Ratio (TSR)	Asphalt Mix Design Moisture Susceptibility Specification	HMA Distress of Concern
0.50%	**5.625 %**	**0.8503**	**≥ 0.80**	If **TSR** value is below **0.70**, Asphalt mixture will have good potential for moisture damage.

Most people don't realize that asphalt roadways are not exactly hard surfaces; or they feel hard when you stand on them, but they actually are designed to have an elastic quality that allows the asphalt to recover under the strain of repeated loads. Over time, moisture combined with freezing temperatures and excessive heat can cause asphalt to become less resilient and more susceptible to rutting and fatigue cracking.

This whole process is based on the method AASHTO T-283. From the above data, the Tensile Strength Ratio with anti-stripping agent (Portland cement) for the Bholaganj stone based AC mix is found 85.03% which is within the typical more the minimum required of TSR value 75% as per ASTM. Considering the American Association of State Highway Transportation Officials (AASHTO) T- 283, a minimum Tensile Strength Ratio of 70 to 90% is generally specified for the Asphalt Concrete Mixture.

CHAPTER 5

COST ANALYSIS OF ROAD MAINTENANCE IN BANGLADESH

5.1Road Transport in Bangladesh

In Bangladesh, the road sector has been playing an increasingly significant role in transporting both passengers and freight. Road transport is preferred for transport of smaller consignments, especially of perishable and fragile goods, due to wide and faster geographical coverage, ability to offer personalized door-to-door service and reduced terminal handling cost. This is the most likely a result of consistently higher levels of investment as well as its inherent advantage of ability to provide point-to-point service and also has the ability to interact with all other modes of transport, such as inland waterways, railways, and airports.

The main stakeholders include the Roads and Highways Department (RHD) and the Local Government Engineering Department (LGED). RHD is responsible for construction of the major road network in the country, including the national, the regional highways, and zila roads, whereas LGED is responsible for development and maintenance of transport infrastructure in rural areas, such as rural roads and minor feeder. In addition to the road network managed by both RHD and LGED, some urban road networks are operated under the jurisdiction of city corporations and development authorities within the Ministry of Local Government, Rural Development, and Cooperatives.

The road network of Bangladesh is classified into four major groups: National Highways, Regional Highways, Feeder Roads, and Local Roads. As of 2015, the total length of the road network is 21,481.25 km under the management of Roads and Highways Department. Of the total road network under the Department, 3544 km is National Highways (16%), 4278 km Regional Highways (20%) and remaining 13,659 Km is Zila roads (64%). Besides, RHD has 7,741 bridges and 13,751 culverts under its jurisdiction. RHD also operates 134 ferryboats and 55 ferry ghats on its road network throughout the country. Bangladesh currently has 5 toll bridges and 3 toll roads.

Tolled bridges are generally bridges that are over 1,000 meters in length, such as the Bangabandhu Bridge. In addition, there are approximately 304,379.31 Kms of rural roads, including 200,000 km earthen roads which are under the jurisdiction of LGED. Besides, there are about 100,000 km of farm-to-market and tertiary roads which are of earthen surface and unsuitable for use by vehicular traffic during rainy seasons.These roads are maintained by different agencies like District, Union Councils and Paurashavas/ Municipalities.

Because of indiscriminate investment in road sector, amount of country's road network has increased from 3,764 Km in 1971 to 425,860Km in 2015 with 125,265 Km of paved road. Due to this polarized development strategy, during the last 45 years the amount of paved road network has expanded by 28 folds; whereas during the same period navigable waterway has shrunk from 24,000Km to 5,200Km and railway tracks and services also reduced. Over reliance on road mode resulted in imbalance and energy hungry transport system.

Table 5.1: Paved Road Network of Bangladesh.

Road Network belongs to RHD		
Road Type	**Length (km)**	**%**
National Highway (N)	3,544	3.38
Regional Highway (R)	4,278	4.08
Zila Road (Z)	13,659	13.04
Total	**21,481**	**20.50**
Road Network belongs to LGED		
Upazilla Road (UPR)	29,797	28.44
Union Road (UNR)	22,478	21.45
Village Road (VR)	31,028	29.61
Total	**83,303**	**79.50**
Grand Total	**104,784**	**100.00**
Access Controlled Tolled Road		
Hatikamrul-Banpara highway	51	0.13 % of total paved road
Jagadishpur-Sherpur highway	68	
Port Link Road	13	
Total	**132**	

Table 5.1 shows that in Bangladesh there is only 0.13% road is access controlled, which essentially suggests that there is an acute shortage of quality road with arterial configuration. For having a balanced pyramid of different category of roads, there should be at least 3-5 percent highways with access controlled configuration.

In India, partial access controlled national highways are less than 2 percent of network but carry 40 percent of total traffic. These multi-lane highways often called expressways are developed connecting the entire major metropolis, industrial, agricultural, cultural centers and ports with golden quadrilateral framework and considered as a backbone road network.

Inefficient and unreliable road transport and a poorly developed road network provide limited mobility of goods and services, thereby constraining economic development. With regards to annual average traffic growth rate, a wide range of variations (11.17 to 21.03%) are also observed along the selected corridors. These values are found to be much higher than the standard growth factor of 10% as considered to the RHD pavement design guide (PDG, 2005). The ever increasing vehicle population and heavy axle loads has been causing substantial damage to roads. Trucks carry loads much in excess of legal axle load limits are largely responsible for poor road conditions in addition to the inadequate structural capacity of pavements and diminishing allocation of funds year after year for maintenance and rehabilitation.

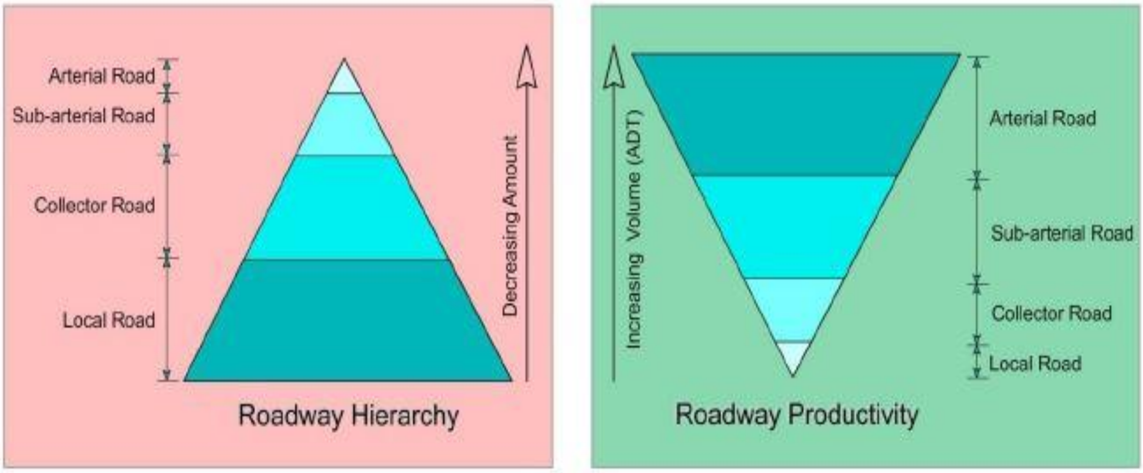


Figure 5.1: Relationship between Roadway Hierarchy vs. Productivity

Note: ADT- Average daily traffic

National highways and regional highways are the most utilized transportation mode for both the movement of passengers and freight. Currently, the Government has given priority to five important corridors for road network development: Dhaka-Chittagong, Dhaka-Northwest, Dhaka-Khulna, Dhaka- Sylhet, and Khulna-Northwest. Based on a 2007 World Bank report, the modal share of roads and highways in Bangladesh transport sector in the carriage of passenger and freight traffic is more than 88% in passenger-Kms and 80% also in ton-Kms, making it the most utilized form of transport in Bangladesh.

5.2 Unit Cost of Flexible Pavement

According to the Road Note-29, for a heavily trafficked highway taking over 10,000 AADT on a good soil subgrade (CBR 3%) condition, the various layer thickness of 2-lane flexible pavement for 10 years design period are:

- Bituminous wearing course = 50 mm
- Bituminous binding course = 100 mm
- Aggregate base course = 300 mm
- Aggregate sub-base = 200 mm
- **Total = 650 mm**

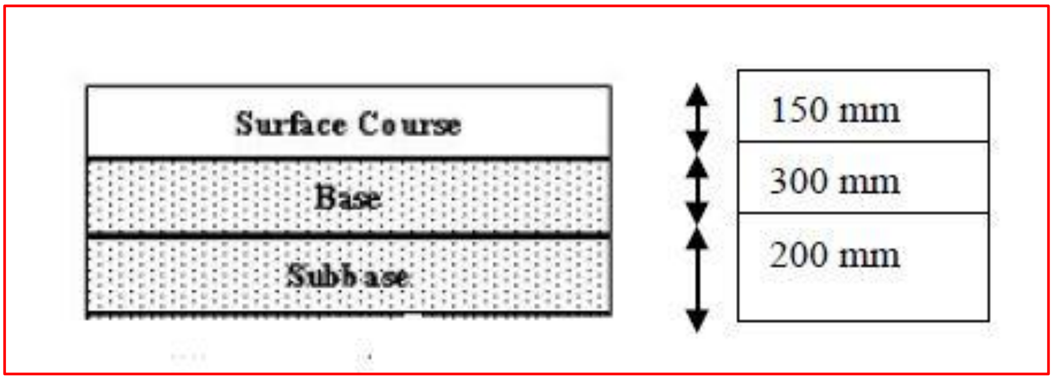


Figure 5.2: Layers of flexible pavement

Taking the unit rates from the **RHD schedule of rates 2018**, the cost of per km cost of 2-lane pavement for 10 years design life is given in the following **Table 5.2**. For further details see **Appendix (F)**.

Table 5.2: Per km Cost of Standard 2-lane Road for 10 years Design Period

Type of Pavement	Cost per Km (in Lac)					
	Without Cement		With Cement			
	60/70 Bitumen	80/100 Bitumen	60/70 Bitumen		80/100 Bitumen	
			0.5% Cement	2.0% Cement	0.5% Cement	2.0% Cement
Flexible	553.80	549.95	555.34	559.95	551.49	556.10

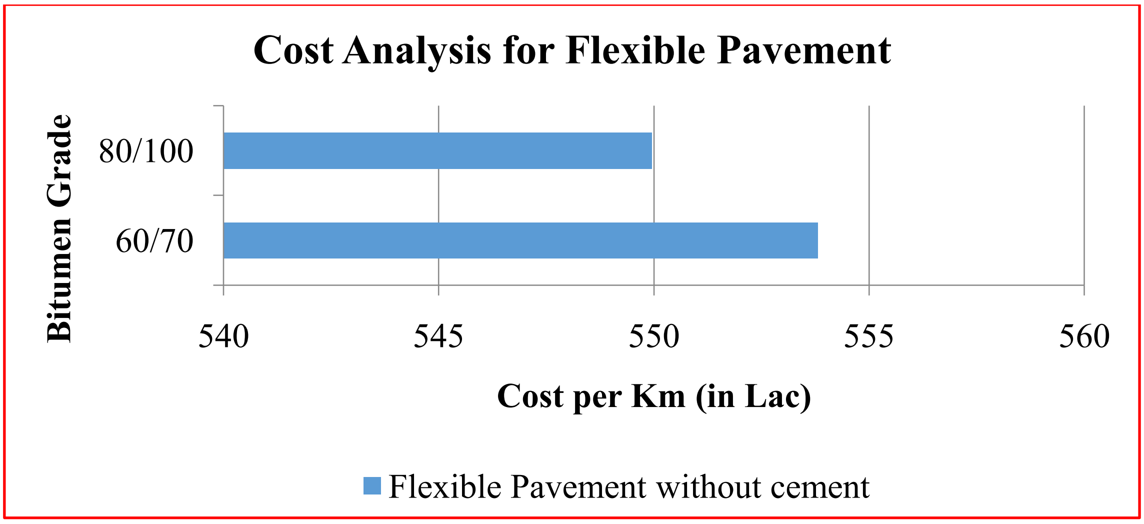


Figure 5.3: Flexible Pavement without Cement

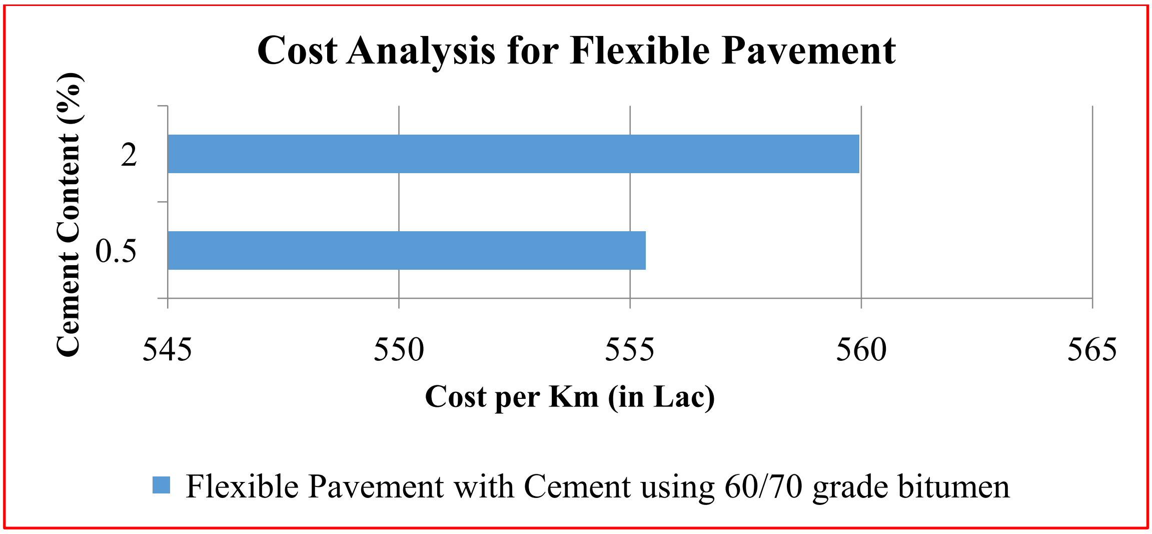


Figure 5.4: Flexible Pavement with Cement using 60/70 grade Bitumen

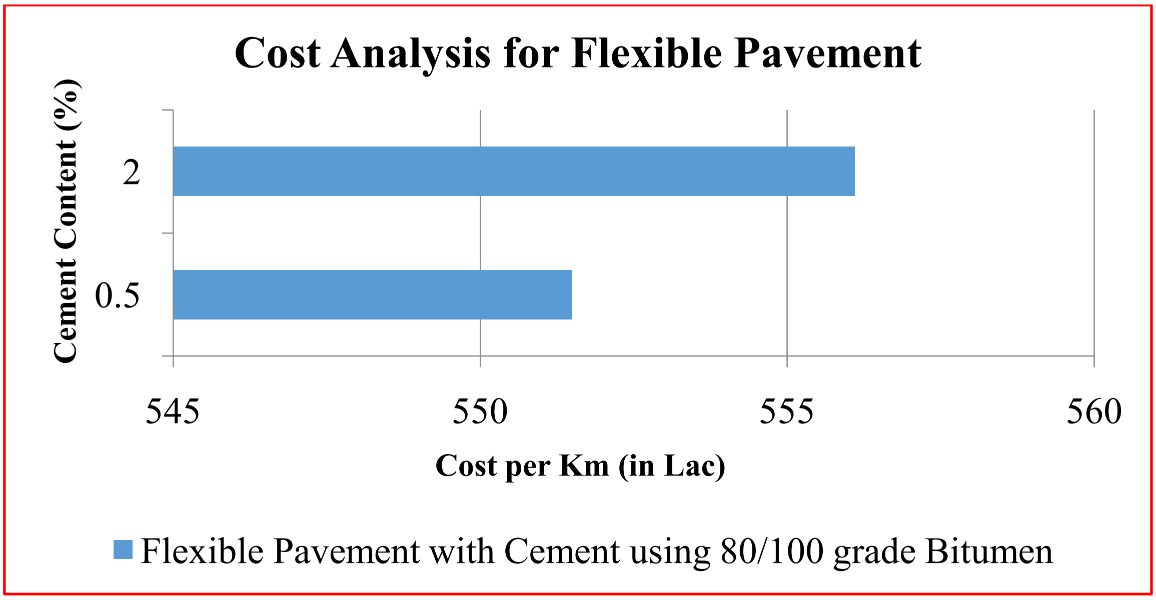
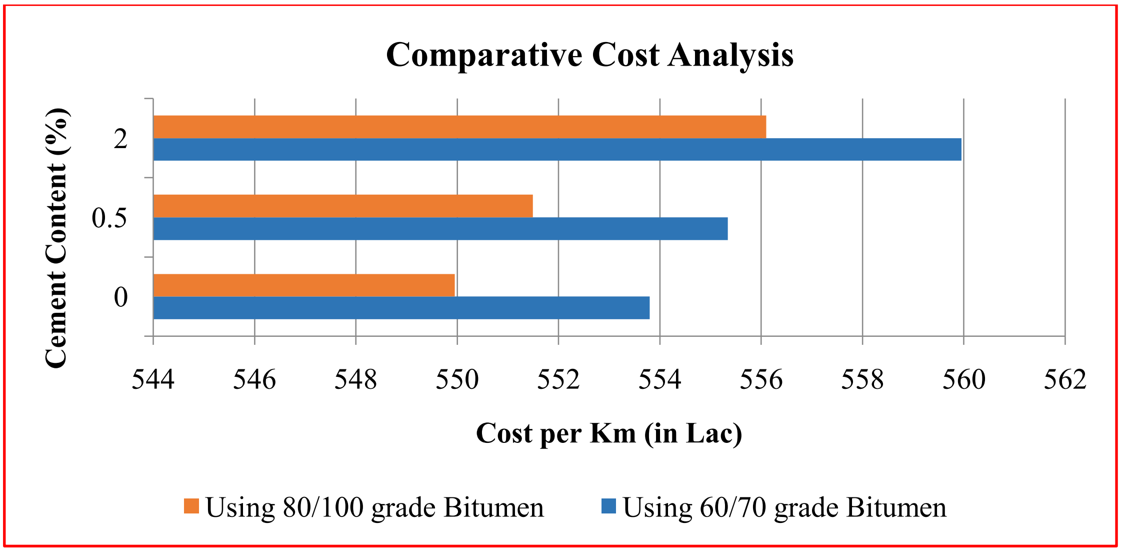


