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Application of Waste Materials in Asphalt Mixtures

By:
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March 2018

Certificate of Authorship/Originality

I certify that the work in this thesis has not previously been submitted for a degree nor has it been submitted as part of requirements for a degree except as fully acknowledged within the thesis.
I also certify that the thesis has been written by me. Any help that I have received in my research work and the preparation of the thesis itself has been acknowledged. In addition, I certify that all information sources and literature used are indicated in the thesis.

Farzaneh Tahmoorian

March 2018

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Abstract

Road networks are increasingly expanding all over the world. Over the past decades, the buoyant economy, growing population and increasing freight volumes have created high demand for new road pavements as well as the maintenance of current road networks. The construction and maintenance of the road pavements require large amounts of aggregates.

Due to considerable usage of various natural aggregates for constructing roads, these materials have started to deplete gradually. At the same time, the generation rate of solid waste in the society is increasing with the increase of population, technological development, and changes in the life style of people. The management of solid wastes has become an acute problem due to enhanced economic activities and rapid urbanization.

Moreover, according to World Commission on Environment and Development (1987), sustainable development is defined as the development that meets the needs of the present without compromising the ability of future generations to meet their own needs. Therefore, referring to the importance of the sustainability and all the above mentioned issues, the utilization of recycled materials and waste materials in the manufacture of new asphalt mixtures is addressed in this research, in order to follow the sustainability concept.

Using waste materials in asphalt mixtures conserves natural aggregates, reduces the impact on landfills, decreases energy consumption, can provide cost savings, and generally results in significant economic and environmental benefits. In spite of the benefits mentioned above, designing asphalt mixtures containing waste materials can be challenging and complex. In this regard, it is necessary to review a wide variety of engineering and geotechnical considerations for asphalt mixture design, as the application of wastes should not influence the structural and functional aspects of the pavements. Among various factors, which would be considered in designing an asphalt mix, permanent deformation, and stress/strain characteristics are the most important factors from the perspective of structural and geotechnical stability.

Evaluation of asphalt mixtures in terms of these engineering properties is necessary as they define the structural performance and strength of an asphalt pavement. In fact, these properties influence the ability of the mix in withstanding the destructive effects of traffic and environment.

In light of this, many researchers have studied the behaviour of asphalt mixtures containing waste materials through different tests and approaches to predict the performance of the asphalt surface layer under traffic load and environmental conditions.

Today, there are many published reports about the application of various waste materials in construction of different layers of pavements including asphalt surface layer. The research studies have been conducted from simple and primary tests to the advanced laboratory investigations, field tests, and constitutive modelling of asphalt surface layer. Due to the wide range of materials that fit into the definition of pavement, this proposed research does not intend to cover the investigation of the application of all types of materials available. Instead, it focuses on the assessment of the utilization of some of the waste materials including Recycled Construction Aggregate (RCA), Glass, plastic, and rubber in Dense Graded Asphalt (DGA) mixture, and it aims to propose a mixture with proper response to traffic loads and environmental conditions as well as appropriate resistance to rutting, cracking, and weathering.

In this report, a critical and comprehensive literature review about the performance of the asphalt mixture incorporating selected waste materials is provided. Moreover, this report includes a brief description of the relevant tests commonly specified to provide confidence of material suitability for the intended application as well as the results of these tests. Based on these tests, it is concluded that these waste materials have serious shortcomings in accounting for the asphalt mixtures components. They are also unable to account for designing asphalt mixture individually, and therefore, it is required that combination of some waste materials in certain percentages be considered in designing the asphalt mixture.

One of the goals of the proposed research is to determine the optimum percentage of RCA and glass in the mixture, and also to investigate the influence of the application of rubber and plastic in binder characteristics. This will be done through a series of preliminary and primary tests to evaluate the properties of individual components of asphalt mixtures and the volumetric properties of the mixture. The information and data collected from these tests will be used to select the adequate samples for further assessment through advanced tests such as resilient modulus test.

Since the variability in the behaviour of RCA used in different construction projects indicates the variability in RCA composition. Therefore, this research investigates the composition and variability of RCA through classification of aggregate samples collected from a recycling centre in Sydney over one year. Furthermore, to demonstrate the effect of the utilization of waste materials on the internal structure of the asphalt mixtures, microstructure studies of RCA and the image analyses on different asphalt mixtures is carried out using a scanning electron microscope (SEM).

This research will also include the simulation of the asphalt surface layer in order to understand the permanent deformation of asphalt surface layer. In addition, the life cycle assessment (LCA) and cost analysis of designed asphalt mixtures will be evaluated in order to compare them with conventional asphalt mixtures.

The results suggest feasible use of RCA in a certain amount as partial aggregate substitution in hot mix asphalt in terms of engineering properties and also environment and economic aspects of asphalt production. Furthermore, the test results reveal that glass waste is a viable material for improving the shortcomings of asphalt mixtures containing RCA.

This dissertation is lovingly dedicated to my precious son

Amir Koorosh Nemati

who really gave me the reason to continue….

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List of Publications

During the course of this research, a number of publications have been prepared which are based on the work presented in this thesis. They are listed here for reference.

Book Chapters:

- Tahmoorian, F., Samali, B. and Yeaman, J. (2018), Modified Asphalt (Book Chapter), Publisher: InTech, Edited by: Dr. Jose Luis Rivera-Armenta Instituto Tecnologico de Ciudad Madero Mexico, ISBN 978-953-51-6005-2.

Journal Papers:

- Tahmoorian, F., Samali, B., Tam, V.W.Y. and Yeaman, J. (2017), "Evaluation of Mechanical Properties of Recycled Material for Utilization in Asphalt Mixtures", Journal of Applied Sciences, Vol. 7, pp: 763-784; doi: 10.3390/app7080763.
- Tahmoorian, F., Samali, B. and Yeaman, J. (2017), "Laboratory Investigations on the Utilization of Recycled Construction Aggregates in Asphalt Mixtures", International Journal of Civil, Environmental, Structural, Construction and Architectural Engineering Vol: 11, No: 8, pp: 1011-1016.
- Tahmoorian, F., Samali, B. (2017), "Experimental and Correlational Study on the Utilisation of RCA as an Alternative Coarse Aggregate in Asphalt Mixtures", Australian Journal of Civil Engineering, https://doi.org/10.1080/14488353.2017.1385964.
- Tahmoorian, F., Samali, B., Yeaman, J. and Crabb, R., (2018) "The Use of Glass to Optimize Bitumen Absorption of Hot Mix Asphalt Containing Recycled Construction Aggregates", Journal of Materials, doi:10.3390/ma11071053.
- Tahmoorian, F. and Samali, B., (2018) "Laboratory Investigations on the Utilization of RCA in Asphalt Mixtures", International Journal of Pavement Research and Technology, https://doi.org/10.1016/j.ijprt.2018.05.002.
- Tahmoorian, F., Samali, B. and Yeaman, J., (2018) "Study of the Characteristics of Different Components of Recycled Construction Aggregate (RCA): Statistical Study in Sydney", International Journal of GEOMATE, Vol.15, No. 52; pp. 84-90, 10.21660/2018.52.58360.

- Tahmoorian, F., Samali, B., Liyanapathirana, S. and Yeaman, J., “Performance of Hot Mix Asphalt Containing Polyethylene: State-of-the-art”, Journal of Transportation Engineering, Part B: Pavements, (Submitted).

<u>Conference Papers:</u>

- Tahmoorian, F., Samali, B. and Yeaman, J. (2017), “Classification and characterization of recycled construction aggregate (RCA)”, 1st International Conference on Structural Engineering Research, Sydney, Australia, 2017.
- Tahmoorian, F., Samali, B. and Leo, C. (2017), “The Effect of Utilization of RCA on Volumetric Properties of Asphalt Mixtures”, 1st International Conference on Geomechanics and Geoenvironmental Engineering, Sydney, Australia, 2017.
- Tahmoorian, F., Samali, B. and Yeaman, J. (2018), “Utilization of Plastic for Pretreating RCA to Eliminate its Binder Absorption in Asphalt Mixture”, The 4th Australasia and South-East Asia Structural Engineering and Construction Conference, Brisbane, Australia, 2018.

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List of Notations and Symbols

The following symbols including their definitions are used in this report:

AADT	Annual Average Daily Traffic
AASHTO	American Association of State Highway and Transportation Officials
AC	Asphaltic Concrete
ADP	Abiotic Depletion Potential
AP	Acidification Potential
APA	Asphalt Pavement Analyzer
ASMM	Advanced Sequential Mixing Method
AV	Air Void
BEES	Buildings for Environmental and Economic Sustainability
BFI	Binder Film Index
BLCC	Building Life Cycle Cost
C	Conventional
CA	Coarse Aggregate Ratio
CBR	California Bearing Ratio
CCAA	Cement Concrete and Aggregate Australia
CDF	Cumulative Damage Factor
CEI	Construction Energy Index
CF	Crystallized Fraction
CGF	Cumulative Growth Factor
CI	Colloidal Instability
CO	Carbon Monoxide
CRM	Crumb Rubber Modifier
CTUe	Comparative Toxic Unit for Aquatic Ecotoxicity
CTUh	Comparative Toxic Unit for Human Toxicity
CV	Coefficient of Variation
D	Dry strength
DESA	Design Number of Equivalent Standard Axles
DF	Direction Factor
DGA	Dense Graded Asphalt
DQO	Data Quality Objectives
DSARm	Design Number of Standard Axle Repetitions
DSC	Differential Calorimetry Scanning
EAPA	European Asphalt Pavement Association
EDS	Energy Dispersive Spectroscopy
EP	Eutrophication Potential
EPS	Environmental Priority Strategies
ESA	Equivalent Standard Axle
FAc	Coarse Fraction Ratio
FAf	Fine Fraction Ratio

FGGA	Fine Gap Graded Asphalt
FI	Flakiness Index
FTIR	Infrared Spectroscopy
GEMIS	Global Emission Model for Integrated Systems
GWP	Global Warming Potential
HDPE	High Density Polyethylene
HMA	Hot Mix Asphalt
HVAG	Heavy Vehicle Axle Group
IDT	Indirect Tensile Test
IRI	International Roughness Index
ISO	International Standards Organization
ITS	Indirect Tensile Strength
LCA	Life Cycle Assessment
LCI	Life Cycle Inventory
LCIA	Life Cycle Impact Assessment
LDF	Lane Distribution Factor
LDPE	Low Density Polyethylene
LLDPE	Linear Low Density Polyethylene
LPB	Liquid Paperboard
LVDT	Linear Variable Differential Transformers
MET	Material, Energy and Toxicity matrix
MPCS	Mixture Primary Control Sieve
MQ	Marshall Quotient
MR	Resilient Modulus
MSW	Municipal Solid Waste
NAPA	National Asphalt Pavement Association
NCAT	National Centre for Asphalt Technology
NMPS	Nominal Maximum Particle Size
NMVOC	Non Methane Volatile Organic Compounds
NOx	Nitrogen Oxides
NREL	National Renewable Energy Laboratory
OBC	Optimum Bitumen Content
ODP	Ozone Depletion Potential
OGA	Open Graded Asphalt
OGFC	Open Graded Friction Course
OGPA	Open Graded Porous Asphalt
OS	Oversized Structure
PAFV	Polished Aggregate Friction Value
PaLATE	Pavement Life-cycle Assessment Tool for Environmental and Economic Effects
PC	Polycarbonate
PCS	Primary Control Sieve
PE	Polyethylene
PET	Polyethylene Terephthalate

PG	Performance Grade
PM	Particulate Matters
PMB	Polymer Modified Binder
POCP	Photochemical Ozone Creation Potential
PP	Polypropylene
PS	Polystyrene
PS	Primary Structure
PSF	Packaging Stewardship Forum
PVC	Polyvinyl Chloride
QADT	Quad Axle with Dual Tyres
RAP	Reclaimed Asphalt Pavement
RCA	Recycled Construction Aggregate
RF	Reliability Factor
RFM	Red Flag Method
S	Advanced Sequential Mixing Method
SADT	Single Axle with Dual Tyres
SAR	Standard Axle Repetition
SAST	Single Axle with Single Tyres
SCS	Secondary Control Sieve
SD	Standard Deviation
SE	Sequential
SEM	Scanning Electron Microscope
SHRP	Strategic Highway Research Program
SMA	Stone Mastic Asphalt
SN	Structural Number
SPI	Society of Plastic Industry
SS	Secondary Structure
SSD	Saturated Surface Dry
TADT	Tandem Axle with Dual Tyres
TAST	Tandem Axle with Single Tyres
TCS	Tertiary Control Sieve
TGA	Thermogravimetric Analysis
TRDT	Triaxle with Dual Tyres
TSR	Tensile Strength Ratio
UNEP	United Nations Environment Program
UTR	Useful Temperature Range
VFB	Voids Filled with Binder
VMA	Voids in Mineral Aggregates
VOC	Volatile Organic Compounds
W	Wet strength
WA	Water Absorption
WIM	Weigh In Motion
WMAPT	Weighted Mean Annual Pavement Temperature

Chapter 1

Introduction

1.1. Research Scope

1.2. Research Objective, Significance, and Innovations

1.3. Organization and Thesis Layout

1.1. Research Scope

With an expanding world and the remarkable growth of industry, the demand for extending the road networks is rapidly increasing. As available natural resources becomes scarce, application of recycled materials for construction purposes including road construction has become increasingly common. Over the past decade, there has been a significant increase in the utilization of the waste materials in pavement industry and construction of asphalt surface layer, as the road construction requires a large amount of materials. Natural resources depletion and increasing cost of quarrying and materials transportation has caused the increase in the cost of natural materials. Therefore, if the application of waste materials in pavement construction can be justified, it will result in substantial advantages in pavement construction coupled with reduction in the construction cost of roads taking into account the sustainability.

The application of recycled materials in road construction is not a recent development. In fact, the utilization of recycled materials in road construction has a long history. However, finding innovative uses for waste materials in road construction is one of the priorities in the pavement industry. In light of this, asphalt surface layers provide unique opportunities for reuse, as using the recycled materials in asphalt surface layer can contribute to improvement of engineering characteristics of the asphalt pavement materials as well as the pavement performance, representing a value application for solid wastes. However, significant development limitations and many relevant considerations must be addressed in this regard. Among various factors and conditions that should be considered in the application of waste materials in asphalt mixture, permanent deformation (rutting) and cracking can be the most important ones and need to be considered in asphalt mixture design. To minimize permanent deformation and cracking, it is necessary to pay extra attention to material selection and mixture design.

In general, asphalt mixtures are composed of coarse aggregates, fine aggregates, filler, and bitumen. The contents and the type of any of these constituents play an important role in the final performance of the asphalt mixture. Therefore, it is essential to design an asphalt mixture considering the properties of all mix components as all of them can have substantial influence on each other and also on the final mixture behaviour which must not be neglected.

In this research, the feasibility of using waste materials as different components of asphalt mixture including coarse and fine aggregates, filler, and binder modifier in Dense Graded Asphalt (DGA) mix are investigated. In order to explore this problem, several testing methods including the preliminary tests on asphalt components, and primary tests for the evaluation of volumetric properties of the mixture are used. Moreover, advanced tests in the form of resilient modulus test have been performed to assess the mixture performance when waste materials are added to it. For this purpose, some waste materials are selected based on their generation rate and different parameters affecting the asphalt mixture behaviour. The selected materials include recycled construction aggregate (RCA) as coarse aggregate and glass as fine aggregate which are planned to be used in the mixture with nominal size of 14 mm. Moreover, according to the pavement requirements, bitumen C320 is used in this study, while high density polyethylene (HDPE) and crumb rubber is considered as binder modifier to improve the mixture performance.

In addition, analytical and numerical studies of the permanent deformation behaviour of asphalt mixture constitute the other major aspect of this study. Analytical and numerical studies include the simulation of an asphalt surface layer and rutting measurement of the asphalt layer under various load and environmental conditions as well as parameter values.

1.2. Research Objectives, Significance, and Innovations

1.2.1. Objectives

Road construction involves a high outlay of investment. An accurate engineering design of pavements can result in the reliable performance of pavements. The importance of road networks and appropriate pavements has drawn the attention of many researchers to study the behaviour of asphalt mixtures and to design different asphalt mixtures incorporating varying components.

On the other hand, over the past decade, there has been a significant increase in generation of waste. The increased generation of municipal solid waste (MSW) is partially caused by population growth and economic developments. In light of this, as natural resources become more scarce and valuable, reuse of materials from different construction industries including pavement construction and particularly asphalt surface layer is becoming more widespread. However, the application of waste materials in asphalt surface layer often requires additional considerations for proper performance, as an effective asphalt mixture should have enough ability to withstand the destructive effects of traffic and environment.

This study aims at designing an asphalt mixture considering the main engineering properties. The main objective of this research is to design an adequate asphalt mixture incorporating some specific waste materials. In order to achieve the research objectives, attention will be given to different parameters and mechanisms that would affect the behaviour of asphalt surface layer. To this end, a series of experimental work covering different tests is considered for this study. These tests include the required tests for the assessment of the individual components of the asphalt mixture and other essential tests for the evaluation of volumetric properties and resilient modulus of the asphalt mixture.

In addition, classification of RCA samples collected from a recycling centre in Sydney into different geological groups (i.e. metamorphic, igneous, sedimentary) is considered as the other important objective of this research in order to create a database containing the composition and characteristics of RCA produced in Sydney in twelve months. Furthermore, in order to measure the structure of these aggregate types, microstructure assessment via digital image analysis has been considered in this research.

In the next step, an analytical study and numerical modelling is carried out by applying available software such as CIRCLY 6.0 for prediction of deformation of asphalt surface layer.

In addition, the life cycle assessment and cost analysis is performed by employing GaBi 6.0 software for determination of environmental and economical aspects of asphalt mixture production.

In summary, the specific aims of this research can be classified into seven groups, as follows:

1) <u>Literature Review:</u>

- Reviewing the current knowledge of road construction, different layers of flexible pavements as well as different types of asphalt mixtures;
- Developing a fundamental knowledge of the factors influencing the design of asphalt surface layer by identifying and considering mix design parameters;
- Surveying the literature for innovative usage of waste and recycled materials in the construction of DGA;
- Studying different aggregate gradation theories to derive the optimum grading target.

2) <u>Preliminary tests to evaluate the asphalt components properties:</u>

- Determining the physical and mechanical properties of the asphalt mixture constituents;
- Determining different aggregate types in recycled construction aggregates (RCA) through classification;
- investigating the effects of using plastic and rubber as binder modifier on selected binder;

3) **<u>Primary tests to evaluate the asphalt mixtures incorporating waste materials and to select the most acceptable asphalt mixtures for further investigation:</u>**
   - Examining the effects of using RCA as coarse aggregate in asphalt mixture in terms of volumetric properties and bitumen content;
   - Investigating the effects of using glass as fine aggregate in mixtures containing different percentages of RCA through assessment of volumetric properties and optimum bitumen content by Gyropac procedure;
   - Investigating the effect of mixing method on asphalt mix design;
   - Developing a framework for selecting the most acceptable mixtures that contain the optimum percentages of RCA and glass for further investigation;
4) **<u>Advanced tests to evaluate the performance of selected asphalt mixtures:</u>**
   - Investigating the performance of the selected mixtures in terms of the their stiffness through conducting the resilient modulus test;
5) **<u>Digital Image Analysis to evaluate the microstructure characteristics of selected asphalt Components :</u>**
   - Performing the microstructure assessment through digital image analysis of the selected mixes and selected asphalt mixture component;
6) **<u>Analytical and Numerical Studies to analyze the test results and simulate the asphalt surface layer performance:</u>**
   - Performing an analytical and numerical study on the behaviour of asphalt surface layer taking into account the properties of the selected mixtures to investigate the influence of time and load;
7) **<u>Life Cycle Assessment (LCA) and Cost Analysis of the proposed asphalt mixture incorporating waste materials:</u>**
   - Evaluation of the effect of asphalt mixture production on the environment in terms of various impact categories
   - Determining the cost advantages and/or disadvantages of using selected waste materials in the production of asphalt mixture.

It is expected that this research can be applied in pavement construction projects to accurately design an asphalt mixture with long term life cycle resulting in substantial advantages in asphalt surface layer construction coupled with reduction in the construction and maintenance costs of roads taking sustainability into account.

1.2.2. Significance

The road network is a vital infrastructure element in all countries, which is increasingly extending in order to meet the societies needs for the transportation of people and goods. Australia has a vast road network compared to other countries of the world (Figure 1.1). The Australian road network consists of 913,000 kilometres of roads, of which 70% is the rural network and 30% belong to urban network. Almost 353,331 kilometres of Australian roadways are paved and the remaining 559,669 kilometres of roads are unpaved. Among the total stretch of paved roads, almost 13,630 kilometres are classified as expressways which interconnect various parts of this large country (Compare Infobase, 2007).

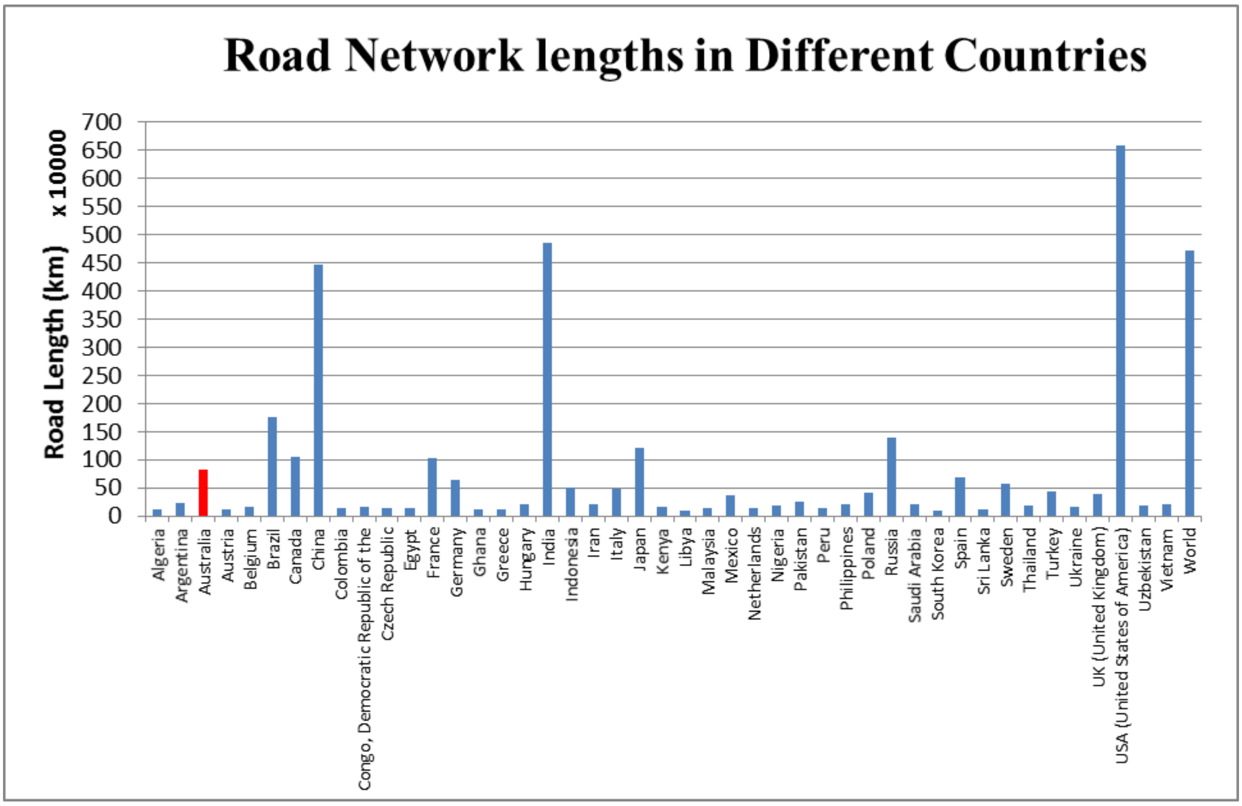

Figure 1.1: Road Network Sizes in Different Countries

In addition to the extensive network of roads in Australia, the relatively small population of Australia has resulted in higher road length per capita in Australia in comparison with the other developed nations. Referring to the available reports and references (Australia's Sustainable Aggregate Industry, 2013 and Austroads Pavement Research Group, 2012), the road length per capita is about 280 m in USA, 130 m in Great Britain, and 90 m in Japan, while the corresponding length is about 450 m in Australia. As in almost all countries, as well as Australia, the governments are taking rapid initiatives to transform the unpaved roads into paved roads.

On the other hand, with the rapid growth in economy and continuously increased consumption, a large amount of waste materials is generated in all countries. The total annual solid waste produced in Australia is estimated at about 60,870,900 tonnes during 2008 to 2009 (Hyder, 2011). The available studies and surveys indicate that the solid waste composition in Australia constitutes main categories including organics, masonry materials, paper and cardboard, metals, glass, plastics, hazardous wastes, and fly ash, as illustrated in Figure 1.2.

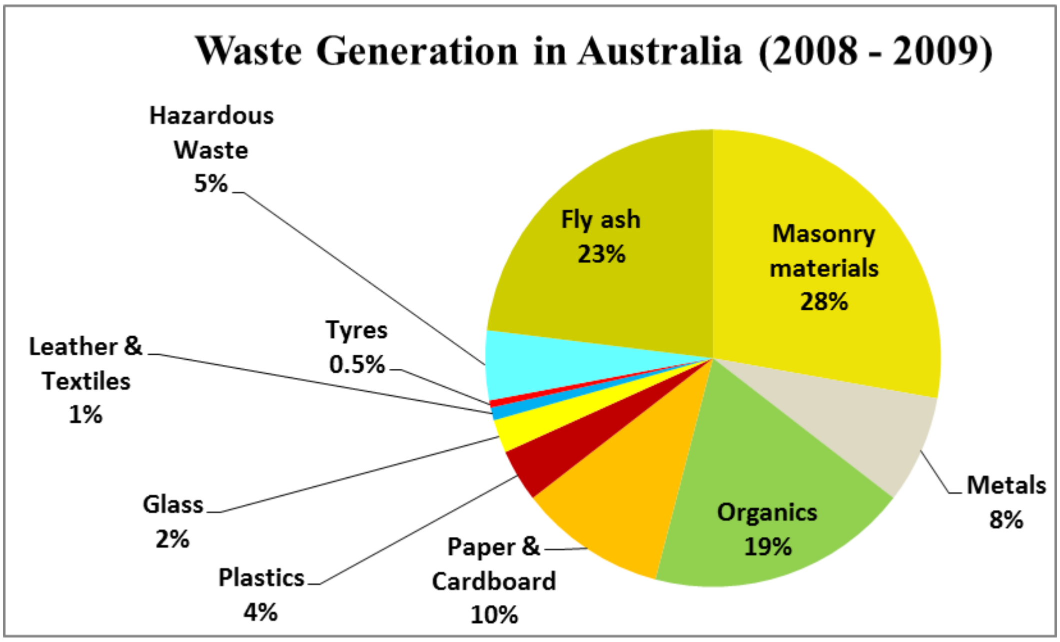

Figure 1.2: Waste Generation in Australia during 2008 to 2009

Referring to the previous discussions and considering above mentioned statistics, application of waste materials in road construction, including the asphalt surface layer remains an attractive route to solve the problems associated with natural resource depletion and solid waste disposal. However, physicochemical and mechanical properties of recycled materials inevitably hinder the beneficial use of such materials in pavement construction, and particularly in asphalt mixtures because the application of waste materials should not influence the structural and functional aspects of the surface (wearing) course.

Therefore, the ability to design an adequate asphalt mix incorporating appropriate waste materials becomes a key issue in the design and construction of pavements, including surface course, in line with sustainable development concept.

1.2.3. Innovation

Design of asphalt mixture incorporating the waste materials is not only critical to reduce the current problem of waste management and disposal but also to moderate the natural materials scarcity. However, the most challenging technical considerations in asphalt mixture design are the permanent deformation and cracking that can take place due to improper use of materials and inadequate asphalt mixture design.

To be able to design a mixture that has adequate resistance to rutting and cracking, knowledge of the effect of mixture composition and properties of the component materials is of paramount importance. Therefore, the main goal of this research is to provide more insight into the effect of incorporation of some selected waste materials in asphalt mixtures on the fatigue behaviour and rutting resistance. In order to meet this goal, it is necessary to investigate the individual contribution of different components in asphalt mixture. As mentioned before, the asphalt mixture is composed of coarse and fine aggregates, binder, filler, and air voids. It is essential to characterize the behaviour of individual components in order to better understand the final performance of asphalt mixture. Accordingly, in this research, a comprehensive study will be carried out on the application of waste materials as different asphalt mixture components. Among these materials, RCA, Reclaimed Asphalt Pavement (RAP), glass, rubber, and plastic (HDPE) are studied precisely through comprehensive experimental works in order to achieve the optimum content and the best combination of materials in a DGA AC 14. The experiments involve the preliminary tests on the asphalt components as well as the primary and advanced tests on asphalt mixture in order to evaluate the asphalt mixture performance more precisely.

Moreover, a numerical study on the asphalt surface layer behaviour will be performed by available software to estimate the asphalt mixture resistance to permanent deformation. Furthermore, the life cycle assessment and cost analysis will be performed on different asphalt mixtures to determine the effect of asphalt mixture production on the environmental and economical aspects.

The outcomes of the study will improve the confidence for design and construction of Dense Graded Asphalt Mixture incorporating waste materials.

1.3. Organization and Thesis Layout

This dissertation investigates the design of new asphalt mixture incorporating waste materials through a comprehensive experimental work and mathematical study of the asphalt surface layer in order to understand the process on the mixture performance in asphalt surface layer considering various related parameters. The organization of this dissertation, including twelve chapters, is as follows:

Chapter 1, which is an introductory chapter, outlines the main problem and describes the significance and innovations of this research. In addition, this chapter provides a brief review of thesis work flow as well as the research scope.

Chapter 2 provides a critical review on the previous studies associated with asphalt mixture containing waste materials as well as a thorough review of asphalt mixture design process.

Chapter 3 describes the research methodologies and experimental work program adopted in this thesis.

Chapter 4 presents the experimental results of the preliminary tests on coarse aggregate, fine aggregate, and filler.

Chapter 5 conducts an analysis of aggregate gradation theories to derive the optimum grading target.

Chapter 6 presents the results of primary tests conducted on asphalt specimens to evaluate the volumetric properties of asphalt mixtures.

Chapter 7 describes different sample preparation methods and presents the results of laboratory investigation and image processing analysis conducted on these methods in order to propose the best sample preparation method.

Chapter 8 contains a brief description about the concept employed for performing resilient modulus test as well as the results of resilient modulus test conducted on selected specimens to evaluate their behaviour in terms of their stiffness.

Chapter 9 provides the detailed information regarding the binder and the preliminary test results on bitumen and modified binders.

Chapter 10 is dedicated to a brief description of the models used as the basis for the prediction of asphalt surface layer behaviour. It involves the introduction of different equations employed for prediction of asphalt performance, as well as the assumptions and the governing equations used in the considered model.

Chapter 11 provides the detailed information regarding the life cycle assessment and cost analysis of asphalt mixture production. This chapter describes the thorough procedure of assessment in GaBi Software and presents the results of this life cycle assessment.

Chapter 12 presents a summary of this thesis, including the concluding remarks that can be drawn from the research and its contributions. This chapter also suggests several ideas for related future work.

Chapter 2

Literature Review

2.1. Introduction

The demand for public infrastructure is commensurate with growth. With an expanding world and the remarkable growth of freight volumes, construction of more public infrastructure and, therefore, more aggregates is required. Asphalt is used extensively in road construction worldwide (Figure 2.1).

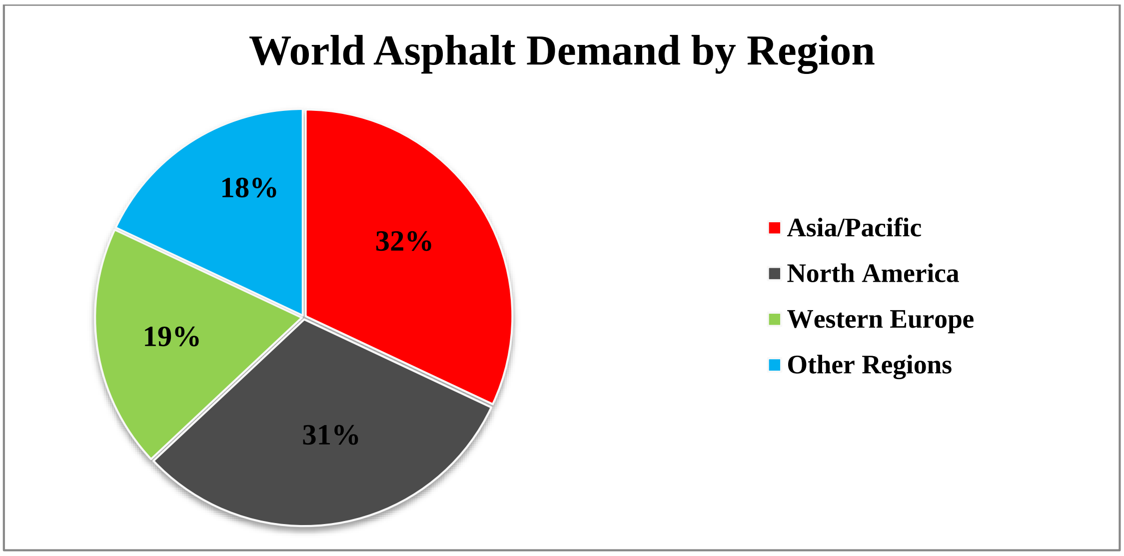

Figure 2.1: World Asphalt Demand by Region (Freedonia Group, 2015)

The worldwide demand for asphalt is estimated to expand by 2.8% annually to 122.5 million metric tonnes, by 2019 (Freedonia Group, 2015). This amount will continue to increase due to population growth, urbanization and economic growth. However, while asphalt production continues to grow and contribute towards socio-economic development, escalating negative and irreversible impact on the environment will be inevitable. These negative impacts can be decreased by using suitable materials for constructing sustainable roads. An accurate engineering design of pavements can result in the reliable performance of road pavements. Accordingly, investigation on the feasibility of the incorporation of certain waste materials as alternative materials in road construction has attracted the attention of many researchers in the pavement industry.

Therefore, the main purpose of this chapter is to provide a review of literature in the field of flexible pavements. However, general topics including pavement types, flexible pavement layers, asphalt components, asphalt mix types, asphalt design factors, common tests for the evaluation of asphalt components properties, and asphalt layer distress modes are initially discussed in order to gain an understanding of issues specific to asphalt surface layer in flexible pavements.

2.2. Waste Generation in Australia

According to available references (e.g. Hyder, 2011), the solid waste in NSW as well as Australia is composed of different categories. The waste materials covered by each category are presented in Table 2.1.

Table 2.1: Main Waste Categories and Material Types (Hyder, 2011)

Material Category	Material Types
Masonry Materials	Asphalt, Concrete, Bricks, Rubble, Plasterboard and Cement Sheeting (ex. Asbestos Reinforced)
Metals	Steel, Aluminium, Non-ferrous Metals (ex. Aluminium)
Organics	Food Organics, Garden Organics, Timber, Other Organics, Biosolids
Paper & Cardboard	Cardboard, Liquid Paperboard (LPB), Newsprint & Magazines, Office Paper
Plastics	Polyethylene Therephthalat (PET), High Density Polyethylene (HDPE), Polyvinyl Chloride (PVC), Low Density Polyethylene (LDPE), Polypropylene (PP), Polystyrene (PS), Other Plastics
Glass	Glass
Other Wastes	Leather and Textiles, Tyres & Other Rubber
Hazardous Waste	Quarantine, Contaminated Soil, Industrial Waste, Asbestos
Fly ash	Fly ash

The data related to waste generation in NSW and Australia during 2008 and 2009 is presented in Table 2.2.

Table 2.2: Waste Generation in NSW and Australia during 2008 and 2009 (Hyder, 2011)

Component	NSW		Australia	
	Waste Generation (tonnes)	Waste Generation (%)	Waste Generation (tonnes)	Waste Generation (%)
Masonry materials	5,374,100	25.88	16,933,900	27.82
Metals	1,785,100	8.596	4,649,100	7.638
Organics	3,471,500	16.72	11,336,900	18.62
Paper & Cardboard	2,595,500	12.5	6,362,700	10.45
Plastics	957,900	4.613	2,272,600	3.733
Glass	627,400	3.021	1,427,700	2.345
Leather & Textiles	174,200	0.839	556,500	0.914
Tyres	29,400	0.142	310,900	0.511
Hazardous Waste	1,274,400	6.137	2,994,000	4.919
Fly ash	4477900	21.56	14026500	23.04
Total	**20,767,400**	**100**	**60,870,800**	**100**

As can be seen in these tables, the large amount of construction and demolition waste generation justifies the idea of using RCA in new asphalt mixtures, which is the main focus of the current research.

2.3. Demand for aggregates and public infrastructure

Asphalt plays an important role in transportation infrastructure worldwide and deals with economic growth and social well-being in all countries (Mangum, 2006). Asphalt contains approximately 95% aggregate and 5% bitumen. Referring to Ektas and Karacasu (2012), construction of asphalt layer with 15 cm thick and 10 m wide for one kilometre of road requires almost 3,750 tonnes of asphalt mixture composed of aggregate and bitumen. In 2007, about 1.6 trillion metric tonnes of asphalt were produced worldwide (EAPA and NAPA, 2009), considering the important role and high proportion of aggregates in asphalt mixtures, it can be estimated that the large quantities of aggregates are required for road construction.

According to a report from Cement Concrete & Aggregates Australia (CCAA) (2013), 25,000 tonnes of crushed rock will be used for construction of one kilometre of highway, and 5,000 tonnes of crushed rock are required for construction of one kilometre of suburban roadway. The demand for aggregate will continue to increase with population growth. According to the Australian Bureau of Statistics, Australia's population will reach 27.2 million by 2026 and it is expected to grow by 9 million people to 36 million in 2056. This population growth will necessitate the infrastructure provision which requires a substantial increase in the aggregates and other construction materials supply. Referring to Australian Quarry Industry, the annual production of aggregates is currently about 130 million tonnes. However, it will be needed that extractive industry produces 210 million tonnes of aggregates per year by 2056 if the current demand trend continues with population growth (Australia's Sustainable Aggregate Industry, 2013).

2.4. Road Pavements

Generally, the road pavements are categorized into two main groups, called rigid pavement and flexible pavement, among which the flexible pavement is the most common way of pavement construction. The structure and the typical cross section of both types of pavements are illustrated in Figures 2.2.

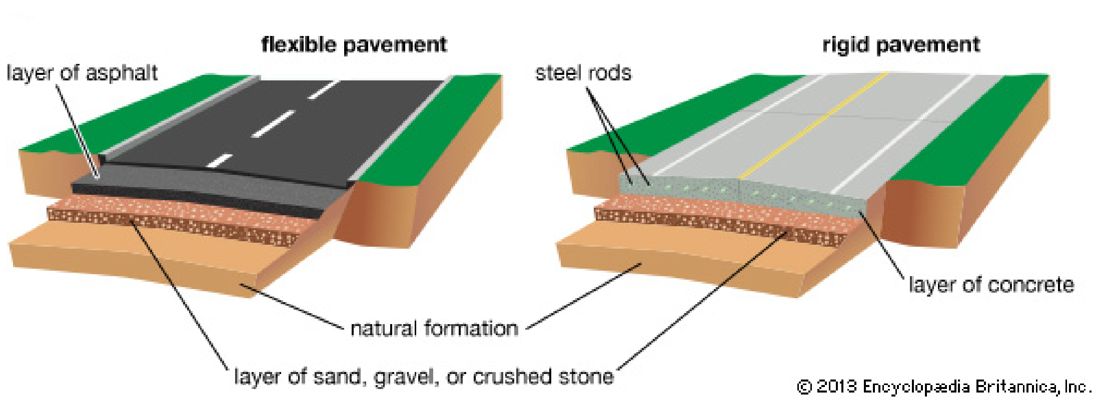

Figure 2.2: Pavement Structure (Encyclopaedia Britannica Inc., 2013)

The traffic loads on the pavement must transfer from the surface layer to the subgrade through base and sub-base layers. Therefore, the main aim of the pavement design is that the pavement be able to transfer the load properly while it resists the rutting beyond a certain level. Accordingly, the properties of the materials applied in pavement layers play a major role in the effective stress distribution to avoid excessive deformations.

Figure 2.3 illustrates how the load is transferred from the pavement surface to the underlying aggregate structure in flexible and rigid pavements. As shown in this figure, the rigid pavements have wide distribution of load in comparison with the narrow distribution of load in the flexible pavements. Moreover, the features of the flexible pavement structure shows that the induced stress is higher in the upper layers and diminish with depth. In other words, after each layer received the loads from the above layer, spreads the load and then pass them on to the under layer.

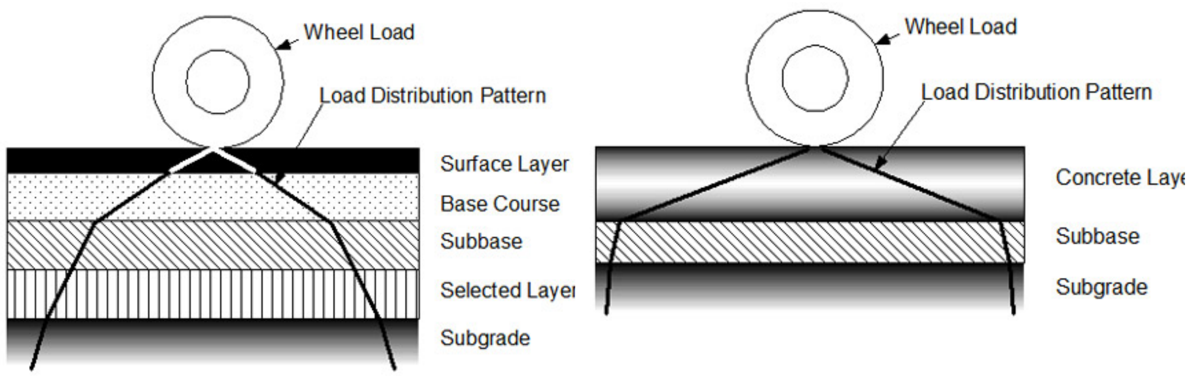

Figure 2.3: Typical Load Distribution for Flexible and Rigid Pavement Layers (The South African National Road Agency Website)

Therefore, the materials for different layers of flexible pavements are usually selected in order of descending load bearing capacity. It means that materials with the highest load bearing capacity material and usually most expensive materials are considered for the top layer, whereas the materials with the lowest load bearing capacity and least expensive materials are used for the bottom layer (Illston and Domone, 2001).