Figure 5.5: Flexible Pavement with Cement using 80/100 grade Bitumen

From the above analysis, it can be concluded that for 10 years design period of per km standard 2-lane pavement, construction cost of **flexiblepavementusing cement** is almost **1.12% higher** than the**flexible pavement without cement**. It is to be noted that in the comparison purposes here only the construction and maintenance costs are considered. If the cost of traffic delay associated with the poor riding quality, workable under submerged condition as well as traffic disruption and diversion during overlay works of the flexible pavement were considered, the **flexiblepavement using cement** would have more economical and favorable because of having good **Tensile Strength Ratio (TSR)** Value. For details on TSR value of different cement content, see **Chapter 4** at **section 4.7.1**.



Type of Pavement	Cost per Km (in Lac)					
	Without Cement		With Cement			
	60/70 Bitumen	80/100 Bitumen	60/70 Bitumen		80/100 Bitumen	
			0.5% Cement	2.0% Cement	0.5% Cement	2.0% Cement
Flexible	553.8	549.95	555.34	559.95	551.49	556.10
Cost of Cement	0.00	0.00	1.54	6.15	1.54	6.15
% of cement cost in total cost using both 60/70 and 80/100 Grade bitumen	0.00	0.00	0.28	1.10	0.28	1.11

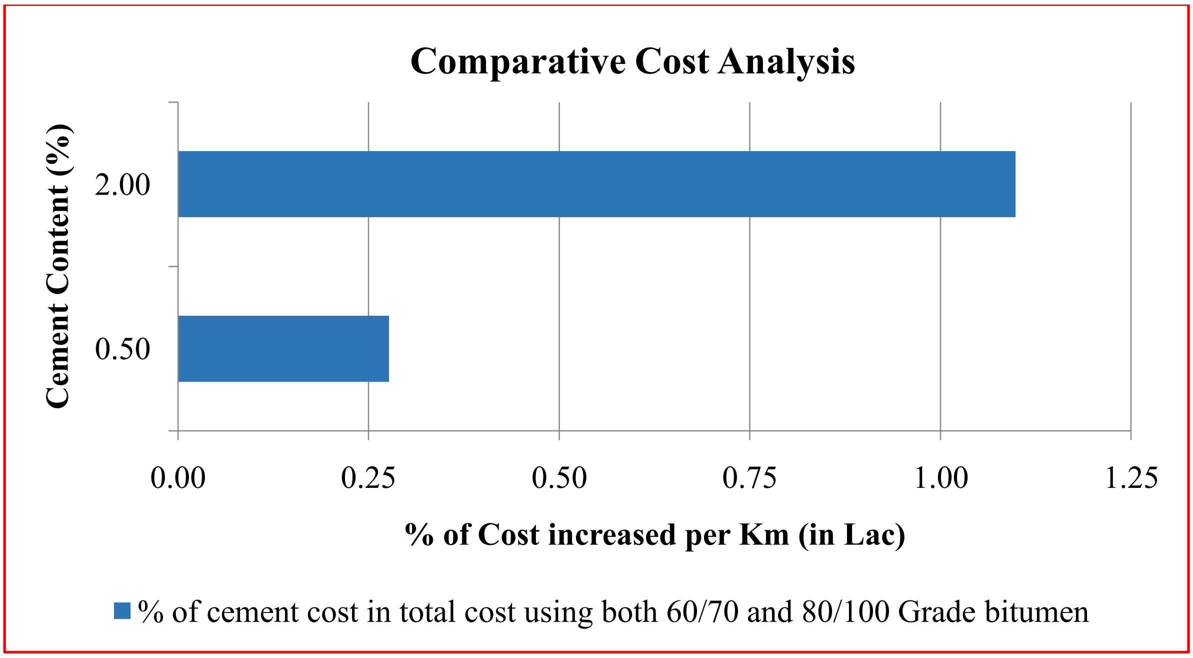


Figure 5.6: Comparative Cost Analysis of Flexible Pavement.

5.3 Economic Analysis

In order to make a true cost comparison between flexible pavements using cement or not, in this section an economic analysis is made by using price escalation of two types pavements' costs i.e. change of unit costs of flexible pavements using cement or no tand materials-bitumen and cement for couple of consecutive years.

Price Escalation of Binders and Mixes

In order to see the price escalation of bitumen and cement as well as bituminous and concrete pavements, unit costs of these items were collected from **the RHD rate of Schedule for the years 2011, 2015 and 2018**. The unit costs and price escalations of binder and mixes over the span of **three-year periods** are presented in the **Table 5.3** and graphically shown in the **Figure 5.7**.

Table 5.3: Unit Cost (in Tk.) of Bitumen and Cement

Year	Bitumen (Ton)	% Increase	Cement (Ton)	% Increase
2011	45000.00	--	8200.00	--
2015	73000.00	38.36	10764.00	23.82
2018	56000.00	-30.36	9000.00	-19.60

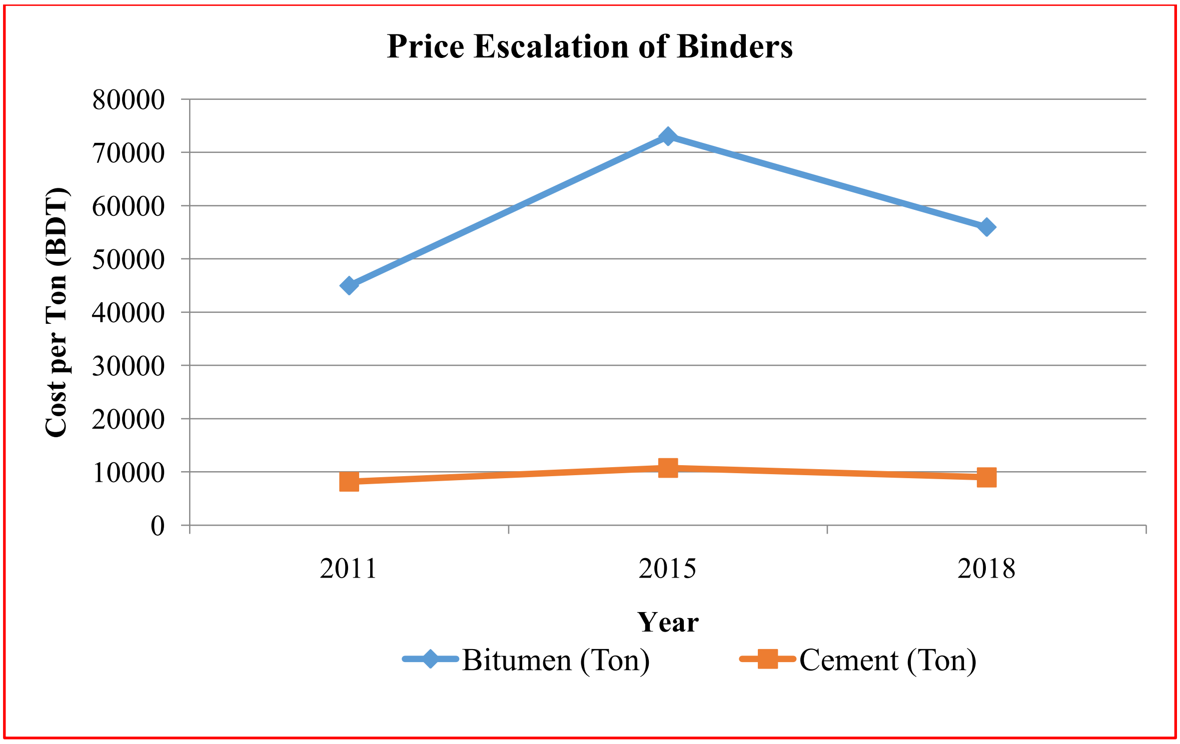


Figure 5.7: Price Escalations of Bitumen and Cement

From the **Table 5.3** it can be seen that over a period of three years the unit cost of bitumen has increased by almost one and halftimes as compared to the unit cost of cement till 2015.On other hand, the unit cost of bitumen has decreased by almost one and half times as compared to the unit cost of cement from 2018.

From the **Figure 5.7** it is clearly evident that the rate of change of bitumen price is very high as compared to the cement. Inferring the present trend it can be reasonably concluded that in the coming year this asphaltic binder would be more economic rather than previous year. Though in the RHD rate of Schedule 2018, the unit cost of flexible paving mix without cement for wearing course is shown as BDT 24,325.00 per cum, but in reality present cost is more than that of the Schedule rate.

Recently, the cost of flexible paving mix using cementis decreasing at very low rate as compared to the flexible paving mix without cement. But it is still higher than conventional flexible paving mix.

This essentially suggests that if the performance of flexible paving mix using cement under moisture condition which is measured by Tensile Strength Ration (TSR) value is good enough than conventional flexible paving mix, then it is more rational and feasible to construct flexible pavement using cement for Bangladesh road network.

5.3.1 Road Maintenance Cost Analysis

Like all structures, roads deteriorate over time. Heavy vehicles and intensive traffic are the main reasons for pavement deterioration. **Table 5.4** represents resurfacing frequency based on the recommendation in the "Viðhaldsaðferðir" report (Valgeirsson, Hjartarson, Guðfinnsson, & Jóhannesson, 2003). Table was recently updated by one of the authors of the report, due to some changes in input parameters: decreased use of studded tires, less initial rut depth, according to the latest measurements results. Updated tables from the report are presented in the **Appendix-(F)**; rehabilitation frequency depends on the traffic density, traffic speed and allowed rut depth. Pavement durability decreases with increased speed.

Maintenance includes all activities needed to keep a country's road network operating indefinitely (Garber and Hole, 2009):

- **Routine maintenance** (restoring drainage, filling potholes and cracks, maintaining edges)
- **Periodic maintenance** (resealing, about every 5 years, to rejuvenate the surface)
- **Rehabilitation** (overlaying, about every 15 years, to restore smoothness and durability)

Table 5.4: Asphalt rehabilitation frequency, when design speed is 70km/h

Treatment Activity		Expected Service Life (Year)
Minor Rehabilitation/ Preservation*	Cold mix with sealing course*	(5-10)
	Distortion corrections	(5-10)
	Drainage improvements	(7-10)
	Frost treatments	(3-5)
	Roadside slopes and erosion control	(3-7)
Major Rehabilitation	Full depth removal & resurfacing	(8-12)
	Full depth reclamation / pulverization	(12-15+)
	Pulverization with expanded asphalt stabilization	(12-15+)
	White topping	(5-10)
	Unbonded Concrete Overlays	(25+)

Treatment Activity		Expected Service Life (Year)
Routine Maintenance	Pothole Repair	< 1
	Roadside maintenance	(1-5)
	Drainage maintenance	(2-5)
	Spray patching	(2-5)
	Localized distortion repair	(2-5)
Minor Rehabilitation / Preservation*	Rout & crack sealing*	(1-5)
	Hot mix patching*	(5-10)
	Surface sealing* (sealcoat, slurry seal, micro-surfacing, chip seal / surface treatment)	(3-7)
	Texturization* (micro-milling, shot blasting, sand blasting)	(1-6)
	Asphalt strip repair* / full depth patching	(5-10)
	Hot mix resurfacing*	(5-12)
	Partial depth removal (milling) & resurfacing*	(8-12)
	In-place recycling* (H1R, CIR, CIREAM)	(7-15+)

The road condition can be grossly categorized into descriptive bands based on roughness. Thesecategories are shown in **Table 5.5**. Different ranges are adopted for each road class to reflect theirrelative importance and the level of service that should be expected from each road class.

Table 5.5: Qualitative descriptors of IRI values (Maintenance and Rehabilitation Needs Report of 2016 - 2017 for RHD Paved Roads)

Condition	National Highway	Regional Highway	Zila Road
Initial Roughness Index (IRI) Values			
Good	0-3.9	0-4.9	0-5.9
Fair	4.0-5.9	5.0-6.9	6.0-7.9
Poor	6.0-7.9	7.0-8.9	8.0-8.9
Bad	8.0-9.9	9.0-10.9	10.0-11.9
Very Bad	>=10.0	>=11.0	>= 12.0

5.4 Maintenance Strategies

Planned maintenance is generally preferred to unplanned (demand) maintenance, and preventive maintenance is preferred to corrective maintenance. **Figure 5.9** shows the relationship between condition and the life of the pavement. The pavement starts in very good shape and deteriorates slowly at first. Maintenance repairs done early in the life of the pavement are much le. Expensive. **Figure 5.10** shows the relationship between pavement condition and the various levels of maintenance. These two figures show that routine and preventive maintenance are the most economical options. Reconstruction techniques are the most expensive, and are usually done when there is no other choice.