2.5. Flexible Pavement Layers

As discussed previously, a flexible pavement structure has typically several layers of material. As the stresses induced by traffic loads reduces with depth, from practical and economical considerations, relatively weak materials are used for bottom layers whereas stronger materials are used in the top layers (Croney and Croney, 1998).

The typical flexible pavement structure consists of three layers, including asphalt surface layer, base layer, and subbase layer. Figure 2.4 shows typical layers of flexible pavement structure and their functional and structural requirements of the layers.

As shown in this figure, the asphalt layer consists of three tiers which constitute the top layer of the road structure. These tiers include surface course (asphalt wearing surface), binder course (asphalt intermediate layer), and asphalt base layer.

There are a wide range of materials that are used in surface course. However, the asphalt wearing surface must have sufficient stability and strength to withstand the traffic loads and detrimental environmental conditions adequately. The binder course is an intermediate layer in asphalt layer structure which is designed in order to reduce permanent deformation and withstand the highest stresses occurring under the asphalt wearing course. Binder course typically consists of larger aggregate size (19-38 mm) and lower binder content for providing the stability as well as durability. The last asphalt layer is the asphalt base layer which is not exposed to the environment. Therefore, it is made of a maximum aggregate size (up to 75 mm) and lower binder content in order to provide adequate durability (Mathew and Rao, 2007).

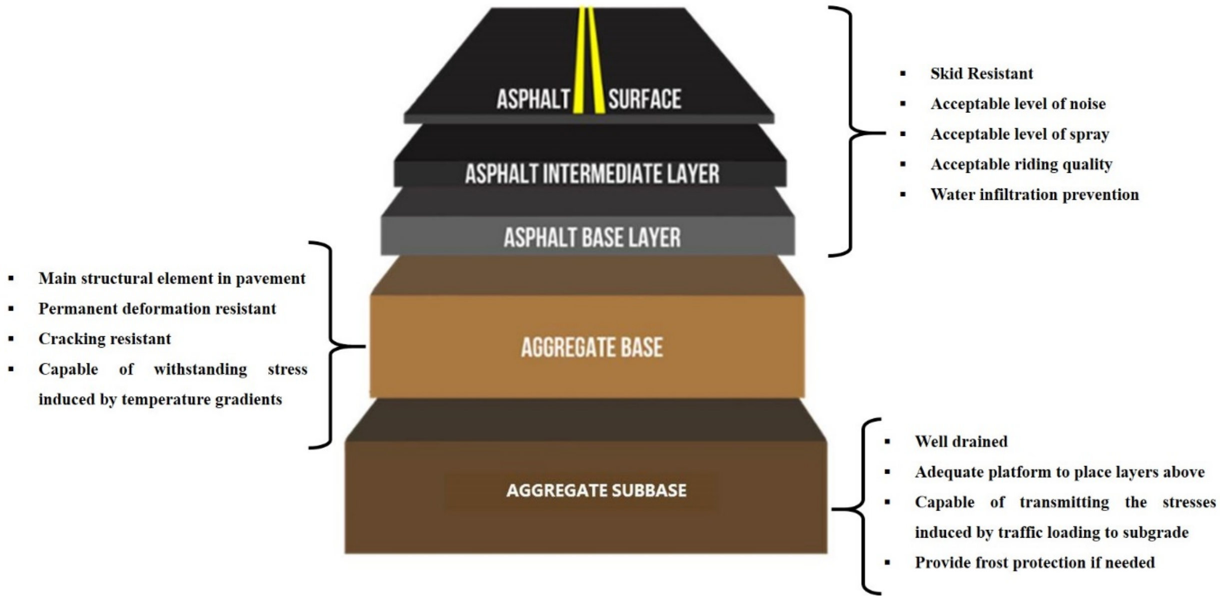

Figure 2.4: Typical Layers of Flexible Pavement and their Requirements

The base course is the main load carrying layer within the pavement which is placed immediately beneath the asphalt surface layer. The base course contributes to drainage and provides the frost resistance while distributes the load.

The sub-base course is the layer consisting of low quality materials in comparison with the base layer materials and it is placed on top of the sub-grade.

2.6. Asphalt Mixture Design

In designing a new asphalt mixture, the following factors must be considered in order to propose an appropriate asphalt mixture which can cover desired requirements:

- Structural performance and strengthening requirements
- Operating environment (user expectations, traffic level, climate and weather, surface characteristics requirements, etc.)
- Life cycle cost

These factors will subsequently affect the selection of the following parameters in asphalt mixture designs:

- Type of asphalt mix
- Mix size (nominal size)
- Layer thickness
- Compaction cycle
- Asphalt ingredients

Referring to the importance of these factors and parameters, and their substantial influence on the type of asphalt mixture as well as its performance, some of them which can be considered as the basic concepts in asphalt mixture design are initially discussed in this section in order to gain an understanding of issues specific to asphalt mixture design, while the other parameters and factors (e.g. asphalt mix type and asphalt components) are described in the following sections and chapters.

2.6.1. Structural Performance and Strengthening Requirements

The engineering properties of the asphalt mixture define the structural performance and strength of an asphalt pavement. In fact, these properties influence the ability of the mix for withstanding the destructive effects of traffic and environment.

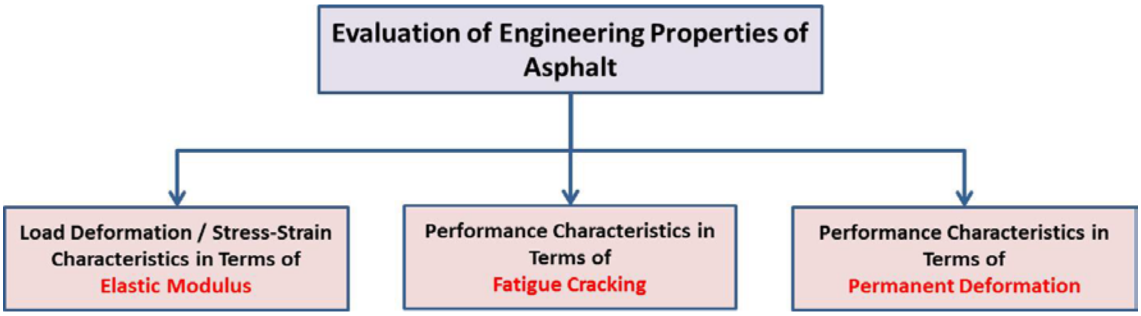

Figure 2.5: Main Characteristics for the Evaluation of Asphalt Engineering Properties

As illustrated in Figure 2.5, the engineering properties of asphalt involve the following characteristics:

- Stiffness which is the stress-strain characteristics
- Performance characteristics related to fatigue cracking
- Performance characteristics related to permanent deformation (rutting)

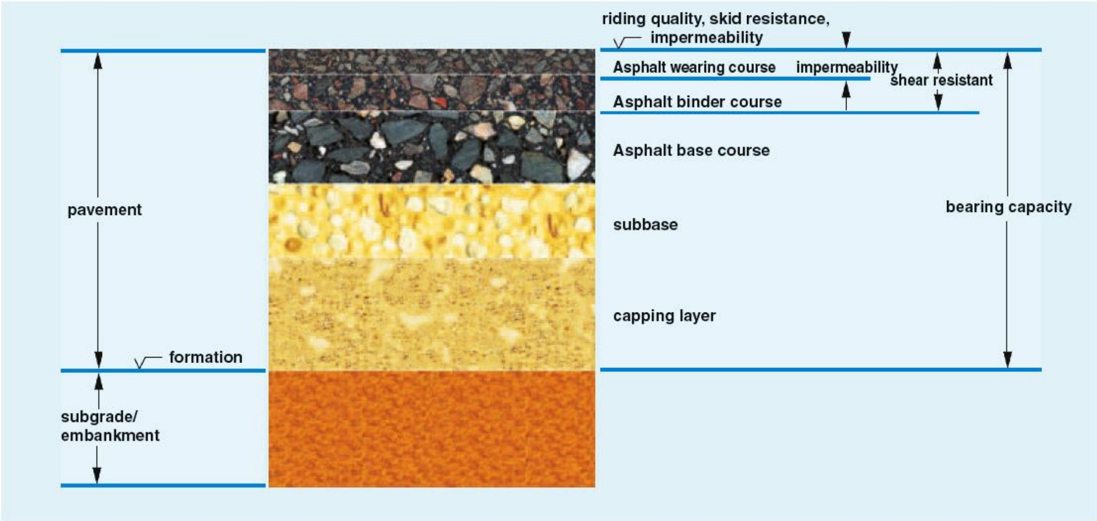

Figure 2.6: Typical Structure and Properties of Flexible Pavement (BOMAG, 2009)

However, as discussed previously and also presented in Figure 2.6, the surface (wearing) course requires two major characteristics:

- Good Resistance to shear forces
- Pavement skid resistance which depends on:
    - ❖ Microtexture which mainly depends on aggregate mineralogy and its shape characteristics,
    - ❖ Macrotexture which is a function of aggregate gradation, compaction method and mix properties (AASHTO, 1976)

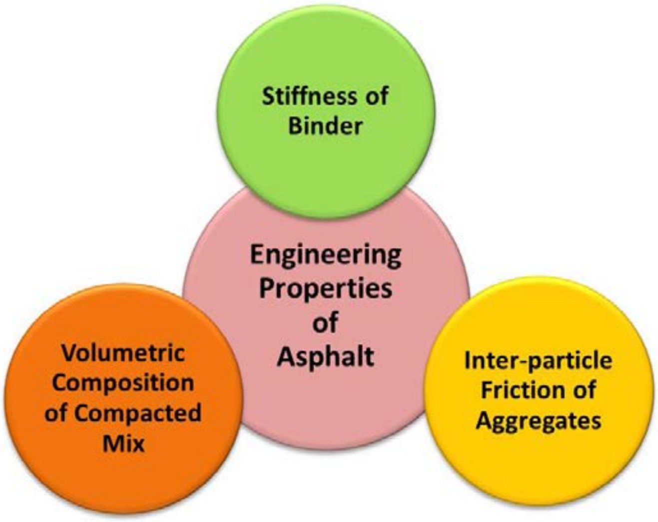

Figure 2.7: Main Parameters Affecting the Asphalt Engineering Properties

As shown in Figure 2.7, there are many factors that influence the above mentioned properties. However, these engineering properties are mainly attributed to the following parameters:

- The binder stiffness
- The aggregates inter-particle friction
- The volumetric characteristics of the mix including the volume of air voids, binder and voids in mineral aggregate

Therefore, the choice of asphalt mix type and the design parameters are of high importance in pavement design.

2.6.2. Traffic Level

Traffic loads cause stresses and strains in pavement structures. Traffic level is an important parameter in asphalt mix design, as different traffic levels require different combinations of aggregate, filler, and binder in terms of their type, proportion and size, in order to provide adequate levels of flexibility, durability, deformation resistance, structural stiffness, permeability, and surface texture. In this regard, the main traffic related factors that should be taken into account in the asphalt mixture design, are:

- Traffic type
- Traffic volumes
- Amount of turning and braking traffic
- Traffic speed

These factors will substantially affect mix types, volumetric properties, and binder types, and subsequently will influence the mixture durability and its resistance to deformation.

Table 2.3: Traffic Category (Austroads, 2014)

Indicative Traffic Volume		Traffic Category	
Commercial Vehicles/Lane/Day	**Structural Design Level**	**Free Flowing Vehicles**	**Stop/Start or Climbing Lane or Slow-moving**
< 100	**5×10^5 ESAs[1]**	Light	Medium
100 to 500	**5×10^5 ESAs to 5×10^6 ESAs**	Medium	Heavy
501 to 1000	**5×10^6 ESAs to 2×10^7 ESAs**	Heavy	Very Heavy
> 1000	**2×10^7 ESAs**	Very Heavy	Very Heavy

[1] Equivalent Standard Axles

In addition, high tyre pressure causes greater stress on the asphalt concrete layer (Austroads, 2014). Therefore, the traffic category need to be selected prior to any mixture design, as presented in Table 2.3.

2.6.3. Mix Size (Nominal Size)

The nominal size of an asphalt mix indicates the maximum aggregate size which is present in an asphalt mixture. The nominal size of a mix can influence the stability and the workability of asphalt layers.

Table 2.4: Typical Mix Sizes for Various Applications (Austroads, 2014)

Application	Typical Mix Size
Dense graded wearing course ▪ Lightly trafficked pavements ▪ Medium to heavily trafficked pavements ▪ Highway pavements ▪ Heavy duty industrial pavements	 7 or 10 mm 10 or 14 mm Generally 14 mm (also 10 mm) 14 or 20 mm
Dense graded intermediate course	In general, it is better to use the largest size practicable where the wearing course is dense graded asphalt. Where the asphalt surface is open graded asphalt, the largest size mix used in this application is typically 14 mm.
Dense graded base course	Normally 20 mm. 28 mm may also be used depending on layer thickness and availability. 40 mm has been used in the past but now largely discontinued through difficulties associated with increased segregation in larger sized mixes and general unavailability.
Dense graded corrective course	5, 7, 10, 14 or 20 mm
Stone mastic asphalt wearing course	7, 10 or 14 mm
Open graded wearing course	10 or 14 mm
Open graded base course (drainage layers)	14, 20 or 28 mm
Fine gap graded asphalt	Generally 10 mm (also 7 mm)
Minor patching **Major patching**	10 mm (also 5, 7, 14 and 20 mm) All sizes as appropriate

According to Roberts et al. (1996), in Hot Mix Asphalt (HMA), selection of excessively small nominal size may result in instability, whereas excessively large mix size may result in poor workability and/or segregation of asphalt mixture. Therefore, the nominal size of an asphalt mix is one of the most important factors in asphalt mix design which can be selected based on the location of asphalt course in pavement, asphalt layer thickness, and the performance requirements of the mix. Typical nominal sizes for various performance functions are summarized in Table 2.4.

2.6.4. Layer Thickness

The thickness of the asphalt layer does influence its compactability. Typically, a layer thickness of not less than 2.5 times and preferably 3 times the nominal size of the mix is needed in order to avoid poor mechanical interlock of aggregate particles and to facilitate compaction.

Table 2.5: Typical Asphalt Layer Thickness (Austroads, 2014)

Nominal Mix Size (mm)	Compacted Layer Thickness (mm)
5	15 to 20
7	20 to 30
10	25 to 40
14	35 to 55
20	50 to 80
28	70 to 110

Referring to Austroads (2014), the maximum thickness of compacted layer is generally limited to between four and five times the nominal mix size. However, it is more cost-effective to use the mixture with larger nominal size if the thickness of layer is greater than four times the nominal size as this will increase the flexural stiffness and deformation resistance. The typical asphalt layer thickness is presented in Table 2.5.

2.6.5. Compaction Cycle

The level of compaction in asphalt mixture production depends on the traffic level as given in Table 2.6.

Table 2.6: Typical Asphalt Layer Thickness (Austroads, 2014)

Traffic Level	Level of Compaction (Cycle)
Light Traffic	50
Medium Traffic	80
Heavy Traffic	120
Voids at Maximum Cycle	250 or 350

2.7. Asphalt Mix Types

Asphalt mixtures are classified into different groups based on their particle size distribution (grading), which subsequently, each grading type defines some variations in terms of material types, binder type, and the proportions of component materials. The asphalt mix types are mainly categorized into the following groups:

- Dense Graded Asphalt (DGA)[1]
- Stone Mastic Asphalt (SMA)
- Open Graded Asphalt (OGA)[2]
- Fine Gap Graded Asphalt (FGGA)

The typical particle size distribution and typical mix components (by volume) for these mix types are illustrated in Figures 2.8 and Figure 2.9, respectively.

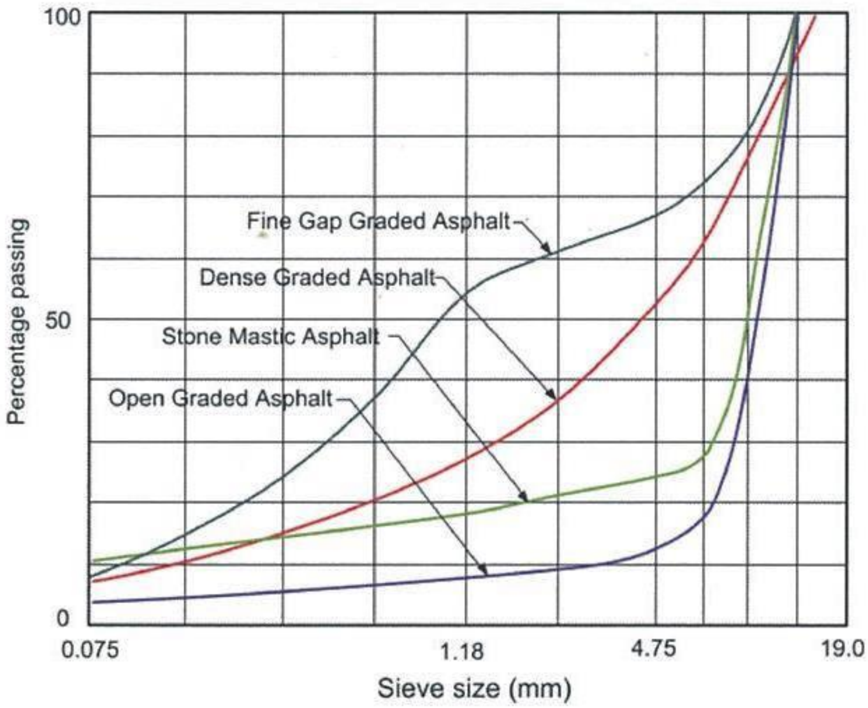

Figure 2.8: Typical Particle Size Distributions for Various Mix Types (Austroads, 2014)

The principal asphalt mix types are briefly described in the following sections.

[1] This asphalt mix type is also called Asphaltic Concrete (AC)
[2] This asphalt mix type is also called Open Graded Porous Asphalt (OGPA) or Open Graded Friction Course (OGFC)

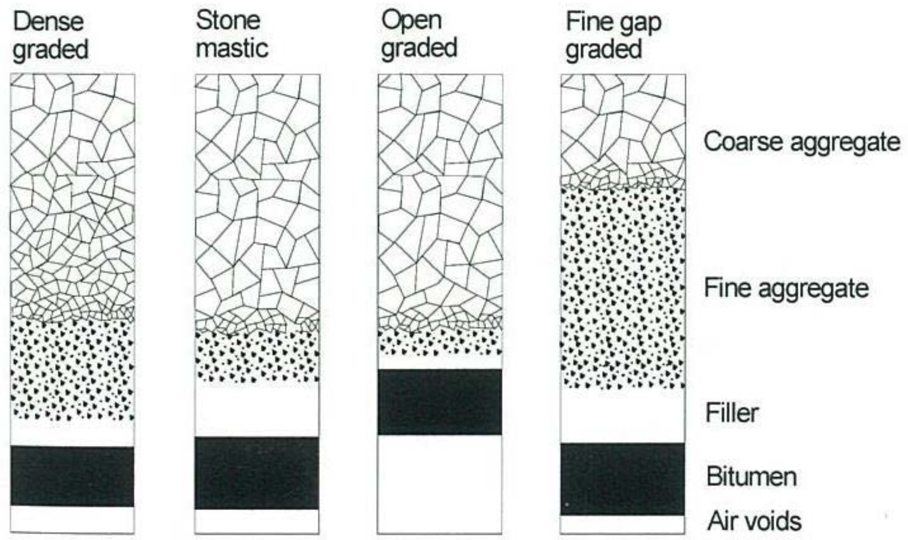

Figure 2.9: Typical Mix Components by Volume (Austroads, 2014)

2.7.1. Dense Graded Asphalt (DGA)

Dense graded asphalt (DGA) involves a continuous aggregate gradation, as shown in Figure 2.8. This type of asphalt mix has a low design air void contents, generally in the range of 3% to 7% (Austroads, 2014). Dense graded asphalt has the greatest load carrying capacity and its characteristics can provide appropriate properties for a wide variety of pavement applications.

2.7.2. Stone Mastic Asphalt (SMA)

Stone Mastic Asphalt (SMA) contains a large proportion of coarse aggregate in order to increase texture, and subsequently reduce water spray and noise (Figure 2.9). This type of asphalt mix has a high deformation resistance. However, the materials cost and construction complexities are higher in comparison with dense graded asphalt mixtures.

2.7.3. Open Graded Asphalt (OGA)

In Open Graded Asphalt (OGA), the tyre road noise and surface water spray are reduced through its permeable surface. Therefore, this type of asphalt mix provides a quieter and safer surface while the structural stiffness and durability are reduced compared with dense graded asphalt and stone mastic asphalt.

2.7.4. Fine Gap Graded Asphalt (FGGA)

Fine Gap Graded Asphalt (FGGA) is generally used in light traffic situations. This type of asphalt mix contains a high proportion of fine aggregates and may be high in binder content to provide a durable, smooth textured and easily compacted asphalt mixture, as shown in Figure 2.9.

2.8. Asphalt Components

Asphalt mixture consists of three basic parts of:

- Aggregate, including coarse and fine aggregates,
- Filler,
- Binder,

The above mentioned components are briefly described in the following sections.

2.8.1. Asphalt Aggregates

Aggregate typically makes up 90% to 95% by mass of most asphalt mixes. By volume, the corresponding ratio is 75% to 85% (Austroads, 2014). Therefore, the aggregate properties play an important role in the performance of road pavements, as the aggregate provides a substantial proportion of the load-bearing capacity of the asphalt mixture. The physical properties of aggregates significantly affect the performance of asphalt mixtures. Therefore, the aggregates properties and the interactions between aggregates and binder are of great importance in order to design appropriate asphalt mixes.

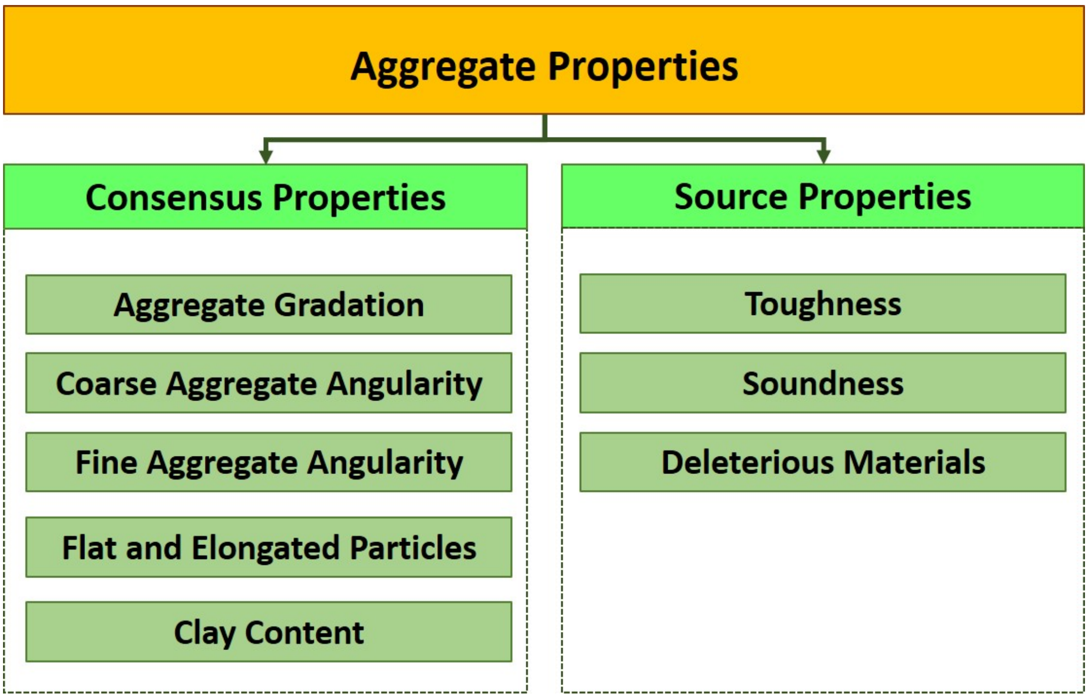

Figure 2.10: Aggregate Properties Based on SHRP Classification

The Strategic Highway Research Program (SHRP) divides aggregate properties into two groups of consensus properties and source properties. As shown in Figure 2.10, consensus properties involves aggregate gradation, fine aggregate angularity, coarse aggregate angularity, flat and elongated particles and clay content, whereas source properties include soundness, toughness, and deleterious materials.

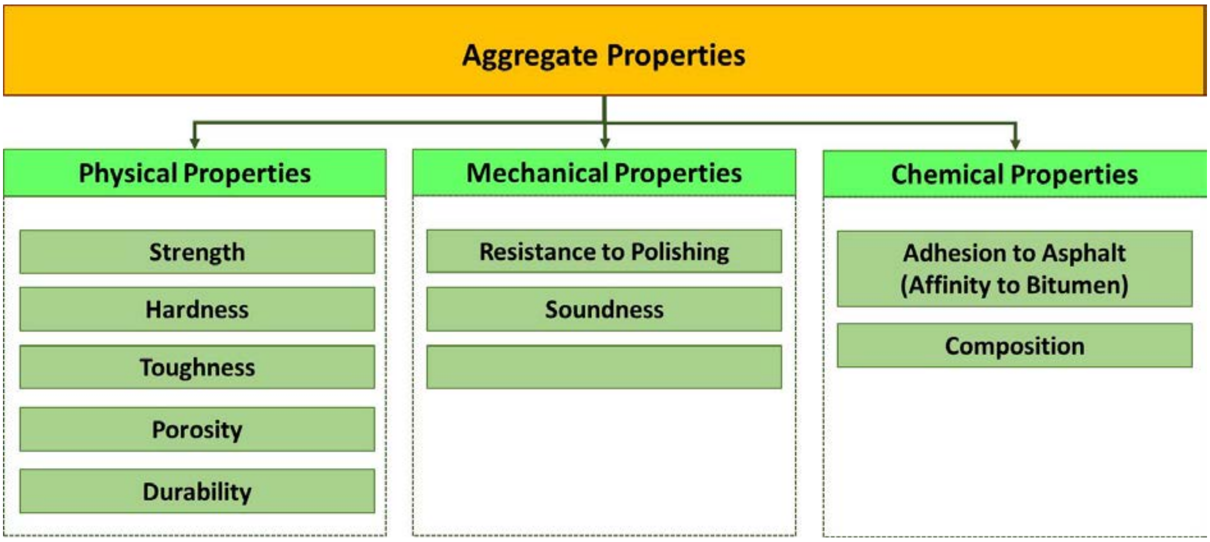

Figure 2.11: Aggregate Properties Classification

In another classification, the aggregate properties are classified into the physical, mechanical, and chemical properties (Figure 2.11). These properties demonstrate the mineralogy of the aggregates as well as the processes of aggregate production. For a material with a specific chemical composition and structure, the mineralogy of the aggregates influences some of the mechanical and physical properties of the aggregates as well as some of the chemical and physiochemical properties. In addition to mineralogy, the alteration and production processes can affect the aggregate properties. The performance of an asphalt mixture is considerably influenced by all of these properties of the used aggregates.

However, many researchers have identified aggregate gradation, aggregate angularity, and filler content as the most important properties having significant effect on the asphalt mixtures performance, particularly permanent deformation resistance of mixtures (Gabra, 2002). Accordingly, this section involves the subjects regarding the aggregate properties, the commonly used tests to determine the aggregate characteristics, and the influence of aggregate properties on the performance of asphalt pavements.

2.8.1.1. Particle Size Distribution and Aggregate Gradation

Aggregate gradation reflects the percentage of each of the aggregate sizes in a blend. This characteristic is one of the most important aggregate properties which are directly related to the performance of an asphalt mixture, as the mechanical interlock of the aggregate skeleton substantially affect the shear strength and the rutting resistance of asphalt mixtures. Generally, poor gradation results in loss of stability in asphalt mixtures. The dense graded layers usually have higher strength or shear resistance due to the increased contact between the larger particles and hence higher frictional resistance to shear forces.

2.8.1.2. Particle Geometry

The particle geometry can be represented in terms of three different independent properties, namely form (shape), angularity (roundness), and surface texture (as illustrated in Figure 2.12). Particle shape is related to the variations in the particle dimensions. Angularity is related to the variations at the particle's corners. Surface texture describes the surface irregularity and relative roughness or smoothness of the particle. Surface texture is primarily a function of the aggregate mineralogy, whereas the particle shape and angularity are influenced by crushing techniques. The aggregate shape characteristic influences the frictional characteristics of the pavement surface. Moreover, the particle shape highly affects the mechanical stability of the asphalt mixture. In general, the flaky and elongated aggregates are not desirable in asphalt mixtures as they influence the durability of asphalt mixtures due to their tendency to break down during construction and production. In addition, flat and elongated aggregates impede compaction and may produce a mix with high in situ air voids, which subsequently lacks strength and is more susceptible to deterioration because of the ageing of the binder and stripping. Furthermore, rough textured particles increase the internal frictions, and form stronger mechanical bonds with the binder in comparison to smoother particles. This will subsequently result in asphalt mixture with higher strength.

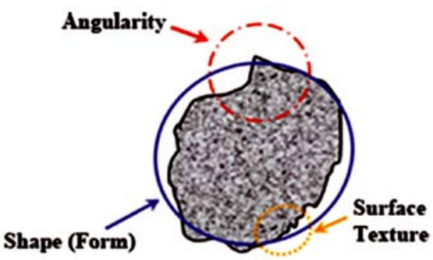

Figure 2.12: Components of Aggregate Geometry (Kuo and Freeman, 2000)

A number of studies (e.g. Perdomo et al., 1992; Sanders and Dukatz, 1992; Kobayashi et al., 1997; Kim et al., 1992; Masad et al., 2007) have shown that utilization of angular aggregates with rough texture will improve the strength and stability of asphalt mixtures as a result of the stronger bond between particles and binder, higher resistance of aggregates to slide over each other, and higher strength of particles. In this regard, the angularity of fine aggregate is more critical than the angularity of coarse aggregate. However, it should be noted that the aggregate angularity is just one property among many other aggregate properties that influence the asphalt mixtures performance.

2.8.1.3. Durability

Durability can be related to chemical, physical and mechanical properties of the aggregates. In terms of the chemical property, the durability refers to the aggregate resistance to the chemical changes and decomposition due to the interaction between the aggregate and the surrounding environment. As a physical property, the durability is defined as the aggregate resistance to disintegration mainly caused by the temperature changes and the presence of moisture, as they result in changes in the volume of aggregates, and subsequently the breakage of the aggregate.

2.8.1.4. Soundness or Cleanliness

The aggregate soundness can be determined based on the amount of deleterious or foreign substances including clay lumps, weathered and weak materials, friable particles, soft particles, and organic matters. The aggregates containing these substances will provide less strong bond with binder resulting in ravelling and stripping problems in asphalt mixture. In addition, these materials are prone to disintegration under wetting and drying conditions or traffic loading.

2.8.1.5. Particle density

The particle density of the aggregates is an essential property of the aggregates which play a crucial role in the asphalt mix design procedure.

2.8.1.6. Absorption

The water absorption of aggregates demonstrates the pore structure of the aggregates and can be considered as an indication of porosity. The pore structure of aggregates reflects the volume, shape and size of the void spaces within a particle. The aggregates used in an asphalt mixture should be dense and of low porosity, as the large volume pores of make the aggregate more susceptible to degradation under repeated cycles of freezing and thawing. Moreover, in HMAs, a porous aggregate result in a dry and less cohesive asphalt mixture due to the high absorption of binder by aggregates.

2.8.1.7. Hardness or Abrasion Resistance

Aggregates are subjected to the abrasion and impact forces during production, compaction, and also under traffic loads. The abrasion resistance of aggregates is an essential characteristic in providing the proper particle interlock and skid resistance under traffic loads. Generally, the aggregates with higher degree of resistance to abrasion and impact forces is desired for most construction applications, and especially pavement construction, as the aggregates must have adequate resistance to the crushing and abrasion forces during manufacture, placing , and compaction of asphalt mixtures.

2.8.1.8. Resistance to Polishing

The aggregates polish under traffic load in various degrees depending on the aggregates type, road geometry and traffic conditions. This characteristic of aggregates influences the skid resistance of asphalt pavements. In HMAs, the friction characteristics or the degree of polishing is mainly affected by the coarse aggregates.

2.8.1.9. Strength

The strength property of aggregate is related to its resistance to degradation due to induced stresses in the pavement layers. Today, this characteristic of aggregates can be considered as an essential property in the assessment of asphalt since pavements are more subjected to high tyre pressures, high traffic intensity, or heavy wheel loads. In general, the strength and stiffness of aggregate are two important indicators used for the evaluation of aggregates in terms of their resistance to degradation. Strength is referred to the maximum compressive or tensile stress that aggregate particles can tolerate prior to failure, whereas is quantified by the modulus of elasticity and refers to the resistance of aggregate particles to deformation. It is important to utilize the aggregates with higher degree of stiffness in pavement construction, particularly in the upper pavement layers as the aggregate materials in these layers are subjected to higher stresses.

2.8.1.10. Adhesion to asphalt (affinity to bitumen)

The abrasion resistance of aggregates is an essential characteristic in providing the proper particle interlock and skid resistance under traffic loads. Generally, the aggregates with higher degree of resistance to abrasion and impact forces are desired for most construction applications, and especially pavement construction.

2.8.1.11. Aggregate Tests

Knowledge of the aggregate characteristics can help avoid the use of improper materials in asphalt mixture, and hence improve the asphalt pavement performance.

Table 2.7: Common Tests for Evaluation of Aggregate Properties (Austroads, 2014; AS 2758.5, 2009)

Test Category	Requirement	Test Name	Test Method	Property
Dimensional Requirements	**Grading Requirement**	**Dry Sieving Method**	AS 1141.11	▪ Particle size distribution ▪ Gradation
	Specification of Shape	**Particle Shape by proportional calliper**	AS 1141.14	▪ Proportion of misshapen particles
		Flakiness Index Test	AS 1141.15	▪ Particle shape ▪ Aggregate flakiness index
	Crushed Particles	**Crushed Particles of Coarse Aggregates**	AS 1141.18	▪ Proportion of crushed faces on an aggregate particle
Durability	**Durability Assessment**	**Wet Strength and Wet/Dry Strength Variation**	AS 1141.22	▪ wet/dry strength variation and wet strength
		Aggregate Crushing Value Test	AS 1141.21	▪ Strength
		Sodium Sulphate Soundness (SSS)	AS 1141.24	▪ Durability
		Los Angeles Abrasion Test	AS 1141.23	▪ Hardness and abrasion resistance of an aggregate
Weak Particles	**Proportion of Weak Particles**	**Friable Particle Test**	AS 1141.32	▪ Proportion of friable particles in coarse aggregate
		Unsound Particle Test	AS 1141.30	▪ Proportion of weak particles in coarse aggregate
Resistance to Stripping	**Resistance to Stripping Assessment**	**Resistance to Stripping of Cover Aggregates from Binders**	AS 1141.50	▪ Adhesion between the aggregates and bituminous binders
Frictional Characteristics	**Susceptibility of Aggregate to Polishing**	**Laboratory polishing of aggregates using the vertical road wheel machine**	AS 1141.40	▪ Loss of texture due to polishing of the aggregate under traffic ▪ Polished Aggregate Friction Value (PAFV), previously known as the polished stone value
		Laboratory polishing of aggregates using the horizontal bed machine	AS 1141.41	
		Pendulum Friction Test	AS 1141.42	
Particle Density	**Density Characteristics**	**Particle Density Test**	AS 1141.6 AS 1141.5	▪ Particle density
Water Absorption	**Water Absorption Characteristics**	**Water Absorption Test**	AS1141.6 AS 1141.5	▪ Water absorption of aggregates

In fact, the precise evaluation of aggregate properties is required for the selection of a suitable aggregate. This evaluation can be performed through a series of tests based on the relevant requirements of the applicable standards, as presented in Table 2.7.

Moreover, typical limits for some common tests regarding the evaluation of properties of coarse and fine aggregates are presented in Table 2.8 and Table 2.9, respectively.

Table 2.8: Typical Limits of Common Tests for Evaluation of Coarse Aggregate Properties (Austroads, 2014; AS 2758.5, 2009; RMS 3152, 2015)

Properties Type	Test Method	Typical Limit	Description
Particle Shape	AS 1141.14	10% (max)	Using a 3:1 calliper ratio
		35% (max)	Using a 2:1 calliper ratio
Flakiness Index	AS 1141.15	25% (max)	Heavy/very heavy traffic mix types
		35% (max)	Other mix types
Crushed Particles	AS 1141.18	75% (min)	By mass of particles with at least two crushed faces
Wet Strength	AS 1141.22	150 kN (min)	Heavy/very heavy traffic mix types
		100 kN (min)	Other mix types
Wet/Dry Strength Variation	AS 1141.22	35% (max)	Heavy/very heavy traffic mix types
		35% (max)	Other mix types
Weak Particles	AS 1141.32	1% (max)	For all mixes
Resistance to Stripping	AS 1141.50	10% (max)	For all mixes
Polished Aggregate Friction Value (PAFV)	AS 1141.42	48 (min) 44 (min)	Heavy/very heavy traffic mix types Wearing course All other courses
		45 (min)	Other mix types
Water Absorption	AS 1141.6	2% (max)	Heavy/very heavy traffic mix types
		2.5% (max)	Other mix types
Unsound Stone Content	AS 1141.30	3% (max)	Heavy/very heavy traffic mix types
		5% (max)	Other mix types
Marginal and Unsound Stone Content	AS 1141.30	8% (max)	Heavy/very heavy traffic mix types
		10% (max)	Other mix types

However, as a general rule, both coarse and fine aggregates must be dry, clean, hard, durable, tough, sound, and free from deleterious matter such as clay, dust and dirt.

Table 2.9: Typical Limits of the Common Tests for the Evaluation of Fine Aggregate Properties (Austroads, 2014; RMS 3152, 2015)

Properties Type	Test Method	Unit	Typical Limit	Description
Water Absorption	AS 1141.5	%	3 (max)	All mixes
Soundness	AS 1141.24	%	12 (max)	All mixes

2.8.2. Asphalt Filler

Fillers are usually defined as materials passing the sieve no. 200 (0.075mm sieve size). Fillers are used in asphalt mixtures to play two roles simultaneously in order to impart greater stability and strength. Firstly, in the asphalt mixtures, fillers basically fill up the voids between the aggregates, namely the coarse and fine aggregates, resulting in increase in the density and strength of compacted mixture. In addition, the fine filler particles form mastic by suspending in the binder and absorbing binder components resulting in an increase in the binder viscosity and subsequently, the asphalt mixtures toughness. There are various materials that can be used as fillers such as quarry dust, Portland cement, hydrated lime, etc. The effect of filler content and type of filler on asphalt mixtures characteristics have been investigated by many researchers.

Al-Suhaibani (1992) evaluated the effect of three different filler type (hydrated lime, Portland cement, and limestone dust) at different rate on properties of asphalt mixtures. The research results revealed that the amount and characteristics of the mineral fillers can have an effect on the rutting susceptibility of flexible pavements, and that the use of hydrated lime can improve resistance of the mixtures to rutting. In another research, Mohammad and Gokman (1998) studied the performance of hot asphalt mixtures containing hydrated lime as filler. The study showed that the application of hydrated lime in asphalt mixtures improves the performance of asphalt mixtures. Kavussi and Hicks (1997) performed a comprehensive study on the properties of asphalt mixtures containing different types of fillers. Their study revealed that the filler size directly influences the mastic viscosity. In addition, the type and content of filler substantially affects the flexural properties of asphalt mixture in such a way that an increase in the filler content will result in an increase in flexural stiffness of the asphalt mixture. However, the maximum toughness of asphalt mixture defines the optimum filler content in asphalt mixtures.

2.8.2.1. Effect of Filler Properties on Asphalt Performance

It is well recognized that the fillers play an important role in improving the performance of asphalt mixtures. Fillers are different in terms of their size distribution, mineral composition, surface area, particle shape, voids content, surface texture, and other physicochemical properties (Bahia et al. 2011), and hence they affect the properties of asphalt mixtures differently. For example, the fillers with larger surface area may absorb more binder and therefore their interaction with binder may result in different performance of asphalt mixture (Taylor 2007; Liao 2007; Lesueur 2009).

Referring to available literature (e.g. Anderson et al., 1992; Al-Suhaibani et al., 1992; Gorkem and Sengoz 2009; Geber and Gomze 2010; Wu et al. 2011), it is desirable to maximize the mastic viscosity particularly at higher field temperatures, as the mastic viscosity affects the pavement resistance to deformation. Generally, at a given temperature, the mastic viscosity can be increased either with an increase in filler content or application of an effective filler. However, there is a limit to the amount of filler that can be used in the asphalt mixtures as high amount of filler requires more binder for covering the surface area of filler particles. Moreover, it should be noted that different fillers have different effects on asphalt mixture performance, as some fillers are bitumen extenders (they increase the binder volume) whereas other fillers may influence the stiffening potential of binder. According to Austroads (2014), in DGA mixtures, the filler makes up 4% to 6% by mass of aggregates, whereas the ratio of filler to binder (by mass) is between 0.6 and 1.2.

2.8.2.2. Filler Tests

The standard test methods for evaluation of fillers in asphalt mixtures are listed in Table 2.10.

Table 2.10: Typical Limits of the Common Tests for the Evaluation of Filler Properties (AS 2150, 2005; RMS 3152, 2015)

Properties Type	Test Method	Unit	Typical Limit	Description
Voids Dry Compacted Filler	AS 1141.17	%	38 (min)	Medium, heavy and very heavy mix types
Grading	AS 1152	%	0.600 mm, 0.300 mm and 0.075 mm sieves	All mixes
Moisture Content	AS 4489.8.1	%	3 (max)	All mixes

In general, filler should be consistent in dry compacted air voids and mineral composition. Filler should be dry and free from clay, lumps and other deleterious material. Fillers should comply with their relevant standards as listed in Table 2.11.