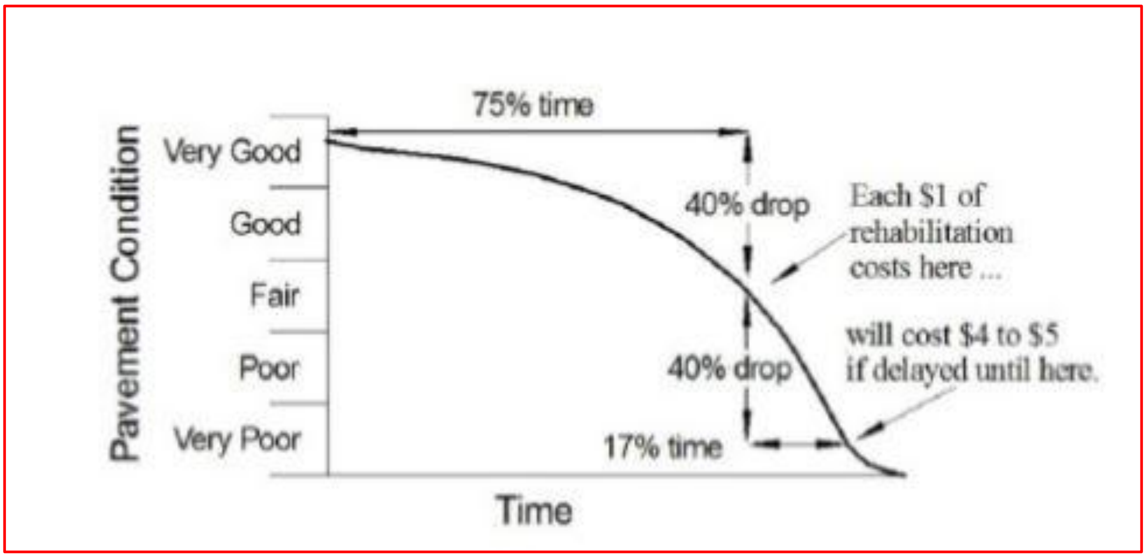


Figure 5.9: Pavement deterioration curve (Life Cycle Cost Analysis of Asphalt and Concrete Pavements by Asta Guciute Scheving, Master of Science Thesis, 2011)

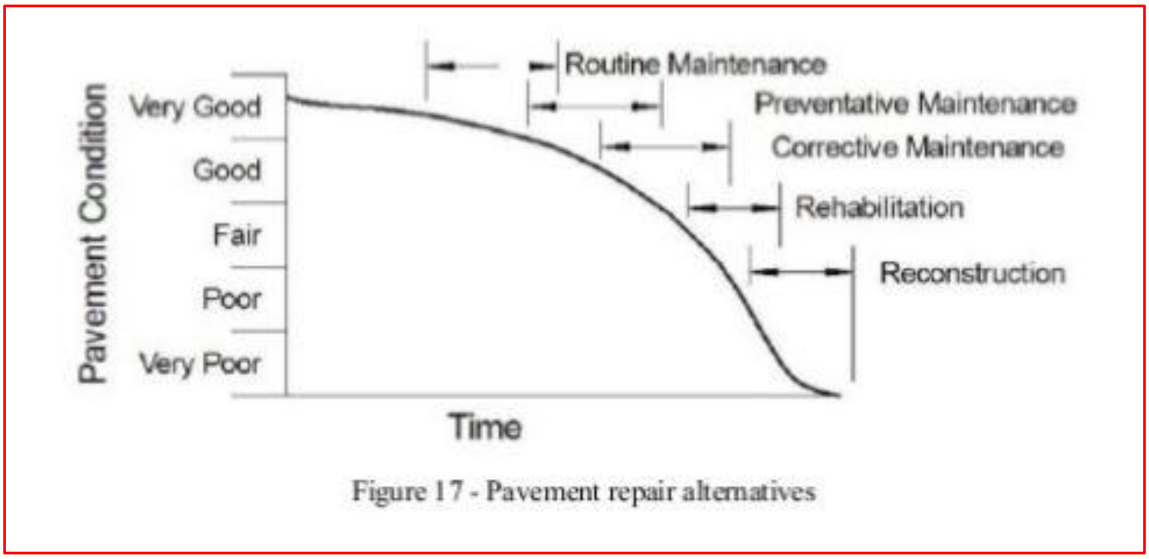


Figure 5.10: Pavement repair alternatives(Life Cycle Cost Analysis of Asphalt and Concrete Pavements by Asta Guciute Scheving, Master of Science Thesis, 2011)

Road condition evaluation by visual assessment method is not accurate. But it is very much effective for surveying the overall condition of the road network. There are different methods through which we can identify the road condition such as

- RHD method
- DRIVe. method
- Indian Road Congress Method

Here we discuss about the RHD method to evaluate the road condition. In order to identify the condition of the road, we have to first identify the cracks, distresses, surface defects and deformation. Guidelines for the estimation of Pavement Condition Rating and Priori, for Flexible Pavements and a typical pavement rating form are also provided as **Figure 5.11**.

A Guide for the Estimation of

Pavement Condition Rating and Priority for Flexible Pavements

Recommended Treatment	Rating	Pavement Condition
Reconstruct or recycle within 2 years	0-20	Pavement is in poor to very poor condition with extensive severe cracking, alligator and channeling. Ride ability is poor and the surface is very rough and uneven.
Reconstruct or recycle within 2 -3 years.	20-30	Pavement is in poor condition with moderate alligator and extensive severe cracking and channeling. Ride ability is poor and the surface is very rough and uneven.
Overlay, recycle or reconstruct within 3 – 4 years	30-40	Pavement is in poor to fair condition with frequent moderate alligator and extensive moderate cracking and channeling. Ride ability is poor to fair and surface is moderately rough and uneven.
Reconstruct in 4 -5 years or resurface within 2 years with extensive leveling	40-50	Pavement is in poor to fair condition with frequent moderate cracking and channeling, and intermittent moderate alligator. Ride ability is poor to fair and surface is moderately rough and uneven.
Resurface within 3 years.	50-65	Pavement is in fair condition with intermittent moderate and frequent slight cracking, and with intermittent slight or moderate alligator and channeling. Ride ability is fair and surface is slightly rough and uneven.
Resurface in 3 -5 years	65-80	Pavement is in fairly good condition with frequent slight cracking, slight or very slight channeling and a few areas of slight alligator. Ride ability is fairly good with intermittent rough and uneven sections.
Normal maintenance only.	80-100	Pavement is in good condition with frequent very slight or slight cracking. Ride ability is good with a few slightly rough and uneven sections.

Figure 5.11: Pavement rating form

BITUMINOUS PAVEMENT RATING FORM

STREET OR ROUTE ______________ CITY OR COUNTY __________

LENGTH OF PROJECT _____________ WIDTH ___________________

PAVEMENT TYPE _______________ DATE _____________________

(Note: A rating of "0" indicates defect does not occur)

DEFECTS	RATING	
Transverse Cracks	0-5	
Longitudinal Cracks	0-5	
Alligator Cracks	0-10	
Shrinkage Cracks	0-5	
Rutting	0-10	
Corrugations	0-5	
Raveling.	0-5	
Shoving or Pushing.	0-10	
Pot Holes. : .	0-10	
Excess Bitumen	0-10	
Polished Aggregate ...	0-5	
Deficiency in Drainage	0-10	
Overall Riding Quality (0 is excellent; 10 is very poor):	0-10	
	Sum of Defects	

Condition Rating = 100 -Sum of Defects

= 100 - _________

Figure 5.11: Pavement rating form

5.4.1 Principles of Road Maintenance Cost Analysis

The Road maintenance cost analysispredicts the pavement conditions (performance), the requiredtreatments and costs and benefits over a specified period (in this case 20 years) under a user-definedmaintenance strategy. The costs used in this analysis include cost of capital investment, maintenancecosts and vehicle operating costs.

The costs of two scenarios are compared:

- The "do minimum maintenance" scenario (either routine maintenance or a "holding strategy").

Details of treatments considered in Bangladesh can be seen in **Table 5.6**. Maintenance strategies wereset for these treatments based on road condition, traffic and roughness data for different classes ofroads (see **Table 5.7**). Holding strategy has been included which means that DBST /carpeting has to be provided instead of going for higher treatment if there is shortage of funds to keepthe roads at maintainable condition. DBST was considered for National and Regional roads andcarpeting for Zilla roads.

The benefits and costs of the above scenarios are compared for a Highway Development and Management (HDM-4) life cycle analysis of 20years. The Net Present Value (NPV)/costs were utilized to prioritize treatment options at a 12%discount rate. NPV/cost was chosen to obtain maximum benefits as it produces highest benefits whenthere is crisis in funding.

5.4.2 Description of Treatments

The HDM analysis considers a number of treatments representing the most commonly used types ofmaintenance work items in Bangladesh. **Table 5.6** provides details of these treatments and theassumptions made for HDM.

Table 5.6: Maintenance and rehabilitation treatments and assumptions used in HDM-4

Routine Maintenance	Off-pavement works	Includes all regular works along a road such as maintaining shoulders, roadside vegetation control, cleaning side drains and pipe culverts, maintenance of signs and signals.
	Patching	Repair of potholes based on a standard pothole unit of0.01m3 per pothole. The quantity of pothole repairing shall not be more than 1% of the total surface.
	Crack Sealing	Sealing to cracks using Seal Coat/Fog Seal. It assumes a maximum in any one kilometer of 5% area affected.

Periodic Maintenance	Preparatory Patching	Patching potholes and regulating surface irregularities prior to undertaking the treatments like DBST or DBS Overlay. Should not be more than 2% of the total quantity of overlay for National roads and maximum of 5% for Regional roads.
	Preparatory Edge Repair	Allows for restoring pavement edges that have been damaged by vehicles leaving the road to drive onto the shoulder prior to undertaking the treatments like DBST or DBS Overlay.
	DBST	Applying two layers of surface treatments on the prepared road surface. This is applied in medium to highly trafficked road. Life expectancy assumed to be 3 years.
	Bituminous Carpeting	This is a 40 mm thick manual overlay used in low trafficked roads in place of dense bituminous overlay. Life expectancy has been taken as 2 to 4 years.
	Overlay	Machine laid premixed dense bituminous surfacing overlay 40 – 80 mm thick used in medium to highly trafficked roads. Carefully controlled overlay may be applied in response to badly damaged road surface or high roughness so as to obtain a predefined roughness level (2.5 to 3 IRI). Life expectancy assumed to be 5 years.
Rehabilitation	Partial Reconstruction	Reconstruction of the upper pavement layers following scarification of the existing damaged surface and re-compaction. Normally a 150-200 mm crushed aggregate base with a dense bituminous surfacing of between 75 and 195mm, depending on traffic level. This is a treatment to overcome higher roughness or higher levels of surface cracking resulting from delayed maintenance. Life expectancy should be 10 years prior to major periodic maintenance. Full design of the pavement must be undertaken prior to treatment. Shoulder rehabilitation would also be provided where necessary.
	Complete Reconstruction	A major reconstruction on the existing alignment and within the same overall dimension limits. The road is not widened. The pavement must be fully designed prior to construction and shoulder rehabilitation provided where necessary. Life expectancy should be 10 years before major periodic maintenance. Applied where there are extremely high levels of roughness and extensive cracking.
Holding Treatment		DBST triggered when rehabilitation is required but budget constraints do not permit the preferred treatment. Expected to last for 3 years

Table 5.7 shows the compound maintenance standards adopted for HDM-4 analysis for the different classes of roads. These standards are based on experience and analysis of road conditions in Bangladesh, and are considered to be a reliable basis for HDM-4 to estimate economic performance of the network. Final treatment designs must be separately established.

Compound maintenance standards have been modified slightly, but are similar to the previous year’sstandards. The slight modification relates to the introduction of a DBST in the holding strategy of National and Regional roads when the roughness will exceed 12 IRI. Similarly carpeting was introduced in the holding strategy of Zilla roads for roughness greater than 12 IRI.

Corridor roads (N1, N2, N3, N4, N5, N6, N7 and N8) were given high priority and hence they were analyzed separately as they cover the major traffic and will be the part of the Asian Highway Network in the near future. Hence, periodic maintenance was considered at 4 IRI. The other National highways, Regional highways and Zilla roads were considered for periodic maintenance at 5, 5.5 and 6 IRI respectively. “Holding maintenance strategy” was considered to maintain roads using DBST when funding is limited and higher treatments cannot be provided. Application of DBST can then delay further road deterioration.

Table 5.7: Compound maintenance standards for HDM-4 program analysis by RHD in 2016-17

<table>
<tr><th colspan="7">Holding Standard without Reconstruction for National and Regional Roads</th></tr>
<tr><th rowspan="2">Roughness Range (IRI)</th><th rowspan="2">All Damage (%)</th><th colspan="5">Traffic Range (MT-AADT)</th></tr>
<tr><th>100 - 1999</th><th>2000 - 3999</th><th>4000 - 5999</th><th>6000 - 9999</th><th>> 10000</th></tr>
<tr><td rowspan="5"><12</td><td><5%</td><td colspan="5">Routine</td></tr>
<tr><td>5 - 10%</td><td colspan="3">Routine</td><td rowspan="4" colspan="2">DBST</td></tr>
<tr><td>10 - 20%</td><td>Routine</td><td>Routine</td><td rowspan="3">DBST</td></tr>
<tr><td>20 - 30%</td><td>Routine</td><td rowspan="2">DBST</td></tr>
<tr><td>> 30%</td><td>DBST</td></tr>
<tr><td>> 12.00</td><td>All</td><td colspan="5">DBST</td></tr>
</table>

<table>
<tr><th colspan="7">Holding Standard with Reconstruction for National and Regional Roads</th></tr>
<tr><th rowspan="2">Roughness Range (IRI)</th><th rowspan="2">All Damage (%)</th><th colspan="5">Traffic Range (MT-AADT)</th></tr>
<tr><th>100 - 1999</th><th>2000 - 3999</th><th>4000 - 5999</th><th>6000 - 9999</th><th>> 10000</th></tr>
<tr><td rowspan="5"><12</td><td><5%</td><td colspan="5">Routine</td></tr>
<tr><td>5 - 10%</td><td colspan="3">Routine</td><td rowspan="4" colspan="2">DBST</td></tr>
<tr><td>10 - 20%</td><td colspan="2">Routine</td><td rowspan="3">DBST</td></tr>
<tr><td>20 - 30%</td><td>Routine</td><td rowspan="2">DBST</td></tr>
<tr><td>> 30%</td><td>DBST</td></tr>
<tr><td>> 12.00</td><td>All</td><td>Full Recon 110mm</td><td>Full Recon 135mm</td><td>Full Recon 150mm</td><td>Full Recon 180mm</td><td>Full Recon 195mm</td></tr>
</table>

Table 5.7: Compound maintenance standards for HDM-4 program analysis by RHD in 2016-17

<table>
<tr><th colspan="7">Holding Standard with Reconstruction for Zilla Roads</th></tr>
<tr><th rowspan="2">Roughness Range (IRI)</th><th rowspan="2">All Damage (%)</th><th colspan="5">Traffic Range (MT-AADT)</th></tr>
<tr><th>100 - 999</th><th>1000-1999</th><th>2000-2999</th><th>3000-3999</th><th>>4000</th></tr>
<tr><td rowspan="4"><12</td><td>0 -10%</td><td>Routine</td><td>Routine</td><td>Routine</td><td>Routine</td><td>Routine</td></tr>
<tr><td>10-20%</td><td>Routine</td><td>Routine</td><td colspan="3" rowspan="3">Overlay 40mm</td></tr>
<tr><td>20-30%</td><td>Routine</td><td rowspan="2">Overlay 40mm</td></tr>
<tr><td>>30%</td><td>Overlay 40mm</td></tr>
<tr><td>> 12.00</td><td>All</td><td colspan="5">Full Rec 75mm</td></tr>
</table>

<table>
<tr><th colspan="7">Compound Maintenance Standards for National Corridor Roads</th></tr>
<tr><th rowspan="2">Roughness Range (IRI)</th><th rowspan="2">Cracking Range (%)</th><th colspan="5">Traffic Range (MT-AADT)</th></tr>
<tr><th>100 - 1999</th><th>2000 - 3999</th><th>4000 - 5999</th><th>6000 - 9999</th><th>> 10000</th></tr>
<tr><td rowspan="2">< 4.0</td><td>< 25%</td><td colspan="5">Routine</td></tr>
<tr><td>>= 25%</td><td colspan="4">DBST</td><td>Overlay 50mm</td></tr>
<tr><td>4.00 - < 7.00</td><td>All</td><td colspan="3" rowspan="2">Overlay 50mm</td><td>Overlay 50mm</td><td>Overlay 80mm</td></tr>
<tr><td>7.00 - < 9.00</td><td>All</td><td>Overlay 60mm</td><td>Overlay 80mm</td></tr>
<tr><td>9.00 - < 12.00</td><td>All</td><td>Rehab 120mm</td><td>Rehab 140mm</td><td>Rehab 150mm</td><td>Rehab 180</td><td>Rehab 195mm</td></tr>
<tr><td>> 12.00</td><td>All</td><td>Full Rec 120mm</td><td>Full Rec 140mm</td><td>Full Recon 150mm</td><td>Full Recon 180mm</td><td>Full Recon 195mm</td></tr>
</table>

<table>
<tr><th colspan="7">Compound Maintenance Standards for Other National Roads</th></tr>
<tr><th rowspan="2">Roughness Range (IRI)</th><th rowspan="2">Cracking Range (%)</th><th colspan="5">Traffic Range (MT-AADT)</th></tr>
<tr><th>100 - 1999</th><th>2000 - 3999</th><th>4000 - 5999</th><th>6000 - 9999</th><th>> 10000</th></tr>
<tr><td rowspan="2">< 5.0</td><td>< 25%</td><td colspan="5">Routine</td></tr>
<tr><td>>= 25%</td><td colspan="4">DBST</td><td>Overlay 50mm</td></tr>
<tr><td>5.00 - < 7.00</td><td>All</td><td colspan="3" rowspan="2">Overlay 50mm</td><td>Overlay 50mm</td><td rowspan="2">Overlay 80mm</td></tr>
<tr><td>7.00 - < 9.00</td><td>All</td><td>Overlay 60mm</td></tr>
<tr><td>9.00 - < 12.00</td><td>All</td><td>Rehab 110mm</td><td>Rehab 135mm</td><td>Rehab 150mm</td><td>Rehab 180 mm</td><td>Rehab 195mm</td></tr>
<tr><td>> 12.00</td><td>All</td><td>Full Recon 110mm</td><td>Full Recon 135mm</td><td>Full Recon 150mm</td><td>Full Recon 180mm</td><td>Full Recon 195mm</td></tr>
</table>

Table 5.8 shows the unit cost of different work prepared in accordance with the RHD Schedule of Rates 2018.

Table 5.8: Unit costs of Treatment

Work Type	Work Class	Work Description	Financial Costs (BDT)	Unit
Routine	Routine Maintenance	Routine	80000	Per Km
		Patching	1573	Per m^2
		Edge Repair	1573	Per m^2
		Crack Sealing	252	Per m^2
Periodic	Resurfacing	Seal Coat	252	Per m^2
		DBST	494	Per m^2
	Asphalt Mix Resurfacing	Overlay 40mm	894	Per m^2
		Overlay 50mm	1050	Per m^2
		Overlay 60mm	1251	Per m^2
		Overlay 80mm	1653	Per m^2
		Overlay 100mm	2056	Per m^2
		Overlay 120mm	2458	Per m^2
Rehabilitation	Partial Reconstruction	Partial Recon 75mm	2993	Per m^2
		Partial Recon 100mm	3496	Per m^2
		Partial Recon 110mm	3697	Per m^2
		Partial Recon 120mm	3898	Per m^2
		Partial Recon 135mm	4528	Per m^2
		Partial Recon 140mm	4629	Per m^2
		Partial Recon 150mm	4830	Per m^2
		Partial Recon 180mm	5762	Per m^2
		Partial Recon 195mm	6064	Per m^2
Reconstruction	Full Reconstruction	Full Recon 75mm	4347	Per m^2
		Full Recon 100mm	4733	Per m^2
		Full Recon 110mm	4934	Per m^2
		Full Recon 120mm	5136	Per m^2
		Full Recon 135mm	5464	Per m^2
		Full Recon 140mm	5866	Per m^2
		Full Recon 150mm	6068	Per m^2
		Full Recon 180mm	6999	Per m^2
		Full Recon 195mm	7301	Per m^2

5.4.3 Maintenance Cost of Flexible Pavement

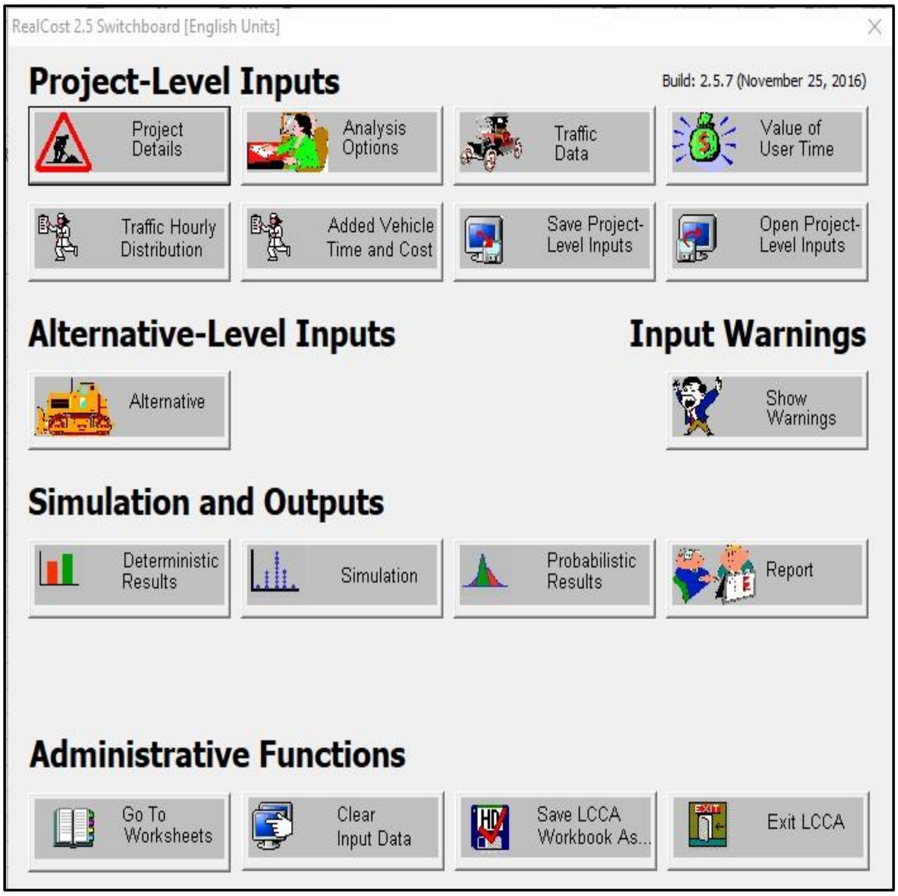
Maintenance cost for a heavily trafficked highway taking over 10,000 AADT on a soil sub grade (CBR 3%) condition, the various layer thickness of 2-lane flexible pavement for 10 years design period are given in **Table 5.9**.

Table 5.9: Different maintenance treatment cost of flexible pavement (per Km)

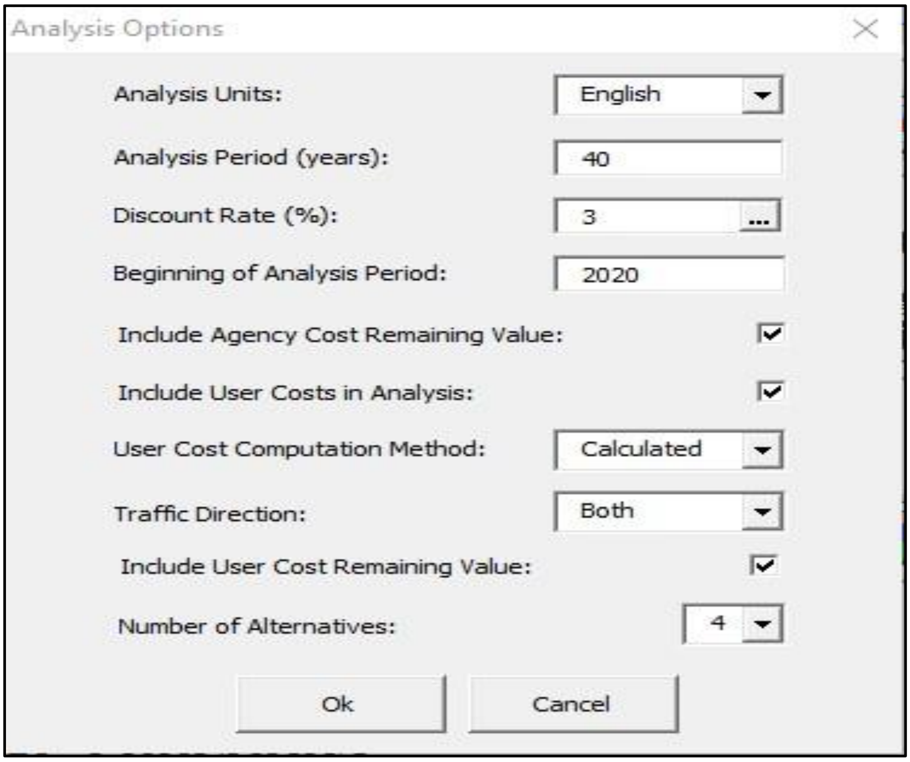
Work Type	Work Class	Work Description	Financial Costs (BDT)	Unit	Total Costs per Km (in Lac BDT)
Routine	Routine Maintenance	Routine	80000	Per Km	0.80
Periodic	**Resurfacing**	**DBST**	**494**	**Per m^2**	**36.06**
	Asphalt Mix Resurfacing	Overlay 40mm	894	Per m^2	244.73
		Overlay 50mm	1050	Per m^2	287.44
		Overlay 60mm	1251	Per m^2	342.46
		Overlay 80mm	1653	Per m^2	452.51
Rehabilitation	Partial Reconstruction	Partial Recon. 195mm	6064	Per m^2	1660.02
	Full Reconstruction	Full Recon. 180mm	6999	Per m^2	1915.98
		Full Recon. 195mm	7301	Per m^2	1998.65

From the initial cost of flexible pavement construction with or without cement as filler material and maintenance costs as per design life, we can conclude that the initial cost of flexible pavement with cement is higher than the normal pavement.

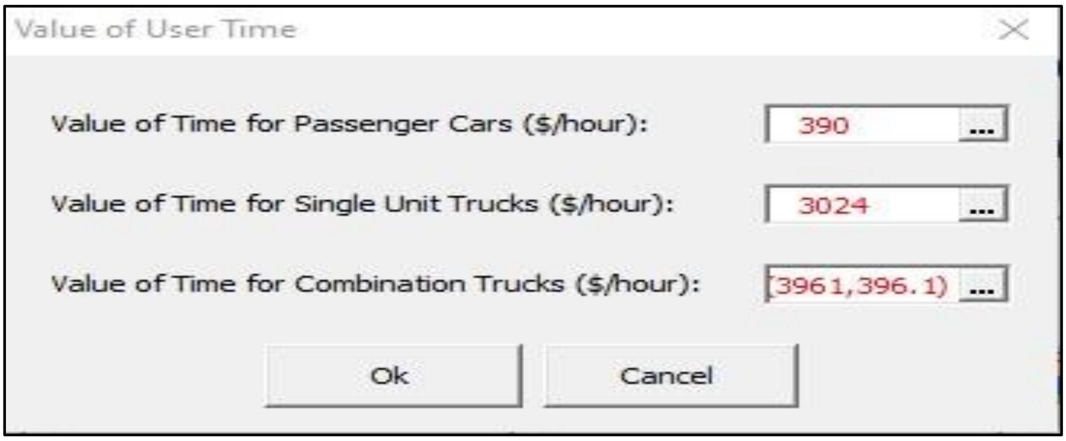
LCCA analysis (Design Data) using Real Cost 2.5 module