Table 2.11: Standards for Materials Used as Filler (AS 2150, 2005; RMS 3152, 2015)

Properties Type	Test Method	Description
Hydrated Lime	AS 1672.1	-
Fly ash	AS 3582.1	-
Cement Kiln Dust	-	Cement kiln dust should be solid material extracted from flue gases in the manufacture of Portland cement, having a maximum water-soluble fraction of 20% by mass and grading

		of 0.600 mm sieve (100%), 0.300 mm sieve (95-100%) and 0.075 mm sieve (75-100%)
Slag	AS 3582.2	-
Ground Limestone	-	Ground limestone should consist of rock dust derived from grinding of sound limestone and must have grading of 0.600 mm sieve (100%), 0.300 mm sieve (95-100%) and 0.075 mm sieve (75-100%)
Cement	As 3972	-

2.8.3. Binder

Binder is a general term for the adhesive which is used in asphalt pavement, and it includes asphalt binders (or bitumen), multi-grade bitumen, and the modified binders. However, there are other kinds of binders which are used in pavement industry, namely asphalt emulsions, asphalt cutbacks and foamed asphalt.

Generally, in flexible pavements, binder is the component that gives the pavement its flexibility, binds the aggregates together, and provides waterproofing properties for the pavement. Therefore, it should function as a water resistance, thermoplastic, and viscoelastic adhesive, which can provide appropriate flexibility and ductility to the asphalt mixture. In addition, it should act as a strong adhesion to bind the aggregates in order to produce a cohesive asphalt mixture. Moreover, it must have adequate durability for providing proper resistance to weathering.

Due to crucial function of asphalt binder in pavement construction, and as this research covers the application of some waste materials as binder modifier in asphalt mixture, this subject will be further discussed in Chapter 9. However, the following sections provide an introduction to binder in asphalt mixtures.

2.8.3.1. Binder Types

Bitumen is a complex mixture of hydrocarbons of various molecular weight, which can be obtained from petroleum refining or natural deposits. However, the bitumen is mostly manufactured from crude oil through different processes, involving distillation, blowing, and blending. As a result of these processes, the components of crude oil which have the highest molecular weight and chemical complexity accumulate in bitumen.

The chemical composition of bitumen is extremely complex and it varies widely depending on the source of crude oil from which the bitumen is manufactured. However, as presented in Table 2.12, the bitumen obtained from different crude oils is composed of a large amount of organic compounds of hydrocarbons consisting of primarily hydrogen and carbon, and a small amount of other atomic particles, called heteroatoms, such as sulphur, oxygen, nitrogen, and metals. These heteroatoms will exist in bitumen in small quantities in comparison to hydrocarbons but they still affect the interactions among molecules and subsequently the bitumen properties. the source of crude oil affects the amount of metals such as vanadium, nickel, and iron and their distribution in bitumen.

Table 2.12: Elemental Composition of the Representative Petroleum Bitumen (Peterson, 1984)

Element	Unit	Mexican Blend	Arkansas – Lousiana Blend	Boscon Blend	California Blend
Carbon	%	83.7	85.78	82.90	86.77
Hydrogen	%	9.91	10.19	10.45	10.93
Nitrogen	%	0.28	0.26	0.78	1.10
Sulphur	%	5.25	3.41	5.43	0.99
Oxygen	%	0.77	0.36	0.29	0.20
Vanadium	ppm	180	7	1380	4
Nickel	ppm	22	0.4	109	6

In pavement construction, multi-grade bitumen and modified binders can be used in order to improve the performance of the asphalt mixture.

Multi-grade bitumen is a chemically modified bitumen that is less susceptible to the temperature changes than the conventional bitumen. Multi-grade bitumen improves deformation resistance of asphalt mixture at high service temperatures, while providing the appropriate flexibility and reduced stiffness at low service temperature because they involve the properties of a harder class of asphalt grade at high temperatures coupled with the properties of softer class of asphalt grade at low temperatures, as illustrated in Figure 2.13.

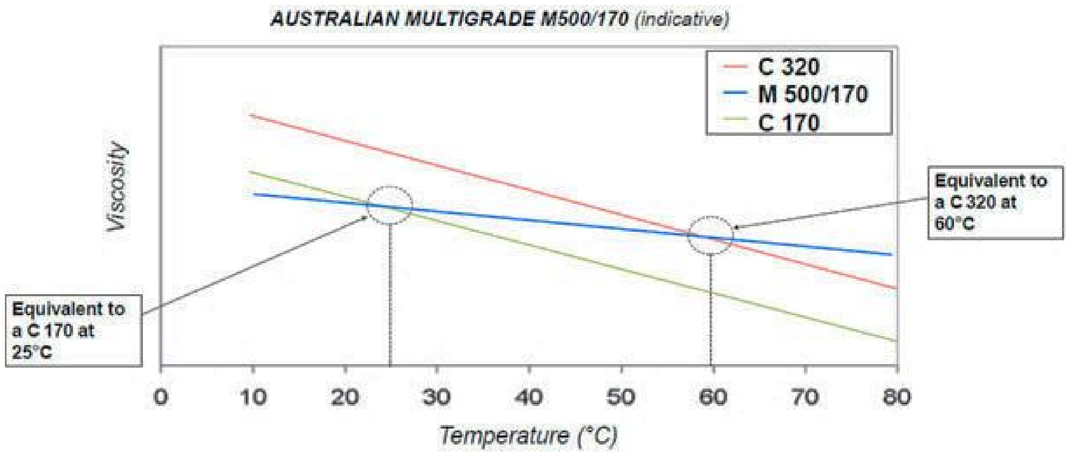

Figure 2.13: The Temperature Sensitivity of Multigrade Bitumen (M500) vs Conventional Bitumens (Ref: www.bitumina.co.uk)

Polymer modified binders are used with increasing frequency for the pavement construction, primarily due to their ability in improving the binder ductility and durability. To achieve this purpose, currently, there are a wide range of polymer types as well as synthetic polymers which are being successfully incorporated into conventional bitumen, in order to improve the cohesion, flexibility, and deformation resistance of asphalt mixtures at high temperatures. Due to the importance of the modified binders in this research, this topic will be further discussed in Chapter 9.

2.8.3.2. Binder Properties and Their Effects on Asphalt Performance

As previously stated, it has been well established that the physical and rheological properties of binder affect the pavement performance. Therefore, the key issue in determining fundamental binder properties is to evaluate the physical properties of binder. The most important physical properties of the binders are mainly classified as:

- **Rheological Characteristics:** This property which varies with temperature, describes the deformation and flow behaviour of the binder, as binder is a viscoelastic material which possesses viscous properties at high temperatures, and demonstrates elastic properties at low temperatures. The asphalt binders which are too viscous, will flow at high temperatures and cause pavement deformation. In contrast, those asphalt binders which are too stiff, become progressively stiffer and eventually brittle at low temperatures, providing asphalt which is susceptible to fatigue cracking. Therefore, rheological properties of the binder can be considered as an important characteristic which can directly affect the pavement performance.
- **Durability (Ageing):** Durability refers to the change in the binder properties due to influence of temperature and oxygen over time. In fact, during the ageing process, bitumen hardens, stiffens, and becomes more viscous. It will also become brittle, and loses their ductility and their adhesiveness. These changes are mainly caused by the influence of different mechanisms, including volatilization, oxidation, molecular restructuring over time, and ultraviolet light effects. the phenomenon of ageing (or durability) can be considered as one of the most important properties of the binder as the rheological properties of binder change during asphalt mixture production and continue to change during the time that pavement is in service,
- **Safety:** When binder is heated to a high enough temperature, the presence of an open flame or a spark will cause the release of vapour to ignite (flash). The flash point is the temperature to which binder can be safely heated without the danger of

instantaneous flash when exposed to an open flame, is called. In pavement construction, it is necessary to measure and control the flash point for safety considerations.

- **Purity:** Binders should consist of almost pure bitumen, as impurities may be detrimental to asphalt performance. There are some tests for the evaluation of the purity of binders utilized in asphalt mixtures.

2.8.3.3. Tests on Binder

The major part of the bitumen which is used in paving applications, is obtained from crude oil through different processes. As crude oils differ in their physical and chemical properties depending on their sources, the bitumen will consequently have different properties, which subsequently affect the asphalt mixture performance. Therefore, understanding the binder characteristics can play an important role in improving the asphalt pavement performance. In light of this, the binder properties can be evaluated through a series of tests, which are all discussed in Chapter 9.

2.9. Recycled Materials in Asphalt Surface Layer

Road networks are expanding worldwide during the past few decades to meet the increasing freight volumes created by the population growth and industrial development. Moreover, the maintenance requirements for existing road networks are increasing. Hence there is a great demand for efficient utilization of materials used for the construction as well as maintenance of road networks reducing the overall cost, especially natural materials such as aggregates and bitumen which is produced from petroleum.

At the same time, the rate at which solid waste is generated in the society is increasing with population growth, technological development, and changes in the people life style. Therefore, the management of solid wastes has become an acute problem. These challenges have attracted the attention of many researchers to investigate the effect of the incorporation of waste materials in road construction, particularly flexible pavements.

Among different layers of flexible pavement, asphalt surface layer should be able to withstand traffic loads and detrimental changing environmental conditions. Moreover, the asphalt surface layer is of high importance in safe and comfortable driving. Therefore, it plays a fundamental and crucial role in flexible pavement structure systems. Today, many waste materials such as tyres, plastics, waste glass, etc. are used for construction of different layers of pavements including asphalt surface layer. Utilization of solid wastes in asphalt layer not only reduces the adverse impacts of waste disposal but also the demand for natural and virgin asphalt binder as well as the coarse and fine aggregates which will subsequently results in cost savings and economical advantages. Moreover, using the recycled materials in asphalt surface layer can contribute to more improvement of engineering characteristics of the asphalt pavement materials as well as the pavement performance, representing a value add application for solid wastes. However, the selection of waste materials using for road construction, particularly the surface course is of high importance as the incorporation of wastes should not affect the functional and structural aspects of the pavements.

Therefore, referring to the importance of the application of waste materials in asphalt surface layer, this section presents an overview of a number of attempts which are made by researchers worldwide regarding the application of different wastes in surface layers of flexible asphalt pavements and their effect on the final performance of asphalt mixtures containing these recycled materials.

2.9.1. Recycled Materials as Aggregate

Since aggregates represent the major portion of the asphalt concrete, from the viewpoint of effective use of resources and environmental aspects, focus on the recycled aggregates can provide enormous benefits. Due to the importance of the aggregates in asphalt concrete, the studies on the utilization of the recycled aggregates have increased worldwide over the past two decades. Accordingly, this section presents a background of the literature of waste materials used as aggregate in the construction of asphalt concrete.

2.9.1.1. Recycled Construction Aggregates (RCA)

As world’s population grows, the demand for public infrastructure and hence the construction materials increases. Today, the demand for aggregates is much greater than the amount of virgin aggregates that could possibly be supplied. Referring to the Australian Quarrying Industry, the average annual consumption of aggregates across Australia is about 7 tonnes per person and will grow in the future with the population growth and infrastructure development. The biggest waste generator in Australia is the construction and demolition sector who is responsible for of generation of 19 million tonnes of construction and demolition wastes annually. This amount corresponds to about 40% of all Australian waste material (CCAA, 2008a). If this entire amount were treated as waste, at least 30 major landfill facilities were needed to operate all year round (Australia's Sustainable Aggregate Industry, 2013). Thankfully however, of the 160 million tonnes of annual aggregate demand, less than 10% is being covered by sustainable aggregate products that contain recycled materials.

In Australia, RCA has been used in construction projects as coarse and fine aggregates especially in recent years. RCA is available in Australian markets principally in Sydney and Melbourne. Figure 2.14 illustrates the sources of RCA in Australia, noting that Man Sand stands for "manufactured sand".

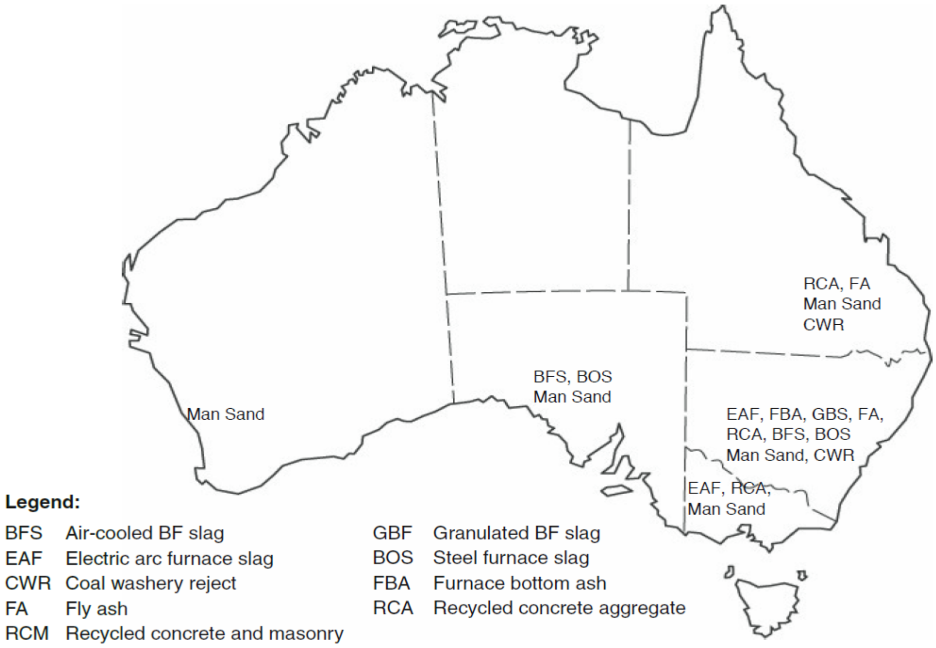

Figure 2.14: Sources of RCA in Australia (CCAA, 2008b)

Choosing RCA can be considered as a smart option because it reduces the need to excavate more virgin aggregates while extracting value from the waste materials which generally dispose. Despite the possibility that there is not any aggregate shortage in Australia, quarrying of virgin materials will still have a range of environmental and social issues. Based on a life cycle analysis (LCA) undertaken by RMIT University, the production of sustainable aggregates from RCA will have 65% less greenhouse emission in comparison to virgin aggregates production, mainly due to the less energy consumption. Accordingly, in utilizing the waste materials in construction, RCA can be considered as one potential material due to large amount of construction and demolition wastes. To this point, over the last two decades, several studies have been performed on the various applications of RCA. However, they are mostly focused on the utilization of RCA in concrete, road base and subbase layers. As the RCA properties are relatively different from virgin aggregates, extensive research is required to verify RCA suitability for utilization in asphalt mixtures. Therefore, it can be said that the application of RCA in asphalt mixtures is still a new field. In this regard, a significant amount of research (e.g. Guimarães and Ribeiro, 2005; Paranavithana and Mohajerani, 2006; Wong et al., 2007; Silva, 2009; Mills-Beale and You, 2010; Marinho, 2011; Chen et al., 2011; Pérez et al., 2012; Zhu et al., 2012; Zulkati et al., 2013) have been conducted to investigate the performance and properties of asphalt mixtures containing RCA. The results obtained from these studies are encouraging while indicate the feasibility of RCA utilization in asphalt mixtures.

Shen and Du (2005) evaluated RCA as aggregate in HMA. They used four aggregate types for their research project namely, 100% of virgin aggregates, 100% RCA, 50% coarse and fine RCA with 50% coarse and fine virgin aggregates, and coarse RCA with fine virgin aggregates. Based on test results, the mixture containing coarse RCA and fine virgin aggregates was identified as the best mixture.

Aljassar et al. (2005) investigated the utilization of RCA in asphalt mixtures in Kuwait with percentages of 40% (19mm RCA), and of 30% (10 mm RCA). The results showed that the asphalt mixtures containing RCA meet all the requirements of local specifications.

Wong et al. (2007) investigated the feasibility of partial replacement of virgin aggregate in asphalt mixture by RCA. Three asphalt mixtures were evaluated in this research by substituting granite fillers with 6% RCA filler, 45% untreated RCA with size of less than 3.15 mm, and 45% heat-treated RCA with size of less than 3.15 mm through the Marshall Mix design method. The result of this investigation showed the optimum binder contents of 5.3%, 6.5% and 7.0% of grade penetration 60/70 bitumen, respectively. All three mixtures satisfied the Marshall criteria of the Singapore specification for wearing course. The resilient modulus and creep resistance of mixtures with 6% RCA fillers were comparable to conventional mixtures, while mixtures with higher RCA content showed better performance. They found that RCA decreases the abrasion resistance of asphalt mixtures. The results suggested feasible use of RCA as partial aggregate substitution in HMA.

Paranavithana and Mohajerani (2006) investigated the effects of RCA on asphalt mixtures properties containing virgin aggregates as fine aggregate and RCA as coarse aggregate. The result of this investigation showed that asphalt mixtures containing RCA have quite lower values for all the volumetric properties (excluding the air voids), resilient modulus and creep values in comparison with control samples. In addition, the stripping potential of asphalt mixtures containing RCA was greater than conventional mixtures.

Mills-Beale and You (2010) conducted a research project to investigate the application of RCA in asphalt mixtures for roads with light traffic volume. In this study, virgin aggregates were substituted with RCA at different rate of 25%, 35%, 50% and 75%. In this investigation, Dynamic Modulus (E*), the rutting potential using Asphalt Pavement Analyser (APA), Indirect Tensile Test (IDT), Tensile Strength Ratio (TSR) and the Construction Energy Index (CEI) were evaluated using the Superpave mix performance specifications. All four mixtures had the minimum rutting specification. However, the dynamic modulus of mixtures containing RCA was lower than conventional asphalt mixtures. Moreover, these mixtures had more moisture susceptibility than the control mixtures. The mixture containing 75% RCA failed to meet the specification criterion, indicating that substitution of virgin aggregates with more than 75% RCA results in less resistance of asphalt mixtures to the rutting.

Cho et al. (2010) evaluated the performance of asphalt mixtures containing RCA through Marshall test, indirect tensile strength test, wheel tracking test, Kim test, and indirect tensile strength ratio test. Four types of asphalt mixtures were investigated in this research. The asphalt mixtures included control sample (Mix I), mixtures containing RCA as fine aggregates (Mix II), mixtures containing RCA as coarse aggregates (Mix III) and mixtures containing RCA as fine and coarse aggregate (Mix IV). The results of this investigation showed that Mix II and Mix III demonstrate adequate performance in terms of deformation resistance, whereas Mix IV containing coarse and fine RCA does not meet the requirements.

Rafi et al. (2011) conducted an experimental study on hot mix asphalt mixture containing RCA. Three RCA rates of 25%, 50%, and 75% were considered in this study to prepare asphalt mixtures using the Marshall method. Based on this investigation, all asphalt mixtures containing up to 50% RCA meet the requirements for wearing course in terms of optimum bitumen content, voids in mineral aggregates, stability and flow.

Al-Sarrag et al. (2014) investigated the viability of different percentage of RCA as coarse aggregate in hot mix asphalt. In this study, five mixtures containing RCA at different rates of 0%, 25%, 50%,75%, and 100% by weight of coarse aggregate were investigated to evaluate the effect of RCA on tensile strength, optimum bitumen content, stability, mixture resistance to plastic flow, and the stability loss. The result of experiments showed that the best content of RCA leading to the improvements in the stiffness, Marshall Stability, flow, minimization of stability loss and bulk density of asphalt mixture were 25% followed by 50% by weight of coarse aggregate.

Motter et al. (2014) evaluated the replacement of natural aggregate coarse fraction with RCA in HMA. In this research, coarse virgin aggregates were replaced by RCA at different percentages of 0%, 25%, 50%, 75%, and 100% to evaluate the volumetric and mechanical properties of asphalt mixtures. The results indicated that RCA can be used in asphalt mixtures for low traffic volume roads in spite of its higher Los Angeles abrasion. In addition, RCA has higher bitumen absorption compared to the virgin aggregates, which emphasizes seeking solutions for the binder consumption of mixtures containing RCA as coarse aggregate.

Heins (1986) found that asphalt mixtures containing RCA have stripping potential. Referring to Cross et al. (1996), some studies have showed that RCA particles tend more to break down during preparation and compaction of asphalt mixtures.

Referring to the previous discussions and available literature, it can be concluded that the utilization of RCA in asphalt mixtures influences the performance of asphalt mixture and leads to both advantages and disadvantages, as summarized in Table 2.13.

Table 2.13: Advantages and Disadvantages of Utilization of RCA in Asphalt Mixture

Advantage	**Description**
Increased stability	The angularity of RCA increases the structural stability of asphalt mixtures
Increased resistance to shear forces	Less flakiness index and misshapen particles in RCA can positively influence the inter-particle interlock in asphalt mixture, leading to improvement in shear resistance of asphalt mixtures containing RCA.
Enhanced skid resistance	The presence of both the igneous and metamorphic groups which are generally hard and prone to polishing as well as the existence of the sedimentary group and crushed brick which wear differentially and create an ever changing depth, improves the skid resistance of asphalt mixtures containing RCA.
Waste reduction	RCA utilization in asphalt mixtures offers environmental benefits and saving costs
No need to change asphalt production process	The same construction method used for conventional asphalt mixtures can be used for asphalt mixtures made with RCA
Commercial benefits	Utilization of RCA instead of virgin materials results in savings on costs of materials and construction costs
Availability and efficient supply of construction materials	The efficiency of the construction materials supply is highly determined by location. Availability of RCA as locally supplied construction materials ensures its affordable provision
Disadvantage	**Description**
Increased bitumen absorption	Porosity and high absorptive characteristics of RCA results in high bitumen absorption
Decreased resistance to freezing and thawing	Porous structure of RCA provides higher wet/dry strength variation, resulting in reduced resistance to freezing and thawing

2.9.1.2. Glass

Since glass waste is one of the most important and particularly troublesome components of solid waste because it can not be incinerated or degraded. Therefore, it is required to consider a proper approach for its management. Recycling is the most common method for handling the glass wastes. In fact, the recycling of glass can be done without any loss in the product quality. Recycling the glass wastes will result in substantial savings of energy as well as mineral resources. However, the variations in glass colour have encouraged the authorities to seek for alternative approaches to glass waste management.

Many countries such as United States, Japan, and several European nations have used the glass waste as a substitution for fine aggregate in asphalt mixture (Japan Asphalt Pavement Association, 1996). However, glass as aggregate in asphalt concrete should meet some technical specifications. Accordingly, many researchers (e.g. Issa, 2016; Androjic and Dimter, 2016; Dalloul, 2013) have studied the utilization of glass as aggregate for asphalt mixtures. Arabani and Azarhoosh (2011) studied the behaviour of asphalt mixtures containing glass (glassphalt) at different temperatures and by using different size of glass at different rates. The results of this investigation revealed that adding glass will improve the dynamic behaviour of and the stiffness of asphalt mixtures. In addition, asphalt mixtures containing glass have less temperature sensitivity compared to conventional mixtures. In another study by Jony et al. (2011), the effect of utilizing different fillers (including glass powder) at different rates in asphalt mixtures was investigated. The results of this investigation indicated that using glass powder as filler improves the Marshall Stability of asphalt mixtures in comparison with the asphalt mixtures made with Portland cement or limestone powder as filler. Pereira et al. (2010) conducted a research on the utilization of waste flat glass as filler in asphalt mixtures. This research concluded that waste glass can be effectively used as filler in asphalt mixtures. At another field study, two sections of road using two sizes of crushed glass were constructed in Minnesota. Referring to Marti et al. (2002), the results of rutting test on these roads revealed that the incorporation of waste glass with size of 9.5 mm in asphalt mixtures provides asphalt mixtures with less dynamic stability compared to asphalt mixtures containing waste glass with maximum size of 4.75 mm.

Another research by Arnold et al. (2008) showed that the addition of up to 30% glass waste by mass of aggregates will not significantly change the aggregates performance. Shafabakhsh and Sajed (2014) concluded that asphalt mixtures containing 10% to 15% crushed glass perform satisfactory. Finkle & Ksaibati (2007) reported that waste glass can be used as an alternative to the virgin road base materials. However, the glass content of up to 20% and maximum size of 12 mm was recommended based on this research. Wu et al. (2013) investigated the performance of asphalt mixtures containing waste glass as fine aggregate. The maximum size of 4.75mm and the optimum content of 10% were recommended by this research. In a report published by ARRB Group for the Packaging Stewardship Forum (PSF) of the Australian food and grocery council (2012), the glass content of up to 20% was recommended for the utilization in asphalt mixtures as fine aggregate. This report limits the utilization of waste glass to 30% by mass of the total fine aggregate in asphalt mixture.

Referring to Su and Chen (2002), in a research program in Taiwan, the engineering properties of the asphalt mixtures incorporating the crushed glass waste were studied through the laboratory and field tests. The result of this research revealed that the utilization of glass waste in asphalt mixtures provides substantial economical and engineering advantages. Pioneer Road Services carried out the first glass mix trials in Australia in 2003. The roads were compared with conventional asphalt roads for skid resistance properties. The investigation showed that the skid resistance of the asphalt mixtures containing glass waste is similar to conventional asphalt mixtures. Based on a research by Viswanathan (1996), the waste glass can be used in highway construction.

In general, glass is typically brittle and has very low impact resistance. This physical property of glass has been used positively in crushing the waste glass in desirable sizes with low energy consumption. Furthermore, glass shows high volumetric stability under high temperatures of up to 700°C. The thermal expansion coefficient and softening point of glass are in the range of 8.8 to 9.2 × 10^{-6} cm/cm/°C and 718 to 738 °C, respectively.

According to available literature, it can be concluded that application of glass in asphalt mixtures has both advantages and disadvantages, as summarized in Table 2.12, which introduces some limits on the utilization of glass in asphalt mixture as follows:

- The use of recycled glass is recommended to be limited to 20% as maximum replacement rate in asphalt mixture
- In case of glass content of more than 15% of the total mixture, it is required to add 1% to 2% antistripping agent to the asphalt mixture in order to avoid the stripping problems. Hydrated lime is an effective antistripping agent which can be used in asphalt mixtures containing glass.
- Suitable particle size of glass as aggregates in asphalt mixture is 4.75 mm or smaller.

Table 2.14: Advantages and Disadvantages of Utilization of Glass in Asphalt Mixture

Advantage	Description
Increased road safety	since the glass particles have low water absorption, the pavement surface gets dry faster after rain
Easier to compact and cartage over longer distance	Glass asphalt mixtures hold heat longer compared to conventional asphalt mixtures
Improved night time road visibility	Glass asphalt surfaces are more reflective in comparison with conventional asphalt surfaces
Waste reduction	Glass wastes are not disposed into the landfills offering environmental benefits and saving costs
Improved workability	The presence of long and flat particles will positively affect the workability of asphalt mixtures
Commercial benefits	Raw materials are replaced by glass resulting in savings on costs of materials
No need to change asphalt production process	The same construction method used for conventional asphalt mixtures can be used for asphalt mixtures containing glass
Improved resistance to thermal cracking	The small inflation coefficient of glass improves the thermal cracking resistance
Disadvantage	**Description**
Bleeding problem	Low bitumen absorption and density may cause bleeding problem
Stripping problem	Exceeding smooth surface of glass particles reduces the adhesion of asphalt film to the crushed glass, causing stripping the asphalt mixture.
Decreased transverse stability	The angularity and friction angle of glass particles provides inadequate transverse stability, particularly at braking or start-up
Sensitive to water damage	The high silica content in glass particles will make asphalt mixtures made with glass have more moisture sensitivity depending on the glass particle size or the glass content in asphalt mixture
Abrasion of tires	The presence of long and flat particles (particularly in case of large glass particles size) may result in abrasion of tires.
Decreased skid resistance	High amount of large size glass particles cause a decrease in skid resistance.

Through this information, it can be easily understood that the application of waste materials in asphalt mixtures directly affects the behaviour and performance of asphalt mixture and eventually the asphalt surface layer, leading to both advantages and disadvantages of overall asphalt mixture performance. Recognizing this fact, having knowledge about the properties of each individual component and their combination in asphalt mixtures will lead to the selection of most acceptable combination of aggregates for designing asphalt mixtures.

2.9.2. Recycled Materials as Binder Modifier

Enhancing the properties of asphalt components can result in increasing the quality of the asphalt concrete. One of the approaches to improve the asphalt mixtures quality is modifying bitumen through the utilization of different additives including the polymers. There are a large variety of polymers which are often used for modifying the binder. Modifying the binders will result in the enhancement of binders' properties, including elasticity increase, cohesion improvement, and temperature susceptibility reduction, which they all subsequently lead to the improvement of asphalt mixture performance in terms of flexibility, cohesion, and deformation resistance at high temperatures.

Although modifying the binders will result in the enhancement of binder's properties, but using virgin additives as modifier will increase road construction cost. Therefore, in recent years, many investigations have been conducted on modifying binders using waste materials as additives. Among these waste materials, application of plastic wastes in certain amount (about 5%-10% by weight of bitumen) and crumb rubber as binder modifier can substantially enhance the stability, strength, fatigue life and generally the asphalt performance on one hand (Verma, 2008), and on the other hand it would be an ideal solution for reducing the environmental pollution associated with these non-biodegradable wastes.

Incorporation of modifiers into asphalt pavements can be performed through two primary methods including the wet process and the dry process. In the wet process, the binder is thoroughly mixed with polymer modifiers at high temperature (177°C–210°C) at specific blending conditions to form a homogenous blend prior to adding to the hot aggregates. In wet process, the modifier is substituted as portion of the bitumen in mixture calculations and the interaction between the bitumen and polymer results in the production of a modified binder. This will result in increase of the melting point of binder, and therefore increasing the Marshall Stability by about two or three times compared to pure bitumen (Gawande et al., 2012). In the dry process, a portion of aggregates is replaced with polymers of similar size. In this process, the polymers are added to the aggregate before the addition of binder to the mixture. The aggregates when coated with plastics have better quality in terms of voids, moisture absorption, soundness, and generally binding properties. In a study by Vasudevan et al. (2007), various aggregates were coated with plastics and then moulded into a stable product. On cooling, they were tested for compression and bending strengths. The results of these tests showed that by increasing the plastic percentage, the values of the compression strength and bending strength increase which indicate that the plastics can be used as a binder. Based on this research, they concluded that the application of this process for construction of a single lane road of 1 km length and 3.375m width can consume 1,000,000 carry bags while increase the road strength by 100%.

This section focuses on the application of polyethylene and particularly HDPE and crumb rubber (recycled tyre) as binder modifier through wet process, and the main objective of this section is to summarize many of the most significant studies, including scientific papers, theses, and technical reports that have been published on polyethylene and rubber modified binders and asphalt mixes incorporating these wastes over the last decade in order to draw general conclusions regarding the present state of knowledge of modified binder rheology and hot mix asphalt performance.

2.9.2.1. Plastic

Plastics are durable and non-biodegradable wastes that may persist for hundreds or even a thousand years. The annual consumption of plastic materials has continued to rise for more than 50 years in all countries (PlasticsEurope, 2014). In 2013, the plastic production of 299 million tonnes was reported, representing a 3.9% increase compared to the plastic production in 2012 (UNEP, 2014) which implies that more plastic waste generation. Waste plastics are becoming a major concern in solid waste as they are one of the major constituents of municipal and commercial waste after food waste and paper waste.

In a broad classification based on physical properties, plastics can be divided into two main categories depending on their behaviour in exposure to heat. One type is called thermosetting (e.g. Polyurethane and Polyester) which after its formation, it cannot be redesigned by applying heat (i.e. it doesn’t withstand heat) due to the occurrence of an irreversible chemical reaction. The other type is thermoplastics (e.g. polyethylene) which can be shaped and redesigned by use of heat because they do not have any chemical change in their composition upon heating. Polyolefins are the largest class of thermoplastics made of a simple olefin or an alkene with the general formula of C_nH_{2n}. The two most important polyolefins are polyethylene (PE) and polypropylene (PP). Thermoplastics and thermosets constitute approximately 80% and 20% of total plastic wastes, respectively (Gawande et al., 2012). Thermosetting plastics are not used in pavement construction (Prasad, 2015).

Table 2.15: Plastics Classification based on the Society of Plastics Industry (SPI)

Code	Type	Application
1	**Polyethylene Terephthalate (PET)**	many common household items like beverage bottles, medicine jars, rope, clothing and carpet fibre
2	**High-Density Polyethylene (HDPE)**	containers for milk, motor oil, shampoos and conditioners, soap bottles, detergents, and bleaches
3	**Polyvinyl Chloride (PVC)**	all kinds of pipes and tiles, most commonly found in plumbing pipes
4	**Low-Density Polyethylene (LDPE)**	cling-film, sandwich bags, squeezable bottles, and plastic grocery bags
5	**Polypropylene (PP)**	lunch boxes, margarine containers, yogurt pots, syrup bottles, prescription bottles
6	**Polystyrene (PS)**	disposable coffee cups, plastic food boxes, plastic cutlery and packing foam
7	**miscellaneous types of plastic including Polycarbonate (PC) and Polylactide**	baby bottles, compact discs, and medical storage containers

In another classification, the Society of Plastics Industry (SPI) categorized all plastics based on their chemical properties into seven types of plastics with specific codes, as presented in Table 2.15. Many researchers have used some plastic types as modifiers to improve asphalt pavement performance. Some of these plastic types successfully improved the asphalt mixture behaviour. However, this study focused on polyethylene, as among all plastic types, polyethylene is the most common plastic in the world.

Table 2.16: Characteristics of Different Grades of Polyethylene (Awwad and Shbeeb, 2007)

Characteristics	Unit	Methods	LDPE	HDPE
Density	g/cm^3	D792	0.92	0.95
Water Absorption	%	D570	<0.01	0
Tensile Strength	psi	D638	1800 - 2200	4600
Tensile Elongation at Yield	%	D638	600	900
Flexural Modulus	psi	D790	-	200,000
Heat Deflection Temperature at 66 psi	°C	D648	48	76
Heat Deflection Temperature at 264 psi	°C	D648	36	40
Approximate Melting Temperature	°C	D3418	110	125
Max Operating Temperature	°C	-	71	82

Two main types of polyethylene grades are called low density polyethylene (LDPE) and high density polyethylene (HDPE). The main mechanical and physical properties of these types of polyethylene are presented in Table 2.16.

LDPE has adequate corrosion resistance as well as low moisture permeability. Accordingly, LDPE can be used in those projects where corrosion resistance is more important than other properties such as high temperatures, stiffness and structural strength. HDPE has high impact resistance and high tensile strength. HDPE has low water absorption and light weight.

Recognizing the importance of managing particular waste streams and building the capacity for the implementation of projects regarding the conversion of waste into value added materials, many researchers are investigating more environmentally friendly options for repurposing of plastic wastes, including polyethylene wastes. One viable economic solution for preventing vast quantities of plastic wastes from being landfilled or incinerated is using them in asphalt pavements through reliable methods.

Utilization of polymers from plastic wastes combine the advantages of producing better asphalt pavement as well as plastic waste management and reducing cost for asphalt industries. The addition of 8% by weight of processed plastic saves 0.4% bitumen by weight of mix (Justo and Veeravarana, 2002), which subsequently provides reduction in cost and resources demand. Among plastics, polyethylene forms the largest portion followed by PET. In light of this, this section has provided the reader with an overview of a number of case studies which were conducted by researchers to capture significant properties of incorporating recycled HDPE in asphalt binders.

Punith and Veeraragavan (2004) studied the behaviour of modified binder by preparation of samples containing different percentage of HDPE (0% to 12% by weight of bitumen at an increment of 2%) through wet process. They found that ductility, penetration and specific gravity values of the plastic modified binder decreased with the increase in the plastic content whereas the softening point increased. The results of penetration, softening point and ductility tests indicate that the stiffness of the binder increases with increasing quantity of polyethylene in binder.

Brozyna and Kowalski (2016) also investigated the effect of asphalt binder modification using 5% (by mass of bitumen) of different type of polyethylene type polymers, including HDPE, LDPE, linear low density polyethylene (LLDPE). A reduction in the penetration value and an increase in the softening point was observed for all samples. The highest increase of softening temperature was measured for LLDPE, which was followed by the HDPE.

Chandh and Akhila (2016) investigated the physical properties of the bitumen modified with the mixed plastic. The results of their research indicated that application of plastic waste as modifier in mixtures will improve the binder properties such as binding strength, moisture absorption, soundness, penetration, and softening point, resulting in stronger mixtures with higher stability value. It also slightly increases the flash and fire point of modified binder.

Prusty (2012) studied the behaviour of asphalt mixtures containing HDPE through the aggregate replacement process. It was observed that Marshall Stability value increased with increase of polyethylene value up to 4% and decreased thereafter. In addition, Marshall Flow values decreased upon the addition of HDPE. These results indicate that HDPE modified asphalt mixtures provide higher resistance to deformation under heavy wheel loads. The values of parameters of Voids in Mineral Aggregates (VMA), Air Voids (AV), and Voids filled with Binder (VFB) revealed that the polymer coated aggregates result in less void space in the mix, which subsequently lead to reduction in the mixture moisture absorption and bitumen oxidation by entrapped air and finally more stable and durable asphalt mixture.

Rahman and Wahab (2011) evaluated the effect of HDPE on the rheological properties of the modified binder. In their study, they determined the viscosity of 100/150 , 160/220 grade bitumen and three HDPE modified binder (1%, 3% and 5% by weight of binder) at temperatures ranging from 100 °C to 160°C. The results of this study showed a consistent increase in rotational viscosity with HDPE modification; where viscosity of 100/150 grade bitumen was as similar as 1% HDPE modified binder at 100°C and 3% HDPE modified binder at 160°C. In this research, the penetration and softening point values were also studied. Their findings also showed an increase in softening point and the decrease in penetration values of HDPE modified binder which indicates an increased hardness or stiffness of HDPE modified binders. Habib et al. (2010) studied the rheological properties of binder modified with HDPE, LLDPE, and PP at varying percentages (0.5%, 1%, 1.5%, 2%, 2.5%, 3%, and 5% by weight of bitumen) through empirical tests such as softening point, penetration and viscosity. It was observed that the increase in polymer concentration increases the softening point value and decreases the penetration value. However, the softening temperature for Polymer Modified Binders (PMB) containing up to 1.5% polymer does not significantly change since the binder modification incorporating thermoplastics does not substantially affect the softening point in comparison with the penetration (Read and Whiteoak, 2003). However, the increase in the amount of HDPE and LLDPE showed a rapid increase of softening point compared to PP. the test results revealed that in case of the incorporation of up to 3% polymer, binders modified with LLDPE and PP demonstrate the least variation in softening point and penetration compared to HDPE modified bitumen. This research concluded that the viscoelastic behaviour of bitumen is highly dependent on the penetration grade of the bitumen, polymer concentration and the temperature.

Hinislioglu et al. (2004) investigated the performance of HDPE modified binder under various mixing time, mixing temperature and HDPE content in terms of Marshall Stability, flow and Marshall Quotient (Stability to flow ratio). the powdered HDPE used at different rate of 4%, 6% and 8% by the weight of bitumen and at temperatures of 145°C, 155°C and 165°C considering different mixing time of 5min, 15min and 30 min for modification of binders. The results of this research showed that the specimens containing 4% HDPE prepared at 165 °C mixing temperature and with 30 min mixing time have the highest stability and the smallest flow. They explained that the stability reduction by increasing HDPE content may be as a result of the adhesion reduction. Moreover, the increase in the amount of HDPE has adverse impact on the interior friction of the mixture. In addition, since Marshall Quotient (MQ) is an indicator of the deformation resistance of the asphalt mixtures (Read and Whiteoak, 2003), so that stiffer mixtures have higher value of MQ and hence, more rutting resistant (NCAT, 1991). This research showed that specimens with 4% HDPE mixed at 165 °C for 30 min had the highest MQ. In another research, Awwad and Shbeeb (2007) investigated the utilization of two types of polyethylene (i.e. LDPE and HDPE) in asphalt mixtures through aggregate replacement process in order to determine the best type and optimum content of polyethylene for asphalt mixture modification. In this research, seven polyethylene content (6% to 18% by weight of binder at an increment of 2%), in two forms of grinded and ungrinded, were selected to be tested. The result of this investigation on the asphalt mixture performance containing different type and form of polyethylene indicated that the optimum modifier content was 12%. In addition, the test results revealed that HDPE modified binders improve the asphalt mixture properties much more than LDPE modified binders. Asphalt mixtures modified with optimum content of HDPE provide the highest stability and minimum air voids while satisfying the flow and VMA requirements. Data analysis also proved that asphalt mixtures containing modified binder made of grinded HDPE have better engineering properties compared to ungrinded HDPE modified binder.

Moatasim et al. (2011) investigated the viability of using HDPE (at different ratios by weight of bitumen) as a modifier for asphalt pavements through different tests including tensile strength, Marshall Stability, flexural strength, tensile strength ratio, Marshall Quotient and resilient modulus. The result of this investigation showed that the application of HDPE-modified binders in asphalt mixtures improves the performance of asphalt mixtures. The HDPE content of 5% (by weight of bitumen) was recommended by this research to improve the performance of asphalt mixtures.

Table 2.17: Advantages and Disadvantages of Utilization of HDPE in Asphalt Mixture

Advantage	Description
Improved road performance	increasing softening point and decreasing penetration results in improved hardness and stiffness of binder and subsequently improved performance of road
Improved fatigue cracking	Enhanced stiffness and the elastic behaviour of PE modified binders will improve resistance to fatigue cracking
Increased rutting resistance	Higher value of complex shear modulus and lower phase angle of modified asphalt with PE results in higher rutting parameter and hence improved rutting resistance
Environment Protection	Considering PE as binder modifier is an effective way of disposing of the increasing volume of non-biodegradable wastes, offering environmental benefits
Commercial benefits	Raw materials are replaced by plastic waste resulting in savings on costs of materials
Enhanced durability	Lower air voids in PE modified asphalt mixtures results in reduction in the mixture moisture absorption and hence more durable mixture
Improved strength characteristics	Higher bulk density and stability of asphalt mixtures modified with a certain amount of PE results in improved strength characteristics
Improved resistance to thermal cracking	The tensile strength of the asphalt mixtures at low temperatures increases by the incorporation of PE resulting in reducing the cracking potential of pavements at low temperatures.
Improved moisture susceptibility	The inclusion of PE increases the stripping inflection point increases the moisture susceptibility
Disadvantage	**Description**
Decreased stability	Increasing HDPE content results in the decrease in the adhesion and negative influence on the interior friction of the mixture, and hence decreased stability
Decreased Homogeneity	Increasing HDPE content beyond a certain amount increases the variation of softening point for the top and bottom layer, resulting in less homogeneity of mixture

From the available literature, it is concluded that the use of polyethylene results in improved engineering properties of bituminous mixes, as summarized in Table 2.17. However, most past studies prefer to use reasonable amount of polymer (1% to 5% by binder weight) as modifier optimum content, as the incorporation of PE in excess of this amount may adversely affect the engineering properties of asphalt mixtures.