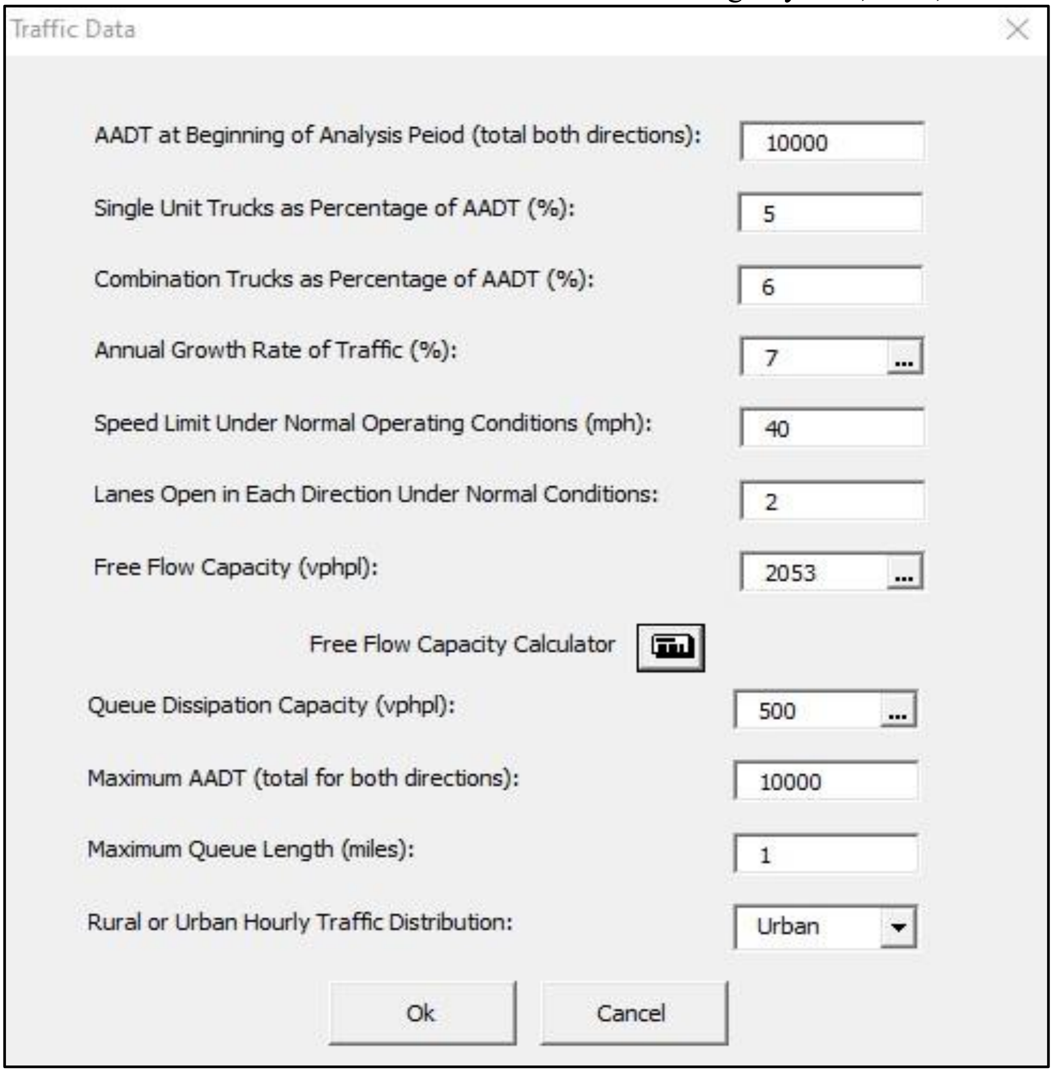
Real Cost 2.5 Interface



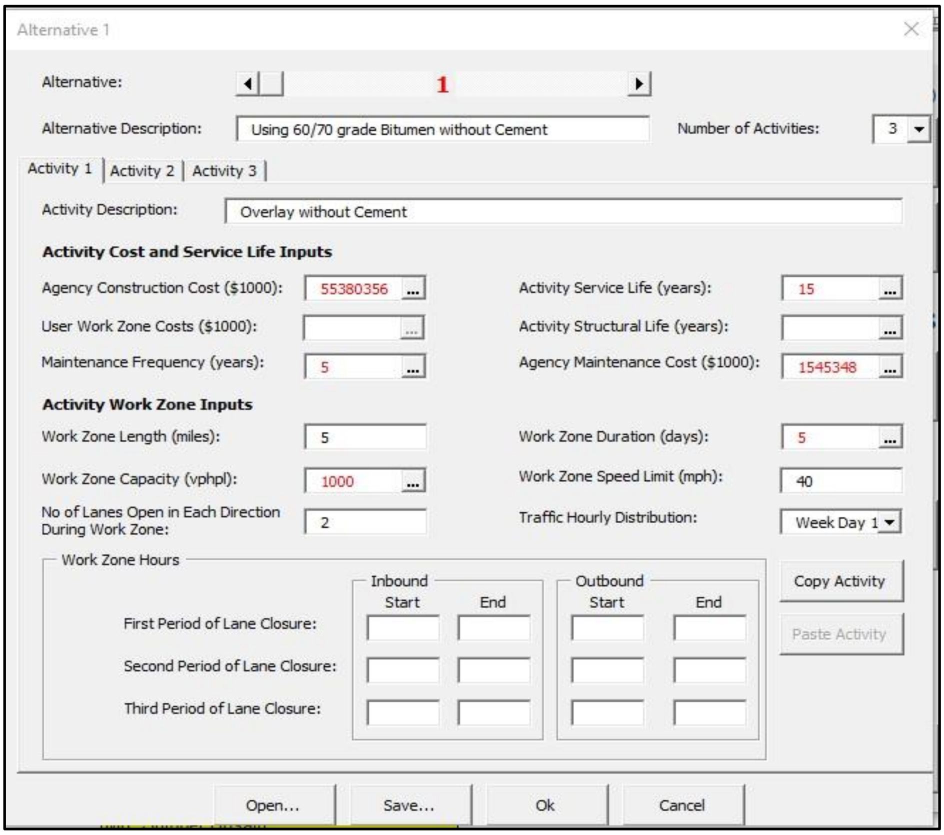
Basic Design Parameter



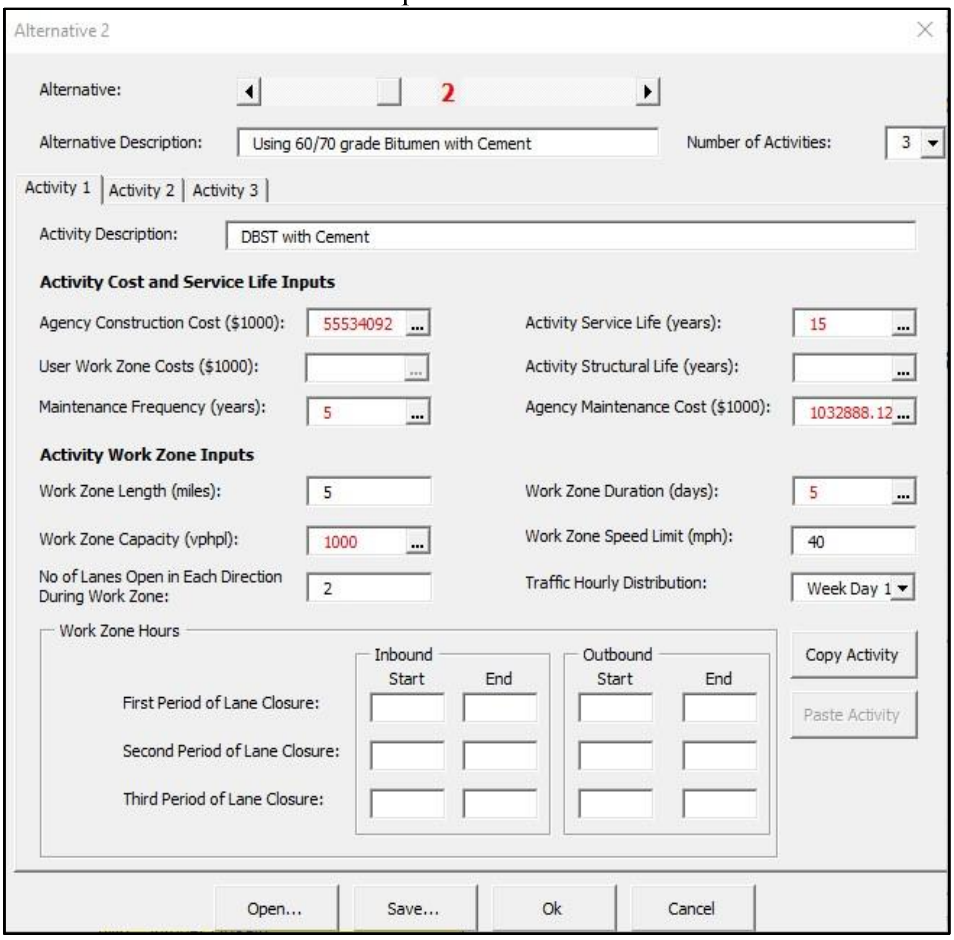
Value collected from RHD Road User Costs Knowledge System, June, 2007



Traffic Data for Analysis



Alternative options-1 for maintenance



Alternative options-2 for maintenance

From LCCA analysis using Real Cost 2.5 module, the life cycle cost result is given below as:

Table 5.10: Deterministic life cycle cost result (cost per Km)

Total Cost	60/70 Bitumen without cement	60/70 Bitumen with 0.50 % cement
	Agency Cost (in BDT)	Agency Cost (in BDT)
Undiscounted Sum	**1738,67,792**	**1717,66,720**
Present Value	**1183,68,672**	**1171,50,440**
EUAC	**51,20,910**	**50,68,206**
Lowest Present Value Agency Cost	**Flexible pavement (per Km) using 60/70 Bitumen with 0.50% cement**	

Table 5.11: Probabilistic life cycle cost result (cost per Km)

Total Cost (Present Value)	60/70 Bitumen without cement	60/70 Bitumen with 0.50 % cement
	Agency Cost (in BDT)	Agency Cost (in BDT)
Mean	**1181,01,948**	**1164,26,956**
Standard Deviation	**80,31,733.93**	**85,98,421.13**
Minimum	**895,99,360**	**957,45,208**
Maximum	**1346,71,824**	**1399,49,520**

But, cement modified flexible pavement requires less maintenance cost than that of flexible pavements. Till 03 years, cement modified pavement needs routine maintenance having BDT 0.80 Lac where as normal pavement needs rehabilitation with cost of BDT 36.86 Lac. However, the initial cost of pavement with cement is BDT 1.54 Lac. For more details, see the **Appendix-(F)**.

CHAPTER 6

CONCLUSION AND RECOMMENDATIONS

6.1 Conclusion

The objective of this study is to investigate the effect of using Portland cement as filler in the Asphalt Binder Course, where the results can be concluded as the following:

- The existence of Portland cement in the asphalt binder course mixed is considered as an eco-friendly material and it can be utilized as a sustainable management against weather susceptibility effects.
- The results of this study apply only to the specific gradation (**C.A: F.A:M.F=55:40:5**) and the type of cement (**Portland cement**) that were used. Other gradations of aggregate or resources (cement or others) may produce different results.
- Portland cementis used in asphalt binder course with the optimum content of **0.50%** of total aggregate.
- The results of Marshall Stability, flow, bulk density and air voids of cement mixture asphalt are consistent with the specifications range at the different percentages of cement contents (0.50, 1.00, 1.50, 2.00 and 2.50 %).
- Marshall Stability and the bulk density achieve the Asphalt Institute specifications requirements with 0.50% cement content.
- At **2.00%** cement content the value of Air Voids (%) slightly lower than the minimum limit of the international specification.
- From the initial cost of flexible pavement construction with or without cement and maintenance costs as per design life, we can conclude that the initial cost of flexible pavement with cement is higher than the normal pavement.
- As per the LCCA analysis, the Deterministic results show that Flexible pavement (per Km) using 80/100 Bitumen with 0.50% cement has Lowest Present Value Agency Cost.
- In the respect of Maintenance cost of the flexible pavement, the life cycle cost of cement modified pavements is lower than the conventional flexible pavement.

- As the maintenance mechanism of cement modified flexible pavement is empirical, the actual scenario of this study helps to provide a qualitative idea regarding long term pavement policy in Bangladesh.

6.2 Recommendations

- Further studies are needed using various types of filler materials such ascement kiln dust, glass, hydrated lime etc.at different percentages of that material.
- More studies are needed to study the effect of cement in wearing course layers of asphalt pavement.
- It is recommended to evaluate using Portland cement from other products like Hydrophobic Cement, Portland Blast Furnace Cement, Air Entraining Cement and High Alumina Cement etc.
- It is recommended for the road authority like RHD, LGED etc. in Bangladesh to permit using Portland cement in asphalt pavements depending on the results of this research and other researches, and to encourage using binding materials in construction fields.
- It is recommended to encourage the field application and evaluation to find out the performance of hot mix asphalt containing recycled waste materials.
- Further studies are needed to study the effect of cement in binder courseagainst weather susceptibility condition.
- Since HDM-4 is basically an economic tool for selection and prioritization of road maintenance options, Field investigations and design verification must be undertaken before finalizing treatment option.
- Routine maintenance has to be done properly and should be the first investment priority.
- Since the prioritization of road maintenance using NPV/Cost favors the more highly trafficked roads,adoption of a separate prioritization process for zilla road is recommended.
- Separate investment should be assigned for routine maintenance, periodic maintenance and rehabilitation in order that monitoring of expenditure can be effectively carried out.

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Appendix (E)

Tensile Strength Ratio (TSR) Test results

Calculations (Interactive Equation) for TSR Test

Calculate the tensile strength as follows:

$$S_t = \frac{2P}{\pi t \ D}$$

Where:

- St = tensile strength (psi)
- P = maximum load (lbs)
- t = sample thickness (inches)
- D = sample diameter (inches)

Express the resistance to moisture damage as a ratio of the unconditioned sample tensile strength that is retained after the conditioning.

Calculate the TSR as follows:

$$TSR = \frac{S_2}{S_1}$$

Where:

- TSR = tensile strength ratio
- S1 = average tensile strength of unconditioned samples
- S2 = average tensile strength of conditioned samples

Bitumen content (By weight of the mix) =	**5.625**
Aggregate Source:	**Bholaganj**
Portland Cement (% by aggregates weight)=	**0.5**

Sample ID				1	2	3	1	2	3
Particulars	Unit	Symbol	Equation	Dry Subset			Moisture Conditioned Subset		
Diameter	mm	D		99	99	100	100	99	98
Thickness	mm	T		68	69	67	69	68	68
Dry mass air	gm	A		1198.8	1201.3	1200.4	1202.8	1200.0	1202.1
Saturated surface dry mass	gm	B		1201.2	1203.0	1202.6	1204.5	1202.8	1204.3
Mass in water	gm	C		672.5	673.6	670.8	675.2	673.2	674.3
Vol of test specimen	cc	E	B-C	528.7	529.4	531.8	529.3	529.6	529.5
Bulk specific gravity	gm/cc	F	A/(B-C)	2.267	2.269	2.257	2.272	2.265	2.270
Max specific gravity (gmm)	gm/cc	G		2.463	2.463	2.463	2.463	2.463	2.463
Air void	%	H	(G-F) /G*100	7.95	7.87	8.36	7.75	8.03	7.83
Vol of air void	cc	I	H*E/100	42.03	41.66	44.45	41.02	42.52	41.45
Crushing load	KN	P		35.0	36.5	34.8			

Table-I: Initial Data on Dry Subset as Well as Moisture Conditioned Subset Samples (Adding 0.5% Cement)

Sample ID				1	2	3	1	2	3
Particulars	Unit	Symbol	Equation	Dry Subset			Moisture Conditioned Subset		
Saturation time	Min	D					15	15	15
Vacuum pressure in Hg	mm	T					445	435	420
Saturated surface dry mass	gm	B’					1222.5	1221.6	1223.2
Mass in water	gm	C’					689.8	689.7	690
Vol of test specimen	cc	E’	B’-C’				532.7	531.9	533.2
Vol of absorbed water	cc	J’	B’-A				19.7	21.6	21.1
Saturation	gm/cc	G	J’/I *100				48.02	50.79	50.9
Swell	%	H	[(E’-E)/E]* 100				0.64	0.43	0.69

Table-II: Data on Moisture – Conditioned Subset: Partial Saturation (Adding 0.5% Cement)

Sample ID				1	2	3	1	2	3
Particulars	Unit	Symbol	Equation	Dry Subset			Moisture Conditioned Subset		
Thickness	mm	T’					69.5	69.0	69.5
Saturated surface dry mass	gm	B”					1225.6	1224.5	1226.8
Mass in water	gm	C”					690.3	690.1	690.8
Vol of test specimen	cc	E”	B”-C”				535.3	534.4	536
Vol of absorbed water	cc	J”	B”-A				22.8	24.5	24.7
Saturation	gm/cc		J”/I *100				55.58	57.61	59.58
Swell	%		[(E”-E)/E]* 100				1.13	0.9	1.22
Crushing load	KN	P’					30.0	32.0	31.5

Table-III: Data on Moisture –Conditioned Subset: Test Specimen in 60° C Water for 24 Hours (Adding 0.5% Cement)

Sample ID				1	2	3	1	2	3
Particulars	Unit	Symbol	Equation	Dry Subset			Moisture Conditioned Subset		
Dry strength	Kpa	Sth	$2*P*10^6/(\pi * T * D)$	3.31	3.40	3.31			
Average dry strength	Kpa			3.34					
Wet strength	Kpa	Stm	$2*P*10^6/(\pi * T * D)$				2.77	2.87	2.89
Average wet strength	Kpa						2.84		
TSR%	%		100x(Stm/Sth)	**85.03**					

Table-IV: Strength Data of Dry and Moisture Conditioned Subsets
(Tensile Strength Ratio -TSR) (Adding 0.5% Cement)

Bitumen content (By weight of the mix) =	**5.625**
Aggregate Source:	**Bholaganj**
Portland Cement (% by aggregates weight)=	**1.0**

Sample ID				1	2	3	1	2	3
Particulars	Unit	Symbol	Equation	Dry Subset			Moisture Conditioned Subset		
Diameter	mm	D		100	99	98	98	99	99
Thickness	mm	T		64	68	67	68	69	67
Dry mass air	gm	A		1199.8	1199.5	1199.4	1202.8	1201.3	1200.5
Saturated surface dry mass	gm	B		1201.2	1202	1202.6	1203.5	1202.8	1202.3
Mass in water	gm	C		674.7	674.1	673.6	676	676.4	676.7
Vol of test specimen	cc	E	B-C	526.5	527.9	529	527.5	526.4	525.6
Bulk specific gravity	gm/cc	F	A/(B-C)	2.278	2.272	2.267	2.276	2.282	2.284
Max specific gravity (gmm)	gm/cc	G		2.463	2.463	2.463	2.463	2.463	2.463
Air void	%	H	(G-F) /G*100	7.51	7.75	7.95	7.59	7.34	7.26
Vol of air void	cc	I	H*E/100	39.54	40.93	42.09	40.03	38.63	38.15
Crushing load	KN	P		38.0	38.8	37.8			

Table-I: Initial Data on Dry Subset as Well as Moisture Conditioned Subset Samples (Adding 1.0% Cement)

Sample ID				1	2	3	1	2	3
Particulars	Unit	Symbol	Equation	Dry Subset			Moisture Conditioned Subset		
Saturation time	Min	D					15	15	15
Vacuum pressure in Hg	mm	T					445	435	420
Saturated surface dry mass	gm	B’					1222.4	1223.8	1222.5
Mass in water	gm	C’					691.7	694.3	693.6
Vol of test specimen	cc	E’	B’-C’				530.7	529.5	528.9
Vol of absorbed water	cc	J’	B’-A				21.6	22.5	22.0
Saturation	gm/cc	G	J’/I *100				53.95	58.24	57.66
Swell	%	H	[(E’-E)/E]* 100				0.60	0.58	0.62

Table-II: Data on Moisture – Conditioned Subset: Partial Saturation (Adding 1.0% Cement)

Sample ID				1	2	3	1	2	3
Particulars	Unit	Symbol	Equation	Dry Subset			Moisture Conditioned Subset		
Thickness	mm	T’					69.0	69.5	68
Saturated surface dry mass	gm	B”					1225.6	1226.5	1224.8
Mass in water	gm	C”					692.8	693.5	692.3
Vol of test specimen	cc	E”	B”-C”				532.8	533	532.5
Vol of absorbed water	cc	J”	B”-A				24.8	25.2	24.3
Saturation	gm/cc		J”/I *100				61.95	65.23	63.69
Swell	%		[(E”-E)/E]* 100				1.00	1.25	1.31
Crushing load	KN	P’					31.5	32.4	32.9

Table-III: Data on Moisture –Conditioned Subset: Test Specimen in 60° C Water for 24 Hours (Adding 1.0% Cement)

Sample ID				1	2	3	1	2	3
Particulars	Unit	Symbol	Equation	Dry Subset			Moisture Conditioned Subset		
Dry strength	Kpa	Sth	$2*P*10^6/(\pi * T * D)$	3.78	3.67	3.66			
Average dry strength	Kpa			3.70					
Wet strength	Kpa	Stm	$2*P*10^6/(\pi * T * D)$				3.01	3.02	3.16
Average wet strength	Kpa						3.06		
TSR%	%		100x(Stm/Sth)	**82.70**					

Table-IV: Strength Data of Dry and Moisture Conditioned Subsets
(Tensile Strength Ratio -TSR) (Adding 1.0% Cement)

Bitumen content (By weight of the mix) =	**5.625**
Aggregate Source:	**Bholaganj**
Portland Cement (% by aggregates weight)=	**1.5**

Sample ID				1	2	3	1	2	3
Particulars	Unit	Symbol	Equation	Dry Subset			Moisture Conditioned Subset		
Diameter	mm	D		99	99	100	99	98	101
Thickness	mm	T		63	65	62	61	62	62
Dry mass air	gm	A		1193.5	1196.6	1198.3	1190.6	1192.6	1191.5
Saturated surface dry mass	gm	B		1194.9	1198.3	1199.9	1192.3	1194.6	1192.9
Mass in water	gm	C		675.0	677.6	676.5	672.4	674.8	673.6
Vol of test specimen	cc	E	B-C	519.9	520.7	523.4	519.9	519.8	519.3
Bulk specific gravity	gm/cc	F	A/(B-C)	2.295	2.298	2.289	2.290	2.294	2.294
Max specific gravity (gmm)	gm/cc	G		2.463	2.463	2.463	2.463	2.463	2.463
Air void	%	H	(G-F) /G*100	6.82	6.69	7.06	7.02	6.86	6.86
Vol of air void	cc	I	H*E/100	35.45	34.83	36.95	36.49	35.65	35.62
Crushing load	KN	P		64.8	61.3	65.9			

Table-I: Initial Data on Dry Subset as Well as Moisture Conditioned Subset Samples (Adding 1.5% Cement)

Sample ID				1	2	3	1	2	3
Particulars	Unit	Symbol	Equation	Dry Subset			Moisture Conditioned Subset		
Saturation time	Min	D					15	15	15
Vacuum pressure in Hg	mm	T					475.0	445.5	435.6
Saturated surface dry mass	gm	B’					1210.7	1212.9	1212.6
Mass in water	gm	C’					688.2	689.8	689.8
Vol of test specimen	cc	E’	B’-C’				522.5	523.1	522.8
Vol of absorbed water	cc	J’	B’-A				20.1	20.3	21.1`
Saturation	gm/cc	G	J’/I *100				55.08	56.94	59.23
Swell	%	H	[(E’-E)/E]* 100				0.50	0.63	0.67

Table-II: Data on Moisture – Conditioned Subset: Partial Saturation (Adding 1.5% Cement)

Sample ID				1	2	3	1	2	3
Particulars	Unit	Symbol	Equation	Dry Subset			Moisture Conditioned Subset		
Thickness	mm	T’					62.0	62.5	63
Saturated surface dry mass	gm	B”					1213.9	1215.8	1215.7
Mass in water	gm	C”					689.8	690.6	691.8
Vol of test specimen	cc	E”	B”-C”				524.1	525.2	523.9
Vol of absorbed water	cc	J”	B”-A				23.3	23.2	24.2
Saturation	gm/cc		J”/I *100				63.85	65.07	67.93
Swell	%		[(E”-E)/E]* 100				0.80	1.03	0.88
Crushing load	KN	P’					51.0	50.6	51.4

Table-III: Data on Moisture –Conditioned Subset: Test Specimen in 60° C Water for 24 Hours (Adding 1.5% Cement)

Sample ID				1	2	3	1	2	3
Particulars	Unit	Symbol	Equation	Dry Subset			Moisture Conditioned Subset		
Dry strength	Kpa	Sth	$2*P*10^6/(\pi * T * D)$	6.61	6.06	6.77			
Average dry strength	Kpa			6.48					
Wet strength	Kpa	Stm	$2*P*10^6/(\pi * T * D)$				5.38	5.30	5.23
Average wet strength	Kpa						5.30		
TSR%	%		100x(Stm/Sth)	**81.79**					

Table-IV: Strength Data of Dry and Moisture Conditioned Subsets
(Tensile Strength Ratio -TSR) (Adding 1.5% Cement)

Bitumen content (By weight of the mix) =	**5.625**
Aggregate Source:	**Bholaganj**
Portland Cement (% by aggregates weight)=	**2.0**

Sample ID				1	2	3	1	2	3
Particulars	Unit	Symbol	Equation	Dry Subset			Moisture Conditioned Subset		
Diameter	mm	D		100	99	98	99	99	98
Thickness	mm	T		61	62	65	67	64	65
Dry mass air	gm	A		1200.1	1196.8	1198.9	1199.5	1201.5	1197.8
Saturated surface dry mass	gm	B		1202.3	1198.9	1200.5	1201.6	1201.5	1199.8
Mass in water	gm	C		680.0	678.7	678.9	680.9	681.8	679.3
Vol of test specimen	cc	E	B-C	522.3	520.2	521.6	520.7	521.2	520.5
Bulk specific gravity	gm/cc	F	A/(B-C)	2.297	2.300	2.298	2.303	2.305	2.301
Max specific gravity (gmm)	gm/cc	G		2.463	2.463	2.463	2.463	2.463	2.463
Air void	%	H	(G-F) /G*100	6.73	6.61	6.69	6.49	6.41	6.57
Vol of air void	cc	I	H*E/100	35.15	34.38	34.89	33.79	33.4	34.19
Crushing load	KN	P		53.2	55.2	54.0			

Table-I: Initial Data on Dry Subset as Well as Moisture Conditioned Subset Samples (Adding 2.0% Cement)

Sample ID				1	2	3	1	2	3
Particulars	Unit	Symbol	Equation	Dry Subset			Moisture Conditioned Subset		
Saturation time	Min	D					15	15	15
Vacuum pressure in Hg	mm	T					415	420	410
Saturated surface dry mass	gm	B’					1224.8	1225.3	1223.5
Mass in water	gm	C’					699.2	701.0	698.1
Vol of test specimen	cc	E’	B’-C’				525.6	524.3	525.4
Vol of absorbed water	cc	J’	B’-A				25.3	23.8	25.7
Saturation	gm/cc	G	J’/I *100				74.87	71.25	75.16
Swell	%	H	[(E’-E)/E]* 100				0.94	0.59	0.94

Table-II: Data on Moisture – Conditioned Subset: Partial Saturation (Adding 2.0% Cement)

Sample ID				1	2	3	1	2	3
Particulars	Unit	Symbol	Equation	Dry Subset			Moisture Conditioned Subset		
Thickness	mm	T’					67.5	65.0	66.5
Saturated surface dry mass	gm	B”					1226.5	1227.4	1225.6
Mass in water	gm	C”					699.4	700.7	700
Vol of test specimen	cc	E”	B”-C”				527.1	526.7	525.6
Vol of absorbed water	cc	J”	B”-A				27.0	25.9	27.8
Saturation	gm/cc		J”/I *100				79.90	77.54	81.31
Swell	%		[(E”-E)/E]* 100				1.22	1.05	0.97
Crushing load	KN	P’					43.0	42.5	43.5

Table-III: Data on Moisture –Conditioned Subset: Test Specimen in 60° C Water for 24 Hours (Adding 2.0% Cement)

Sample ID				1	2	3	1	2	3
Particulars	Unit	Symbol	Equation	Dry Subset			Moisture Conditioned Subset		
Dry strength	Kpa	Sth	$2*P*10^6/(\pi * T * D)$	5.55	5.73	5.40			
Average dry strength	Kpa			5.56					
Wet strength	Kpa	Stm	$2*P*10^6/(\pi * T * D)$				4.13	4.27	4.35
Average wet strength	Kpa						4.25		
TSR%	%		100x(Stm/Sth)	**76.44**					

Table-IV: Strength Data of Dry and Moisture Conditioned Subsets
(Tensile Strength Ratio -TSR) (Adding 2.0% Cement)

APPENDICES

Appendix (A)

Physical properties and sieve analysis of aggregates

Physical properties of used Aggregates

1. **Specific gravity and Absorption**

Where,

A = Weight of oven-dry sample in air, gram

B = Weight of saturated - surface -dry sample in air, gram

C = Weight of saturated sample in water, gram

- **Coarse Aggregate (Pakura Stone)**

  A= 3120.2 gram, B= 3133.33 gram, C= 2075.90 gram

  - Bulk dry S.G= A/(B-C) = 2.95
  - SSD S.G= B/(B-C) = 2.96
  - Apparent S.G= A/(A-C) = 2.98
  - Effective S.G= (Bulk dry+ Apparent)/2 = 2.97
  - Adsorption= {(B-A)/A}*100 = 0.42%

- **Fine Aggregate (Sylhet Sand)**

  A= 3120.2 gram, B= 3133.33 gram, C= 2075.90 gram

  - Bulk dry S.G= A/(B-C) = 2.95
  - SSD S.G= B/(B-C) = 2.96
  - Apparent S.G= A/(A-C) = 2.98
  - Effective S.G= (Bulk dry+ Apparent)/2 = 2.97
  - Adsorption= {(B-A)/A}*100 = 0.42%

- **Fine Aggregate (Sylhet Sand)**

  - Fineness Modulus (F.M.)=

    (Cumulative % retained on #4+#8+#16+#30+#50+#100)/100

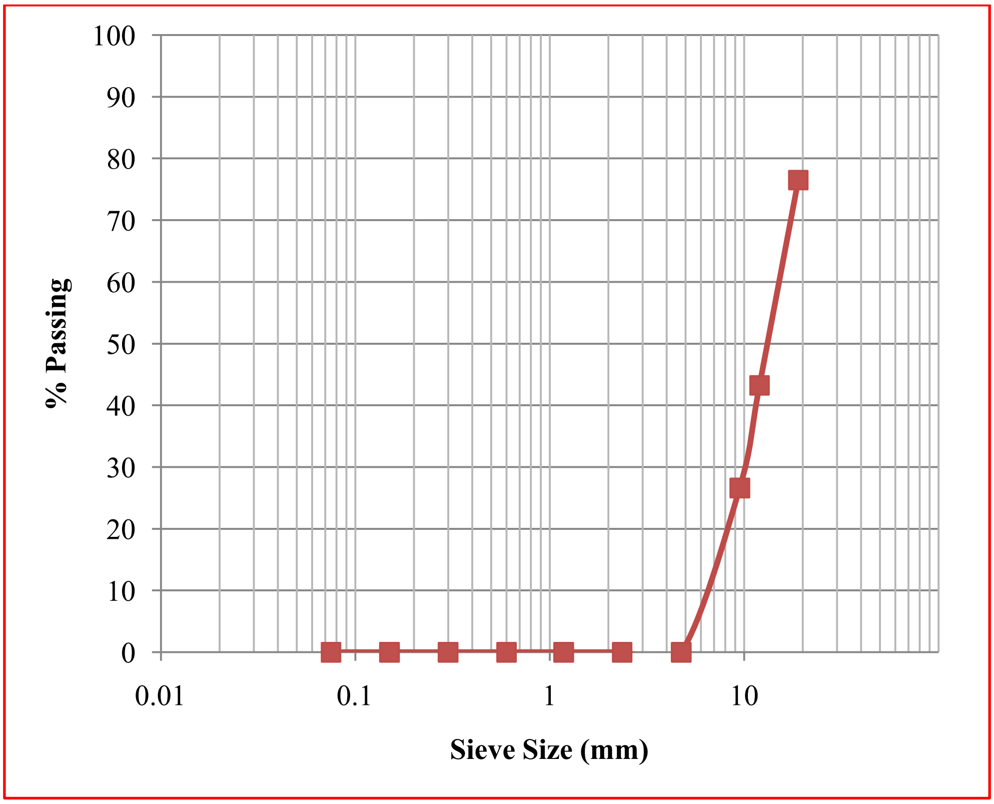
    = (3.27+8.28+28.70+60.40+83.12+93.85)/100

    = 277.57/100 = 2.78

Sieve analysis of used Aggregates

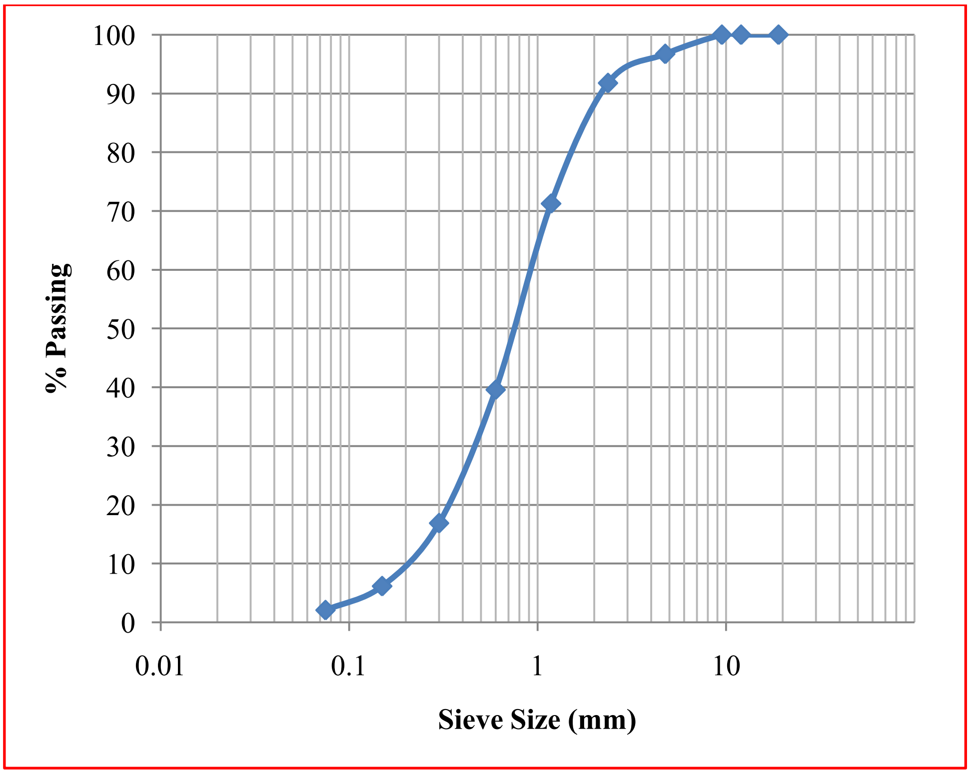
- **Coarse Aggregate (Pakur Stone)**

Sieve size (mm)	Sieve #	Individual retained (gm)	Cumulative retained (gm)	Cumulative retained (%)	Sample passing (%)
25	1"	20	20	0.35	99.65
19	3/4"	1325	1345	23.49	76.51
12	1/2"	1905	3250	56.76	43.24
9.5	3/8"	952	4202	73.38	26.62
4.75	#4	1520	5722	99.93	0.07
2.36	#8	0	5722	99.93	0.07
1.18	#16	0	5722	99.93	0.07
0.60	#30	0	5722	99.93	0.07
0.30	#50	0	5722	99.93	0.07
0.15	#100	0	5722	99.93	0.07
0.075	#200	0	5722	99.93	0.07
Pan		04	5726		
Total=			5726		



- **Fine aggregate (Sylhet Sand)**

Sieve size (mm)	Sieve #	Individual retained (gm)	Cumulative retained (gm)	Cumulative retained (%)	Sample passing (%)
25	1"	0	0	0.00	100
19	3/4"	0	0	0.00	100
12	1/2"	0	0	0.00	100
9.5	3/8"	0	0	0.00	100
4.75	#4	16.4	16.4	3.27	96.73
2.36	#8	24.8	41.2	8.22	91.78
1.18	#16	102.7	143.9	28.70	71.30
0.60	#30	158.9	302.8	60.40	39.60
0.30	#50	113.9	416.7	83.12	16.88
0.15	#100	53.8	470.5	93.85	6.15
0.075	#200	20.41	490.91	97.93	2.07
Pan			10.4		
Total=			501.31		



Appendix (B)

Aggregates Blending

Suggested percentages for binder course aggregates mix

Aggregate	Grain size (mm)											Suggested percent for final aggregate mix
	<0.075	0.075 /0.15	0.15/0.3	0.3/0.6	0.6/1.18	1.18/2.36	2.36/4.75	4.75/9.5	9.5/12.5	12.5/19	19/25	
Filler (Crown Cement)	**60**	**0**	**0**	**0**	**0**	**0**	**0**	**0**	**0**	**0**	**0**	**05**
Fine Aggregate (Sylhet Sand)	**0**	**42**	**42**	**60**	**108**	**120**	**168**	**0**	**0**	**0**	**0**	**40**
Coarse Aggregate (Pakura Stone)	**0**	**0**	**0**	**0**	**0**	**0**	**0**	**216**	**96**	**228**	**60**	**55**
Sum	**60**	**42**	**42**	**60**	**108**	**120**	**168**	**216**	**96**	**228**	**60**	**100**
% passing	**05**	**8.5**	**12**	**17**	**26**	**36**	**50**	**68**	**76**	**95**	**100**	**----**
Sieve size (mm)	**0.075**	**0.15**	**0.3**	**0.6**	**1.18**	**2.00**	**4.75**	**9.50**	**12.50**	**19.00**	**25.00**	ASTM Specifications D 3515 - 01
Binder 0/19 (min)	**2**	**3**	**5**	**6**	**15**	**23**	**35**	**56**	**67**	**90**	**100**	
(Max)	**8**	**14**	**19**	**22**	**37**	**49**	**65**	**80**	**85**	**100**	**100**	

Appendix (C)

Binder Course Job Mix

Used Equations to calculate the mechanical properties of asphalt mix

$V_a = \{(\rho_{bit} - \rho_A) / \rho_{bit}\} * 100$

$V_b = m_b * (\rho_A / d_{25}) \%$

VMA (%) = $V_a + V_b$

VFA (%) = $(V_b/VMA)*100$

Where,

V_b: Percent bitumen volume.

V_a: Air voids contents in total mix.

m_b: Percent of Bitumen.

ρ_A : Density of compacted mix (g/cm^3).

d_{25} : Density of Bitumen at 25^oC.

ρ_{bit} : Maximum Theoretical density.

VMA: Voids in the Mineral Aggregates.