2.9.2.2. Rubber

Two major methods of Crumb rubber manufacture are ambient and cryogenic grinding. The main difference between these two methods is the particle surface texture. Crumb rubber manufactured by the ambient processing have a rough texture with an irregular shape as a result of the tearing and shredding of rubber particles in the cracker mills, whereas the crumb rubber manufactured by the cryogenic method have smooth surfaces. The difference in the surface texture of rubber particles results in the higher surface area of ambient particles compared to cryogenic rubber particles (Putman, 2005).

Referring to Heitzman (1992), the utilization of crumb rubber modifier (CRM) in asphalt mixtures has a long history. CRM has used since 1840s when natural rubber was used in bitumen to increase its engineering performance. The recycled tyre has been used in asphalt mixtures by researchers and engineers since 1960s. The addition of rubber to asphalt binders improves some of binder’s properties.

Muthu (2013) studied the application of crumb rubber and LDPE through the laboratory investigation in a wet process, and as a function of percentages in asphalt mixtures for the evaluation of the performance of asphalt mixture. The results of this study indicated that the incorporation of higher percentages of rubber and LDPE up to a certain amount can provide a lower penetration value and higher viscosity in comparison with the standard bitumen. Moreover, the addition of this material will result in higher softening point, and subsequently higher resistance of binder to the effect of the temperature changes. According to this laboratory investigation, the modified binder will demonstrate a better resistance to rutting and temperature cracking.

Cooper et al. (2007) evaluated the field performance of rubber modified asphalt pavements for a long period of 10 years. Based on this investigation, the rubber modified asphalt pavements demonstrated an overall better field performance compared to the conventional asphalt pavements.

Huang et al. (2002) studied the performance of rubber modified asphalt mixtures through the Marshall Stability and flow tests and indirect tensile resilient modulus (MR) test. Huang et al also investigated the field performance of asphalt pavements in terms of fatigue cracking and rutting. The laboratory investigation showed higher strength for conventional asphalt mixtures compared to the CRM asphalt mixtures. However, the field study showed overall better performance for CRM asphalt pavement in comparison with conventional pavements.

Referring to Airey et al., 2003; Bahia and Davis, 1994; Green and Tolonen, 1977; Kim et al., 2001; and Zanzotto and Kennepohl, 1996, the addition of CRM to asphalt reduces the binder temperature susceptibility of the. When crumb rubber is used in binder, the rubber particles swell and form a viscous gel in the binder which increases the viscosity and stiffness of the binder. Sufficient interaction between crumb rubber and binder results in an adequate rubber modified binder with improved properties.

Lee et al. (2008) investigated the properties of binders modified with crumb rubber at different rate. In this study, the effect of rubber processing method on modified binder characteristics was also investigated. The results of this study showed that the binder viscosity, and the resistance to permanent deformation and low temperature cracking increases with the increase in crumb rubber percentage in modified binder. In addition, the crumb rubber produced through the ambient processing method provides more improvements in terms of viscosity, rutting resistance and cracking susceptibility in comparison with cryogenic processing method.

In another research by Bahia et al. (1998), it was found that the crumb rubber content in the modified binder highly affects the resistance of asphalt mixture to permanent deformation. In addition, Paulo and Jorge (2008) studied the effect of two blending times (30 minutes and 60 minutes) on the performance of rubber modified binders. Their research revealed that the blending time has significant effect on the permanent deformation resistance, so that longer blending time leads to more improvement in modified binder performance. Based on this research, it was concluded that the blending time of 30 minutes does not provide enough time for the complete reaction between the crumb rubber and binder.

Abdelrahman et al. (2013) reported that producing rubber modified binder at an intermediate temperature of 190°C using high shear mixer provides more homogenous modified binder and more improvement in performance of mixture at high temperature conditions.

Zumrawi (2017) studied the physical characteristics of modified binder containing crumb rubber at different rate of 0%, 5%, 10%, 15%, 20%, and 30%. The tests results showed that addition of crumb rubber to binder significantly increases the softening point, viscosity, flash and fire points of binder, while decreases the penetration and ductility of modified binder. Based on this investigation, addition of 15% crumb rubber to binder resulted in 15% increase in softening point and 200% increase in binder viscosity, coupled with 75% and 25% reduction in ductility and penetration, respectively.

Through available literature and information, it could be concluded that the modification of binders with crumb rubber, similar to other waste materials, has both advantages and disadvantages, as summarized in Table 2.18.

Table 2.18: Advantages and Disadvantages of Utilization of Rubber in Asphalt Mixture

Advantage	Description
Decreased temperature susceptibility	interaction between rubber and binders results in an increase in the viscosity and stiffness of the rubber modified binders, which subsequently leads to better resistance to temperature distresses
Increased rutting resistance	increasing softening point and decreasing penetration results in improved hardness and stiffness of binder and subsequently improved performance of road
Improved ageing performance	The significant carbon black content of rubber improves the ageing performance of binders modified with rubber
Improved fatigue cracking	Interaction between rubber and binder and high amount of carbon enhances stiffness and the elastic behaviour of rubber modified binders, which will subsequently results in improved resistance to fatigue cracking
Bleeding prevention	High binder viscosity prevents flushing, bleeding and drain-down
Improved road performance	Rubber increases both the high and low service temperature performance and hence results in asphalt enhancement
Environment Protection	Utilization of recycled rubber as binder modifier is an effective way of disposing of the increasing volume of non-biodegradable wastes, offering environmental benefits
Commercial benefits	Raw materials are replaced by recycled rubber resulting in savings on costs of materials
Noise reduction	Limited research has shown that the addition of rubber to binder results in noise reduction
Decreased water sensitivity	binders modified with rubber have greater cohesion and less water sensitivity
Disadvantage	**Description**
Challenging construction	Construction is challenging as temperature requirements are critical when using rubber modified binder
Decreased Homogeneity	Increasing rubber content beyond a certain amount increases the variation of softening point for the top and bottom layer, resulting in less homogeneity of mixture
Needing careful design	Due to diversity of rubber and their different compositions, the rubber modified binders must be properly designed and produced

2.10. Asphalt Pavement Distress Modes

The traffic loads on the pavement must transfer from the surface layer to base and sublayers and finally to the subgrade. Therefore, the main aim of the pavement design is that the pavement layers be able to transfer the load in a tolerable stress level at different seasonal environmental conditions. This role of pavement is called serviceability. The properties of the materials applied in pavement layers have a major effect on the effective stress distribution and pavement serviceability. However, a number of failures result in reduction of the pavement serviceability, of which the most important ones include rutting, fatigue cracking, thermal cracking, moisture damage, and ageing. Referring to a survey in Europe for determination of the most common type of pavement deterioration (European Commission, 1999), European countries were asked to rank the most common failure modes on their roads, as shown in Figure 2.15. To reduce these forms of deterioration, it is necessary to pay more attention to the selection of materials, properties of the asphalt mixture components, mix design and the effect of mixtures' volumetric properties on mixture performance. These issues are the main areas of this research work.

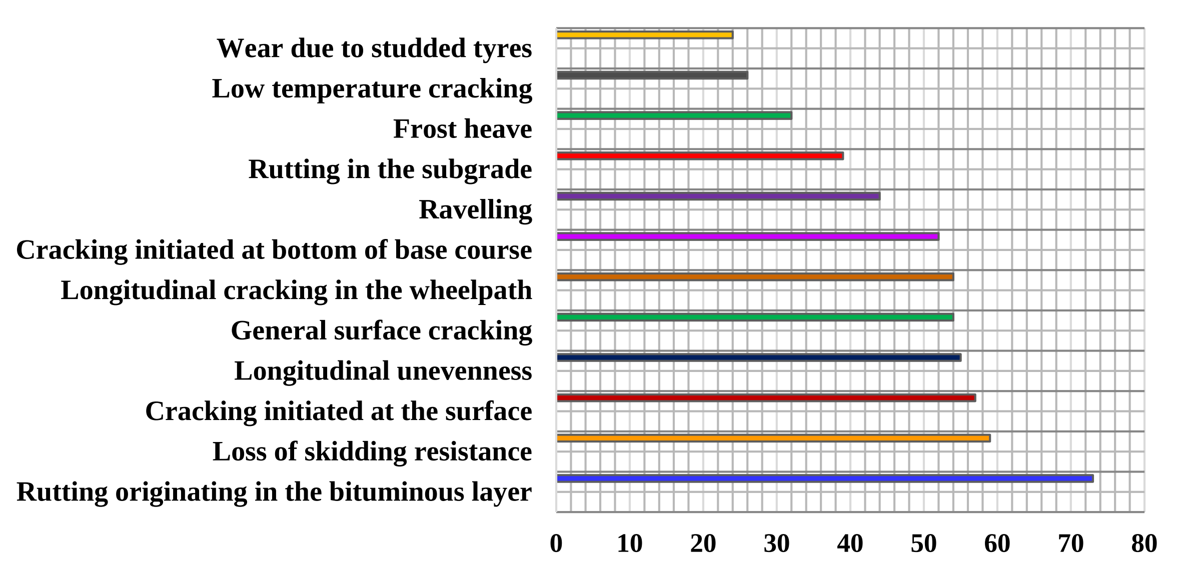

Figure 2.15: Rating of Observed Deterioration

2.10.1. Permanent Deformation and Shear-related Distress

Shear-related distress is one of the possible failures in asphalt surfaces. Shear distress occur as a result of shear stress from moving vehicles. Permanent deformation often occurs as a result of inadequate resistance of asphalt pavement to shear stress. However, in case of well construction of asphalt pavement, the high horizontal shear stresses result in horizontal creep without rutting (White, 2016).

Accumulation of the small irreversible deformation in the pavement is referred to as rutting (Illston and Domone, 2001) which depends on aggregate mechanical properties, aggregate gradation, viscoelastic characteristics of binder and the subgrade characteristics. Referring to Eisemann and Hilmar (1987), the asphalt pavement deformation phenomenon occurs in the following two major steps (as illustrated in Figure 2.16):

1) The first stage in which the pavement compaction under traffic is the main mechanism causing rutting. At this stage, the wheel loads cause irrecoverable deformation below the tyres which is greater than the upheaval zones increase.

2) The second stage in which shear deformation is the main reason for further rutting. In this stage, the volume increase in the upheaval zones is almost equal to the volume decrease below the tyres.

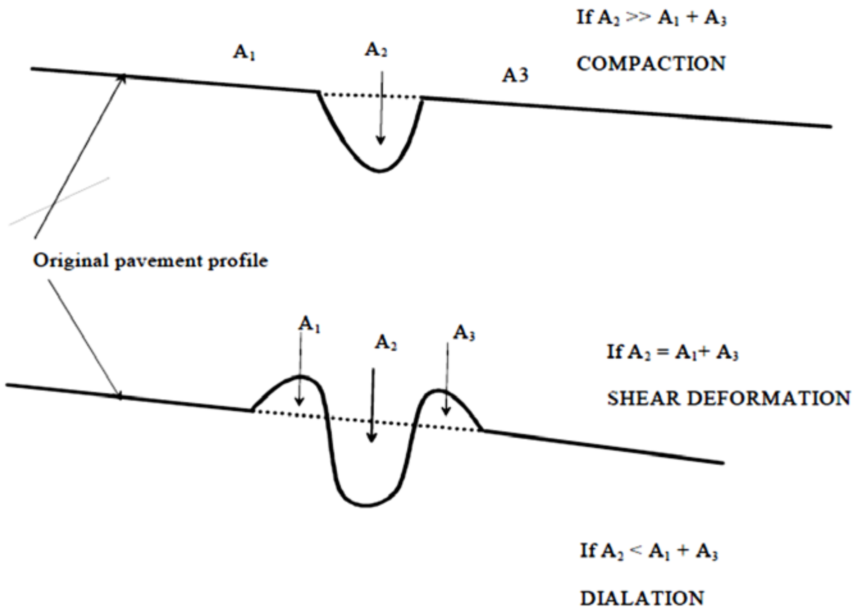

Figure 2.16: Illustration of the Rutting Mechanism (Eisemann and Hilmar, 1987)

There may be volume increase (dilation) in asphalt pavements under load due to the debonding between aggregate and binder which results in the pavement deterioration.

In general, permanent deformation in asphalt pavements can occur due to either accumulation of rutting in the asphalt surface layer or subgrade rutting. The subgrade rutting was considered as the main reason of pavement permanent deformation in the past. However, much research has shown that most of deformation occurs in the upper part of asphalt layer. According to Brown and Cross (1992), the majority of rutting occurs in the top 75mm to 100 mm of the asphalt layer and the subgrade deformation is very small in comparison to asphalt layer deformation. It should be noted that the permanent deformation is generally calculated by the sum of rutting in all layers.

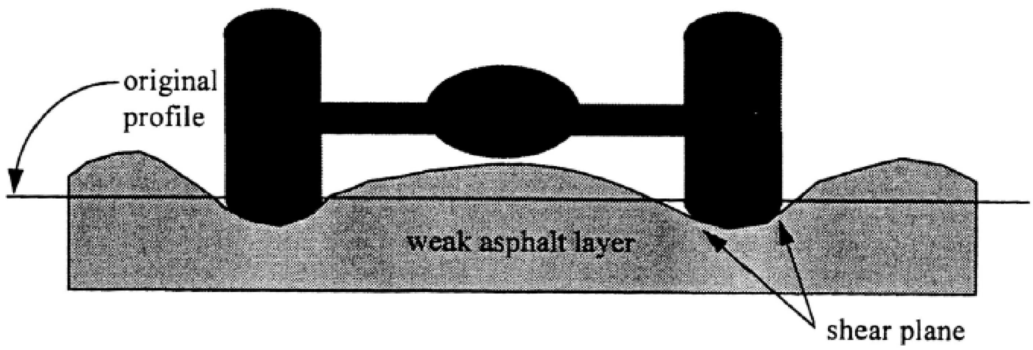

Figure 2.17: Rutting Caused by Weak Asphalt Layer (McGennis et al., 1995)

Inadequate shear strength of asphalt mixture against the repeated traffic loads is the main reason of rutting in asphalt layers, whereas insufficient thickness of pavement layers for reduction of the induced stress to a tolerable level as well as inadequate strength of subgrade materials are the main reasons of rutting in the subgrade. Accordingly, rutting in subgrade is named as structural rutting since it is mainly because of structural problem of the pavement than the materials problem. Figures 2.17 and 2.18 illustrate rutting from weak subgrade.

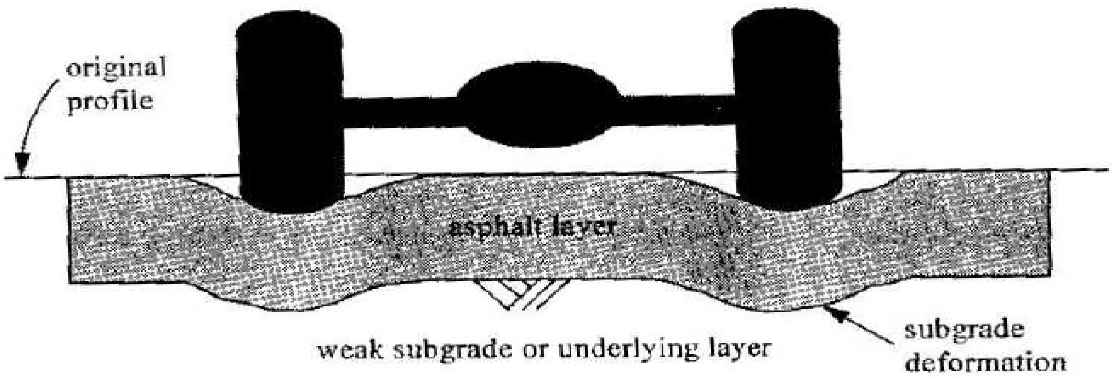

Figure 2.18: Rutting from the Weak Subgrade (McGennis et al., 1995)

2.10.2. Fatigue Cracking

Fatigue cracking like rutting occurs as a result of repeated heavy loads which exceed the pavement endurance limit. Fatigue cracking mostly happens at low or moderate pavement service temperatures due to the binder behaviour at these temperatures. At moderate service temperatures, binder is more brittle and stiff, and hence it is prone to cracking rather than deformation. Fatigue cracking in asphalt usually occurs when vertical compressive loads create horizontal compressive stresses on the top half of asphalt layers and horizontal tensile stresses on the lower half of the asphalt layer. Fatigue cracks are longitudinal cracks along the wheel path in early stages of formation. As they progress, they join together to form a pattern like alligator's skin.

In asphalt layers, cracking occurs due to the tensile strain in the layers as a result of traffic load. Therefore, the tensile strain magnitude and the number of load repetition play role in the occurrence of fatigue cracking. Cracking, in thinner pavements, commonly initiates at the bottom of the asphalt layer with the maximum tensile stress and strain, and then propagate upwards due to the repeated loading (Brown, 2000). In thicker pavements, cracking initiates at surface of asphalt layer and then propagate downward.

In addition, referring to NCHRP (2011), the binder stiffness and the pavement structure affect the pavement resistance to fatigue cracking. In asphalt layers with less than 76 mm thickness, the resistance to both top-down and bottom-up fatigue cracks decreases with an increase in the high temperature binder stiffness, whereas, the resistance to bottom-up fatigue cracks increases with an increase in the high temperature binder stiffness in asphalt layers with thickness of 127 mm or more. Thus, by selecting binder modifiers that act as an elastic and soft material at intermediate temperature, fatigue cracking can be enhanced (Bahia and Anderson, 1995).

2.10.3. Thermal Cracking

Thermal cracking, unlike rutting and fatigue cracking occurs because of adverse environmental conditions not traffic loading. Thermal cracking appears as transverse cracks spaced consistently. These cracking are perpendicular to the traffic direction and they are controlled by the environmental factors and binder properties at low temperature (NCHRP, 2011). Binder modifiers can significantly enhance the pavement resistance to thermal cracking, as some of them can soften the binder at lower temperatures (Bahia and Anderson, 1995).

2.10.4. Moisture Damage

Moisture damage or stripping is one of the main failures in pavements resulting from the lack of cohesion between binder and aggregates in asphalt mixture. Different types of additives are used to improve the adhesion between the binder and aggregates.

2.10.5. Binder Ageing

Asphalt binder behaviour can be altered during asphalt mixture production as well as during pavement's in-service stage due to environmental conditions and traffic loading. This alteration is called binder ageing and is believed to affect significantly the pavement performance. according to many references (e.g. Vallerga et al. 1957; Finn, 1967; Johansson, 1998), binder ageing is dependent on many factors such as mixture type, aggregate gradation, climatic conditions, air void content and its distribution in mixture. However, oxidation, exudation, volatilization, and physical hardening are identified as the main parameters affecting the binder ageing.

2.11. Summary of Literature Review

This chapter has provided a critical review on the previous studies about the utilization of some of the waste materials in asphalt mixtures. Due to the wide range of materials that fit into the definition of road pavement, this document has not been written to cover the supply of all types of materials available. Instead it provides the reader with a brief description of how a pavement is constructed, some details of the way in which materials are used in asphalt mixtures, the relevant tests commonly specified to provide confidence of material suitability for the intended application, and the result of experimental studies by many researchers to evaluate the properties of some selected waste materials (i.e. RCA, glass, plastic and rubber) as an alternative to virgin materials in asphalt mixture under different percentages and processes.

From the existing literature, in comparison to conventional asphalt mixtures containing the natural materials, the asphalt mixture design incorporating waste materials are complex and need extra attention. Therefore, the materials should be evaluated precisely in terms of their individual properties as well as their reaction in combination with the other materials. These evaluations include the assessment of the mechanical, physical, and rheological properties of the materials and also the performance behaviour of the final mixture consisted of these materials. Based on the research, it can be concluded that application of waste materials in asphalt mixtures has both advantages and disadvantages, which introduces some limits on the utilization of these materials in asphalt mixture as discussed in this chapter.

In addition, according to available literature, the major failure modes of asphalt mixtures were identified as permanent deformation, fatigue cracking, moisture damage, binder ageing and thermal cracking. As the application of waste materials in asphalt mixtures is relatively and conceptually new, it has a huge potential for further research and field applications.

Chapter 3

Design of New Asphalt Mixture

3.1. Introduction

The aim of pavement design is the selection of the most suitable composition and economical thickness which can provide a satisfactory level of service for the considered traffic. The design of any asphalt mixtures usually involves the selection of aggregates, bituminous materials and additives, asphalt mixture testing, and finally determining the optimum mix design which can provide the best performance in service. Therefore, this chapter presents the general methodology for designing a new asphalt mixture including the waste materials.

3.2. Selection of Appropriate Asphalt Mix Type

In accordance with AS 2150 (1995) for Hot Mix Asphalt (HMA), the asphalt mix types used in Australia include Dense Graded Asphalt (DGA), Stone Mastic Asphalt (SMA), Open Graded Asphalt (OGA), Fine Gap Graded Asphalt (FGGA). As discussed previously, the required aggregate and materials vary in different asphalt mix types and will depend on the performance criteria and traffic conditions and can be classified into the following categories:

- FGGA for residential functions
- DGA for highway construction
- OGA and SMA for special surfacing mixtures

As roads with traffic loading of greater than 2×10^7 equivalent standard axle (ESA) repetitions has been considered in this research, therefore, this study is more concentrated on DGA. For this type asphalt mix, the following performance requirements shall be considered:

- high stability and resistance to deformation
- optimum binder content to provide sufficient cohesion and resistance to fatigue

In designing a Dense Graded Asphalt, different considerations must be taken into account. These factors are the basis of asphalt mixture design process and can be listed as follows:

- Traffic level
- Nominal size
- Compaction level
- Aggregates
- Fillers
- Binder

Since these factors can be considered as primary parameters in asphalt mixture design based on detailed discussion in Chapter 2, an appropriate value has been considered for some of the above mentioned factors, as summarized in Table 3.1.

Table 3.1: Primary Parameters in Asphalt Mix Design

Factor	Considered Limit	Description
Traffic Level	Very Heavy	it is assumed that the proposed asphalt mixture must provide suitable structural performance for traffic loading of greater than 2×10^7 ESAs repetitions which is related to the traffic category of very heavy (Table 2.3).
Nominal Size	14 mm	For highway pavements based on detailed information in Table 2.4
Compaction Level	120 cycles	For heavy traffic level based on Table 2.6

In addition, the following sections discuss the materials selected in this research as aggregates, filler, and binder.

3.3. Selected Materials in This Research

The asphalt mixture design is a multi-step process consisting of the material selection and determination of the materials' proportions in order to provide an adequate performance under traffic loading and climate conditions. In this research, a dense graded asphalt mixture is composed of a blend of aggregates continuously graded from the maximum size, which is 14 mm in this study, through the filler particles which are finer than 0.075 mm.

Table 3.2: Materials Considered in Asphalt Mix Design

Component	Material
Coarse Aggregate	RCA , RAP , Basalt
Fine Aggregate	Glass, Basalt
Filler	Hydrated Lime, Cement Portland
Binder Modifier	Rubber, HDPE
Bitumen	C320

A certain amount of bitumen is then added to the mixture to make an effectively impervious mixture with acceptable elastic properties. Considered materials in this research are summarized in Table 3.2.

3.3.1. Selected Coarse Aggregates

In the present study, RCA, RAP, and basalt passing through 20 mm and retained on 4.75 mm I.S. sieve have been used throughout the experiments as coarse aggregates (Figure 3.1).

RAP material used in this research was stockpiled RAP collected from Boral Asphalt Plant (Prospect, NSW, Australia) which is generated from milling and being used in their asphalt projects. It was plant-screened material retained on 19 mm sieve size.

Figure 3.1: Used Coarse Aggregates Including RCA, Basalt, and RAP (from left to right)

The crushed virgin basalt aggregate was obtained from a local supplier. These virgin aggregates were transported from a local quarry (Nepean Quarries) in the vicinity of Sydney. In addition, RCA was collected from a local recycling centre called Revesby Recycling Centre (Revesby, NSW, Australia), a licensed waste facility and transfer station which accepts all construction and demolition waste from both the residential and commercial waste streams. In this centre, RCA is produced through the first sorting process for removing of contaminants such as wood, plastic, metal and glass, then crushing of construction wastes, and finally screening for removal of contaminants such as reinforcement, wood, plastics and gypsum.

3.3.2. Selected Fine Aggregates

Fine aggregates considered in this study are basalt and recycled glass (Figure 3.2). Recycled glass used in this research was clear crushed glass made from recycled glass and passed through 4.75 mm sieve size and was obtained from Schneppa Glass (Burwood, VIC, Australia).

Figure 3.2: Used Fine Aggregates Including Basalt, and Recycled Glass (from left to right)

Fine virgin aggregate is basalt passed through 4.75 mm I.S. sieve which was obtained from a local supplier.

3.3.3. Selected Filler

The fillers considered in this research are hydrated lime and Portland cement (Figure 3.3). Using the correct amount of hydrated lime, approximately 2% by weight, in mix designs will enhance the durability of mixtures and will minimize the problem of stripping, particularly in asphalt mixtures made with partial glass substitution.

Figure 3.3: Used Filler Including Hydrated Lime, and Portland Cement (from left to right)

3.3.4. Selected Binder and Binder Modifier

Bitumen is a visco-elastic material which performs as a binding agent to the aggregates and fillers in asphalt mixtures. Bitumen must be selected on the basis of different factors such as pavement performance requirements, climatic conditions, etc. in order to cause a proper adhesion between the particles and provide the impermeability while fills the voids.

According to the pavement requirements, the original bitumen studied in this research corresponds to C320, which is the most common binder for wearing courses subjected to heavy loading and/or in hot climates.

Figure 3.4: Used Binder Modifier Including HDPE, and Crumb Rubber (from left to right)

In addition, some materials can be used as binder modifier and additives for improving the binding property of the bitumen, and subsequently providing a pavement with higher quality and better performance characteristics. Binder modifiers investigated in this research are recycled HDPE and crumb rubber (Figure 3.4). Recycled HDPE was obtained from plastic recycling plant (International Moulded Plastics, Taren Point, NSW, Australia) and is in the granular form with the particle size of 2.36 mm. Crumb rubber used in this research was obtained from tyre recycling plant which processes the car tyres into crumb rubber through the ambient grinding method. The crumb rubber was provided in granules form.

3.4. Design Procedure for Dense Graded Mix

The mixture design aims to determine the amount of coarse aggregates, fine aggregates, filler and bitumen for production of a durable, workable and economical mixture.

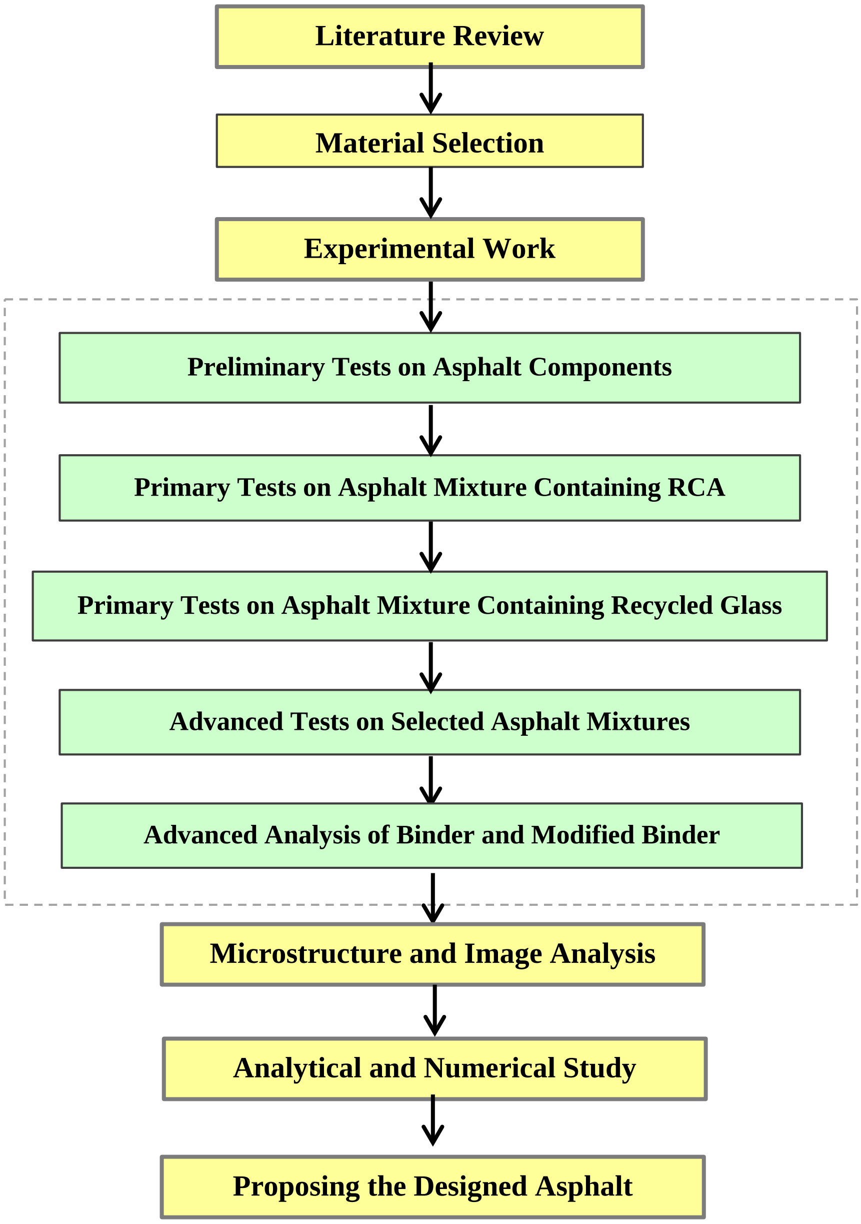

Figure 3.5: Research Methodology Flowchart

The mix design procedure in this study includes different parts mainly categorized as the experimental work to design an asphalt mixture incorporating waste materials and analytical and numerical study (Figure 3.5). Experimental work will be considered as the main target, which involves a comprehensive laboratory investigation at different levels including preliminary, primary, and advanced tests.

As it is almost impossible to design an efficient asphalt mix without having sufficient data and information on the engineering properties of the asphalt components, this research involves preliminary tests. The preliminary tests include the tests on asphalt components individually in order to understand their properties (Figure 3.6). These tests would be essential to enhance the overall properties of the final mixture through the selection of the most appropriate waste materials and the best combination of these materials.

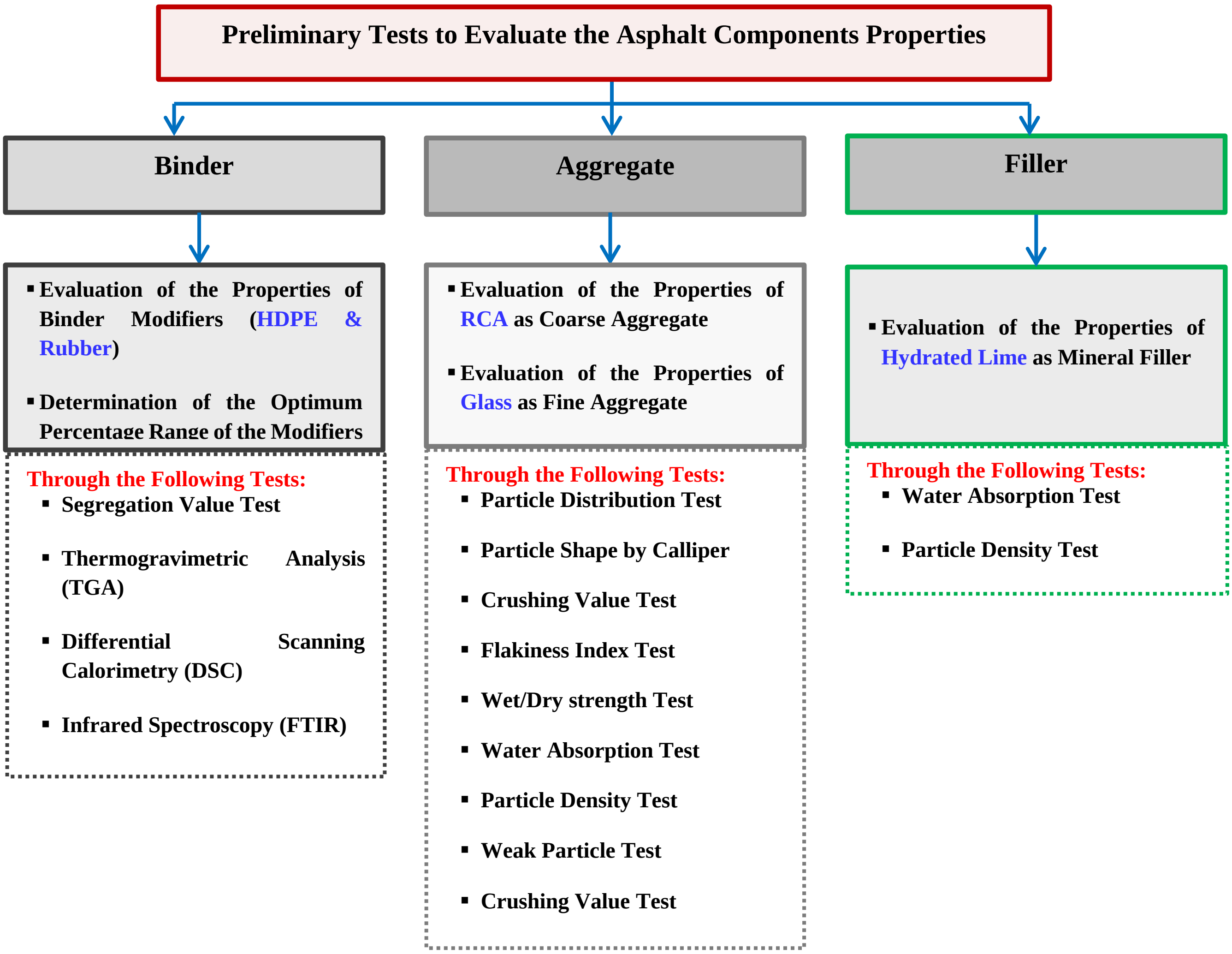

Figure 3.6: Flowchart of Preliminary Tests on Asphalt Components

Since RCA and glass are considered to be replaced as a certain percentage of coarse and fine aggregates, respectively, their mechanical and engineering properties would be evaluated through a range of preliminary tests including water absorption and particle density tests, wet/dry strength variation test, crushing value test, weak particles test, flakiness index test, particle shape test, particle size distribution, etc.

The primary tests are divided into two steps, as presented in Figure 3.7. First, a comprehensive primary test program considering variations of RCA in the asphalt mixture to illustrate the role of RCA variations in asphalt characteristics by comparing them with control samples. Then, a series of tests will be carried out to investigate the influence of glass on the properties of the asphalt mixtures incorporating RCA.

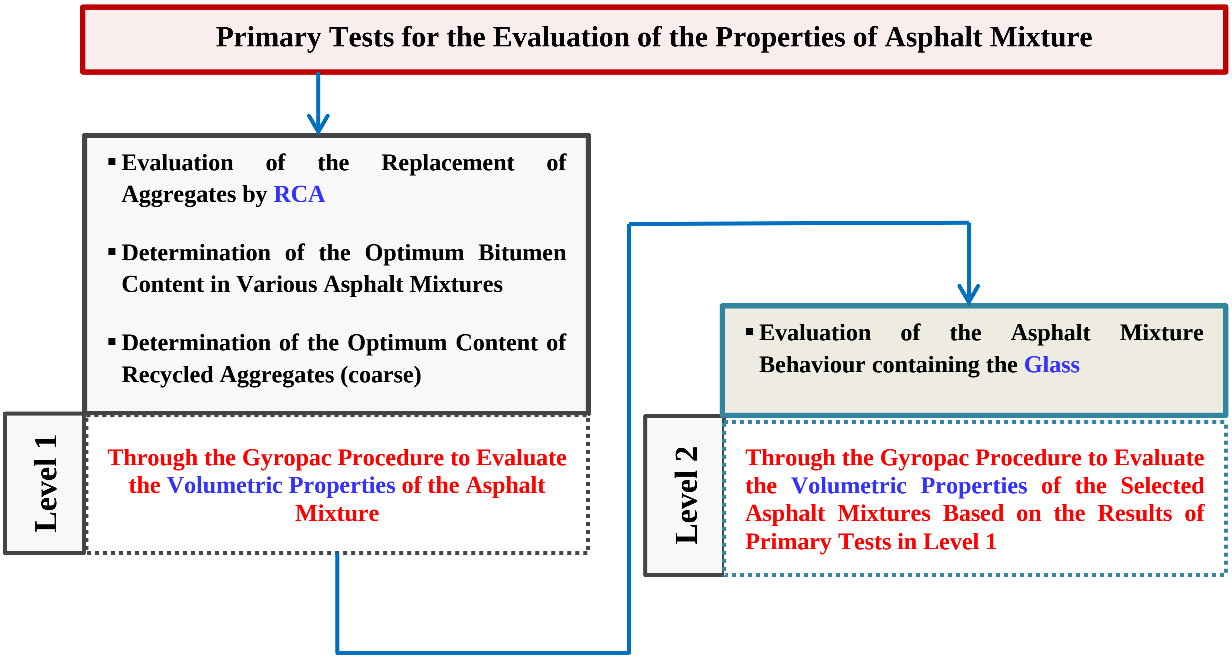

Figure 3.7: Flowchart of Primary Tests on Asphalt Mixtures

Following the preliminary and primary tests, the most appropriate samples will be selected for further investigation through Indirect Tensile Strength Test to determine their stiffness (Figure 3.8).

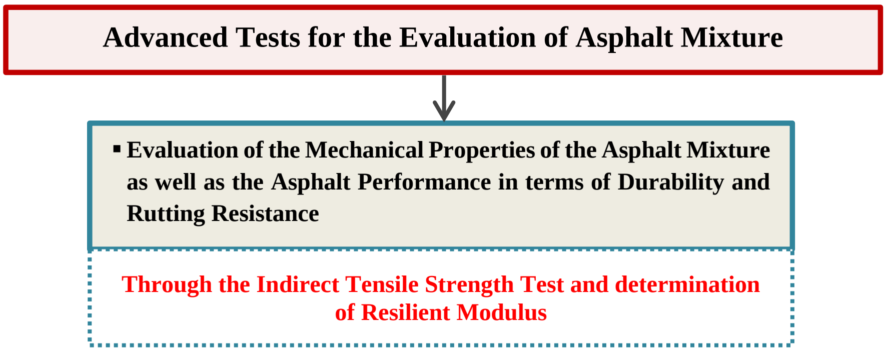

Figure 3.8: Flowchart of Advanced Tests on Asphalt Mixtures

According to procedure described above, for this research, a group of specimens were made without recycled materials (0%) as reference to specimens made with RCA and glass at different rates (Figure 3.9).

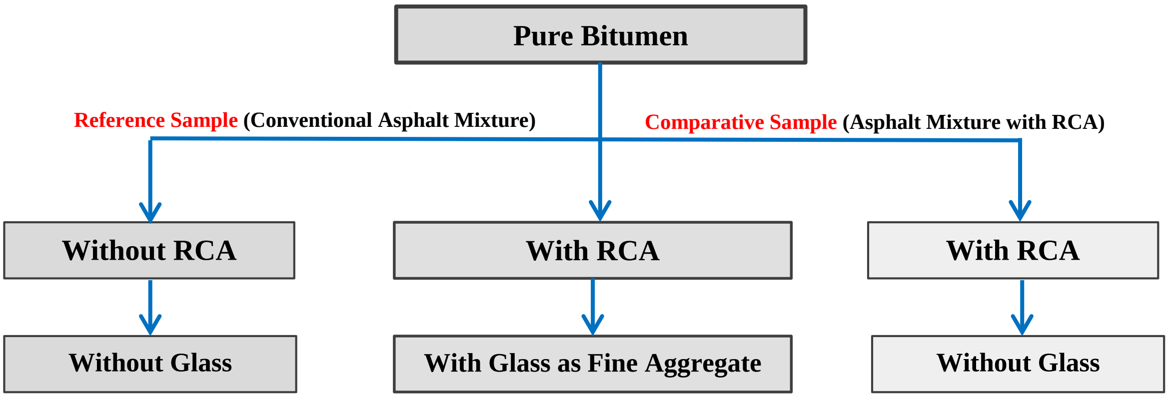

Figure 3.9: Flowchart of Primary Tests on Asphalt Mixtures

Therefore, 87 samples (29×3) of the following asphalt mixtures were prepared for this investigation through various tests including primary and advanced tests, as indicated in Table 3.3. The following chapters provide the results of the investigation based on this research methodology.

Table 3.3: Modes of Asphalt Mixtures Containing Different Percentages of Materials

Mode	Coarse Aggregate		Fine Aggregate		Filler		Bitumen
	RCA	Basalt	Glass	Basalt	Portland Cement	Hydrated Lime	
1	0	100	0	100	100	0	4.5
2	0	100	0	100	100	0	5.0
3	0	100	0	100	100	0	5.5
4	0	100	0	100	100	0	OBC[1]
5	25	75	0	100	60	40	5.0
6	25	75	0	100	60	40	5.5
7	25	75	0	100	60	40	6.0
8	25	75	0	100	60	40	OBC
9	25	75	10	90	60	40	5.0
10	25	75	10	90	60	40	5.5
11	25	75	10	90	60	40	6.0
12	25	75	10	90	60	40	OBC
13	25	75	20	80	60	40	5.0
14	25	75	20	80	60	40	5.5
15	25	75	20	80	60	40	6.0
16	25	75	20	80	60	40	OBC
17	50	50	0	100	60	40	5.0
18	50	50	0	100	60	40	5.5
19	50	50	0	100	60	40	6.0
20	50	50	0	100	60	40	6.5
21	50	50	0	100	60	40	OBC
22	50	50	10	90	60	40	5.0
23	50	50	10	90	60	40	5.5
24	50	50	10	90	60	40	6.0
25	50	50	10	90	60	40	OBC
26	50	50	20	80	60	40	5.0
27	50	50	20	80	60	40	5.5
28	50	50	20	80	60	40	6.0
29	50	50	20	80	60	40	OBC

[1] Optimum Bitumen Content

Chapter 4

Tests on Asphalt Components - Aggregates

4.1. Introduction

The high proportion of aggregate materials in volumetric design of asphalt mixes inherently links aggregate properties to the strength, stiffness, and generally the performance of the asphalt surface layer.