VFA: Voids Filled with Asphalt

Determination of the maximum theoretical density for the asphalt mix

Calculation of the theoretical asphalt mix density it can be done by using the Pycnometer or by calculation using specific gravities for all aggregates. In this research, the calculation method was used to find out the theoretical density of the asphalt mix.

Calculation method:

$$\boxed{\rho_{\min} = \frac{100}{\frac{m_1}{\rho_{\min 1}} + \frac{m_2}{\rho_{\min 2}} + \ldots\ldots + \frac{m_n}{\rho_{\min n}}}} \qquad \boxed{\rho_{bit} = \frac{100}{\frac{m_b}{d_{25}} + \frac{100 - m_b}{\rho_{\min}}}}$$

Where,

ρ_{bit} : Max. Theoretical Density.

m_b: % of bitumen by total mix.

d_{25}: Density of bitumen.

m_1: The percentage of aggregate type (1) in the aggregates blend.

$\rho_{min\ 1}$: Density of aggregate type (1).

Aggregate type	Percentage in aggregate mix m %	Aggregate density ρ_{min}(g/cm^3)	m /ρ_{min}
Pakura Stone	55	2.95	5.36
Sylhet Sand	40	2.65	6.63
Mineral Filler/ Portland Cement	5	1.26	25.20
		Sum=	37.19

- **Effective Specific gravity for aggregate mix ρ_{min} = 100 / 37.19 = 2.69 (g/cm^3)**

Loading Conditions	Bitumen percentage m_b %	Bitumen density d_{25} (g/cm^3)	Aggregate blend density ρ_{min} (g/cm^3)	Maximum Theoretical Density ρ_{bit} (g/cm^3)
Heavy Load (75 Blows)	4.0	1.037	2.69	2.53
	4.5	1.037	2.69	2.51
	5.0	1.037	2.69	2.49
	5.5	1.037	2.69	2.47
	6.0	1.037	2.69	2.45

Bitumen content (By weight of the mix) = **4.00**

Number of blows on each side : 75 blows (for Heavy Loads)

Mixing temperature : 150 ºC

Bitumen grade: 70/80 and Density of Bitumen at 25 ºC= **1.037**

CA:FA:MF= **55:40:05**

Test	Samples Results			Average
	H-4.0-1	H-4.0-2	H-4.0-3	
Weight of sample in air (gm)	1227.9	1228.6	1229.5	1228.67
Weight in water (gm)	737.5	736.9	737.2	737.20
SSD weight (gm)	1231	1229.6	1231.4	1230.67
Bulk volume (cm^3)	523.37	535.19	531.54	530.03
Density of compacted mix ρ_A (g/cm^3)	2.35	2.30	2.31	2.32
Max. theoretical density ρ_{bit} (g/cm^3)	2.53	2.53	2.53	2.53
Average sample hight (mm)	66.01	67.5	67.04	66.85
Stability read value	449	452	447	449.33
Stability (KN)	11.90	11.98	11.84	11.91
Stability correction factor	0.96	0.96	0.96	0.96
Corrected stability (KN)	11.42	11.50	11.37	11.43
Flow (0.01 in)	8.26	8.24	8.28	8.26
Stiffness (KN/0.01 in)	1.38	1.40	1.37	1.38
Air voids content in total mix V_a (%)	7.27	9.26	8.57	8.37
Percent bitumen volume V_b (%)	9.05	8.85	8.92	8.94
Voids in Mineral Aggregate (VMA) (%)	16.32	18.12	17.50	17.31
Voids Fill with Asphalt (VFA) (%)	55.46	48.87	51.00	51.78

Bitumen content (By weight of the mix) = **4.50**
Number of blows on each side : 75 blows (for Heavy Loads)
Mixing temperature : 150 °C
Bitumen grade: 70/80 and Density of Bitumen at 25 °C= **1.037**
CA:FA:MF= **55:40:05**

Test	Samples Results			Average
	H-4.5-1	H-4.5-2	H-4.5-3	
Weight of sample in air (gm)	1235.30	1236.10	1235.90	1235.77
Weight in water (gm)	743.50	743.80	742.90	743.40
SSD weight (gm)	1238.70	1239.10	1237.80	1238.53
Bulk volume (cm^3)	520.33	528.13	527.27	528.39
Density of compacted mix ρ_A (g/cm^3)	2.37	2.34	2.34	2.35
Max. theoretical density ρ_{bit} (g/cm^3)	2.51	2.51	2.51	2.51
Average sample hight (mm)	66.02	67.01	66.90	66.64
Stability read value	495.00	497.00	493.00	495.00
Stability (KN)	13.12	13.17	13.07	13.12
Stability correction factor	0.96	0.96	0.96	0.96
Corrected stability (KN)	12.59	12.64	12.54	12.59
Flow (0.01 in)	9.53	9.55	9.51	9.53
Stiffness (KN/0.01 in)	1.32	1.32	1.32	1.32
Air voids content in total mix V_a (%)	5.42	6.75	6.61	6.26
Percent bitumen volume V_b (%)	10.30	10.16	10.17	10.21
Voids in Mineral Aggregate (VMA) (%)	15.72	16.91	16.79	16.47
Voids Fill with Asphalt (VFA) (%)	65.54	60.06	60.59	62.07

Bitumen content (By weight of the mix) = **5.00**
Number of blows on each side : 75 blows (for Heavy Loads)
Mixing temperature : 150 ºC
Bitumen grade: 70/80 and Density of Bitumen at 25 ºC = **1.037**
CA:FA:MF= **55:40:05**

Test	Samples Results			Average
	H-5.0-1	H-5.0-2	H-5.0-3	
Weight of sample in air (gm)	1237.30	1238.50	1240.10	1238.63
Weight in water (gm)	749.50	749.50	749.50	749.50
SSD weight (gm)	1238.00	1238.00	1238.00	1238.00
Bulk volume (cm^3)	525.80	526.82	523.93	525.52
Density of compacted mix ρ_A (g/cm^3)	2.35	2.35	2.37	2.36
Max. theoretical density ρ_{bit} (g/cm^3)	2.49	2.49	2.49	2.49
Average sample hight (mm)	67.25	67.38	67.01	67.21
Stability read value	586.00	585.00	588.00	586.33
Stability (KN)	15.53	15.51	15.59	15.54
Stability correction factor	0.96	0.96	0.96	0.96
Corrected stability (KN)	14.91	14.89	14.96	14.92
Flow (0.01 in)	10.03	10.05	10.01	10.03
Stiffness (KN/0.01 in)	1.49	1.48	1.49	1.49
Air voids content in total mix V_a (%)	5.50	5.59	4.94	5.34
Percent bitumen volume V_b (%)	11.35	11.34	11.41	11.36
Voids in Mineral Aggregate (VMA) (%)	16.84	16.92	16.35	16.71
Voids Fill with Asphalt (VFA) (%)	67.37	66.99	69.78	68.05

Bitumen content (By weight of the mix) = **5.50**
Number of blows on each side : 75 blows (for Heavy Loads)
Mixing temperature : 150 ºC
Bitumen grade: 70/80 and Density of Bitumen at 25 ºC= **1.037**
CA:FA:MF= **55:40:05**

Test	Samples Results			Average
	H-5.5-1	H-5.5-2	H-5.5-3	
Weight of sample in air (gm)	1250.00	1245.00	1239.50	1244.83
Weight in water (gm)	756.80	760.20	749.10	755.37
SSD weight (gm)	1251.40	1255.00	1259.60	1255.33
Bulk volume (cm^3)	527.76	531.20	531.75	530.23
Density of compacted mix ρ_A (g/cm^3)	2.37	2.34	2.33	2.35
Max. theoretical density ρ_{bit} (g/cm^3)	2.47	2.47	2.47	2.47
Average sample hight (mm)	67.50	67.94	68.01	67.82
Stability read value	465.00	469.00	467.00	467.00
Stability (KN)	12.32	12.43	12.38	12.38
Stability correction factor	0.96	0.96	0.96	0.96
Corrected stability (KN)	11.83	11.93	11.88	11.88
Flow (0.01 in)	11.05	11.07	11.03	11.05
Stiffness (KN/0.01 in)	1.07	1.08	1.08	1.08
Air voids content in total mix V_a (%)	4.11	5.11	5.63	4.95
Percent bitumen volume V_b (%)	12.56	12.43	12.36	12.45
Voids in Mineral Aggregate (VMA) (%)	16.67	17.54	17.99	17.40
Voids Fill with Asphalt (VFA) (%)	75.35	70.86	68.72	71.65

Bitumen content (By weight of the mix) = **6.00**
Number of blows on each side : 75 blows (for Heavy Loads)
Mixing temperature : 150 ºC
Bitumen grade: 70/80 and Density of Bitumen at 25 ºC= **1.037**
CA:FA:MF= **55:40:05**

Test	Samples Results			Average
	H-6.0-1	H-6.0-2	H-6.0-3	
Weight of sample in air (gm)	1253.70	1243.90	1249.80	1249.13
Weight in water (gm)	758.20	756.10	759.80	758.03
SSD weight (gm)	1254.20	1254.20	1254.20	1254.20
Bulk volume (cm^3)	531.75	531.28	529.71	530.91
Density of compacted mix ρ_A (g/cm^3)	2.36	2.34	2.36	2.35
Max. theoretical density ρ_{bit} (g/cm^3)	2.45	2.45	2.45	2.45
Average sample hight (mm)	68.01	67.95	67.75	67.90
Stability read value	410.00	415.00	413.00	412.67
Stability (KN)	10.86	11.00	10.94	10.93
Stability correction factor	0.96	0.96	0.96	0.96
Corrected stability (KN)	10.43	10.56	10.50	10.50
Flow (0.01 in)	12.58	12.53	12.59	12.57
Stiffness (KN/0.01 in)	0.83	0.84	0.83	0.84
Air voids content in total mix V_a (%)	3.77	4.43	3.70	3.97
Percent bitumen volume V_b (%)	13.64	13.55	13.65	13.61
Voids in Mineral Aggregate (VMA) (%)	17.41	17.98	17.35	17.58
Voids Fill with Asphalt (VFA) (%)	78.36	75.34	78.68	77.46

Appendix (D)
Cement mix tests results

Determination of the maximum theoretical density for the cement mix

Pycnometer method

(W_{P+W}) = Weight of Pycnometer filled with water

(W_S) = Weight of the asphalt sample

(W_{s+P+W}) = Weight of Pycnometer filled with water and the crushed sample

$$\rho_{bit} = \frac{W_s}{W_s - (W_{s+p+w} - W_{p+w})}$$

Loading Conditions	Cement (%)	W_{P+W} (g)	W_S (g)	W_{S+P+W} (g)	ρ_{bit} (g/cm^3)
Heavy Load (75 Blows)	0.50	1783.34	450.73	2050.75	2.46
	1.00	1783.34	475.63	2064.07	2.44
	1.50	1783.34	475.83	2063.84	2.44
	2.00	1783.34	452.40	2050.65	2.44
	2.50	1783.34	478.00	2065.33	2.43

Bitumen content (By weight of the mix) = 5.10
Number of blows on each side : 75 blows (for Heavy Loads)
Mixing temperature : 150 ºC
Bitumen grade: 70/80 and Density of Bitumen at 25 ºC= 1.037
CA:FA:MF= 55:40:05
Portland Cement (% by aggregates weight)= 0.50

Test	Samples Results			Average
	HC-0.5-1	HC-0.5-2	HC-0.5-3	
Weight of sample in air (gm)	1247.20	1245.50	1246.80	1246.50
Weight in water (gm)	752.00	751.90	752.00	751.97
SSD weight (gm)	1249.30	1247.30	1248.50	1248.37
Bulk volume (cm^3)	520.33	528.13	527.27	528.39
Density of compacted mix ρ_A (g/cm^3)	2.40	2.36	2.36	2.37
Max. theoretical density ρ_{bit} (g/cm^3)	2.46	2.46	2.46	2.46
Average sample hight (mm)	66.02	67.01	66.90	66.64
Stability read value	505.00	488.00	495.00	496.00
Stability (KN)	13.38	12.93	13.12	13.15
Stability correction factor	0.96	0.96	0.96	0.96
Corrected stability (KN)	12.85	12.42	12.59	12.62
Flow (0.01 in)	8.75	8.44	9.21	8.80
Stiffness (KN/0.01 in)	1.08	1.04	1.06	1.06
Air voids content in total mix V_a (%)	2.56	4.13	3.88	3.52
Percent bitumen volume V_b (%)	11.79	12.10	12.10	11.99
Voids in Mineral Aggregate (VMA) (%)	14.35	16.23	15.97	15.52
Voids Fill with Asphalt (VFA) (%)	82.14	74.53	75.73	77.47

Bitumen content (By weight of the mix) =	**5.10**
Number of blows on each side : 75 blows (for Heavy Loads)	
Mixing temperature : 150 ºC	
Bitumen grade: 70/80 and Density of Bitumen at 25 ºC=	**1.037**
CA:FA:MF= 55:40:05	
Portland Cement (% by aggregates weight)=	**1.00**

Test	Samples Results			Average
	HC-1.0-1	HC-1.0-2	HC-1.0-3	
Weight of sample in air (gm)	1249.20	1248.20	1250.10	1249.17
Weight in water (gm)	752.50	753.40	752.00	752.63
SSD weight (gm)	1251.50	1249.80	1252.10	1251.13
Bulk volume (cm^3)	520.33	528.13	527.27	528.39
Density of compacted mix ρ_A (g/cm^3)	2.40	2.36	2.37	2.38
Max. theoretical density ρ_{bit} (g/cm^3)	2.44	2.44	2.44	2.44
Average sample hight (mm)	66.02	67.01	66.90	66.64
Stability read value	439.00	389.00	401.25	409.75
Stability (KN)	11.63	10.31	10.63	10.86
Stability correction factor	0.96	0.96	0.96	0.96
Corrected stability (KN)	11.17	9.89	10.21	10.42
Flow (0.01 in)	6.51	5.95	6.50	6.32
Stiffness (KN/0.01 in)	1.11	0.98	1.02	1.04
Air voids content in total mix V_a (%)	1.61	3.14	2.83	2.53
Percent bitumen volume V_b (%)	11.81	12.00	12.00	11.94
Voids in Mineral Aggregate (VMA) (%)	13.41	15.14	14.83	14.46
Voids Fill with Asphalt (VFA) (%)	88.02	79.27	80.91	82.73

Bitumen content (By weight of the mix) = **5.10**
Number of blows on each side : 75 blows (for Heavy Loads)
Mixing temperature : 150 ºC
Bitumen grade: 70/80 and Density of Bitumen at 25 ºC= **1.037**
CA:FA:MF= **55:40:05**
Portland Cement (% by aggregates weight)= **1.50**

Test	Samples Results			Average
	HC-1.5-1	HC-1.5-2	HC-1.5-3	
Weight of sample in air (gm)	1242.50	1244.90	1245.10	1244.17
Weight in water (gm)	749.80	749.60	750.10	749.83
SSD weight (gm)	1244.20	1246.70	1247.60	1246.17
Bulk volume (cm^3)	520.33	528.13	527.27	528.39
Density of compacted mix ρ_A (g/cm^3)	2.39	2.36	2.36	2.37
Max. theoretical density ρ_{bit} (g/cm^3)	2.44	2.44	2.44	2.44
Average sample hight (mm)	66.02	67.01	66.90	66.64
Stability read value	438.00	355.00	405.70	399.57
Stability (KN)	11.61	9.40	10.75	10.59
Stability correction factor	0.96	0.96	0.96	0.96
Corrected stability (KN)	11.14	9.03	10.32	10.16
Flow (0.01 in)	4.95	5.10	3.95	4.67
Stiffness (KN/0.01 in)	0.98	0.79	0.91	0.90
Air voids content in total mix V_a (%)	2.14	3.39	3.22	2.92
Percent bitumen volume V_b (%)	11.74	12.00	12.00	11.91
Voids in Mineral Aggregate (VMA) (%)	13.88	15.39	15.22	14.83
Voids Fill with Asphalt (VFA) (%)	84.61	77.95	78.84	80.47

Bitumen content (By weight of the mix) = **5.10**
Number of blows on each side : 75 blows (for Heavy Loads)
Mixing temperature : 150 ºC
Bitumen grade: 70/80 and Density of Bitumen at 25 ºC= **1.037**
CA:FA:MF= **55:40:05**
Portland Cement (% by aggregates weight)= **2.00**