Because of the important impact of aggregate on the properties of asphalt mixture, a better understanding of the aggregates characteristics is essential in selecting the appropriate materials to optimize the asphalt mixture for strength and durability, and subsequently design a pavement with enough resistance to permanent deformation and cracking.

The most important physical and mechanical characteristics of aggregates include size and gradation, shape and angularity, surface texture, absorption, particle density, durability, toughness and hardness, resistance to polishing, soundness, cleanliness and the deleterious materials contained. Many research studies (e.g., Dahir, 1979; Brown et al., 1989; Brown and Bassett, 1990; Button et al., 1990; Elliot et al., 1991; Krutz and Sebaaly, 1993; Oduroh et al., 2000; Chen and Liao, 2002; Sengoz et al., 2014; Masad et al., 2009; Wu and King, 2011) have been conducted to link the properties of the aggregates to the performance of asphalt concrete pavement.

The results of these studies have shown that the physical and mechanical properties of the aggregates significantly affect the performance of the asphalt pavements.

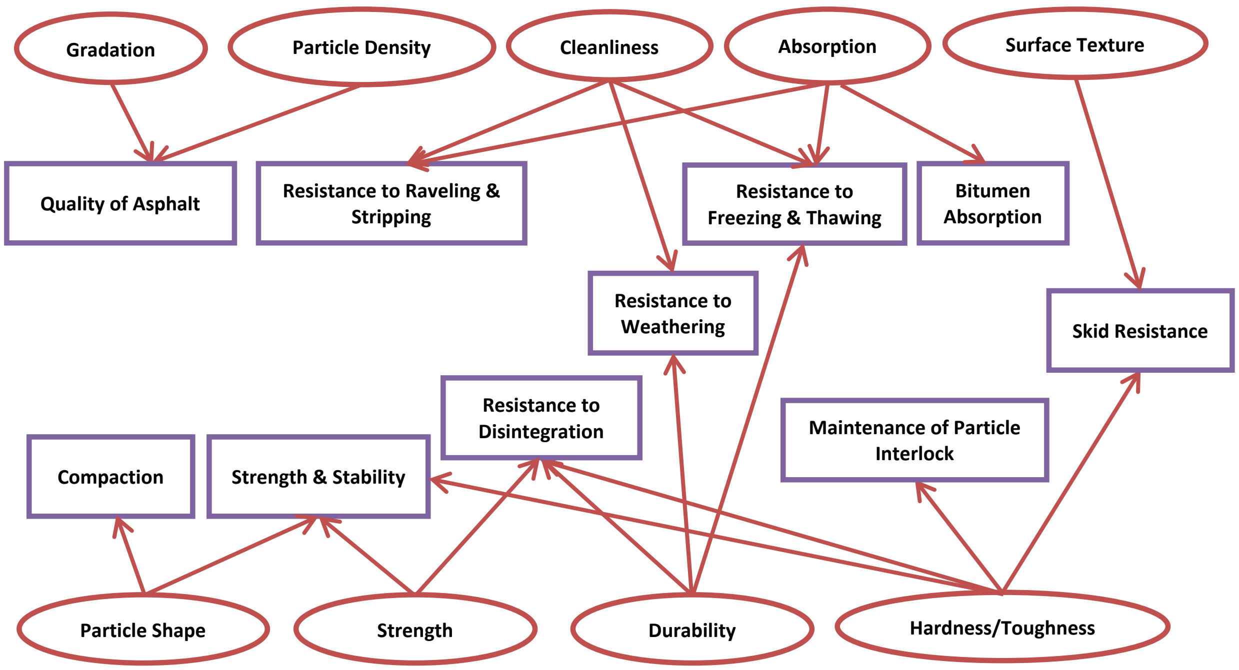

Figure 4.1: A Summary of the Effects of Aggregate Properties on the Asphalt Performance

Referring to the literature and the research conducted to relate aggregate properties and HMA performance, Figure 4.1 is generated to illustrate a generalized pattern and a summary of the effects of aggregate properties on the asphalt performance. The figure is the result of extensive literature review during the course of this research study and could be used by the practicing engineers as well as researchers to further improve their understanding of the effects of aggregate constituents on asphalt system performance. The reported relations and correlations shown in Figure 4.1 exemplify the complexities of mix design issues and considerations involved. This is certainly not unexpected considering the heterogeneity of the asphalt mixes. For example, as shown in this figure, different aggregate properties affect different aspects of asphalt mixture performance, which consequently define pavement service life. Accordingly, in order to design asphalt mixtures with longer service lives and lower production and maintenance costs, the aggregate must have appropriate characteristics.

Therefore, in this chapter and following chapter, the properties of the asphalt mixture components have been evaluated through the laboratory investigation and by conducting different tests on these components. Accordingly, the purpose of this chapter is to describe briefly the tests conducted on aggregates including coarse and fine aggregates and filler and to discuss the results of these tests.

4.2. Tests for Evaluation of Coarse Aggregates Properties

Aggregates constitute the largest part of bituminous materials, as they contribute up to 90% to 95% of the mixture weight. With such a big proportion in asphalt mixture, the aggregate would have important effects on the strength and stiffness of bituminous mixtures. In general, the aggregate in asphalt mixtures can be considered to perform the following functions:

- Supplying appropriate stability to the asphalt mixture through the proper interlock of aggregate particles
- Providing adequate skid resistance properties through the suitable surface texture and enough resistance of aggregates to polishing under traffic
- Producing a durable asphalt mixture with adequate abrasion resistance to withstand the environmental condition and traffic loading
- Spreading the wheel loads to the lower layers of the pavement

According to the important role of aggregates in the properties of asphalt mixture including the load bearing and strength characteristics of the mixture, there are some requirements for aggregates used in asphalt mixtures, which are addressed in available references (e.g.

Australian standards), as discussed in Chapter 2. To achieve these requirements, some tests have been performed on the aggregates in order to evaluate their properties.

This section reports the laboratory investigation on coarse aggregates (i.e. RCA, RAP and basalt) and fine aggregates (i.e. glass and basalt), in order to obtain comprehensive information and data of their properties and to compare these properties with the requirements specified in the standards as well as with the properties of the virgin aggregate. The key properties investigated in this experimental study are presented in Table 4.1.

Table 4.1: The Key Properties Investigated in the Experimental Study

Property	Test Method	Test Name
Gradation and particle size distribution	AS 1141.11.1	**Particle Size Distribution (Sieving Method)**
Flakiness index of aggregate	AS 1141.15	**Flakiness Index**
Proportion of misshapen particles	AS 1141.14	**Particle Shape by proportional calliper**
Water absorption of aggregate	AS1141.6.1	**Particle Density and Water Absorption of coarse aggregate**
Variation in strength of aggregate in wet/dry condition	AS 1141.22	**Wet/Dry Strength Variation**
Particle density of aggregate	AS 1141.6.1	**Particle Density and Water Absorption of coarse aggregate**
Strength and crushing value of aggregate	AS 1141.21	**Aggregate Crushing Value**
Percentage of weak particles in coarse aggregate	AS 1141.32	**Weak particles (including clay lumps, soft and friable particles) in coarse aggregates**

In addition, based on the test results on the individual aggregates, necessary tests were conducted on different combinations of these aggregates. The results of these tests are shown in the following sections. It should be noted that three samples were performed for each test and the average of the three samples was reported as the test result.

4.2.1. Particle Size Distribution (Sieve Analysis)

The gradation of aggregate to be used in asphalt mixtures are evaluated through Particle Size Distribution Test (Figure 4.2).

Figure 4.2: Aggregate Gradation by Particle Size Distribution Test

This test is conducted in accordance with AS 1141.11.1 (2009) and the gradation curves obtained from this test for different coarse aggregates considered in this research, including RCA, RAP and basalt, are shown in Figure 4.3.

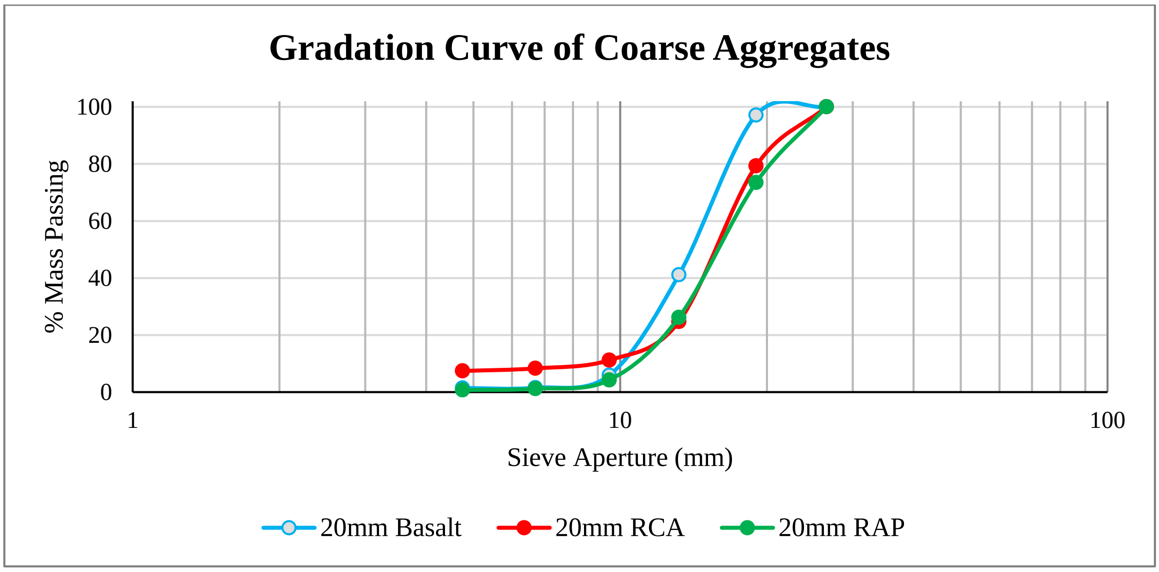

Figure 4.3: The Results of Particle Size Distribution Test for Coarse Aggregates

Moreover, the results of the particle distribution test are presented in Tables 4.2 to 4.4 for RCA, RAP and basalt, respectively.

Table 4.2: The Results of Particle Size Distribution Test for RCA (20 mm)

Sieve Size	Mass Passing for RCA (%)			Average Mass Passing for RCA (%)
	Sample 1	Sample 2	Sample 3	
26.5	100	100	100	100
19	79.6	79.9	78.3	79.3
13.2	25.8	26.6	21.8	24.7
9.5	10.7	12.3	10.6	11.2
6.7	8	8.7	8.3	8.3
4.75	7.2	7.5	7.6	7.4

Table 4.3: The Results of Particle Size Distribution Test for RAP (20 mm)

Sieve Size	Mass Passing for RAP (%)			Average Mass Passing for RAP (%)
	Sample 1	Sample 2	Sample 3	
26.5	100	100	100	100
19	73.1	76.5	70.7	73.4
13.2	19.7	29.0	29.9	26.2
9.5	3.2	4.6	5.0	4.2
6.7	0.9	1.2	1.5	1.2
4.75	0.6	0.7	0.9	0.7

Table 4.4: The Results of Particle Size Distribution Test for Basalt (20 mm)

Sieve Size	Mass Passing for Basalt (%)			Average Mass Passing for Basalt (%)
	Sample 1	Sample 2	Sample 3	
26.5	100	100	100	100
19	94.8	96.5	99.9	97.1
13.2	41	37.2	45.1	41.1
9.5	5.2	6.1	6.3	5.9
6.7	1.5	1.4	1.6	1.5
4.75	1.3	1.3	1.5	1.4

4.2.2. Particle Shape Test

The results of the studies on aggregate have shown that the aggregate physical shape properties significantly affect both the strength and stability of asphalt mixes (Kandhal and Cooley, 2001). Therefore, in order to design asphalt mixtures with long service lives, the aggregate must have the proper gradation and shape. The particle shape of aggregate substantially influences the mechanical stability of asphalt mix. The presence of excessive flaky and elongated particles is undesirable in asphalt mixtures as they tend to break down during the production and construction, and thus affect the durability of HMAs. Therefore, it is preferable to have rough and angular aggregates rather than smooth and round aggregates.

In this study, the proportion of misshapen aggregates, including the flat particles, elongated particles and, flat and elongated particles found in coarse aggregates is evaluated through the Particle Shape Test (Figures 4.4).

Figure 4.4: Classification of Coarse Aggregate Based on Particle Shape Test

The particle shape test is carried out by proportional calliper, using a 2:1 calliper ratio and based on AS 1141.14 (2007). The results of this test on three samples for each aggregate type (i.e. RCA, RAP and basalt) and the average values are given in Table 4.5.

Table 4.5: The Results of Particle Shape Test for Coarse Aggregates and Misshapen Percentage Limits for Dense Graded Asphalt Based on Australian Standards

Sample Number	Misshapen Particles (%)			Australian Standards Misshapen Percentage Limits (%)
	RCA	RAP	Basalt	
Sample 1	5.3	12.0	19.4	35% (max) For heavy and very heavy traffic
Sample 2	6.2	7.7	18.3	
Sample 3	7.0	8.7	17.3	
Average Misshapen Particles Percentage (%)	**6.2**	**9.5**	**18.3**	

As presented in Table 4.5, basalt materials show more of misshapen particles than RAP and RCA while still below the 35% limit of the Australian standard.

4.2.3. Flakiness Index Test

Some aggregates, on account of their shape, would be unsuitable for asphalt mixture as they would have low potential for developing inter-particle interlock. The percentage by mass of this type of aggregates, namely flaky aggregates is determined by the most commonly used test, called Flakiness Index Test (Figure 4.5). In this test, the flakiness index is determined by direct measurement using a special slotted sieve, from the ratio of the mass of material passing the slotted sieve to the total mass of the size fraction.

Figure 4.5: Conducting Flakiness Index Test for Coarse Aggregates

The flakiness index test is performed based on AS 1141.15 (1999) and the results of this test on three samples for each aggregate type (i.e. RCA, RAP and basalt) and the average flakiness index for each aggregate type are given in Table 4.6.

Table 4.6: The Results of Flakiness Index Test for Coarse Aggregates and Flakiness Index Limits for Dense Graded Asphalt Based on Australian Standards

Sample Number	Flakiness Index (%)			Australian Standards Flakiness Index Limits (%)
	RCA	RAP	Basalt	
Sample 1	5.6	12.8	21.3	25% (max) For heavy and very heavy traffic
Sample 2	8.1	10.1	21.1	
Sample 3	7.1	8.4	14.7	
Average Flakiness Index (%)	**6.9**	**10.4**	**19.0**	

The results of flakiness index test shows that RCA has less flakiness index than basalt and RAP which can positively affect the inter-particle interlock in asphalt mixture.

4.2.4. Particle Density and Water Absorption Test

The absorption is an indication of porosity in aggregate which demonstrates the pore structure of the aggregate. In asphalt mixtures, a porous aggregate increases the binder absorption, resulting in a dry and less cohesive asphalt mixture. In addition, the particle density of the aggregate is an essential property of the aggregate which plays an important role in the whole procedure of asphalt mix design.

Figure 4.6: Conducting Particle Density and Water Absorption Test on Coarse Aggregates

Therefore, in this research, the particle density and water absorption test is conducted on coarse aggregates (i.e. RCA, RAP and coarse basalt) based on the procedure described in AS 1141.6.1 (2000), as presented in Figure 4.6.

Table 4.7: The Results of Particle Density and Water Absorption Test on Coarse Aggregate and Water Absorption Limits for Dense Graded Asphalt Based on Australian Standards

Sample Name	Apparent Particle Density (gr/cm^3)	Particle Density on a Dry Basis (gr/cm^3)	Particle Density on a SSD Basis (gr/cm^3)	Water Absorption (%)	Australian Standards Limits for Water Absorption (%)
RCA 1	2.375	2.211	2.352	6.39	2 % (max) For heavy and very heavy traffic
RCA 2	2.375	2.211	2.352	6.39	
RCA 3	2.361	2.214	2.349	6.12	
Average Values for RCA	**2.370**	**2.212**	**2.351**	**6.30**	
RAP 1	2.539	2.422	2.468	1.89	
RAP 2	2.544	2.437	2.479	1.72	
RAP 3	2.540	2.433	2.475	1.73	
Average Values for RAP	**2.541**	**2.431**	**2.474**	**1.78**	
Basalt 1	2.635	2.528	2.568	1.60	
Basalt 2	2.630	2.521	2.562	1.64	
Basalt 3	2.654	2.542	2.584	1.67	
Average Values for Basalt	**2.640**	**2.530**	**2.571**	**1.64**	

In this test, the amount of water which a dried sample will absorb is measured. This test is performed on three trials and the related test results on RCA, RAP and basalt are given in Table 4.7, under apparent, dry, and saturated surface dry (SSD) conditions. The results of the particle density and water absorption test on different coarse aggregates (i.e., RCA, RAP and basalt) and their average value, as presented in Table 4.7, indicate the high absorption of RCA in comparison with RAP and basalt. The RCA water absorption exceeds the limit set by the Australian Standard.

As this research aims to investigate the feasibility of the application of RCA as a recycled material for potential partial replacement of coarse virgin aggregates (basalt) in asphalt mixtures, the particle density and water absorption test is also conducted on the following mix of coarse aggregates (i.e. RCA, RAP and coarse basalt) considering different percentages of these materials (Figure 4.7):

- Mix of basalt (75%) and RCA (25%)
- Mix of basalt (50%) and RCA (50%)
- Mix of basalt (25%) and RCA (75%)
- Mix of basalt (80%) and RAP (20%)
- Mix of basalt (25%), RAP (50%) and RCA (25%)
- Mix of basalt (25%), RAP (25%) and RCA (50%)

Such undertaking was needed in order to get a better understanding of an acceptable range of mix proportions in terms of water absorption.

Figure 4.7: Particle Density and Water Absorption Test on Mix of Coarse Aggregates

The results of Particle Density and Water Absorption test on six different mixes of RCA, RAP and basalt are given in Table 4.8.

It should be noted that despite the fact that above mixes (except when there is no RCA) have water absorption of more than 2%, the use of RCA is still a viable option as discussed in previous Sections.

Table 4.8: The Results of Particle Density and Water Absorption Test for the Mix of Coarse Aggregates

Sample Name		Apparent Particle Density (gr/cm^3)	Particle Density on a Dry Basis (gr/cm^3)	Particle Density on a SSD Basis (gr/cm^3)	Water Absorption (%)
75% Basalt & 25% RCA	1	2.579	2.394	2.466	2.98
	2	2.589	2.406	2.476	2.94
	3	2.601	2.420	2.489	2.88
Average Values for 75% Basalt & 25% RCA		**2.590**	**2.407**	**2.477**	**2.93**
50% Basalt & 50% RCA	1	2.520	2.296	2.385	3.86
	2	2.535	2.322	2.406	3.62
	3	2.527	2.313	2.397	3.65
Average Values for 50% Basalt & 50% RCA		**2.527**	**2.310**	**2.396**	**3.71**
25% Basalt & 75% RCA	1	2.477	2.224	2.326	4.57
	2	2.471	2.207	2.313	4.84
	3	2.480	2.234	2.333	4.44
Average Values for 25% Basalt & 75% RCA		**2.476**	**2.222**	**2.324**	**4.62**
80% Basalt & 20% RAP	1	2.727	2.607	2.651	1.68
	2	2.719	2.600	2.644	1.68
	3	2.724	2.596	2.643	1.80
Average Values for 80% Basalt & 20% RAP		**2.723**	**2.601**	**2.646**	**1.72**
25% Basalt & 25% RCA & 50% RAP	1	2.579	2.411	2.476	2.70
	2	2.581	2.401	2.471	2.89
	3	2.585	2.412	2.479	2.78
Average Values for 25% Basalt & 25% RCA & 50% RAP		**2.582**	**2.408**	**2.475**	**2.79**
25% Basalt & 50% RCA & 25% RAP	1	2.606	2.380	2.466	3.63
	2	2.592	2.356	2.447	3.85
	3	2.596	2.355	2.447	3.94
Average Values for 25% Basalt & 50% RCA & 25% RAP		**2.598**	**2.364**	**2.453**	**3.81**

4.2.5. Crushing Value Test

Aggregates used in road construction should be strong enough to resist crushing under traffic wheel loads (Mohajerani, 1997). The strength of the coarse aggregates can be evaluated by the Aggregate Crushing Value Test. In this test, the aggregate were crushed by a compression testing machine with a load rate of 40 kN/min to reach the peak load of 400 kN. The percentage of particles produced when the aggregate is crushed under this load and which pass a 2.36 mm sieve is called Aggregate Crushing Value.

The aggregate crushing value provides a relative measure of resistance to crushing under a gradually applied compressive load. To achieve a high quality pavement, it is preferred to utilize the aggregate possessing low crushing value.

Figure 4.8: Conducting Crushing Value Test for Coarse Aggregates

In this research, the crushing value of RCA, RAP and basalt is assessed through the Aggregate Crushing Value Test in accordance with AS 1141.21 (1997), as presented in Figure 4.8.

Table 4.9: The Results of Aggregate Crushing Value Test for Coarse Aggregates and Crushing Value Limits for Dense Graded Asphalt Based on Australian Standards

Sample Number	Crushing Value (%)			Australian Standards Aggregate Crushing Value Limits (%)
	RCA	RAP	Basalt	
Sample 1	29.53	7.04	9.32	35% (max) For heavy and very heavy traffic
Sample 2	28.88	7.76	8.51	
Average Crushing Value (%)	**29.21**	**7.40**	**8.91**	

This test was performed in two trials, as required in the standard, and the related test results on RCA, RAP and basalt and the average crushing value for each aggregate type are given in Table 4.9.

4.2.6. Weak Particle Test

The aggregate cleanliness refers to the presence of foreign or deleterious substances such as soft particles, weak and weathered materials, friable particles, clay lumps, and organic matters. The presence of these materials in the used aggregate can lead to stripping and ravelling in HMAs, as these materials adversely affect the bond between the aggregate and asphalt, and subsequently the stability of the pavement structure. Moreover, these substances disintegrate under traffic loading and wetting and drying cycles.

Figure 4.9: Conducting Weak Particle Test for Coarse Aggregates

The cleanliness of aggregate can be evaluated based on the Weak Particles Test. In this test, the percentage of weak particles in coarse aggregate is determined. These particles will deform under finger pressures when wet. In this study, the percentage of weak particles in RCA, RAP and basalt are determined through the Weak Particle Test in accordance with AS 1141.32 (2008).

Table 4.10: The Results of Weak Particle Test for Coarse Aggregate and Weak Particles Percentage Limits for Dense Graded Asphalt Based on Australian Standards

Sample Number	Weak Particles (%)			Australian Standards Aggregate Crushing Value Limits (%)
	RCA	RAP	Basalt	
Sample 1	0.21	0.05	0.29	1% (max) For heavy and very heavy traffic
Sample 2	0.25	0.03	0.16	
Average Weak Particles (%)	**0.23**	**0.04**	**0.23**	

The weak particle test is conducted on two samples, as specified in the related standard, and the results of this test on RCA, RAP and basalt and the average weak particle percentage for each type of aggregate are presented in Table 4.10. The test results show that RCA and basalt have higher percentage of weak particles. However, all aggregates still meet the Standard’s requirements.

4.2.7. Wet/Dry Strength Variation Test

Strength is an important aggregate property which is related to the satisfactory resistance to crushing under the roller during construction, and adequate resistance to surface abrasion under traffic (Prowell et al., 2005). Therefore, aggregates used in pavement construction should be strong enough to resist crushing during mixing, laying process, compaction, consolidation and during its service life period when they are subjected to various loads applied by traffic (Dickinson, 1984). In this research, the variation in strength of aggregate is evaluated by conducting the Wet-Dry Strength Variation Test on RCA, RAP and basalt in accordance with AS 1141.22 (2008), as shown in Figure 4.10.

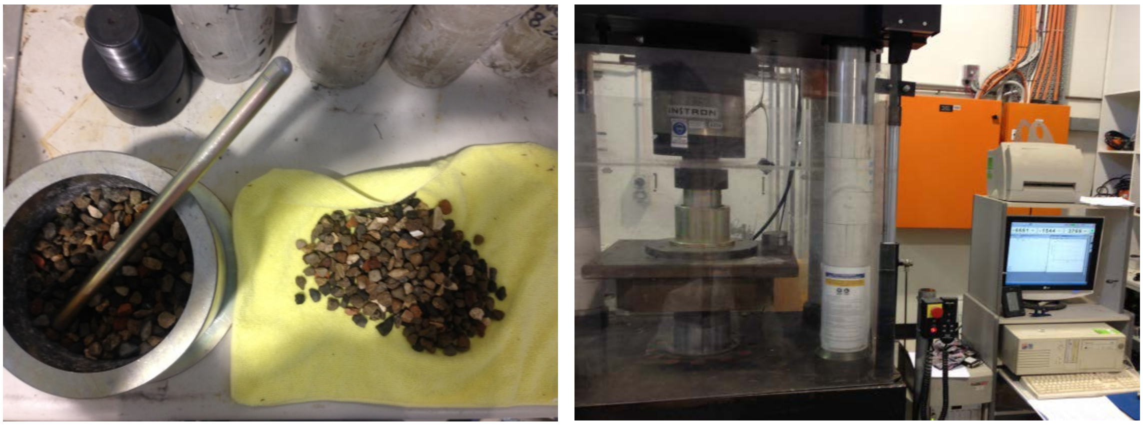

Figure 4.10: Conducting Wet/Dry Strength Variation Test on Coarse Aggregates

This test determines the variation in strength of the aggregates tested after drying in an oven and then saturated yet with a dry surface. Based on the available standards, the wet/dry strength variation of less than 35% indicate a durable material but values as high as 60% could be used in undemanding circumstances.

In this research, the wet/dry strength variation test was conducted on the RCA, RAP and basalt fraction passed through 13.2 mm and retained on 9.5 mm I.S sieve. Different loading was used in order to adjust the applied load for providing the fines within the range of 7.5% and 12.5%. The results of these tests for coarse aggregates are illustrated in Figures 4.11 and 4.12 under dry condition and saturated surface dry condition (SSD), respectively.

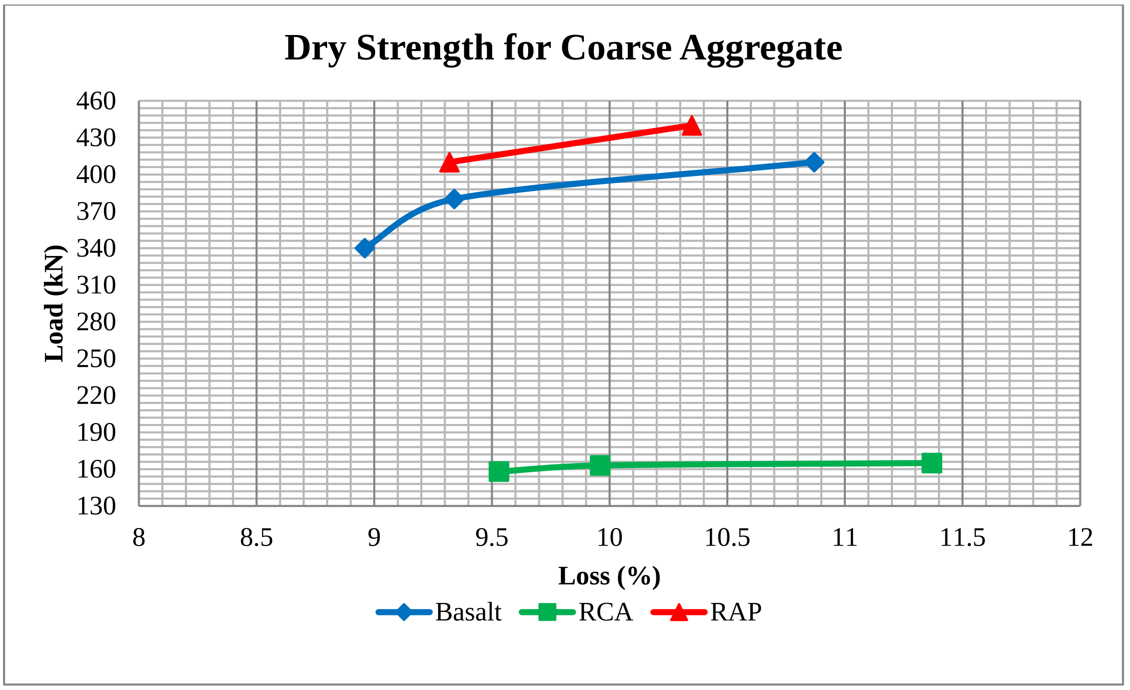

Figure 4.11: Results of Wet/Dry Strength Test for Coarse Aggregate (Dry Strength)

The related test results on RCA, RAP, and basalt are given in Tables 4.11.

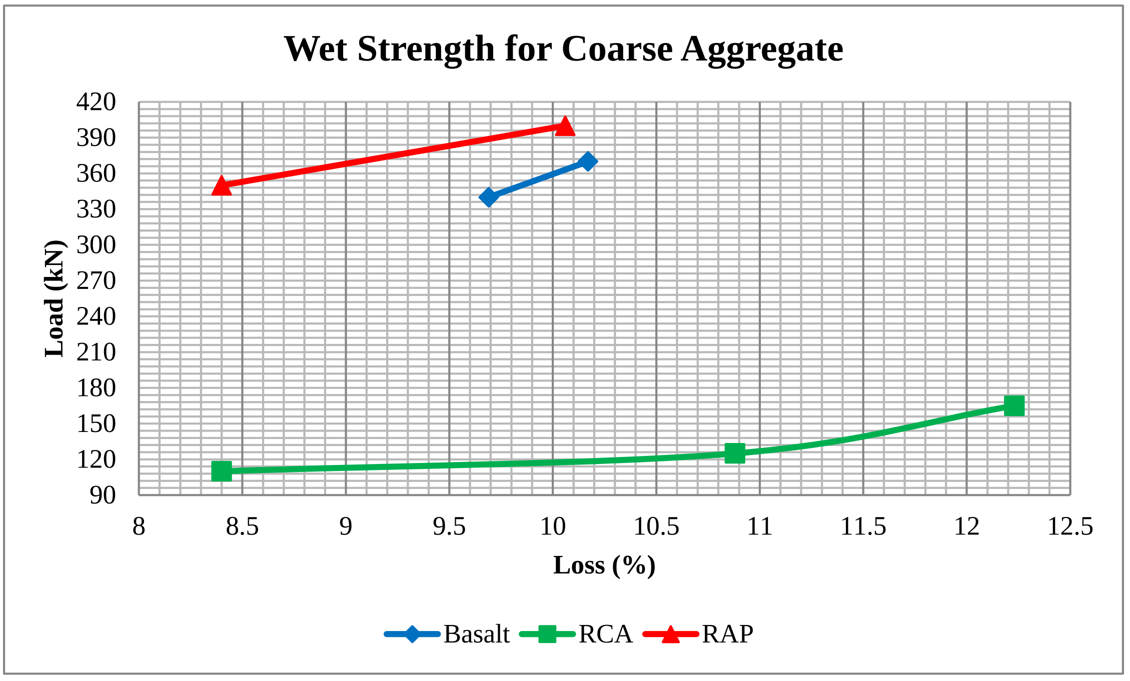

Figure 4.12: Results of Wet/Dry Strength Test for Coarse Aggregate (Wet Strength)

Table 4.11: The Dry Strength (Dry Condition) and Wet Strength (SSD Condition) for Coarse Aggregates

Sample Name	Size (mm)	Dry Condition		Saturated Surface Dry Condition	
		Force (kN)	Produced Fines (%)	Force (kN)	Produced Fines (%)
RCA 1	13.2^{-} 9.5^{+}	158	9.53	110	8.4
RCA 2	13.2^{-} 9.5^{+}	163	9.96	125	10.88
RCA 3	13.2^{-} 9.5^{+}	165	11.37	165	12.23
RAP 1	13.2^{-} 9.5^{+}	410	9.32	350	8.4
RAP 2	13.2^{-} 9.5^{+}	440	10.35	400	10.06
Basalt 1	13.2^{-} 9.5^{+}	410	10.87	370	10.17
Basalt 2	13.2^{-} 9.5^{+}	380	9.34	340	9.69
Basalt 3	13.2^{-} 9.5^{+}	340	8.96	-	-

The wet and dry strengths can be inferred from the test results shown in these figures. Based on the obtained data, the wet/dry strength variation was calculated as follows:

$$\mathbf{Wet/dry\ strength\ variation} = \frac{\mathbf{D - W}}{\boldsymbol{D}} \times \mathbf{100} \qquad (\mathbf{4.1})$$

where D is the dry strength in kilonewtons, and W is the wet strength in kilonewtons.

The results of the calculations for wet strength, dry strength, and wet/dry strength variation for basalt, RAP and RCA are presented in Table 4.12.

Table 4.12: The Results of Wet/Dry Strength Variation Test for Coarse Aggregates and Strength Limits for Dense Graded Asphalt Based on Australian Standards

Material	Dry Strength, D (kN)	Wet Strength, W (kN)	Wet/Dry Strength Variation (%)	Australian Standards Wet/Dry Strength Limits (%)
RCA	163.1	119.7	26.6	35% (max) For heavy and very heavy traffic
RAP	429.8	398.2	7.4	
Basalt	392.9	359.4	8.5	

As the results of wet/dry strength test shows, the wet/ dry strength variation of RCA is substantially more than the corresponding values for RAP and basalt. Therefore, as mentioned previously, it appears plausible to further investigate the feasibility of the application of RCA for the replacement of part of basalt in asphalt mixtures.

Accordingly, the wet/dry strength variation test was also conducted on different mix of coarse aggregates (i.e. RCA, RAP and coarse basalt) considering different percentages of these materials. The related test results on several mixes of coarse aggregates are given in Table 4.13.

Table 4.13: The Results of Wet/Dry Strength Test for Mix of Basalt (75%) and RCA (25%)

Sample Name (Number)		Dry Condition		Saturated Surface Dry Condition	
		Force (kN)	Produced Fines (%)	Force (kN)	Produced Fines (%)
RCA 25% & Basalt 75%	1	350	11.77	265	11.13
	2	310	10.71	250	9.97
	3	270	9.18	-	-
RCA 50% & Basalt 50%	1	300	12.9	215	11.23
	2	270	11.13	200	9.54
	3	225	8.86	-	-
RCA 75% & Basalt 25%	1	240	12.5	180	10.57
	2	215	10.25	165	9.9
	3	205	9.71	-	-
RCA 25% , Basalt 25% & RAP 50%	1	350	10.58	345	11.36
	2	320	9.15	315	10.52
RCA 50% , Basalt 25% & RAP 25%	1	270	11.76	210	11.87
	2	230	9.88	190	9.79
Basalt 80% & RAP 20%	1	410	10.3	373	10.31
	2	375	9.35	352	9.24
	3	350	8.69	-	-

In addition, Figures 4.13 and 4.14 illustrate the results of the wet/dry strength test for different mixes of RCA, RAP and basalt in dry condition and saturated surface dry condition respectively.

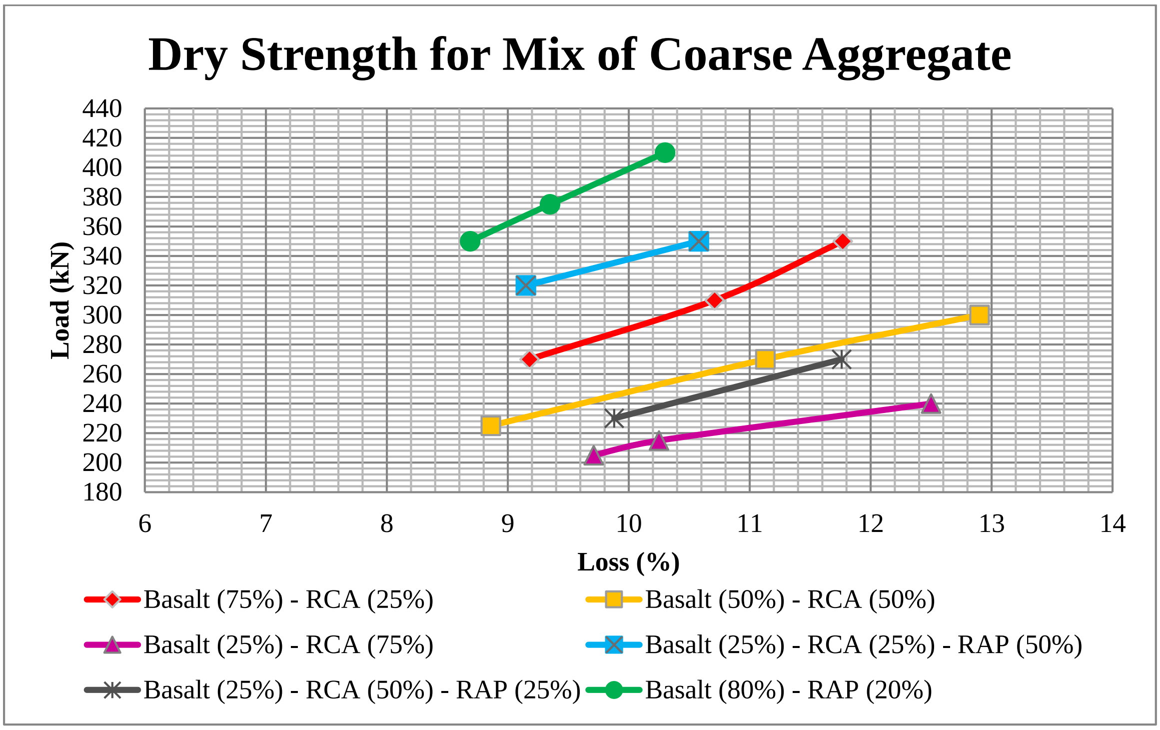

Figure 4.13: Results of Wet/Dry Strength Test for Mix of Coarse Aggregates (Dry Strength)

Based on the obtained results from these graphs, the wet strength (W) and the dry strength (D) can be determined and subsequently the wet/dry strength variation can be calculated as shown previously.

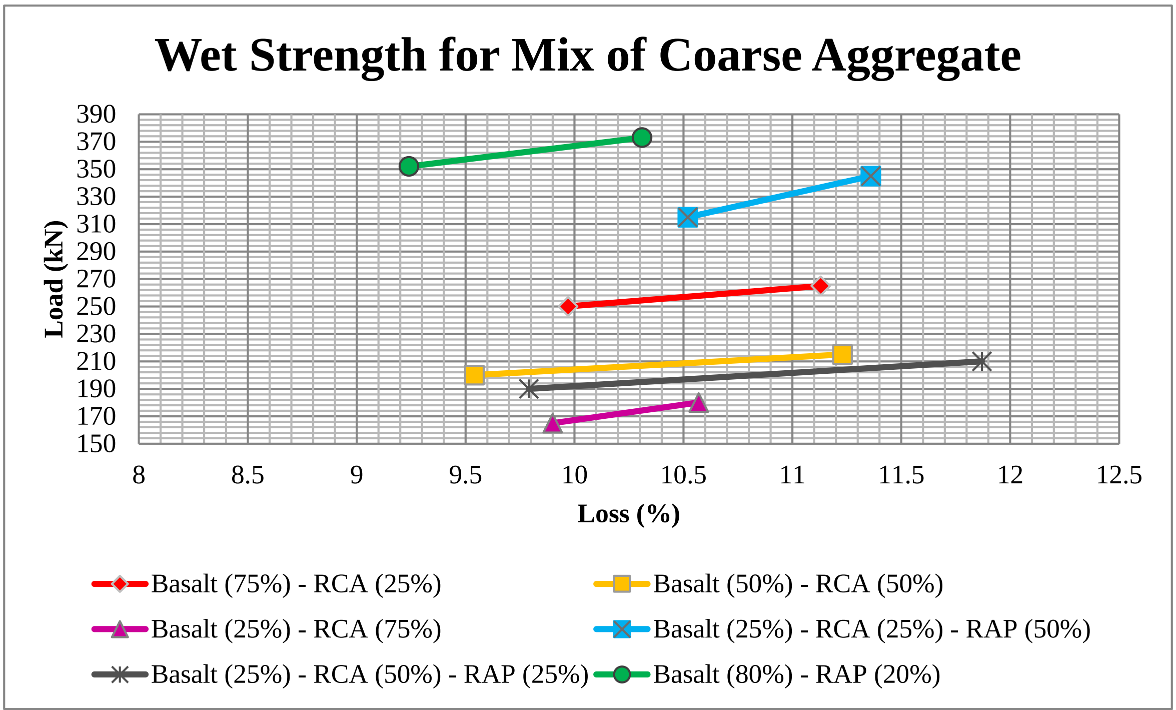

Figure 4.14: Results of Wet/Dry Strength Test for Mix of Coarse Aggregates (Wet Strength)

The results of the calculations for wet strength, dry strength, and wet/dry strength variation on different mix of RAP, basalt and RCA are presented in Table 4.14. The results indicate that all mixes satisfy the maximum 35% limit set by the Australian Standards.

Table 4.14: The Results of Wet/Dry Strength Variation Test for Different Mix of Coarse Aggregate

Material	Dry Strength, D (kN)	Wet Strength, W (kN)	Wet/Dry Strength Variation (%)	Australian Standards Wet/Dry Strength Limits (%)
Basalt (75%) & RCA (25%)	291.4	250.4	14.1	35% (max) For heavy and very heavy traffic
Basalt (50%) & RCA (50%)	247.6	204.1	17.6	
Basalt (25%) & RCA (75%)	210.4	167.2	20.5	
Basalt (80%) & RAP (20%)	398.9	366.9	8.0	
Basalt (25%), RCA (25%) & RAP (50%)	337.8	296.4	12.3	
Basalt (25%), RCA (50%) & RAP (25%)	232.6	192	17.5	

4.2.8. Recycled Construction Aggregate (RCA) Classification

The asphalt mixture performance can vary significantly depending on the type, percentages, and the properties of the materials. When it comes to aggregates, the mechanical, physical and chemical properties of the aggregates, resulting from the geological origin and mineralogy of the potential source and its subsequent weathering or alteration, play an important role on final product performance.

Aggregates can be classified in three groups reflecting the origin, formation and history of their rock:

- Igneous rocks which are generally of high strength with low directional differences in their mechanical properties (Palmstrom, 1995).
- Sedimentary rocks greatly vary in strength and behaviour. As the minerals in sedimentary rocks are cemented together not interlocked, their aggregation are usually weaker than igneous rocks. Sedimentary rocks usually have significant anisotropy in their physical properties.
- Metamorphic rocks have a great variation in characteristics, composition and structure. The metamorphism has often led to hard minerals and hence rocks with high strength. Strength and resistance to weathering of metamorphic rocks make them suitable for use in construction projects.

The human-made materials like cement, concrete (as human made sedimentary rock) brick and tile (as human made metamorphic rock) that can be found in construction and demolition wastes do not differ substantially from natural sedimentary rocks since they are all made from natural rocks. For example, cement is produced from limestone, concrete is composed of sand, gravel and cement. Brick and tile are made from baked clay.

Study on properties of all these rock groups indicates that each geological group has its own advantages and disadvantages in terms of engineering properties.

RCA is made up of these three different aggregate types in terms of geological classification, and hence can provide proper level of function for asphalt surface layer. For example, a matrix of Portland cement concrete which will vary between basalt (i.e. Basic Igneous) and granite (i.e. Acidic Igneous) depending on the source of material and the age of the building from which it came, will form the igneous part of RCA. Sandstone or an agglomerate of sand and cement paste involves the sedimentary part of RCA, and metamorphic part of RCA could be quartz or hornfels depending on the source rock in the concrete, or could be “man-made” metamorphic rock such as ceramic, glass or brick.

As each of the aggregate types (i.e. igneous, sedimentary, and metamorphic) has different properties, their proportion in RCA significantly affects the properties of RCA, and subsequently the final performance of asphalt mixture. For instance, the aggregate proportion influences the bitumen absorption of asphalt mixtures. If RCA contains a lot of sedimentary rock, the RCA would be too absorbent and the binder content will be reduced by absorption. Consequently, the asphalt will be too dry and crack and ravel. In contrast, if the RCA contains a very large proportion of basalts and metamorphic group such as glass and ceramics, it would be very low in absorption, and subsequently the mix will be wet and lack shear strength and shove. It should be mentioned that crushed brick could be low or high in absorption depending on the amount of firing (clinker or callow).

Moreover, the skid resistance will be impacted by the aggregate composition. Asphalt concrete with crushed brick will provide differential wearing of the asphalt by creating a fresh and rugose surface and subsequently will enhance skid resistance (Chen and Liao, 2002). Therefore, RCA will positively affect the skid resistance of the asphalt concrete, as:

- Both the Igneous and Metamorphic groups will be generally hard and prone to polishing,
- The Sedimentary group and crushed brick will wear differentially and create an ever changing depth.

In light of this, asphalt surface layers provide unique opportunities for RCA reuse, as using RCA in asphalt surface layer can contribute to improvement of engineering characteristics of the asphalt pavement materials as well as the pavement performance, representing a value add application for RCA. However, significant developmental limitations and many relevant considerations must be addressed in this regard. For example, the variability in the behaviour and performance of RCA used in different construction projects indicates the variability in RCA composition. Therefore, the classification of RCA in terms of their geological history is considered as one of the preliminary tests in order to investigate the composition and variability of recycled construction aggregates through classification of aggregate samples collected from a recycling centre in Sydney. For this purpose, the RCA is collected at different dates over one year, and is categorized into different geological groups of igneous, metamorphic, and sedimentary, respectively (from left to right), as illustrated in Figure 4.15.

Figure 4.15: Classification of Recycled Construction Aggregate (RCA)

It is intended to create a database containing the composition and characteristics of RCA produced in Sydney in twelve months. The results of sorting RCA samples into different geological groups are presented in Table 4.15.

Table 4.15: Summary of Test Results for the Classification of Recycled Construction Aggregate (RCA)

Sample	Time	Aggregate Type in Sample				Test Results (%)		
		No of Igneous Type in the Sample	No of Sedimentary Type in the Sample	No of Metamorphic Type in the Sample	Total	Igneous	Sedimentary	Metamorphic
1	**17 Feb 2016**	37	62	8	107	34.6	57.9	7.5
2	**4 Mar 2016**	30	69	15	114	26.3	60.5	13.2
3	**7 Apr 2016**	32	84	19	135	23.7	62.2	14.1
4	**24 Apr 2016**	36	121	23	180	20	67.2	12.8
5	**24 May 2016**	23	127	32	182	12.6	69.8	17.6
6	**25 July 2016**	26	151	37	214	12.1	70.6	17.3
7	**10 Aug 2016**	41	142	31	214	19.2	66.3	14.5
8	**29 Aug 2016**	67	188	65	320	20.9	58.8	20.3
9	**14 Sep 2016**	37	172	91	300	12.3	57.3	30.4
10	**28 Sep 2016**	45	168	40	253	17.8	66.4	15.8
11	**13 Oct 2016**	21	147	59	227	9.3	64.7	26.0
12	**7 Nov 2016**	38	193	82	313	12.1	61.7	26.2
13	**14 Nov 2016**	24	124	39	187	12.8	66.3	20.9
14	**2 Dec 2016**	37	181	70	288	12.9	62.8	24.3
15	**19 Dec 2016**	43	189	86	318	13.5	59.4	27.1

As the results of classification shows, the sedimentary rocks in RCA are the greatest part and significantly influence the RCA properties. However, the man made metamorphic rocks such as bricks and ceramics involve about 20% of RCA. These types of man-made aggregates can enhance both the strength (due to a good shape) and the durability due to low absorption as well as skid resistance. In an investigation by Yeaman (1976), it has been shown that the addition of small quantities of crushed brick to asphalt mixture, improves the skid resistance of this material (Yeaman, 1976). Therefore, the variability in RCA composition can result in making a superior HMA to natural aggregate mixtures.

4.2.9. Image Analysis of Recycled Construction Aggregate (RCA)

In the present study, microstructure studies of RCA are carried out to characterize the nature of RCA aggregates. For this purpose, three different aggregate types, as illustrated in Figure 4.16, were selected and were analysed using a scanning electron microscope (SEM).

Figure 4.16: Studied Rocks from RCA Sample (a) Sedimentary, (b) Igneous, (c) Metamorphic

Figure 4.17 shows the scanned images of a typical sedimentary rock in RCA at different resolutions.

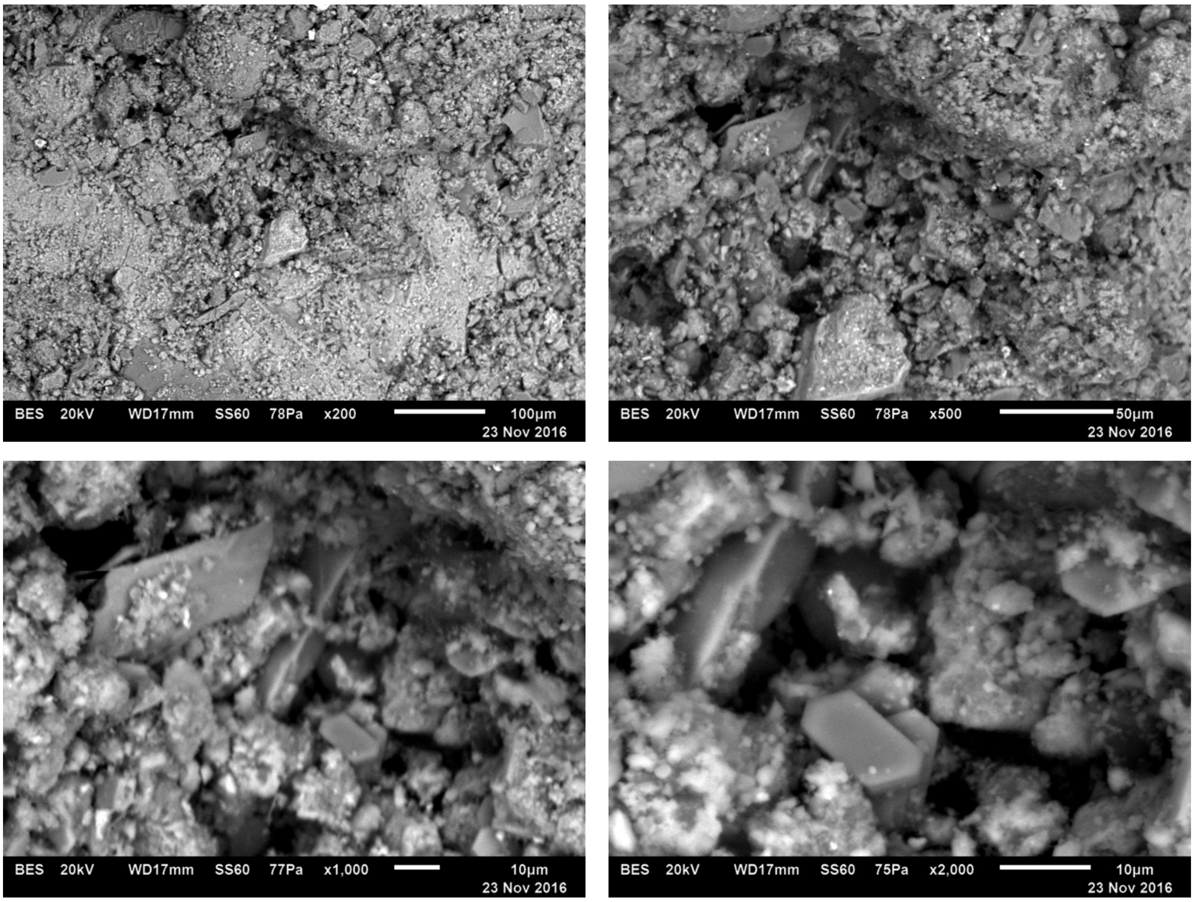

Figure 4.17: Scanned Images of Sedimentary Rock in RCA at Resolution of (a) 200 dpi, (b) 500 dpi, (c) 1,000 dpi, (d) 2,000 dpi

In addition, Figures 4.18 and 4.19 illustrate the scanned images of a typical igneous and metamorphic rock from a RCA sample, respectively.

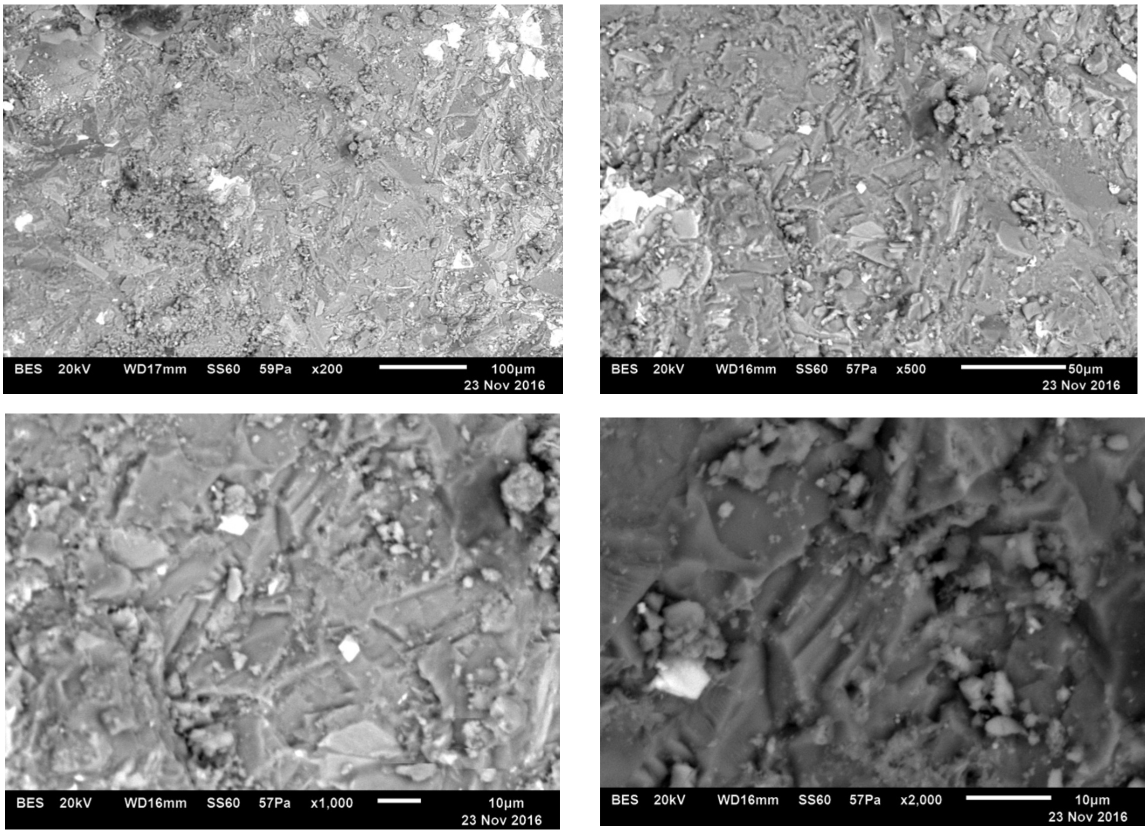

Figure 4.18: Scanned Images of Igneous Rock in RCA at Resolution of (a) 200 dpi, (b) 500 dpi, (c) 1,000 dpi, (d) 2,000 dpi

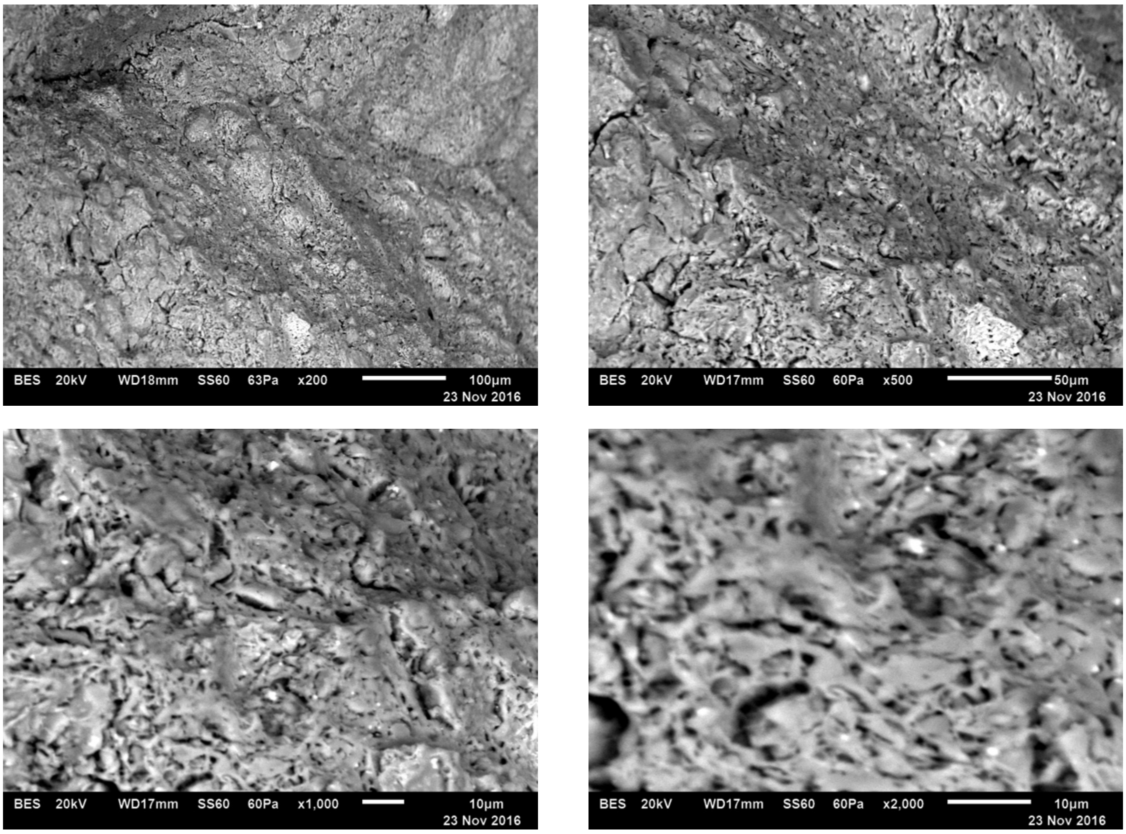

Figure 4.19: Scanned Images of Metamorphic Rock in RCA at Resolution of (a) 200 dpi, (b) 500 dpi, (c) 1,000 dpi, (d) 2,000 dpi

As shown in the scanned images of different rocks, the porous structure of sedimentary rocks can clearly be seen which is responsible for high absorption of RCA due to great proportion of these aggregates (about 60%) in RCA samples. In addition, the micro cracks in metamorphic rock are also observable in Figure 4.19.

4.3. Tests for Evaluation of Fine Aggregates Properties

To achieve the goals of this research, some other materials such as recycled glass is also selected to compensate for RCA for some of its shortcomings, and this research will seek the optimum combination of these materials. Accordingly, the study of properties of recycled glass as part of fine aggregates in combination with basalt has been considered as part of this research. In light of this, different tests have been conducted on recycled glass and basalt (passed through 4.75 mm sieve). These tests and their results analysis are described in this section. It should be noted that three trials were performed for each test and the mean of the three trials were chosen as the test result.

4.3.1. Particle Size Distribution (Sieve Analysis)

The gradation of fine aggregates to be used in asphalt mixtures (i.e. glass and basalt) are evaluated through Particle Size Distribution Test (Figure 4.20).

Figure 4.20: Glass Gradation by Particle Size Distribution Test

This test is conducted in accordance with AS 1141.11.1 (2009) and the gradation curves obtained from this test for different fine aggregates considered in this research, including recycled glass and basalt, are shown in Figure 4.21.

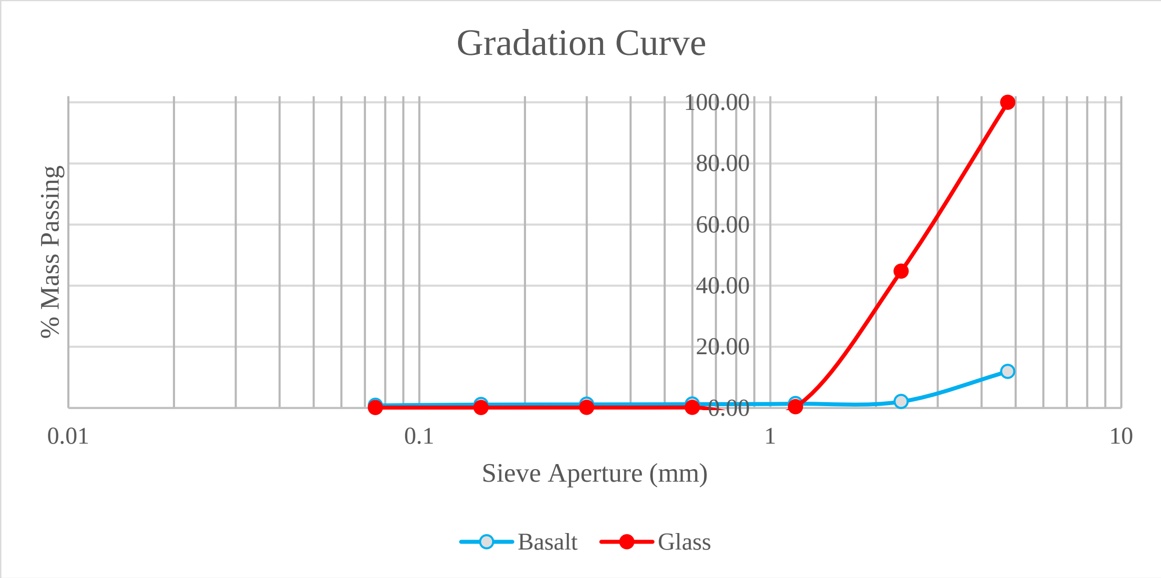

Figure 4.21: The Results of Particle Size Distribution Test for Fine Aggregates

Moreover, the results of the particle distribution test are presented in Tables 4.16 and 4.17 for recycled glass and basalt, respectively.

Table 4.16: The Results of Particle Size Distribution Test for Recycled Glass

Sieve Size	Mass Passing for Glass (%)			Average Mass Passing for Glass (%)
	Sample 1	Sample 2	Sample 3	
4.75	99.93	99.98	99.95	99.95
2.36	47.21	43.38	43.59	44.73
1.18	0.49	0.18	0.34	0.34
0.600	0.21	0.09	0.12	0.14
0.300	0.19	0.08	0.10	0.12
0.150	0.18	0.06	0.09	0.11
0.075	0.15	0.06	0.08	0.10

Table 4.17: The Results of Particle Size Distribution Test for Basalt (10 mm)

Sieve Size	Mass Passing for Basalt (%)			Average Mass Passing for Basalt (%)
	Sample 1	Sample 2	Sample 3	
4.75	11.35	12.73	11.67	11.92
2.36	1.81	2.4	2.01	2.07
1.18	1.19	1.59	1.38	1.39
0.600	1.04	1.41	1.26	1.24
0.300	0.97	1.33	1.19	1.16
0.150	0.94	1.26	1.09	1.10
0.075	0.82	0.99	0.7	0.84

4.3.2. Particle Density and Water Absorption Test

The particle density and water absorption of the fine aggregates are determined based on the procedure described in AS 1141.5 (2000), as presented in Figures 4.22 and 4.23.

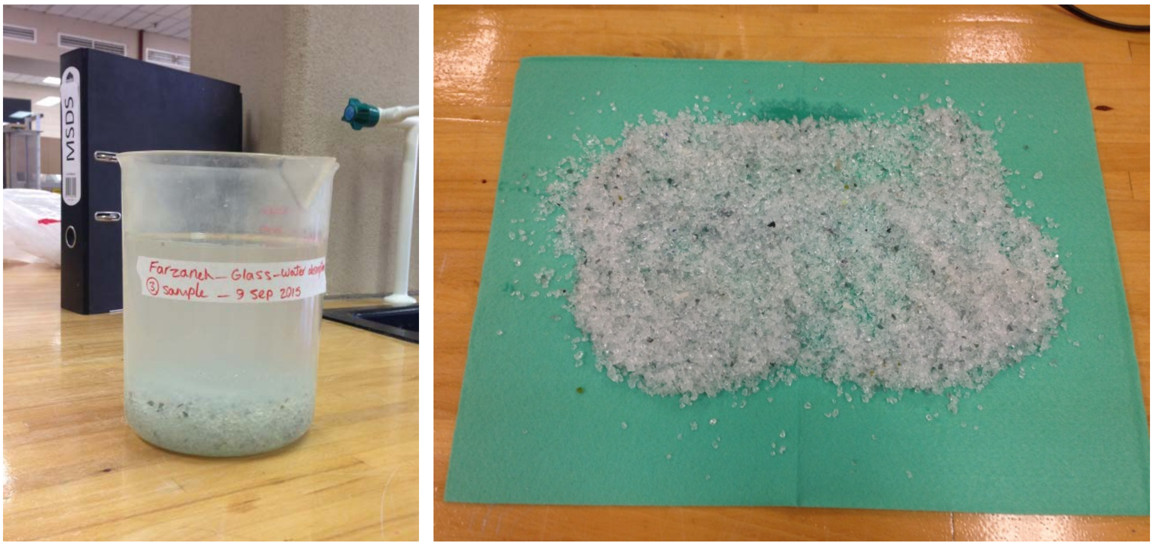

Figure 4.22: Particle Density and Water Absorption Tests for Glass

In this study, the particle density and water absorption tests are conducted on glass and basalt (10 mm) for three trials and the corresponding test results are presented in Tables 4.18 and 4.19.

Figure 4.23: Conducting Particle Density and Water Absorption Tests on Glass

Table 4.18: The Results of Particle Density and Water Absorption Test for Glass and Water Absorption Limits for Dense Graded Asphalt Based on Australian Standards

Sample Number	Apparent Particle Density (gr/cm^3)	Particle Density on a Dry Basis (gr/cm^3)	Particle Density on a SSD Basis (gr/cm^3)	Water Absorption (%)	Australian Standards Limits for Water Absorption (%)
Sample 1	2.508	2.501	2.504	0.10	3 % (max) For heavy and very heavy traffic
Sample 2	2.452	2.446	2.448	0.10	
Sample 3	2.507	2.502	2.504	0.09	
Average Values	**2.489**	**2.483**	**2.485**	**0.10**	

Table 4.19: The Results of Particle Density and Water Absorption Test for Basalt and Water Absorption Limits for Dense Graded Asphalt Based on Australian Standards

Sample Number	Apparent Particle Density (gr/cm^3)	Particle Density on a Dry Basis (gr/cm^3)	Particle Density on a SSD Basis (gr/cm^3)	Water Absorption (%)	Australian Standards Limits for Water Absorption (%)
Sample 1	2.689	2.538	2.589	2.26	2 % (max) For heavy and very heavy traffic
Sample 2	3.001	2.532	2.593	2.43	
Sample 3	2.947	2.759	2.821	2.35	
Average Values	**2.879**	**2.610**	**2.668**	**2.35**	

4.4. Tests for Evaluation of Filler Properties

As discussed previously, the fillers used in this research are hydrated lime and Portland cement which are common types of fillers. However, as the particle density of filler is important in evaluating the volumetric properties of asphalt mixtures, in this research, the apparent particle density of selected fillers is determined in accordance with AS/NZS 1141.7 (2014) on three trials and its related results are presented in Table 4.20.

Table 4.20: The Results of Apparent Particle Density Test for Fillers

Material	Sample 1	Sample 2	Sample 3	Average Value (gr/cm^3)
Portland Cement	3.131	3.132	3.128	3.130
Hydrated Lime	2.333	2.330	2.331	2.331

4.5. Summary and Conclusions

Due to the important role and high portion of aggregates in asphalt concrete, utilization of recycled aggregates can provide the enormous benefits from the viewpoint of environmental sustainability and effective use of resources. Among the recycled aggregates, RCA offers a good solution to design a sustainable asphalt mixture due to large amount of construction and demolition wastes as well as to provide proper level of function for wearing course because RCA is made up of three different aggregate types. However, it is important to assess the engineering properties of RCA comprehensively.

Accordingly, in this research, the properties of RCA have been thoroughly evaluated through the laboratory investigation and tests. The test results are summarized in Table 4.21.

Table 4.21: Summary of the Tests Results for the Evaluation of Coarse Aggregate Properties

Test	Test Method	Aggregate			Typical Limit Based on Australian Standards
		RCA	RAP	Basalt	
Particle Distribution Test	AS 1141.11.1	As presented in relevant Figures and Tables			-
Flakiness Index Test	AS 1141.15	6.91	10.42	19.03	30% (max)
Particle Shape Test	AS 1141.14	6.16	9.47	18.34	35% (max)
Water Absorption	AS 1141.6.1	**6.30**	1.78	1.64	2% (max)
Particle Density	AS 1141.6.1	2.370	2.541	2.640	-
Particle Density on Dry Basis	AS 1141.6.1	2.212	2.431	2.530	-
Particle Density on SSD Basis	AS 1141.6.1	2.351	2.474	2.571	-
Aggregate Crushing Value	AS 1141.21	29.21	7.40	8.91	35% (max)
Weak Particles	AS 1141.32	0.23	0.04	0.23	1% (max)
Wet/Dry Strength Test	AS 1141.22	26.6	7.4	8.5	35% (max)
Wet Strength	AS 1141.22	119.7 kN	398.2 kN	359.2 kN	150 kN (min)
Dry Strength	AS 1141.22	163.1 kN	429.8 kN	392.9 kN	-

In addition, to have better comparisons between the aggregate properties, the test results on different aggregates as well as the standard limits are also illustrated in Figure 4.24.

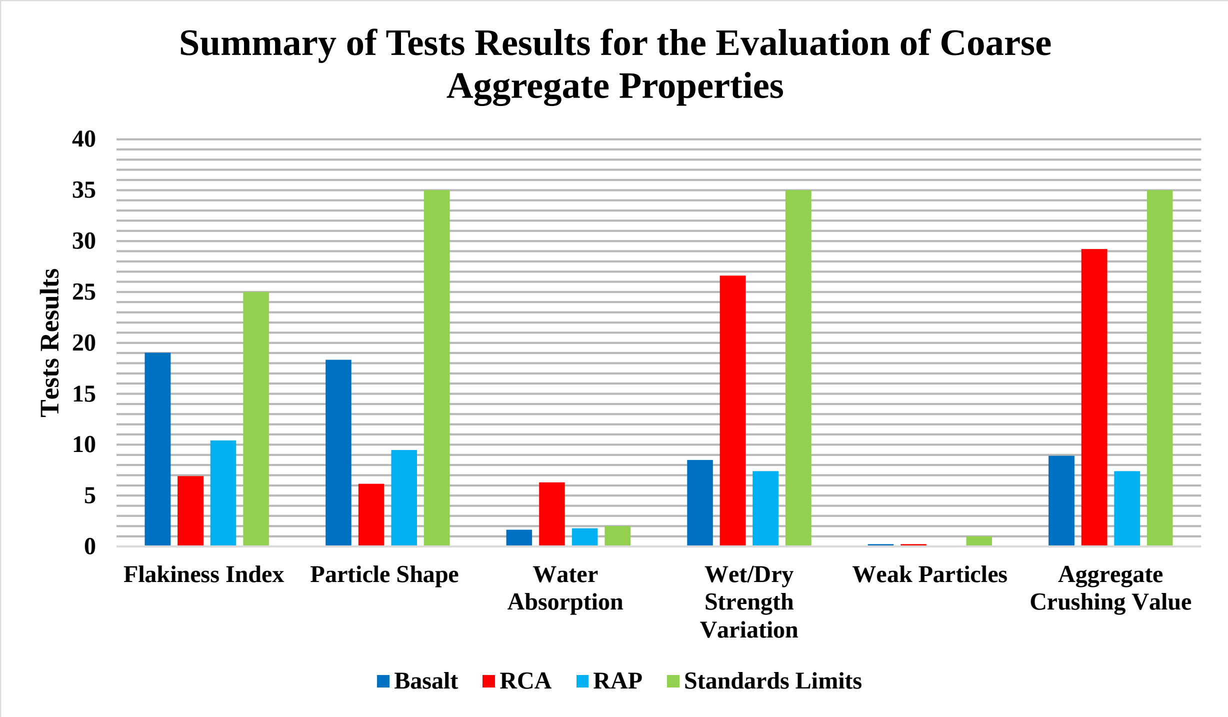

Figure 4.24: Comparison of Different Aggregate Properties with Standard Limits

As shown in Figure 4.24, the results of preliminary tests on coarse aggregates indicate that all properties of RCA, except for water absorption, are within the limits specified by relevant Australian Standards and hence deemed appropriate for use as aggregate in the asphalt mixture. However, for some parameters such as flakiness index and particle shape which are two dominant characteristics having significant impact on asphalt mixture strength and stability; RCA displays smaller value in comparison with basalt and RAP. This can be one of the strong points of RCA as flakiness index and particle shape are the two important properties for proper compaction, deformation resistance, and workability of asphalt mixture (Masad et al., 2007)

In addition, as can be observed in Table 4.21 and Figure 4.24, the water absorption of RCA is higher than the corresponding value of RAP and basalt and the Australian Standards limit, because it is well known that water absorption requires linked and open cracks in the structure of aggregate and RCA contains cracks due to the crushing processes. Moreover, the great amounts of impurities in RCA can increase the water absorption of RCA. The high water absorption of RCA may result in high bitumen absorption in asphalt mixtures, and hence plays an important role in asphalt mixture design.

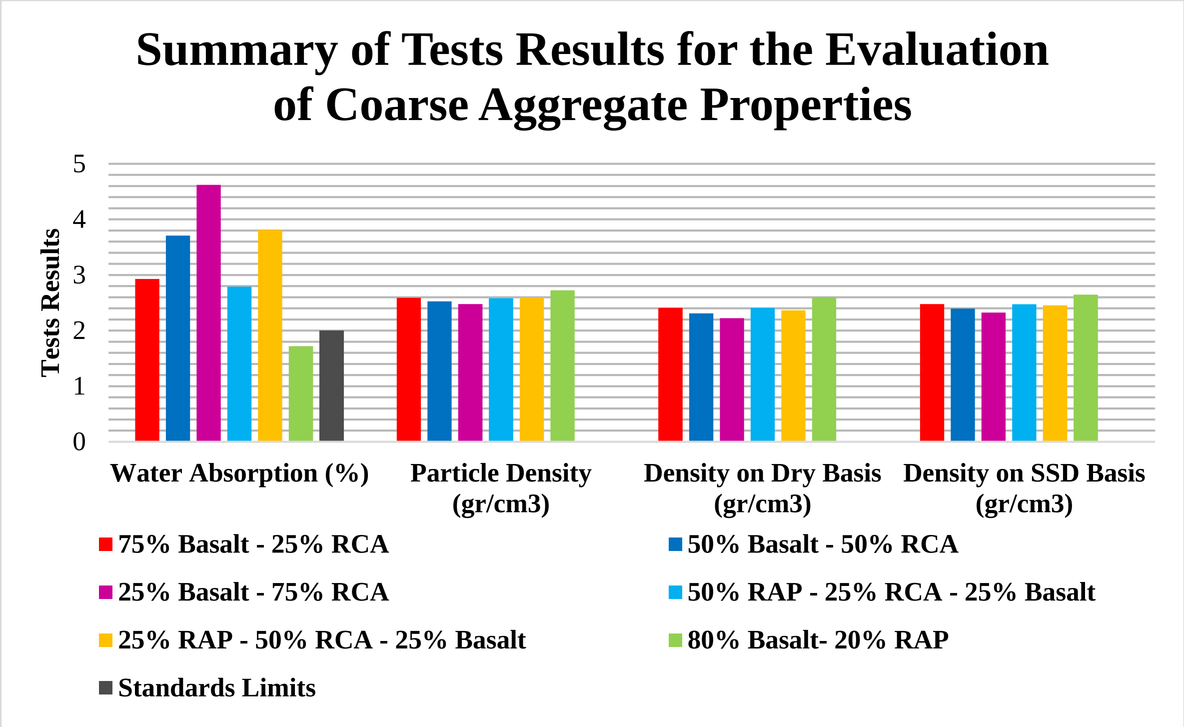

Figure 4.25: Comparison of Water Absorption and Particle Density of Different Mixes of Coarse Aggregates

Accordingly, since this research aims to investigate the feasibility of the application of RCA for the partial replacement of coarse virgin aggregate (basalt) and in combination with other recycled aggregate (i.e. RAP) in asphalt mixtures, the particle density and water absorption tests were conducted on different mix of coarse aggregates while considering different percentages of these materials. The results of these tests are presented in Figure 4.25. As can be observed in Figure 4.25, increasing RCA in the mix does not make any substantial change in mix density in comparison with water absorption. In other words, by increasing RCA from 0% to 100% in the mix, the density decreases by 7%, whereas water absorption increases by 74%.

In addition, although wet/dry strength variation of RCA meets the requirements of Australian standards, the test results show that this value is higher than the corresponding value of RAP and basalt. As the wet/dry strength variation is related to the principal mechanical properties which are required for asphalt aggregate, it is of high importance in asphalt mixture design. Therefore, wet/dry strength variation test was also conducted on different mixes of coarse aggregate. Figure 4.26 shows the comparison of wet strength, dry strength, and wet/dry variation in different mixes of RCA, RAP and basalt.

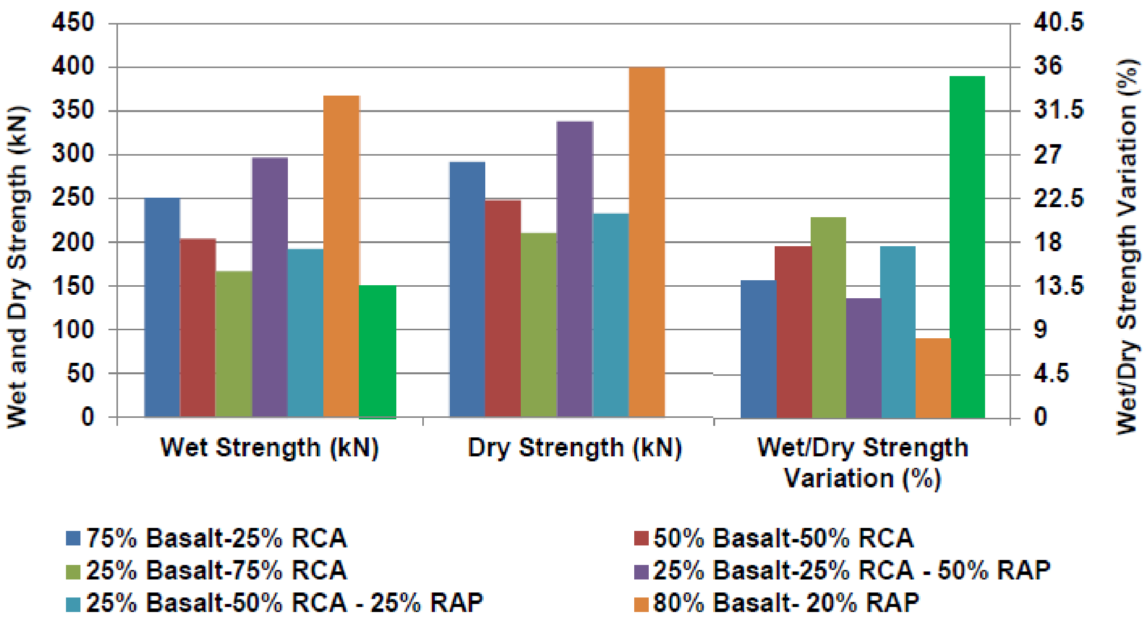

Figure 4.26: Comparison of the Wet Strength and Dry Strength of Different Mixes of Coarse Aggregates

As illustrated in this figure, the wet/dry strength variation of mix of RCA/basalt increases by increase of the percentage of RCA in the mix, so that the increase of RCA from 0% to 100% will result in 20% increase in wet/dry strength variation. The results of these two tests (i.e. water absorption and particle density test, and wet/dry strength variation test) on mix of coarse aggregates are summarized in Table 4.22.

Table 4.22: Summary of Tests Results for Evaluation of Mix of Coarse Aggregates Properties

Mix	Test Method	Water Absorption (%)	Particle Density (kN/m^3)	Wet/Dry Strength Variation (%)
Basalt (75%) + RCA (25%)	AS 1141.15	2.93	2.590	14.1
Basalt (50%) + RCA (50%)	AS 1141.14	3.71	2.527	17.6
Basalt (25%) + RCA (75%)	AS 1141.6.1	4.62	2.476	20.5
Basalt (80%) + RAP (20%)	AS 1141.6.1	1.72	2.723	8.0
Basalt (25%) + RCA (25%) + RAP (50%)	AS 1141.21	2.79	2.582	12.3
Basalt (25%) + RCA (50%) + RAP (25%)	AS 1141.32	3.81	2.598	17.5

The results of tests on mix of coarse aggregate showed that in all cases of RCA ratios, RCA increase causes a decrease in wet and dry strength and an increase in water absorption. This will necessitate the proper selection and optimum combination of RCA and other aggregates.

The coefficient of variation is used as an indication to measure the heterogeneity of test results. The results of calculation of standard deviation (SD) and coefficient of variation (CV) for each set of aggregate mixes are presented in Table 4.23.

Table 4.23: Coefficient of Variation and Standard Deviation for Mix of Coarse Aggregates Properties

Mix	Coefficient of Variation	Standard Deviation
Basalt (100%) + RCA (0%)	2.14	0.035
Basalt (75%) + RCA (25%)	1.71	0.050
Basalt (50%) + RCA (50%)	3.52	0.131
Basalt (25%) + RCA (75%)	4.42	0.204
Basalt (0%) + RCA (100%)	2.47	0.156

As can be observed in Table 4.23, the coefficient of variation for each data set reveals that the test results dispersion is low and the tests are conducted consistently.

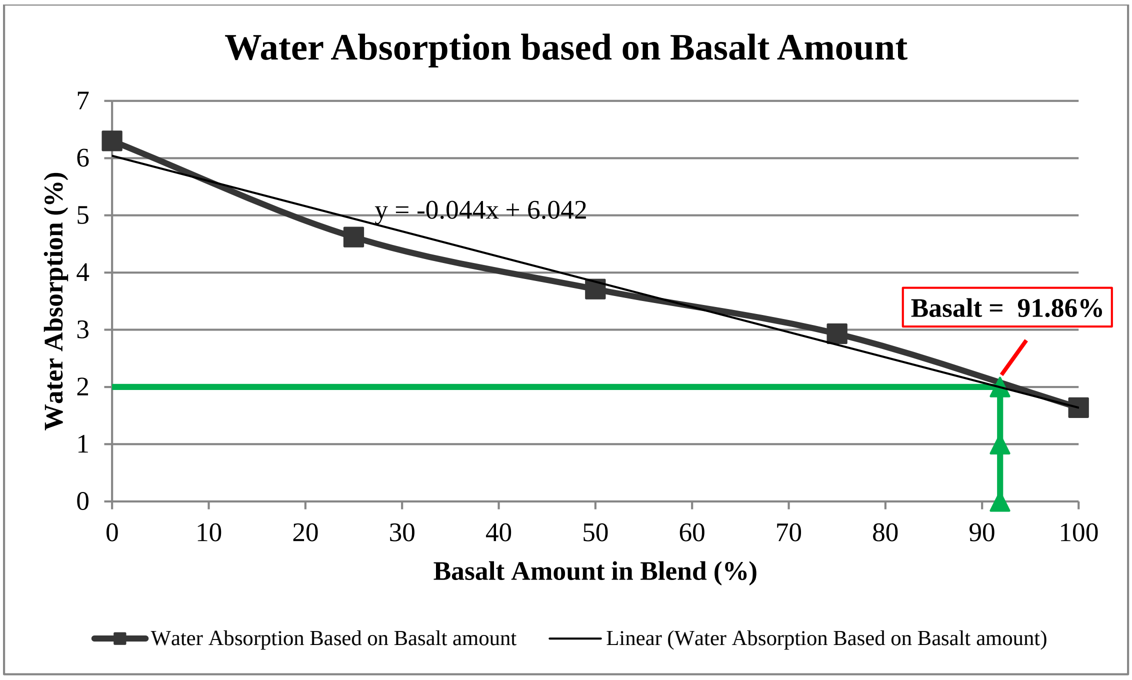

Figure 4.27: Regression Analysis for Determination of Optimum Basalt Amount

Furthermore, regression analysis is typically applied to the water absorption test results for different combination of RCA and basalt, to show the typical amount of RCA and basalt in a blend to give 2% water absorption which is the standard limit of water absorption based on Australian standards (Figures 4.27 and 4.28).