Test	**Samples Results**			**Average**
	HC-2.0-1	**HC-2.0-2**	**HC-2.0-3**	
Weight of sample in air (gm)	1266.00	1251.60	1255.40	1257.67
Weight in water (gm)	764.80	753.50	755.20	757.83
SSD weight (gm)	1268.00	1252.90	1258.10	1259.67
Bulk volume (cm^3)	520.33	528.13	527.27	528.39
Density of compacted mix ρ_A (g/cm^3)	2.43	2.37	2.38	2.39
Max. theoretical density ρ_{bit} (g/cm^3)	2.44	2.44	2.44	2.44
Average sample hight (mm)	66.02	67.01	66.90	66.64
Stability read value	475.00	440.00	449.00	454.67
Stability (KN)	12.59	11.66	11.90	12.05
Stability correction factor	0.96	0.96	0.96	0.96
Corrected stability (KN)	12.08	11.19	11.42	11.57
Flow (0.01 in)	3.85	3.15	3.45	3.48
Stiffness (KN/0.01 in)	1.19	1.09	1.12	1.13
Air voids content in total mix V_a (%)	0.28	2.88	2.42	1.86
Percent bitumen volume V_b (%)	11.97	12.00	12.00	11.99
Voids in Mineral Aggregate (VMA) (%)	12.25	14.88	14.42	13.85
Voids Fill with Asphalt (VFA) (%)	97.68	80.67	83.22	87.19

Bitumen content (By weight of the mix) = **5.10**
Number of blows on each side : 75 blows (for Heavy Loads)
Mixing temperature : 150 ºC
Bitumen grade: 70/80 and Density of Bitumen at 25 ºC= **1.037**
CA:FA:MF= **55:40:05**
Portland Cement (% by aggregates weight)= **2.50**

Test	**Samples Results**			**Average**
	HC-2.5-1	**HC-2.5-2**	**HC-2.5-3**	
Weight of sample in air (gm)	1248.40	1233.80	1235.40	1239.20
Weight in water (gm)	755.10	743.70	745.20	748.00
SSD weight (gm)	1249.50	1234.80	1236.40	1240.23
Bulk volume (cm^3)	520.33	528.13	527.27	528.39
Density of compacted mix ρ_A (g/cm^3)	2.40	2.34	2.34	2.36
Max. theoretical density ρ_{bit} (g/cm^3)	2.43	2.43	2.43	2.43
Average sample hight (mm)	66.02	67.01	66.90	66.64
Stability read value	525.00	422.00	486.00	477.67
Stability (KN)	13.91	11.18	12.88	12.66
Stability correction factor	0.96	0.96	0.96	0.96
Corrected stability (KN)	13.36	10.73	12.36	12.15
Flow (0.01 in)	3.95	2.67	3.21	2.94
Stiffness (KN/0.01 in)	1.26	1.01	1.17	1.14
Air voids content in total mix V_a (%)	1.27	3.86	3.58	2.90
Percent bitumen volume V_b (%)	11.80	11.95	11.95	11.90
Voids in Mineral Aggregate (VMA) (%)	13.07	15.81	15.53	14.80
Voids Fill with Asphalt (VFA) (%)	90.31	75.58	76.95	80.95

Appendix (F)

Comparative Cost Analysis of Road Construction

Unit Cost of Flexible Pavement

According to the Road Note-29, for a heavily trafficked highway taking over 10,000 AADT on a good soil subgrade (CBR 3%) condition, the various layer thickness of 2-lane flexible pavement for 10 years design period are:

- Bituminous wearing course = 50 mm
- Bituminous binding course = 100 mm
- Aggregate base course = 300 mm
- Aggregate sub-base = 200 mm
- **Total = 650 mm**

The cost of per km cost of 2-lane pavement having **Bitumen Grade 60/70** for 10 years design life **without using Cement in Wearing Course** is given:

Item Code in RHD schedule of rates, 2018	Name of the Item	Thickness (m)	Length (m)	Width (m)	Quantity (cum)	Unit	Rate in RHD schedule of rates, 2018 (in BDT)	Amount (in Lac)
03/02/01 (b)	Sub-Base (with Paver) (200 mm thick)	0.20	1000.00	7.30	1460.00	CUM	5588.00	81.58
03/03/01 (b)	Aggregate Base Type-I (with Paver) (300 mm thick)	0.30	1000.00	7.30	2190.00	CUM	9020.00	197.54
03/10/02 (b)	Dense Bituminous Surfacing- Wearing Course (Plant Method) (Bitumen Grade 60/70)	0.15	1000.00	7.30	1095.00	CUM	24325.00	266.36
03/06/1a	Bituminous Prime Coat (Plant Placed)	----	1000.00	7.30	7300.00	Sq.Meter	114.00	8.32
							Total Cost (per Km)=	553.80

The cost of per km cost of 2-lane pavement having **Bitumen Grade 80/100** for 10 years design life **without using Cement in Wearing Course** is given:

Item Code in RHD schedule of rates, 2018	Name of the Item	Thickness (m)	Length (m)	Width (m)	Quantity (cum)	Unit	Rate in RHD schedule of rates, 2018 (in BDT)	Amount (in Lac)
03/02/01 (b)	Sub-Base (with Paver) (200 mm thick)	0.20	1000.00	7.30	1460.00	CUM	5588.00	81.58
03/03/01 (b)	Aggregate Base Type-I (with Paver) (300 mm thick)	0.30	1000.00	7.30	2190.00	CUM	9020.00	197.54
03/10/02 (a)	Dense Bituminous Surfacing- Wearing Course (Plant Method) (Bitumen Grade 80/100)	0.15	1000.00	7.30	1095.00	CUM	23972.99	262.50
03/06/1a	Bituminous Prime Coat (Plant Placed)	----	1000.00	7.30	7300.00	Sq.Meter	114.00	8.32
							Total Cost (per Km)=	549.95

The cost of per km cost of 2-lane pavement having **Bitumen Grade 60/70** for 10 years design life **using Cement (0.50%) in Wearing Course** is given:

Item Code in RHD schedule of rates, 2018	Name of the Item	Thickness (m)	Length (m)	Width (m)	Quantity (cum)	Unit	Rate in RHD schedule of rates, 2018 (in BDT)	Amount (in Lac)
03/02/01 (b)	Sub-Base (with Paver) (200 mm thick)	0.20	1000.00	7.30	1460.00	CUM	5588.00	81.58
03/03/01 (b)	Aggregate Base Type-I (with Paver) (300 mm thick)	0.30	1000.00	7.30	2190.00	CUM	9020.00	197.54
03/10/02 (a)	Dense Bituminous Surfacing- Wearing Course (Plant Method) (Bitumen Grade 60/70)	0.15	1000.00	7.30	1095.00	CUM	24325.00	266.36
-----	Cement (OPC) (0.50% of the Total Wearing course)	---	---	---	17.08	Ton	9000.00	1.54
03/06/1a	Bituminous Prime Coat (Plant Placed)	----	1000.00	7.30	7300.00	Sq.Meter	114.00	8.32
							Total Cost (per Km)=	555.34

The cost of per km cost of 2-lane pavement having **Bitumen Grade 60/70** for 10 years design life **using Cement (2.00%) in Wearing Course** is given:

Item Code in RHD schedule of rates, 2018	Name of the Item	Thickness (m)	Length (m)	Width (m)	Quantity (cum)	Unit	Rate in RHD schedule of rates, 2018 (in BDT)	Amount (in Lac)
03/02/01 (b)	Sub-Base (with Paver) (200 mm thick)	0.20	1000.00	7.30	1460.00	CUM	5588.00	81.58
03/03/01 (b)	Aggregate Base Type-I (with Paver) (300 mm thick)	0.30	1000.00	7.30	2190.00	CUM	9020.00	197.54
03/10/02 (a)	Dense Bituminous Surfacing- Wearing Course (Plant Method) (Bitumen Grade 60/70)	0.15	1000.00	7.30	1095.00	CUM	24325.00	266.36
-----	Cement (OPC) (2.00% of the Total Wearing course)	---	---	---	68.33	Ton	9000.00	6.15
03/06/1a	Bituminous Prime Coat (Plant Placed)	----	1000.00	7.30	7300.00	Sq.Meter	114.00	8.32
							Total Cost (per Km)=	559.95

The cost of per km cost of 2-lane pavement having **Bitumen Grade 80/100** for 10 years design life **using Cement (0.50%) in Wearing Course** is given:

Item Code in RHD schedule of rates, 2018	Name of the Item	Thickness (m)	Length (m)	Width (m)	Quantity (cum)	Unit	Rate in RHD schedule of rates, 2018 (in BDT)	Amount (in Lac)
03/02/01 (b)	Sub-Base (with Paver) (200 mm thick)	0.20	1000.00	7.30	1460.00	CUM	5588.00	81.58
03/03/01 (b)	Aggregate Base Type-I (with Paver) (300 mm thick)	0.30	1000.00	7.30	2190.00	CUM	9020.00	197.54
03/10/02 (a)	Dense Bituminous Surfacing- Wearing Course (Plant Method) (Bitumen Grade 80/100)	0.15	1000.00	7.30	1095.00	CUM	23972.99	262.50
-----	Cement (OPC) (0.50% of the Total Wearing course)	---	---	---	17.08	Ton	9000.00	1.54
03/06/1a	Bituminous Prime Coat (Plant Placed)	----	1000.00	7.30	7300.00	Sq.Meter	114.00	8.32
							Total Cost (per Km)=	551.49

The cost of per km cost of 2-lane pavement having **Bitumen Grade 80/100** for 10 years design life **using Cement (2.00%) in Wearing Course** is given:

Item Code in RHD schedule of rates, 2018	Name of the Item	Thickness (m)	Length (m)	Width (m)	Quantity (cum)	Unit	Rate in RHD schedule of rates, 2018 (in BDT)	Amount (in Lac)
03/02/01 (b)	Sub-Base (with Paver) (200 mm thick)	0.20	1000.00	7.30	1460.00	CUM	5588.00	81.58
03/03/01 (b)	Aggregate Base Type-I (with Paver) (300 mm thick)	0.30	1000.00	7.30	2190.00	CUM	9020.00	197.54
03/10/02 (a)	Dense Bituminous Surfacing- Wearing Course (Plant Method) (Bitumen Grade 80/100)	0.15	1000.00	7.30	1095.00	CUM	23972.99	262.50
-----	Cement (OPC) (2.00% of the Total Wearing course)	---	---	---	68.33	Ton	9000.00	6.15
03/06/1a	Bituminous Prime Coat (Plant Placed)	----	1000.00	7.30	7300.00	Sq.Meter	114.00	8.32
							Total Cost (per Km)=	556.10

Treatment Activity		Restoring or Improving Pavement Surface in terms of:						Expected Service Life
		Preventing Water Infiltration	Localized Severe Distress	Bleeding, Raveling, or Poor Skid Resistance	Ride Quality	Environmental Deterioration	Structural Capacity & Traffic	
Minor Rehabilitation/ Preservation*	Cold mix with sealing course*	**0**		♦		**0**		(5-10)
	Distortion corrections		**0**		♦	**0**		(5-10)
	Drainage improvements	•	**0**			♦		(7-10)
	Frost treatments		♦		•	•		(3-5)
	Roadside slopes and erosion control	**0**				♦		(3-7)
Major Rehabilitation	Full depth removal & resurfacing			•	♦	**0**	•	(8-12)
	Full depth reclamation / pulverization			**0**	•	•	♦	(12-15+)
	Pulverization with expanded asphalt stabilization			**0**	•	•	♦	(12-15+)
	White topping			**0**	**0**	**0**	♦	(5-10)
	Unbonded Concrete Overlays			**0**	**0**	**0**	♦	(25+)

Recommended Asphalt Rehabilitation Frequency

Treatment Activity		Restoring or Improving Pavement Surface in terms of:						Expected Service Life
		Preventing Water Infiltration	Localized Severe Distress	Bleeding, Raveling, or Poor Skid Resistance	Ride Quality	Environmental Deterioration	Structural Capacity & Traffic	
Routine Maintenance	Pothole Repair	•	•		•			< 1
	Roadside maintenance					•		(1-5)
	Drainage maintenance	•	0			•		(2-5)
	Spray patching	•		0	0			(2-5)
	Localized distortion repair		•		•	0		(2-5)
Minor Rehabilitation / Preservation*	Rout & crack sealing*	•						(1-5)
	Hot mix patching*	•		•	0			(5-10)
	Surface sealing* (sealcoat, slurry seal, micro-surfacing, chip seal / surface treatment)	•		•	•	0		(3-7)
	Texturization* (micro-milling, shot blasting, sand blasting)			•	0	0	0	(1-6)
	Asphalt strip repair* / full depth patching	0	•		0		•	(5-10)
	Hot mix resurfacing*	•		•	0	0	•	(5-12)
	Partial depth removal (milling) & resurfacing*	0	0	•	•	•	•	(8-12)
	In-place recycling* (H1R, CIR, CIREAM)			•	•	0	•	(7-15+)

Legend:
* **Pavement Preservation Treatments**
• **Primary Application**
• **Commonly Used**
0 **May be considered**

Recommended Asphalt Rehabilitation Frequency

Recommended Treatment	Rating	Pavement Condition	Maintenance cost of Flexible pavement (per Km)		Initial cost Flexible pavement (per Km) using 80/100 Bitumen		Total cost for Flexible pavement (per Km)	
			without cement	with cement (0.50%)	without cement	with cement (0.50%)	without cement	with cement (0.50%)
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
Reconstruct or recycle within 03 years	0-20	Pavement is in poor to very poor condition with extensive severe cracking, alligator and channeling. Ride ability is poor and the surface is very rough and uneven.	**2035.51** (Full Recon. 195mm)	**1915.98** (Partial Recon. 195mm)	549.95	551.49	2585.46	2467.47
Reconstruct or recycle within 02 years	20-30	Pavement is in poor condition with moderate alligator and extensive severe cracking and channeling. Ride ability is poor and the surface is very rough and uneven.	**1915.98** (Full Recon. 180mm)	**1660.02** (Partial Recon. 195mm)			2465.93	2211.51
Overlay, recycleor reconstruct within 3-4 years	30-40	Pavement is in poor to fair condition with frequent moderate cracking and channeling. And intermittent moderate alligator. Ride ability is poor to fair and surface is moderately rough and uneven.	**1363.20** (Overlay 80mm+ DBST+ Routine)	**910.69** (Overlay 40mm+ DBST+ Routine)			1913.15	1462.18

Recommended Treatment	Rating	Pavement Condition	Maintenance cost of Flexible pavement (per Km)		Initial cost Flexible pavement (per Km) using 80/100 Bitumen		Total cost for Flexible pavement (per Km)	
			without cement	**with cement (0.50%)**	**without cement**	**with cement (0.50%)**	**without cement**	**with cement (0.50%)**
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
Reconstruct in 4 -5 years or resurface within 2 years with extensive leveling	40-50	Pavement is in poor to fair condition with frequent moderate cracking and channeling. Ride ability is poor to fair and surface is moderately rough and uneven.	**910.69** (Overlay 60mm)	**568.23** (Overlay 50mm)	549.95	551.49	1460.64	1119.72
Resurface within 3 years.	50-65	Pavement is in fair condition with intermittent moderate and frequent slight cracking, and with intermittent slight or moderate alligator and channeling. Ride ability is fair and surface is slightly rough and uneven.	**452.51** (DBST*2+Routine)	**342.46** (DBST+ Routine)			1002.46	893.95
Resurface in 3 -5 years	65-80	Pavement is in fairly good condition with frequent slight cracking, slight or very slight channeling and a few alias of s light alligator. Ride ability is fairly good with intermittent rough and uneven sections.	**287.44** (DBST*3)	**244.73** (DBST*2)			1002.46	893.95

Recommended Treatment	Rating	Pavement Condition	Maintenance cost of Flexible pavement (per Km)		Initial cost Flexible pavement (per Km) using 80/100 Bitumen		Total cost for Flexible pavement (per Km)	
			without cement	with cement (0.50%)	without cement	with cement (0.50%)	without cement	with cement (0.50%)
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
Normal maintenance only.	80-100	Pavement is in good condition with frequent very slight or slight cracking. Rideability is good with a few slightly rough and uneven sections.	**36.06** (DBST)	**0.80** (Routine)	549.95	551.49	586.81	552.29

Asphalt Rehabilitation Frequency wise cost comparison

Appendix (G)
Photographs of Laboratory Work

Figure P.1: Aggregates source

Figure P.2: Used Portland cement source

Figure P.3: Used Aggregates

Figure P.4: Marshal Samples

Figure P.5: Marshal Samples weighting in water

Figure P.6: Cement mixed Marshal Samples **Figure P.7:**Water bath for Marshal Samples

Figure P.8: Testing Marshal Samples for stability and flow