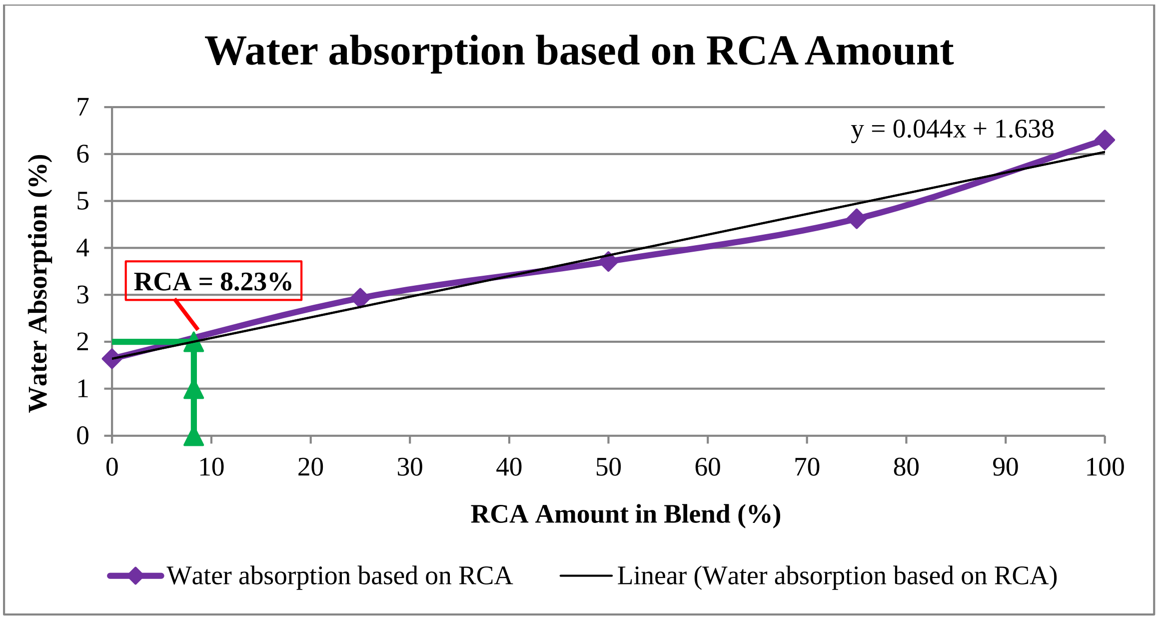

Figure 4.28: Regression Analysis for Determination of Optimum RCA Amount

As illustrated in Figures 4.27 and 4.28, the standard water absorption limit of 2% can be achieved by mixing of almost 8% and 92% of RCA and basalt, respectively. However, based on the available references (Austroads, 2014), if the sample absorbs between 2% and 4% of its mass, it should be carefully examined by other tests. If the sample absorbs in excess of 4% of its mass, it will rarely prove to be an adequate aggregate for asphalt production. Based on the water absorption results, it can be observed that the combination of 25% RCA and 75% basalt would provide water absorption of 2.93%, and also water absorption of the combination of 50% RCA and 50% basalt would be 3.71%, which are still in the range of aggregate water absorption that suggest further research.

It should be noted that the variability in RCA composition has led to the variability in the results of behaviour and performance of RCA used in construction projects. Therefore, the main aim of this chapter was to provide more insight into the contribution of aggregate types (i.e. igneous, sedimentary, and metamorphic) as different components of RCA as well as to create a data base containing the characteristics of RCA produced in nearest recycling units, over twelve months, that can be used in future research using RCA.

To this end, RCA classification has been conducted on different RCA samples collected at different dates over a twelve month period. The results of RCA classification reveals that RCA is composed of mostly sedimentary rocks, igneous rocks and metamorphic rocks (Figure 4.29). All these rocks, with their own properties and their weak and strong points, have made RCA a potential synthetic aggregate for pavement construction depending on the RCA percentage.

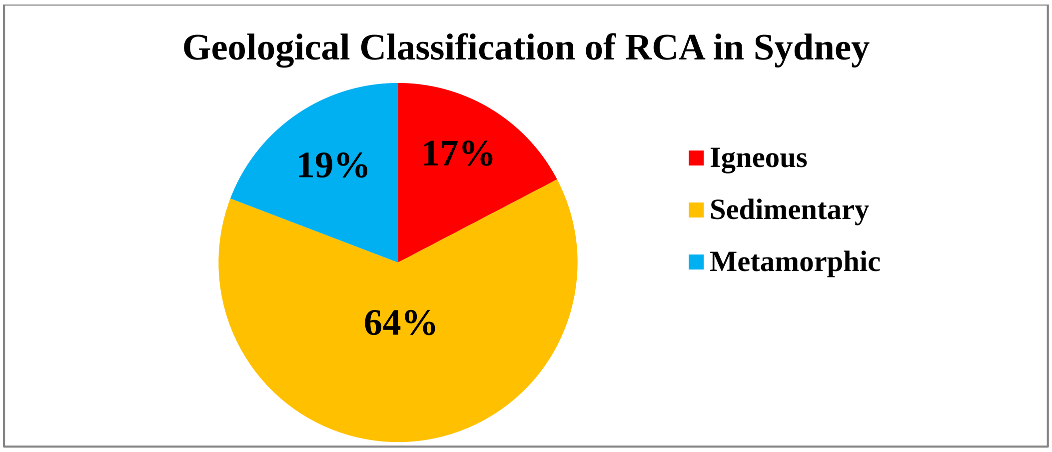

Figure 4.29: The Result of Statistical Study on RCA Classification

In addition, to have better comparisons between the aggregate types in RCA, the image analyses on different aggregates, using a scanning electron microscope (SEM), were conducted which showed the porous structure of sedimentary rock and microcracks in metamorphic rocks. All these tests and analysis emphasize on the selection of optimal combination of RCA and other aggregates to satisfy the relevant standards requirements while taking advantage of other strong points of RCA.

Table 4.24: Summary of the Tests Results for the Evaluation of Fine Aggregate Properties

Test	Test Method	Aggregate		Typical Limit Based on Australian Standards
		Glass	Basalt	
Particle Distribution Test	AS 1141.11.1	As presented in relevant Figures and Tables		-
Water Absorption	AS 1141.5	0.10	2.35	3% (max)
Particle Density	AS 1141.5	2.489	2.879	-
Particle Density on Dry Basis	AS 1141.5	2.483	2.610	-
Particle Density on SSD Basis	AS 1141.5	2.485	2.668	-

Accordingly, in this research, RCA, RAP, and basalt have been considered as coarse aggregate, whereas glass and basalt have been considered as fine aggregates, and various tests have been conducted on each individual component and in combination. This chapter demonstrated the results of the conducted tests leading to the selection of most acceptable combination of aggregates for designing asphalt mixtures. The results of these tests on different fine aggregates and fillers are also summarized in Tables 4.24 and 4.25.

Table 4.25: Summary of the Tests Results for the Evaluation of Filler Properties

Test	Test Method	Filler	
		Portland Cement	Hydrated Lime
Apparent Particle Density	AS 1141.7	3.130	2.331

Chapter 5

Asphalt Mixtures Gradation and Sample Preparation

5.1. Introduction

In general, the most important properties considered in asphalt mix design are strength and rutting resistance as well as durability. Strength and resistance of asphalt mixture to permanent deformation is controlled by the aggregate interlock and proper aggregate packing, whereas the asphalt mixture durability depends on adequate volumetric characteristics and film thickness of asphalt mixture. Accordingly, providing an adequate interlock between aggregates and the aggregate packing is of high importance in designing a new asphalt mixture in order to produce desirable volumetric characteristics in asphalt mixture.

It is well recognized that the asphalt components (i.e. aggregates, binder and air voids) and aggregate gradation substantially affect the asphalt mixture performance. Many studies have shown that the geometrical and physical characteristics of mixture components as well as their relative arrangement will affect the mechanical behaviour of the asphalt mixture. Among these properties, the aggregate gradation is a fundamental characteristic which affects the final performance of asphalt mixture and hence its better understanding is essential prior to any asphalt mixture design. Therefore, the main objective of this chapter is to study different approaches for determination of aggregate gradation and to conclude the optimum gradation for designing the asphalt mixture in this research.

5.2. Gradation of Aggregates

As mentioned previously, aggregate gradation shows the distribution of different sizes of aggregate particles. The aggregate gradation can be identified through particle distribution test (sieve analysis) by determination of the total percentages of aggregates passing through different sieve sizes. Aggregate gradation influences the main characteristics of asphalt mixtures such as stiffness, durability, stability, fatigue resistance, moisture sensitivity, workability and permeability (Brown et al., 2009) and hence it can be considered as one of the most important factors in asphalt mixture design. Referring to McLeod (1937), the aggregate type and gradation significantly affect the important fundamental properties of asphalt mixtures. For example, an adequate gradation of hard aggregates combined with optimum bitumen content results in asphalt mixtures with proper compactability, whereas a uniformly gradation with sufficient amount of fine aggregates reduces the moisture susceptibility as well as the workability of asphalt mixtures.

Historically, the gradation providing the densest blend and highest interparticle contact was recognized as the best gradation for asphalt mixtures by many early studies (e.g. Fuller and

Thompson, 1907; Goode and Lufsey, 1962). However, further studies showed that sufficient air void is required in asphalt mixtures to allow the incorporation of bitumen and to ensure workability, durability, and prevention of rutting or bleeding.

According to the above mentioned information and due to the importance of aggregate gradation, it is essential to develop a method for determination of proper aggregate gradation by considering aggregate interlock and aggregate packing in order to design an asphalt mixture that meets all volumetric requirements while provides an excellent behaviour during construction and in service. In this research, an adequate framework has been considered for aggregate gradation in order to accomplish these objectives. In this framework, the basis for the selection of the aggregate blend gradation is achieving a satisfactory interlock between aggregate particles, a proper skeleton within the asphalt mixture, and acceptable volumetric properties. In light of this, in this research, referring to Vavrik (2000), Das et al. (2014), and Miranda (2012), a proper amount of aggregates at each size fraction is determined based on particle packing theory as well as continuous gradation concept and through a systematic design procedure while considering the local experience for the evaluation of aggregate gradation.

In packing theory, aggregates are assumed as uniform spheres arranged in a unit volume. As the aggregate particles cannot be packed together completely, there are always void spaces between the aggregate particles, as shown in Figure 5.1. The amount of these voids are controlled by the shape of aggregate particles, degree of compaction, gradation of aggregate particles, and surface texture of aggregate particles.

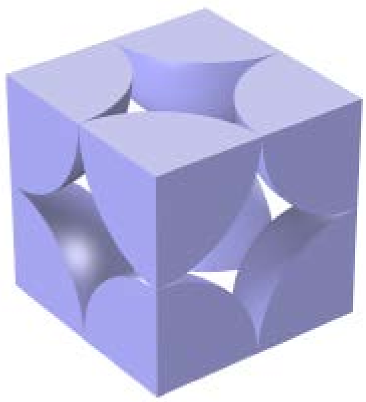

Figure 5.1: The Packing Theory Assumptions (Hesami, 2012)

As mentioned previously, the aggregates in asphalt mixtures are typically composed of three fractions of coarse aggregate, fine aggregate, and filler. The combination of all these three fractions provides an aggregate skeleton within the asphalt mixture to resist against permanent deformation and cracking. Among these fractions, coarse aggregate is the primary component providing resistance to deformation in asphalt mixtures since the interlock between coarse aggregate particles provide a path within the asphalt mixture to transmit the induced stresses to the lower layers of pavement while providing a skeleton for rutting resistance.

Fine aggregates, in asphalt mixture, complete the aggregate structure by filling the void spaces in the coarse aggregate. Mineral filler is used in the aggregate blend for developing mastic, and for filling the voids between the fine aggregates in the mixture.

The above discussion confirms that the coarse aggregate is the main fraction in aggregate blend of asphalt mixture. Therefore, it is necessary to determine the adequate amount of coarse aggregate in order to provide coarse aggregate interlock. In this regard, it is necessary to quantify the maximum and minimum volume of coarse aggregate in the mixture. The maximum volume of coarse aggregate is the amount of coarse aggregate needed to fill the unit volume in a compacted state considering a specified compactive energy, whereas, the minimum volume of coarse aggregate is the required amount to fill the unit volume in its loosest state. Asphalt mixtures containing coarse aggregate between the minimum and maximum values will have adequate interlock between their aggregate particles and resistance to deformation, depending on their relative coarse aggregate amount. In addition, the fine aggregate and filler amount would be determined based on the appropriate volume of fine aggregate and mastic that must fill the voids between the coarse aggregates.

It should be noted that the proper analysis of mixture gradation using the concept of aggregate interlock and packing theory requires changing the traditional definition of coarse and fine aggregates are changed to the following definitions:

- **Coarse aggregates** are the large aggregate particles which create voids when they are placed in a unit volume.
- **Fine aggregates** are the aggregate particles which fill the voids between the coarse aggregate particles.

Considering these definitions, it is necessary to consider some factors as controlling factors for designing an aggregate blend in an optimized way to provide adequate interlock between aggregate particles and acceptable volumetric properties of asphalt mixture. Accordingly, the following section will present the information regarding evaluation of the aggregate gradation in this research based on these controlling factors.

5.3. Grading Target

As mentioned previously, many researchers proposed the densest condition for the aggregate gradation, believing that it will result in the closest aggregate packing and subsequently the best performance. For example, Fuller and Thompson (1907) developed an equation (Equation 5.1) which describes a maximum density gradation. Based on this equation, the aggregate blend with a given maximum aggregate size achieves its maximum density when its gradation follows this equation.

$$\mathbf{P} = \left(\frac{\mathbf{d}}{\mathbf{D}}\right)^{\mathbf{n}} \times \mathbf{100} \qquad (\mathbf{5.1})$$

where D is the maximum aggregate size in the blend, P is the percentage passing through sieve size d, d is the aggregate size being considered, and n is the adjustment parameter which equals 0.5 or 0.45 according to Fuller and Thompson (1907).

However, as explained previously, the realization that adequate void structure is necessary, the gradation achieved by Fuller and Thompson, as presented in Table 5.1, is considered as an initial gradation that should be modified to the grading target based on the packing theory and other present mixture design concepts in order to achieve the densest aggregate gradation with the desired volumetric properties.

Table 5.1: The Aggregate Gradation based on Different Approaches

Sieve Size (mm)	Fuller & Thompson Method		Mixture Morphology Framework				Australian Standards Gradation Limits (%)	
	n = 0.45	n = 0.5	D_{avg}	Min Range for Interaction	Max Range for Interaction	Modified Gradation (%)	Min	Max
19	100	100	11.350	12.827	14.689	100	100	100
13.2	85	83	8.784	9.111	10.385	96	90	100
9.5	73	71	6.997	6.464	7.395	81	72	83
6.7	63	59	4.782	4.230	5.081	68	54	71
4.75	54	50	2.457	2.325	3.025	58	43	61
2.36	39	35	1.393	1.164	1.509	42	28	45
1.18	29	25	0.690	0.587	0.759	30	19	35
0.600	21	18	0.357	0.295	0.383	20	13	27
0.300	15	13	0.178	0.147	0.192	13	9	20
0.150	11	9	0.088	0.061	0.090	8	6	13
0.075	8	6	0.012	0.012	0.026	5.5	4	7

Accordingly, recent approaches regarding the analysis of aggregate gradation rely on the size range carrying the load in the asphalt mixture and various morphological parameters which affect the long-term behaviour of mixture, regardless of the individual material properties.

Based on these theories, in particulate materials, a stress-transmitting path exists that transfers the load through the chain of main particles. In this condition, the smaller particles prevent the buckling of the main chain. Therefore, it is crucial to check the interlock between the aggregate particles of consecutive sieve sizes through the determination of the amount of aggregate particles in each size fractions. The sufficient amount of aggregates at each size fraction ensures the adequate contact between aggregate particles, as aggregate in asphalt mixture must form a continuous network for transferring the load. To achieve this, it is required to consider a minimum value of about 45% for the concentration of load carrying range, in which concentration is defined as:

$$\boldsymbol{\varphi = \frac{W_{ret}^{n}}{W_{tot}}} \qquad (5.2)$$

where W_{ret}^{n} represents the weight of aggregates retaining on sieve n and W_{tot} is the total aggregates weight of. In addition, referring to Miranda (2012), Equation (5.3) can be used to calculate the average particle diameter (D_{avg}) of the two consecutive sieve sizes:

$$\mathbf{D}_{avg} = \frac{\bar{D}_n \varphi_n + \bar{D}_{n+1} \varphi_{n+1}}{\varphi_n + \varphi_{n+1}} \qquad (5.3)$$

where φ_n and φ_{n+1} are the concentration of each sizes and $\bar{D}_n$ and $\bar{D}_{n+1}$ are the mean diameter at sieve sizes which can be defined as follows:

$$\bar{D}_n = B(D_{min} + D_{max}) \qquad (5.4)$$

In Equation (5.4), D_{min} is the opening of the sieve and D_{max} is the opening of the previous size, and B is the parameter which characterizes a continuous size distribution over the materials retained at a certain sieve size.

Two consecutive sieve sizes can be in the load carrying range only if the following equation is satisfied:

$$0.311\bar{D}_n + 0.689\,\bar{D}_{n+1} \le \mathbf{D}_{avg} \le 0.703\,\bar{D}_n + 0.297\,\bar{D}_{n+1} \qquad (5.5)$$

By employing this framework and considering the Australian gradation limits, the aggregate gradation for asphalt mixture using the Fuller and Thompson (1907) will be changed to the modified gradation, as presented in Table 5.1.

In addition, the modified combined gradation of aggregate can be analysed using the particle packing concepts. The use of particle packing involves applying the appropriate parameter to show the void relationships that result from the filling of voids with particles of different size. Referring to Vavrik (2000), the particle diameter ratio can be the most appropriate parameter for the examination of aggregate gradation in HMA mixtures. The particle diameter ratio is defined based on the following equation:

$$\textbf{Particle Diameter Ratio} = \frac{\textbf{Particle Fitting in Void}}{\textbf{Large Particle Creating Void}} \tag{5.6}$$

The particle diameter ratio has a range from 0.155 to 0.42 (Bourbie, 1987). The review of literature has presented evidence that a particle diameter ratio of 0.22 is an appropriate value for the evaluation of the gradation of asphalt mixture. By applying the particle diameter ratio to the standard set of sieves, the primary control sieve (PCS) can be achieved which is the closest sieve to the nominal maximum particle size in millimetres multiplied by 0.22. In this definition, the nominal maximum particle size (NMPS) is the first sieve larger than the first sieve which retains more than 10%. As presented in Table 5.2, the nominal maximum particle size in this research is 14 mm. The list of standard sieve sizes considered in this study and the particle parameters is given in Table 5.2.

Table 5.2: The Gradation Limits and Grading Target for a 14 mm Nominal Size Dense Graded Asphalt Complying with AS 2150

Sieve Size (mm)	Particle Size × 0.22	Primary Control Sieve (PCS)	Gradation (%)	Border	Grading Target (%)	
19	4.18	4.75	100		100	**Coarse Aggregate Portion**
13.2	2.75	2.36	96	**NMPS**	96	
9.5	2.09	2.36	81		81	
6.7	1.47	1.18	68	**Half Sieve** (NMPS × 0.5)	68	
4.75	1.05	1.18	58		58	
2.36	0.52	0.600	42	**MPCS** (NMPS × 0.22)	43	
1.18	0.26	0.300	30		30	**Fine Aggregate Portion**
0.600	0.13	0.150	20	**SCS** (MPCS × 0.22)	21	
0.300	0.07	0.075	13		13	
0.150	0.03	-	8	**TCS** (SCS × 0.22)	8	
0.075	0.02	-	5.5		5.5	

All the parameters used in establishing ratios for the evaluation of the aggregate gradation and the values for this research are presented in Tables 5.2 and 5.3.

In this regard, as shown in Table 5.2, the PCS is the border between coarse and fine aggregate in the total aggregate blend, and therefore it can be named as Mixture Primary Control Sieve (MPCS). In addition, another term called "Half Sieve" is required for providing the optimum packing of coarse aggregates. The half sieve can be obtained by the multiplication of NMPS by 0.5. The particles passing the half sieve are termed as interceptor. The amount of interceptors influences the asphalt mixture voids, mainly through the change in the voids size. Therefore, the determination of interceptors' amount affects the compatibility of asphalt mixture and its resistance to deformation by providing a balanced structure for coarse aggregate portion (Vavrik, 2000).

Moreover, there are two other terms called Secondary Control Sieve (SCS) and Tertiary Control Sieve (TCS) which are required for determination of the appropriate aggregate gradation based on the packing theory and providing information on the fine fraction of aggregates blend. As given in Table 5.2, SCS is defined as MPCS multiplied by 0.22, whereas TCS can be calculated from the multiplication of SCS by 0.22.

In general, the main factor affecting the constructability of asphalt mixtures is the packing of coarse aggregate portion which can be determined by the introduction of the coarse aggregate (CA) ratio. The coarse aggregate ratio can be determined based on the half sieve, as given in Equation 5.7. This parameter is an appropriate parameter for characterizing aggregate voids (Vavrik, 2000).

$$\textbf{CA Ratio} = \frac{\%\ \textbf{Passing Half Sieve} - \%\ \textbf{Passing Primary Control Sieve}}{\textbf{100} - \%\ \textbf{Passing Half Sieve}} \qquad (\mathbf{5.7})$$

The desired value of CA ratio for dense graded mixture is between 0.40 and 0.80. This range ensures a balanced coarse aggregate structure. The asphalt mixtures with the CA ratio less than this range have coarse aggregate containing voids with smaller size, and tend to segregate during construction. Approaching the CA ratio to 1 is an indication of unbalanced coarse aggregate portion because of the increase in the quantity of interceptors. In this condition, the fine fraction of coarse aggregates creates the coarse aggregate skeleton, whereas the larger particles in the coarse aggregate fraction just float between the finer particles while are not significantly involved as part of aggregate structure. This results in some problems in the design and construction of asphalt mixtures.

The packing of the fine aggregate fraction is also an important factor in asphalt mixture design, as it has a substantial influence on VMA of the asphalt mixture because of the creation of voids in the fine portion of the aggregates. The packing of the fine aggregate fraction can be examined using the fine aggregate coarse fraction ratio (FAc). If the fine aggregate fraction of the blend be defined as part of particles passing the MPCS, it would be possible to view this portion in two parts of coarse and fine part of fine aggregate portion. In this regard, FAc is used for characterization of the packing behaviour of the coarse part of fine aggregate portion (Equation 5.8).

$$\mathbf{FA_c\ Ratio} = \frac{\mathbf{\%\ Passing\ Secondary\ Control\ Sieve}}{\mathbf{\%\ Passing\ Primary\ Control\ Sieve}} \qquad (5.8)$$

Decreasing FAc ratio will increase the voids in the mixture. It is desirable to have this ratio between 0.4 and 0.5. The ratio with a value lower than 0.4 will create a gradation that is not uniform, resulting in a mixture with the characteristics of gap gradation in the fine fraction of the blend, and subsequently causing the compaction problems and instability in asphalt mixtures. Increasing this ratio to values higher than 0.5 will tend to produce the tender mixtures, which will overdensify and give early failure under traffic.

On the other hand, the fine part of fine aggregate portion is responsible for filling the voids made by the coarse part of fine aggregate fraction. The packing of the fine part of fine aggregate portion can be obtained by the fine aggregate fine fraction ratio (FA_f) given in Equation 5.9. This ratio characterizes the packing behaviour of the smallest portion in the aggregate blend.

$$\mathbf{FA_f\ Ratio} = \frac{\mathbf{\%\ Passing\ Tertiary\ Control\ Sieve}}{\mathbf{\%\ Passing\ Secondary\ Control\ Sieve}} \qquad (5.9)$$

In typical dense graded asphalt mixtures, FA_f ratio should not be more than 0.50 because the decrease in FA_f ratio results in the increase in the mixture (Vavrik, 2000).

Table 5.3: The Ratios and Controlling Factors based on Packing Theory

Nominal Maximum Particle Size (NMPS)	Mix Primary Control Sieve (MPCS)	Half Sieve	CA Ratio	Secondary Control Sieve (SCS)	FAc Ratio	Tertiary Control Sieve (TCS)	FA_f Ratio
13.2 (14 mm)	2.36	6.7	0.78	0.600	0.49	0.150	0.38

A complete list of controlling sieves and calculation results for different ratios are given in Table 5.3, whereas the finalized grading target modified through the morphology framework and packing theory based on the controlling factors of CA ratio, FA_f ratio, and FAc ratio is presented in Table 5.2.

In addition, to compare the gradation curve obtained from different approaches, including Fuller and Thompson (1907), morphology framework, and finalized grading target controlled by the packing theory ratios, these gradation curves as well as the Australian standard gradation limits are illustrated in Figure 5.2.

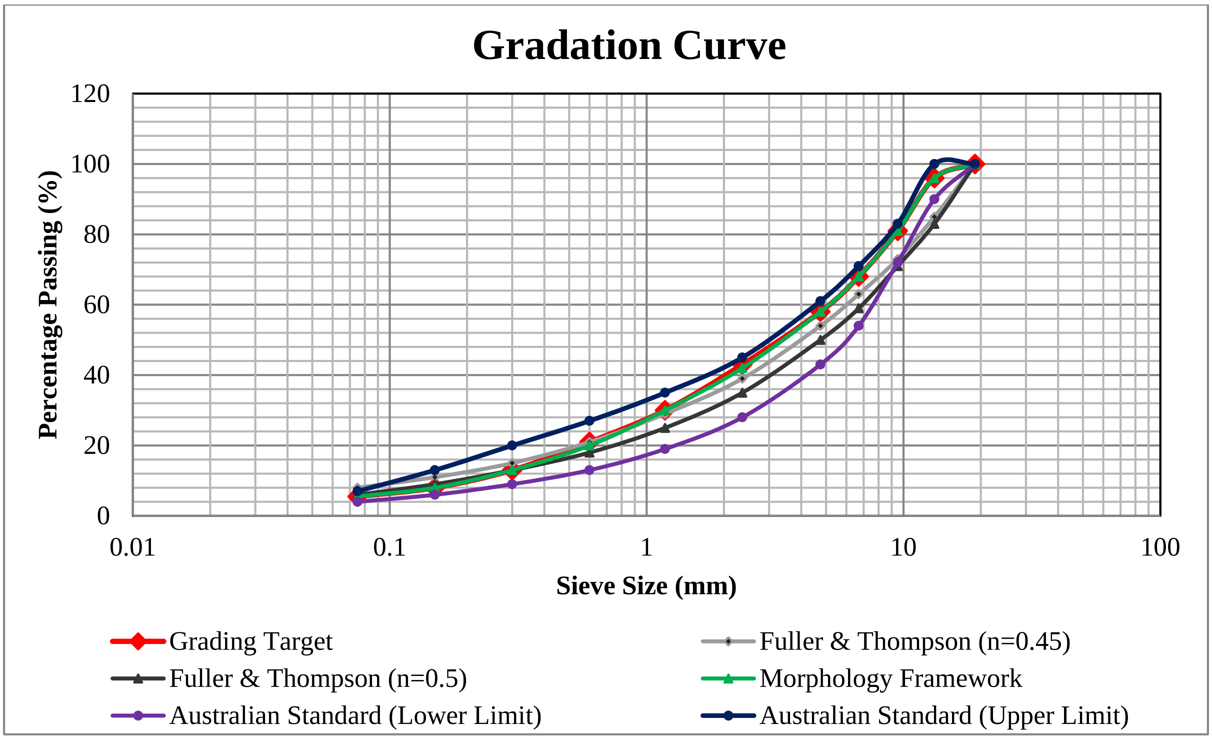

Figure 5.2: The Gradation Curves Based on Different Approaches

Referring to the packing theory and morphology framework (Das et al, 2014), different aggregate size groups are identified within the mineral aggregates, as follows:

- Oversized Structure (OS)
- Primary Structure (PS)
- Secondary Structure (SS)
- Filler particles

Primary Structure (PS) is the portion of aggregate which forms a network and plays a vital role in transferring the load through the mixture. The aggregates between Primary Structure and filler particles in terms of size are termed as Secondary Structure (SS). SS directly affects the PS stability. In addition, particles larger than PS, which are called OS, are not interconnected to the PS and hence do not have any contribution to the load carrying capacity of PS. In asphalt mixture, the filler particles combined with bitumen form a matrix which coats the Secondary Structure. This matrix is termed as mastic. An adequate percentage of all these groups are necessary to provide an asphalt mixture with proper performance properties. Therefore, it is necessary to determine the boundary limits between these sub-structures.

In this research, the ranges and boundary limits between all these structures, as shown in Figure 5.3, are determined through the analysis based on the packing theory and morphology framework to provide the proper contact between aggregate particles as well as the aggregate interlock. As shown in this figure, according to the calculations (Table 5.2), the minimum size for the PS is defined as particles passed the 2.36-mm sieve or retained on the 1.18-mm sieve.

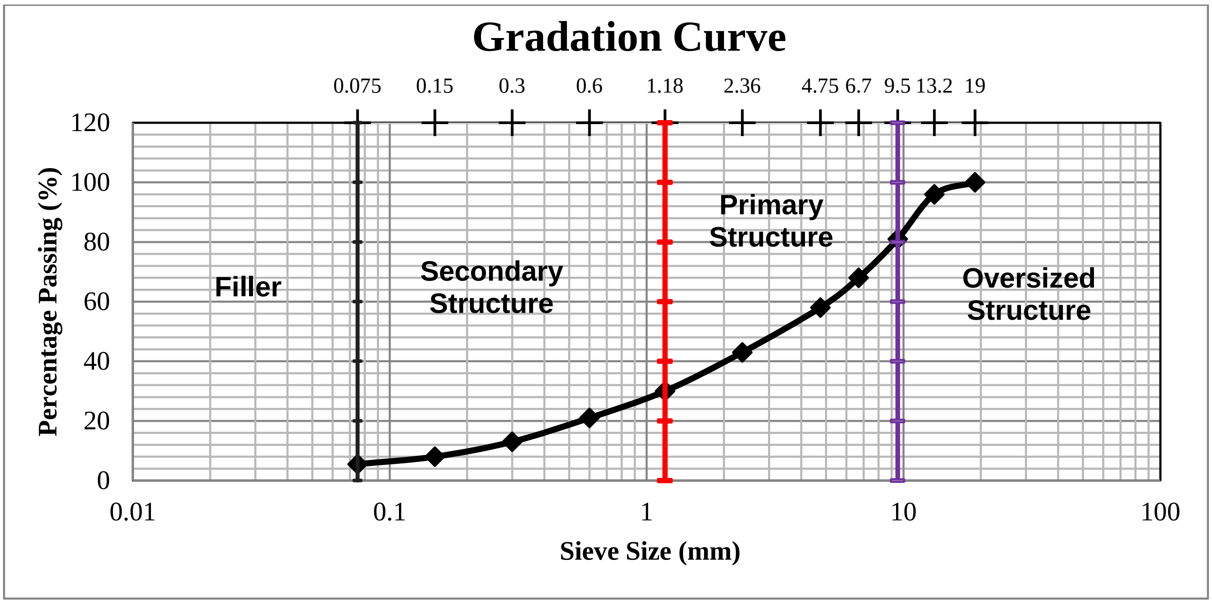

Figure 5.3: Sub-structure Ranges for the Considered Grading Target

Based on the research conducted in this study, a framework has been developed regarding the process of controlling the aggregate gradation as well as determining the Primary Structure range which is summarized in Figure 5.4.

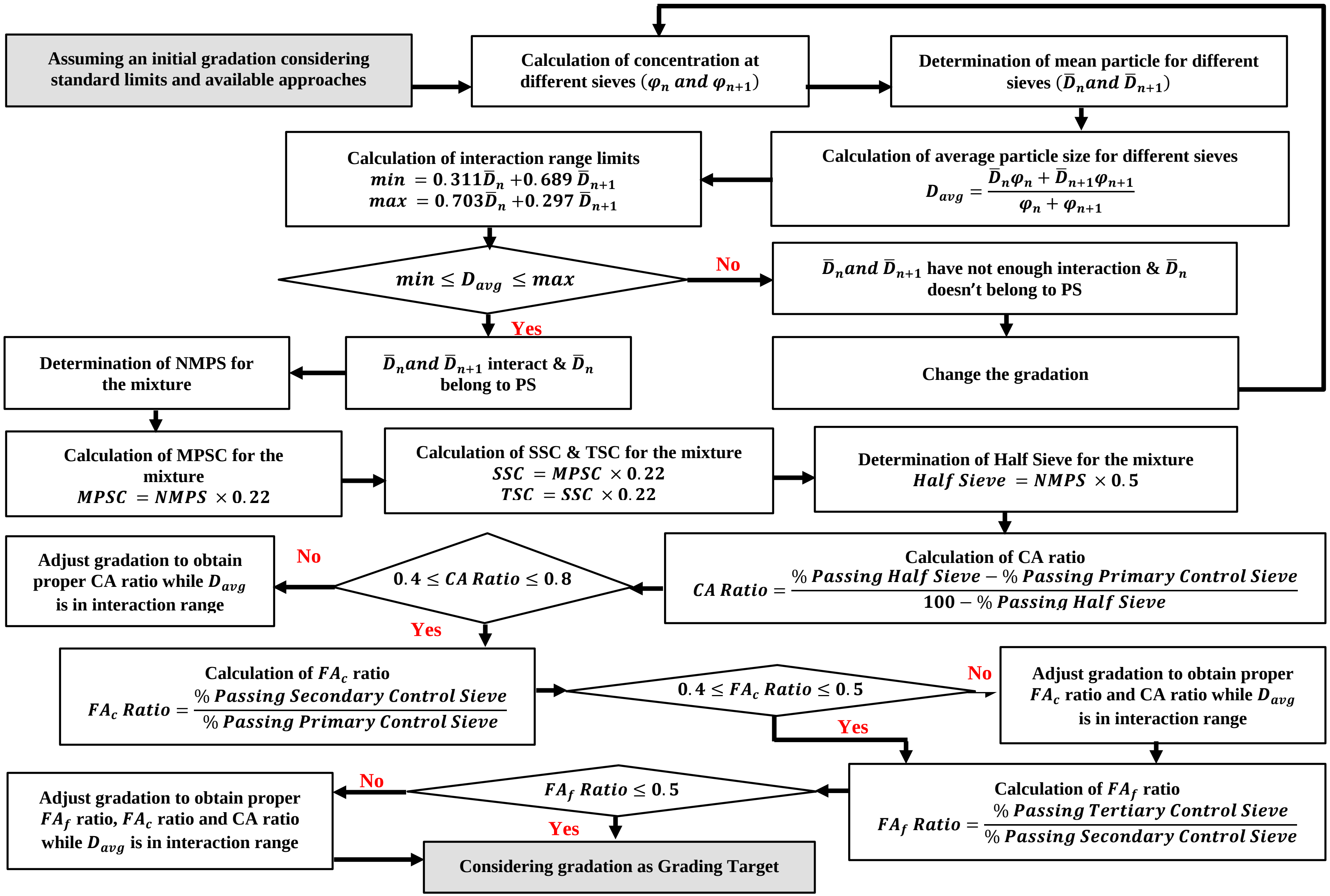

Figure 5.4: Flowchart of Controlling the Aggregate Gradation

5.4. Summary and Conclusions

The objective of asphalt mixture design is optimization of the asphalt mixture properties in terms of rutting resistance, durability, fatigue resistance, stability, flexibility, permeability, skid resistance, and workability. As mentioned in Section 2.8.1.1, many characteristics of asphalt mixtures such as rutting potential and durability are strongly related to the mechanical interlock of the aggregate skeleton and the aggregate gradations, so that, poor gradation in asphalt mixtures results in less durability and lower shear resistance due to less contact between the aggregate particles in asphalt mixtures. Accordingly, since aggregate gradation influences most of the important properties (i.e. stability, workability, durability, etc.) in asphalt mixtures, considering an adequate aggregate gradation based on comprehensive theories will result in improving different aspects of asphalt mixture design, construction, and operation. Unfortunately, although the aggregate gradation affects the asphalt mixture performance, in many cases, it is still selected based on generalized target gradations and local experience. If local experience is not available, the trial and error process will be employed for the selection of target gradation which is time consuming and costly. To this point, enhancing the understanding of the aggregates gradation and its effect on the resulting mixture would lead to a substantial improvement in asphalt mixtures performance.

The concepts discussed in this chapter provide an outline for asphalt mixture design ensuring the existence of a stress-transmitting path and coarse aggregate interlock in asphalt mixture. The establishment of these properties in aggregate blend will put an end to the trial and error process which is normally employed in the target gradation selection while providing an adequate aggregate structure to resist the deformation as well as developing proper volumetric properties in the final asphalt mixture, which will be further discussed in next chapters.

Chapter 6

Primary Tests on Asphalt Mixtures

6.1. Introduction

The durability and the resistance to environmental condition of asphalt mixtures are primarily determined by air voids and binder content, and generally evaluation of volumetric properties of asphalt mixtures. However, the asphalt mix design may include some other requirements than volumetric properties. Therefore, the asphalt mixture design process normally starts with volumetric design of mixture. In addition, some mechanical testing will be conducted for the design verification. In fact, the mechanical testing places a limit on the range of acceptable mixtures that meet the volumetric requirements.

Several studies have shown that there is a relationship between aggregates, binder, and air voids interact with each other and the asphalt mixture response to different loading conditions. Studying and understanding these interactions will result in the optimization of the asphalt mixture design through combining the available materials in the best possible way.

As mentioned previously, in this research, recycled materials are utilized to design an asphalt mixture. It has been well established that predicting the performance of asphalt mixtures containing recycled materials is very difficult given the inconsistency of materials and their complex interaction with natural materials. Therefore, in this research, different mixtures in terms of materials combination as well as the materials amount are considered for primary tests. Primary tests include the tests for the evaluation of asphalt mixtures volumetric properties. Based on the research methodology, volumetric design has been considered as the first level of tests to limit the range of mixtures which are acceptable in terms of volumetric requirements. The selected samples at this level will further be studied through resilient modulus test in order to select the most acceptable samples which satisfy the entire specifications and the requirements defined by the Australian Standards.

Accordingly, this section aims to describe briefly the considered combinations for asphalt mixture preparation, to explain the primary tests, including volumetric properties evaluation of different asphalt mixtures containing different materials with different percentages, and finally to discuss the results of these tests as well as the procedure for the selection of the most acceptable samples.

6.2. Volumetric Properties of Asphalt Mixtures

Asphalt mixtures are composed of three components including air voids, aggregates, and binder. The main volumetric properties of asphalt mixtures are those characteristics which are directly related to the volumetric proportions of the asphalt mixture components. Figure 6.1 clearly illustrates the volumetric properties of asphalt mixture.

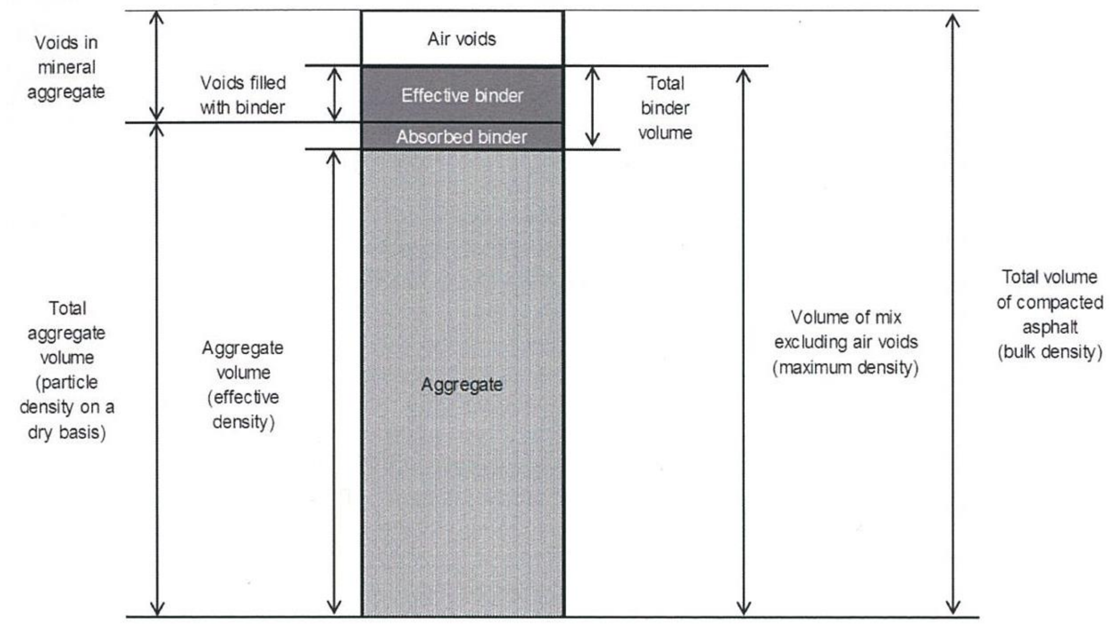

Figure 6.1: Volumetric Constituents of the Asphalt Mixture (AS 2150)

The definitions of the volumetric properties are as follows:

- **Voids in mineral aggregate (VMA)** which is the space between aggregate particles filled with air and asphalt in compacted asphalt mixture. This property is shown in Figure 6.1 as the sum of air volume and the volume of effective binder.
- **Voids Filled with Binder (VFB) or Voids Filled with Asphalt (VFA)** which is the space between aggregate particles occupied with binder.
- **Air Voids content (AV)** which is the percentage of air (by volume) in the intergranular space of a compacted asphalt mixture.
- **Effective Bitumen (B_e)** which is the volume of binder that is not absorbed into the aggregate particles.
- **Absorbed Bitumen (b)** which is the volume of binder that is absorbed into the aggregate particles.

In addition, the relationships for calculating some of the volumetric parameters are given in the following section.

6.3. Evaluation of Volumetric Properties of Asphalt Mix Specimens

As mentioned previously, all asphalt mixes in this research are dense graded asphalt (DGA) with nominal size of 14 (AC14) which are prepared in accordance with RMS T661 and RMS

T662 Test Methods (120 cycles of compaction) which are identical to AS2891.2.1(2014) and AS2891.2.2 (2014), respectively.

Table 6.1: Volumetric Parameters Requirements for DGA AC14

Parameter	Range	Typical Value	Description
Air void	3% - 6%	4%	Mixtures prepared in accordance with RMS T662
VMA	13% - 20%	≥ 15	Mixtures prepared in accordance with RMS T662
VFB	65% - 80%		Mixtures prepared in accordance with RMS T662
Filler-Binder Ratio	0.8 – 1.2		Mixtures prepared in accordance with RMS T662
Binder Film Index		$\geq 7.5\ microns$	Determined in accordance with AG:PT/T237 or AS 2891.8

The requirements for volumetric parameters of this type of mixture are summarized in Table 6.1. In addition, a summary of the relevant tests conducted to investigate the mix properties is explained in the following sections.

6.3.1. Sample Preparation

One of the important aims of sample preparation procedure is to ensure that laboratory prepared specimens have properties as close as possible to asphalt layers. In this study, the asphalt specimens were made from materials mixed in the Centre for Infrastructure Engineering (CIE) laboratory at Western Sydney University. About 4 kg of materials were used to produce three batches of laboratory asphalt mixtures for a finished specimen of diameter 100 ± 2 mm and height of 65 ± 5 mm, in accordance with AS2891.2.1 (2014) and AS2891.2.2 (2014). To prepare samples, both aggregate and filler were heated in an oven to be dried and achieve the required temperature of 180 °C. Binder was also heated in the oven to a temperature not exceeding the conditioning temperature of 150 °C. After mixing the binder and dry components together thoroughly in the mechanical mixer for not more than 3 minutes, the materials were apportioned into three test portions of about 1200 grams and conditioned at the conditioning temperature (150 °C) for about one hour. The test portions were then placed in preheated moulds for being compacted and obtaining compacted specimens for scheduled tests.

An IPC gyratory compactor was used for compacting the samples. The principle of gyratory compaction is illustrated in Figure 6.2. As shown in this figure, during compaction in gyratory compactor, a vertical compressive pressure is applied to the asphalt sample which is confined in a cylindrical mould. The mould is rotated about its vertical axis through a predetermined angle of gyration. The angle of gyration should be constant throughout the compaction process, which was checked before every compaction, in this research. Depending on the ultimate testing of the sample, compaction using gyropac can be terminated at a set height representing a predetermined volume and density of asphalt or after a set number of cycles. In this experiment, the samples were prepared following a specified number of gyrations (120 cycles). The number of gyrations depends on the design traffic loading for the specimen.

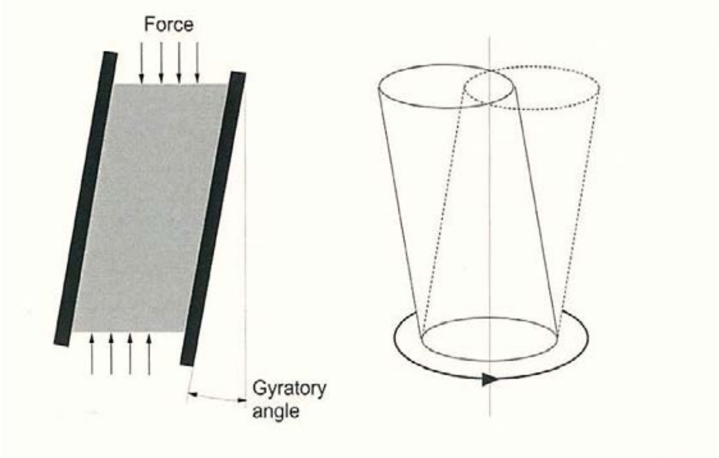

Figure 6.2: Principle of Gyratory Compaction

Sample preparation based on gyratory compaction more closely produces mix characteristics that match those found in the compacted pavement, as this method of compaction is developed to simulate the increasing load and tyre pressures of vehicles operating on pavements (Austroads, 2014). However, in accordance with AS/NZS 2891.2.2, the specimen and testing equipment for gyratory compaction should meet certain requirements. These requirements are presented in Table 6.2 for asphalt mixes with nominal size of 14 mm and medium and heavy traffic categories.

Table 6.2: Specimen and Testing Equipment Requirements (Medium/Heavy Traffic Category)

No.	Specimen and equipment Details	Value
1	Diameter of specimen (mm)	100 ± 2
2	Nominal height of specimen (mm)	65
3	Gyratory angle (°)	2 ± 0.1
4	Vertical loading stress (kPa)	240 ± 10
5	Number of gyration (cycle)	120

Based on discussion presented in Chapter 3, in this research, a group of specimens were made without recycled materials (0%) as reference to specimens made with 25% and 50% RCA and 0% glass substitution. In addition, in order to study the effect of glass on the bitumen absorption of asphalt mixtures containing RCA, two groups of specimens were also prepared with 25% and 50% RCA and glass at the rates of 10% and 20%. The substitution of glass was made on each sieve from #4 down to #8 in the designated substitution percentage.

6.3.2. Bulk Density Determination

Bulk density is an important parameter used for volumetric properties evaluation of asphalt mixtures. Bulk density is defined as the mass per volume of the compacted mixture. Bulk density is used as the basis for calculation of voids relationship and hence the internal air voids are considered in the calculation of bulk density. In this research, the bulk density of compacted specimens is determined using the pre-saturation procedure. This method is suitable for dense graded mixtures with internal air voids that are largely inaccessible to moisture resulting in low permeability. In this method, the mass of specimens were measured in air and water to determine the mixtures water absorption (WA) and bulk density (ρ_{bulk}) in accordance with AS/NZS 2891.9.2 (2014) from Equations 6.9 and 6.10.

$$\boldsymbol{WA = 100 \times \frac{(m_3 - m_1)}{(m_3 - m_2)}} \qquad (6.1)$$

$$\boldsymbol{\rho_{bulk} = \frac{m_1 \rho_w}{m_3 - m_2}} \qquad (6.2)$$

where, WA is water absorption by volume of asphalt specimen, as a percentage ; m_1 and m_3 are mass in air of the dry sample and the saturated sample, respectively, and m_2 is mass in water of the saturated sample, in grams.

6.3.3. Maximum Density Determination

Maximum density is the density of the mixture excluding air voids. The proportional difference between maximum density and bulk density provides the basis for air voids calculation in the mixture. In this research, the maximum density of loose sample of mixture is determined using the methylated spirits displacement procedure, in accordance with AS/NZS 2891.7.3 (2014). Based on this test method, firstly, the density of methylated spirit (ρ_m) was determined as 0.789 tonnes/m^3 using Equations 6.11 and 6.12.

$$\boldsymbol{V = \frac{(m_2 - m_1)}{0.997}} \qquad (6.3)$$

$$\boldsymbol{\rho_m = \frac{(m_3 - m_1)}{V}} \qquad (6.4)$$

where m_1 is mass of flask in grams, m_3 is mass of flask and methylated spirits at 25°C in grams, V is volume of the flask, in millilitres, m_2 is mass of flask and water at 25°C, in grams, and ρ_m is density of methylated spirits at 25°C, in tonnes per cubic metre.

Then, the maximum density (ρ_{max}) for each test portion were determined following the procedures provided in the test method and from Equation 6.13 to the nearest 0.001 tonnes/m^3.

$$\boldsymbol{\rho_{max} = \frac{(m_4 - m_1)\rho_m}{(m_3 - m_1) - (m_5 - m_4)}} \qquad (\mathbf{6.5})$$

where m_4 is mass of flask and test portion of asphalt, in grams and m_5 is mass of flask, test portion and methylated spirits, in grams.

6.3.4. Air Void and Volumetric Parameters Determination

The volumetric properties of asphalt mixture are used as an indicator for understanding the asphalt pavement performance. The durability and the resistance to environmental condition of DGA are mainly evaluated by air voids and other volumetric parameters. Therefore, the optimization of these parameters is crucial to service conditions of asphalt. The air void of asphalt mixture is one of the main parameters influencing the pavement performance throughout its service life. The asphalt mixtures with low voids are more prone to permanent deformation whereas asphalt mixtures with high air voids are more susceptible to oxidation, moisture damage, and cracking. The binder content as well as the degree of compaction during construction and in service affects the amount of voids in the pavement. Since the amount of voids in asphalt mixture have a direct relationship with density, the density of the asphalt mixture is used as surrogate to ensure the acceptable range of voids in asphalt mixtures. The density of asphalt mixture is usually evaluated based on two methods of bulk density measurement and determination of theoretical maximum density.

The voids of asphalt mixtures are determined, in this research, in accordance with Test Method Austroads AG: PT/T237 or AS/NZS 2891.8 (2014) using the results of primary tests and Equation 6.14.

$$\boldsymbol{AV = \frac{(\rho_{max} - \rho_{bulk})}{\rho_{max}}} \qquad (\mathbf{6.6})$$

Other important volumetric parameters which affect the durability and generally the performance of asphalt mixtures are voids filled with bitumen (VFB) and voids in mineral aggregates (VMA), which can be calculated using Equations 6.15 to 6.21.

$$VMA = 100 - \frac{\rho_{bulk}}{\rho_a}(100 - B) \tag{6.7}$$

$$VMA = AV + \left(\frac{\rho_{bulk} B_e}{\rho_b}\right) \tag{6.8}$$

$$VFB = \frac{VMA - AV}{VMA} \times 100 \tag{6.9}$$

$$VFB = \frac{100 B_e}{VMA} \times \frac{\rho_{bulk}}{\rho_b} \tag{6.10}$$

In the above equations, ρ_{max} is maximum density of asphalt (tonnes/m^3) as per AS2891.7.1, AS2891.7.2 or AS2891.7.3, ρ_{bulk} is bulk density of asphalt (tonnes/m^3) as per AS2891.9.1, AS2891.9.2 or AS2891.9.3, B is proportion by mass of binder in total mix, as a percentage, and ρ_b is density of binder (tonnes/m^3) as per AS2341.6 or AS2341.7.

In addition, ρ_a is bulk density of combined mineral aggregates (tonnes/m^3) as per AS2891.8 which can be obtained from the following equation:

$$\rho_a = \frac{100}{\frac{P_c}{\rho_c} + \frac{P_f}{\rho_f} + \frac{P_{fill}}{\rho_{fill}}} \times 100 \tag{6.11}$$

where, P_c , P_f, and P_{fill} are proportion of combined coarse aggregate, combined fine aggregate, and filler, respectively, which are stated as a percentage. In addition, ρ_c , ρ_f and ρ_{fill} are particle density of the combined coarse aggregate, particle density of the combined fine aggregate, and apparent particle density of the filler, respectively, which are obtained from preliminary tests on asphalt components.

Moreover, B_e is proportion by mass of the effective binder, as percentage which can be obtained from the following equation based on AS 2891.8.

$$B_e = B - b \tag{6.12}$$

in which, b is proportion by mass of the binder absorbed, as percentage which can be calculated from the following equation based on AS2891.8:

$$b = B - \rho_b\left(\frac{100}{\rho_{max}} - \frac{(100 - B)}{\rho_a}\right) \tag{6.13}$$

6.3.5. Binder Film Index Determination

Binder Film Index (BFI) is another important parameter that can be considered as an indicator of presence of sufficient binder in the asphalt mixture to provide adequate durability, cohesion, resistance to the effects of moisture and fatigue resistance. BFI can be calculated from the following equation:

$$BFI = \frac{B_e}{100 - B} \times \frac{\rho_a}{2.65 \times A} \times \frac{10^3}{\rho_b} \qquad (6.14)$$

In above equation, A is the surface area factor of aggregate blend, which can be calculated using Equation 6.23.

$$A = (2 + 0.02a + 0.04b + 0.08c + 0.14d + 0.30e + 0.60f + 1.60g) \times 0.20482 \qquad (6.15)$$

where, a, b, c, d, e, f, and g are percentage passing 4.75 mm, 2.36 mm, 1.18 mm, 0.60 mm, 0.30 mm, 0.15 mm, and 0.075 mm sieve, respectively. The surface area of aggregate blend is 5.678 based on the grading target considered in this research.

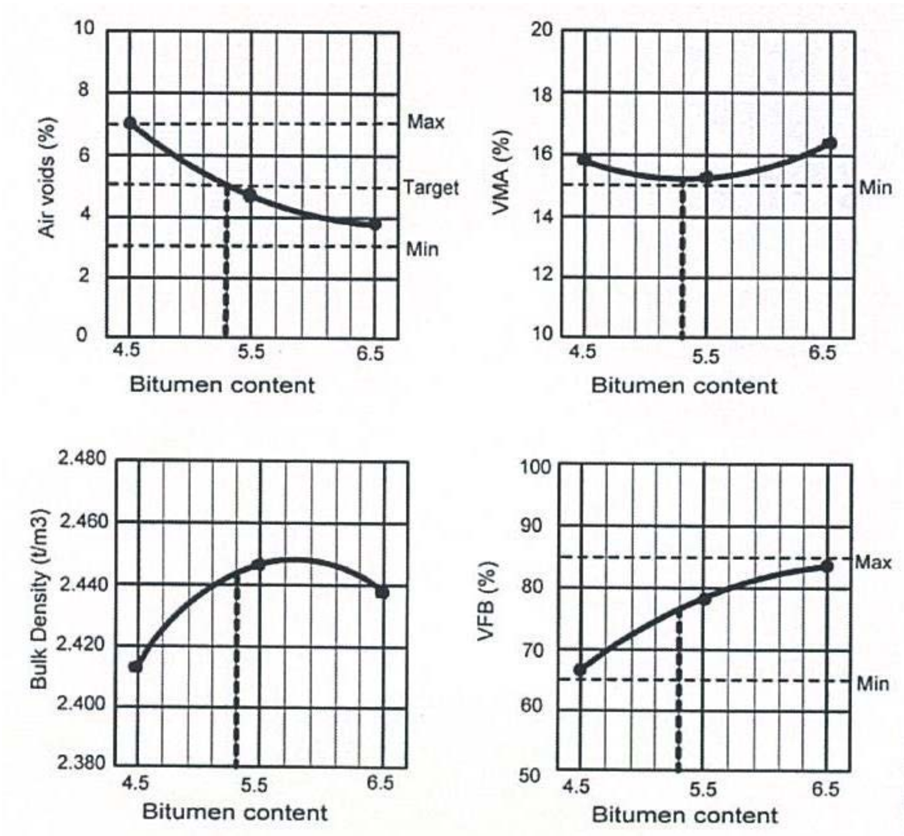

Figure 6.3: Typical Volumetric Properties Results (Austroads, 2014)

The results of the following tests may be presented graphically as shown in Figure 6.3 in order to select the optimum binder content that have air voids within the specified limits and is near the minimum value of VMA.

6.4. Results and Discussion on Volumetric Tests

The volumetric properties of the asphalt mixtures containing waste materials was determined and compared accordingly with the standards specifications. According to Austroads (2014), the essential parameters in the level 1 of mix design (Figure 6.4) include air voids in total mix, VMA, and VFB.

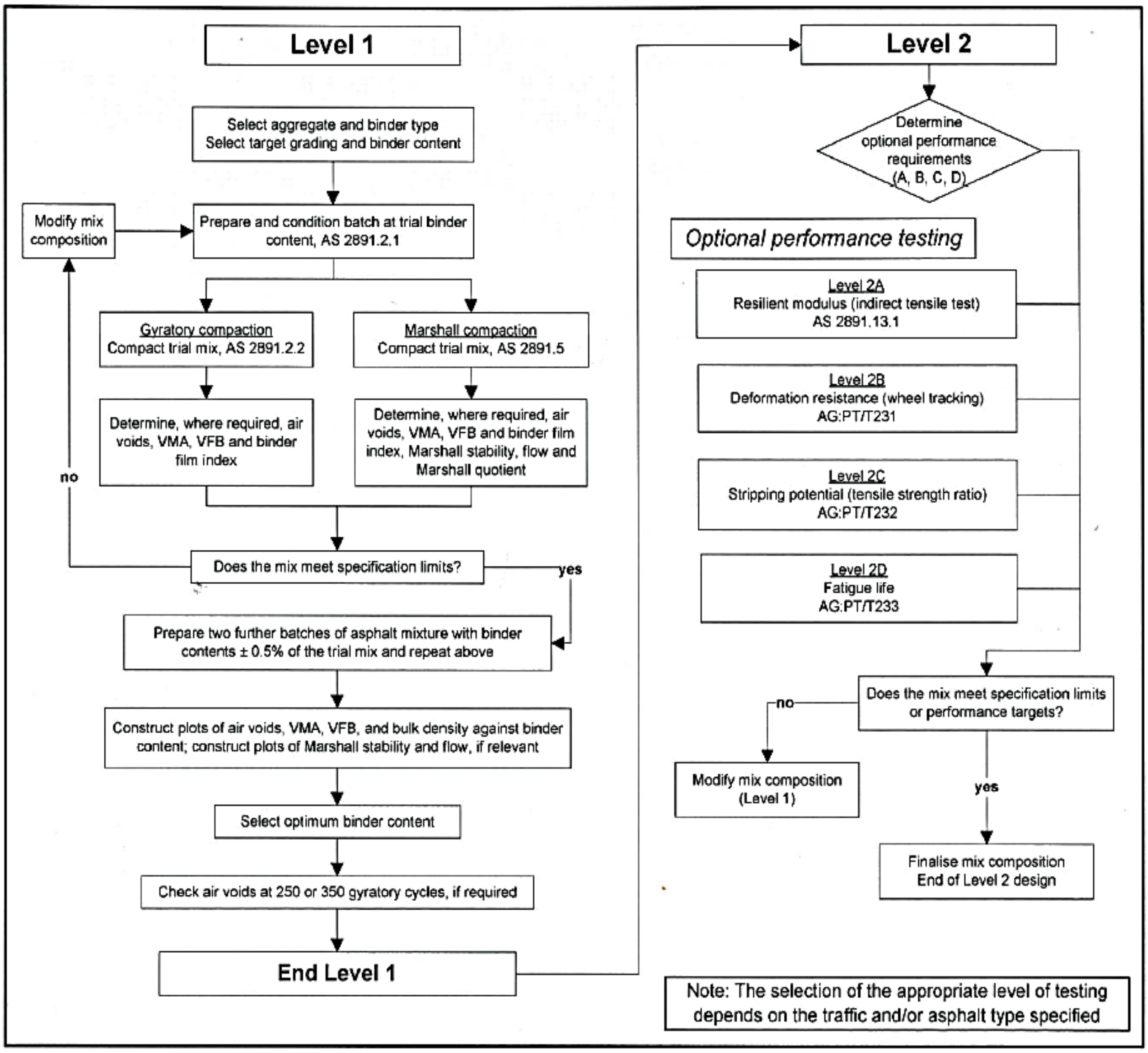

Figure 6.4: Mix Design Procedure (Austroads, 2014)

The volumetric mixture design standards require that for the structural design level of 5×10^6 ESAs to 2×10^7 ESAs, and a nominal maximum aggregate size of 14 mm, VMA should be a minimum of 15% while the VFB specification range is between 60% and 80%. Therefore, in the following sections the volumetric properties of mixtures containing RCA as coarse aggregate (with and without glass as fine aggregate) are analysed.

6.4.1. Volumetric Analysis of Asphalt Mixtures Containing RCA

Table 6.3 presents the properties obtained for asphalt mixtures containing RCA with different bitumen content at selected level of gyrations (120 cycles).

Table 6.3: Volumetric Properties of Asphalt Mixtures Containing RCA

Design Mix	AV (%)	VMA (%)	VFB (%)	Binder Film Index (μm)	Filler-Binder Ratio	Bulk Density (gr/cm^3)	Water Absorption (%)	Height (mm)
B100-4.5	7.2	16.4	59.6	6.9	1.2	2.398	0.46	69.0
B100-5	5.4	15.4	69.0	7.5	1.1	2.439	0.17	67.0
B100-5.5	4.1	15.8	78.6	8.7	1.0	2.441	0.11	66.3
B75-5	6.7	15.3	59.7	6.4	1.1	2.398	0.43	69.7
B75-5.5	5.4	15.3	68.7	7.4	1.0	2.410	0.23	67.6
B75-6	4.8	15.7	73.8	8.2	0.9	2.411	0.15	67.0
B50-5	7.3	15.3	55.3	5.9	1.1	2.355	0.56	73.6
B50-5.5	6.0	15.1	64.0	6.8	1.0	2.371	0.31	71.0
B50-6	5.4	15.5	69.1	7.5	0.9	2.373	0.24	68.9
B50-6.5	4.5	16.5	77.1	9.0	0.9	2.359	0.16	68.2

6.4.1.1. Determination of Optimum Bitumen Content

To determine the optimum bitumen content for the mixture, the procedure indicated by Australian standards, AGPT04B-14, was followed in this research. Three specimens at each bitumen content (4.5, 5, 5.5, 6, and 6.5%) were tested for maximum density, bulk density, and subsequently air voids and VMA calculations. the results of these tests and calculations are used to select the optimum bitumen content that provides air voids within the specified limits and VMA near to the minimum value.

According to the results obtained, Figure 6.5 illustrates the effect of bitumen content and RCA content on air voids of asphalt mixtures.

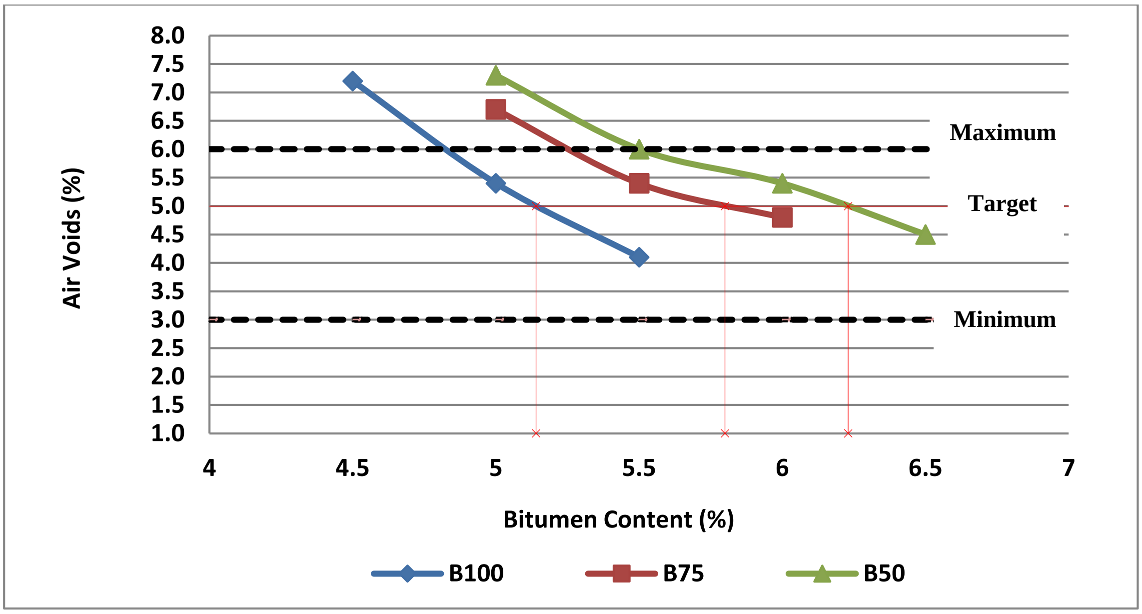

Figure 6.5: Effect of Bitumen Content and RCA Content on Air Voids of Mix I Containing Virgin Aggregate and Mix II and Mix III Containing RCA as Coarse Aggregate

As discussed previously, the air void content in the mix is a function of bitumen content, degree of compaction and VMA. The air void percentage in the mixture affects mix stiffness, fatigue resistance, and durability. As shown in this figure, air voids decrease with the bitumen content increase. Air voids of mixtures made with RCA are substantially higher than the control mixtures because of porous cement paste attached to the virgin aggregates and also porous structure of some aggregates in the RCA.

Generally, asphalt mixtures should be have the lowest practical air voids in order to reduce the binder ageing and the permeability and subsequently stripping problems. However, referring to Austroads (2014), plastic flow and subsequently bleeding, flushing, shoving or permanent deformation of the pavement may occur if the air voids is too low (less than about 2%). Accordingly, as can be observed in Figure 6.5, some of the mixtures (i.e. B100-4.5, B75-5 and B50-5) can not be acceptable in terms of air voids requirements.

As discussed previously, the optimum bitumen content can be obtained based on design air voids of 5%. Based on the results, the optimum bitumen content was found to be 5.1% for reference samples (0% RCA), 5.8% for samples with 25% RCA and 6.2% for samples with 50% RCA, as illustrated in Figure 6.5.

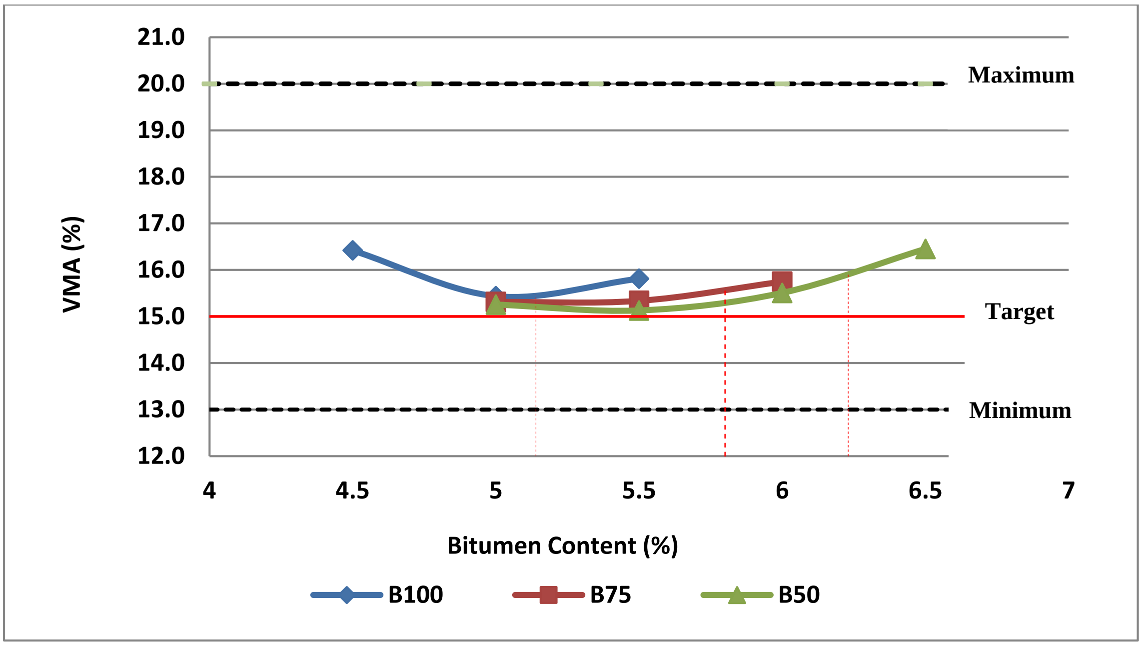

Figure 6.6: Effect of Bitumen Content and RCA Content on VMA of Mix I Containing Virgin Aggregate and Mix II and Mix III Containing RCA as Coarse Aggregate

In addition, VMA is another important volumetric property which should be checked for selection of the final bitumen content. VMA is a function of the gradation and the particle shape and surface texture of the aggregate particles. VMA should be large enough to provide sufficient amount of air voids in the compacted mixture for ensuring the asphalt mixture stability while leaving enough space for binder to ensure the mixture durability. If VMA be too low, binder would be insufficient for cohesion and durability whereas too high VMA results in more costly asphalt mixtures due to increased binder volume to satisfy the air voids requirements. The variation of VMA with bitumen content for Mix I, Mix II and Mix III are shown in Figure 6.6. As can be observed in this figure, VMA increases with bitumen content after a minimum point. Furthermore, VMA of mixtures containing RCA is quite lower than the control samples which can be as a result of higher bitumen absorption of RCA resulting in the lower amount of not absorbed binder (effective binder). As illustrated in Figure 6.6, mixtures at optimum bitumen content meet the requirements of 15% (minimum) for VMA.

6.4.1.2. Determination of Bulk Density and Water Absorption

Bulk density is an important parameter used for volumetric properties evaluation of asphalt mixtures. Bulk density of Mix I, Mix II and Mix III are shown in Figure 6.7. It can be seen that bulk densities of mixtures containing RCA are considerably lower than bulk density of mixtures made with virgin aggregates (Mix I), mainly due to the low density of cement paste and RCA particles.

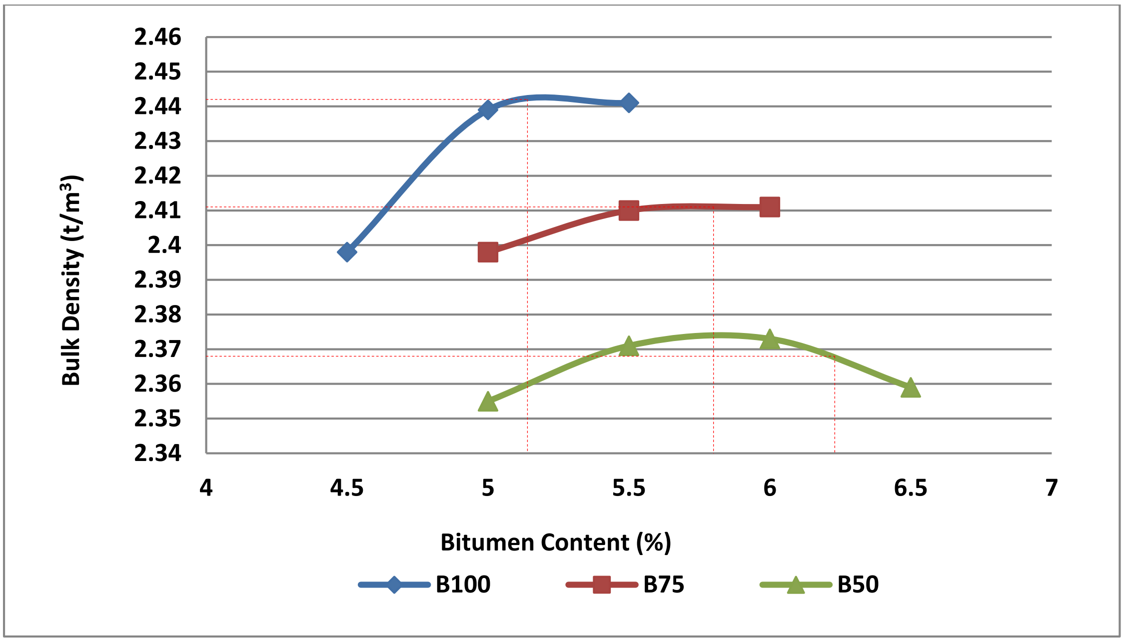

Figure 6.7: Effect of Bitumen Content and RCA Content on Bulk Density of Mix I Containing Virgin Aggregate and Mix II and Mix III Containing RCA as Coarse Aggregate

Furthermore, the experimental results shows that the water absorption of the mixtures increase with the increase in amount of RCA at the same bitumen content due to porous structure of RCA, as expected and shown in Figure 6.8.

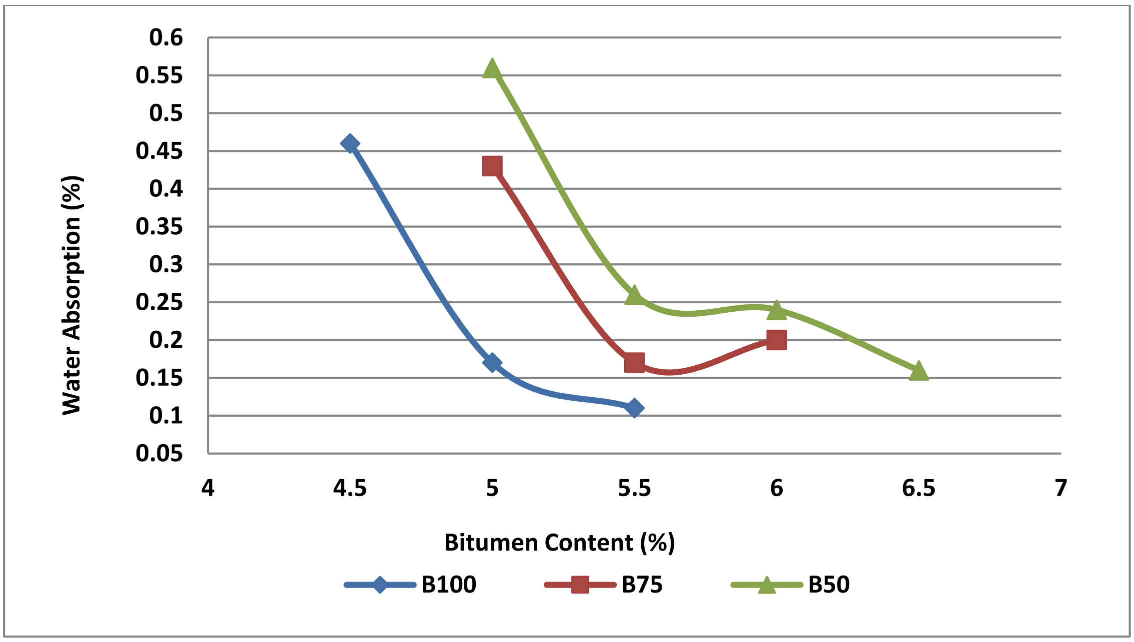

Figure 6.8: Effect of Bitumen Content and RCA Content on Water Absorption of Mix I Containing Virgin Aggregate and Mix II and Mix III Containing RCA as Coarse Aggregate

6.4.1.3. Determination of Voids Filled with Binder (VFB)

Another important volumetric parameter is voids filled with binder (VFB). VFB is defined as the ratio of the effective binder (by volume) and the VMA. Mixtures with low VFB are dry and lack durability, cohesion and fatigue resistance.

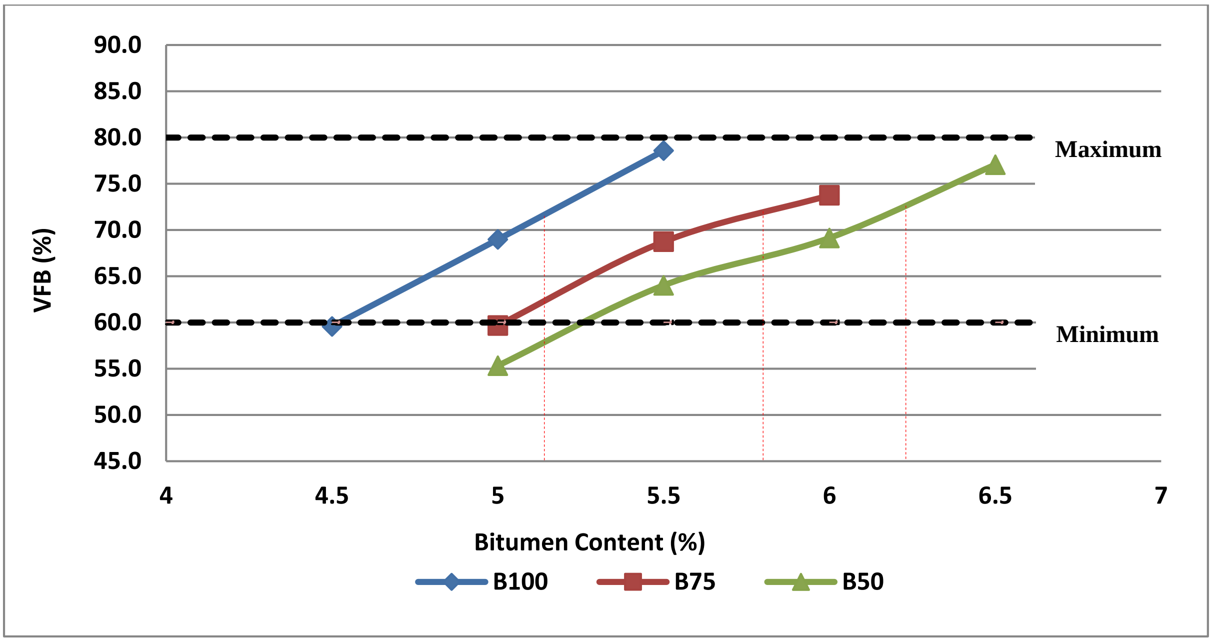

Figure 6.9: Effect of Bitumen Content and RCA Content on VFB of Mix I Containing Virgin Aggregate and Mix II and Mix III Containing RCA as Coarse Aggregate

These mixtures may also be more permeable, whereas asphalt mixtures with too high VFB can become unstable and susceptible to rutting. Figure 6.9 shows the variation of VFB with the RCA and content bitumen content. The VFB values obtained for asphalt mixtures containing RCA are relatively lower than the control samples because of higher absorption of RCA leading to less amount of not absorbed binder (effective binder).

6.4.1.4. Determination of Binder Film Index (BFI)

Binder Film Index (BFI) is another parameter that can be considered at the volumetric design stage as a guide to the incorporation of sufficient binder in the asphalt mixture to ensure adequate durability, cohesion, resistance to the effects of moisture and fatigue resistance (Austroads, 2014). BFI is a function of surface area of filler and the aggregates as well as the effective bitumen content. According to the results obtained, the binder film index of Mix I, Mix II and Mix III are shown in Figure 6.10.

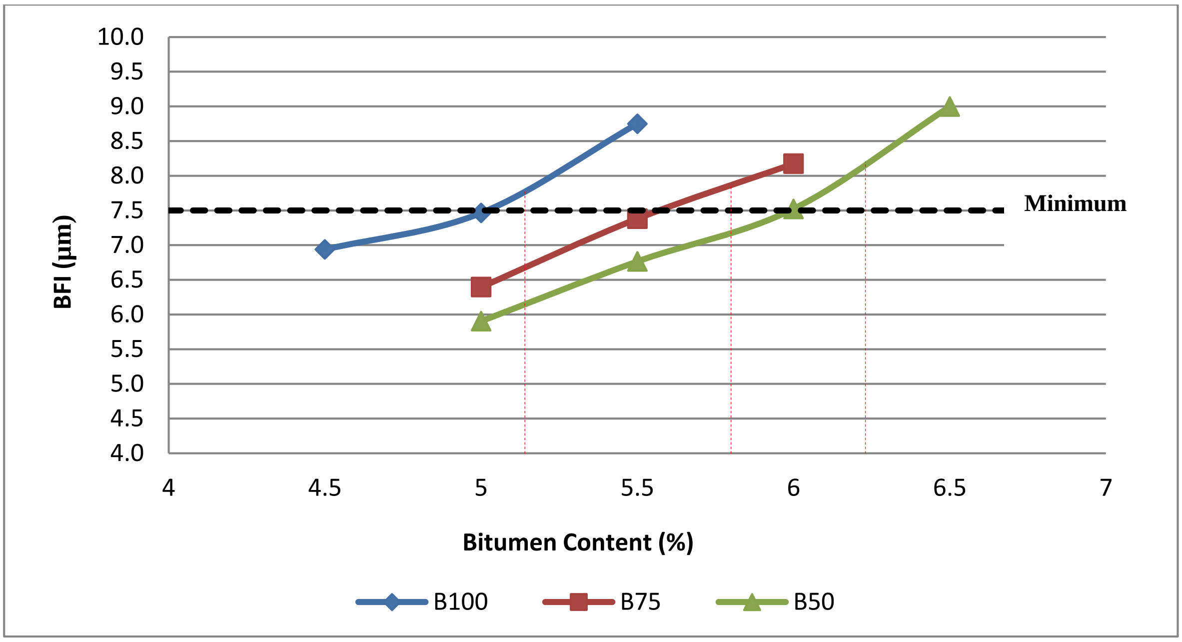

Figure 6.10: Effect of Bitumen Content and RCA Content on BFI of Mix I Containing Virgin Aggregate and Mix II and Mix III Containing RCA as Coarse Aggregate

As can be observed in this figure, BFI values for Mix II and Mix III are lower than BFI for control samples (Mix I) at the same bitumen content since more bitumen is absorbed by the mixtures incorporating RCA resulting in less aggregate particles coating due to the reduction of available binder for this purpose. As can be expected, the BFI is increased with the increase in bitumen content. In addition, as can be seen in Figure 6.10, all samples at their optimum bitumen content meet the minimum requirements of 7.5 µm for BFI.

6.4.2. Volumetric Analysis of Asphalt Mixtures Containing RCA and Glass

Since asphalt mixtures made with RCA have the problem of high absorption, as discussed in previous sections, it is desired to optimize the absorption characteristics of these asphalt mixtures by adding recycled glass, which is one of the objectives of this research work. To this end, different asphalt mixtures containing RCA with three glass contents of 0%, 10% and 20% (by weight of fine aggregates) were prepared for evaluating the effect of addition of glass to RCA-basalt asphalt mixtures.

Table 6.4: Volumetric Properties of Asphalt Mixtures Containing RCA and Glass

Design Mix	AV (%)	VMA (%)	VFB (%)	Binder Film Index (μm)	Filler-Binder Ratio	Bulk Density (gr/cm^3)	Water Absorption (%)	Height (mm)
B75-G10-5	5.9	14.8	63.9	6.6	1.1	2.394	0.29	68.2
B75-G10-5.5	4.7	15.1	73.0	7.7	1.0	2.400	0.16	67.1
B75-G10-6	4.3	15.3	76.3	8.2	0.9	2.406	0.14	66.1
B75-G20-5	5.6	14.5	65.2	6.6	1.1	2.384	0.27	68.1
B75-G20-5.5	4.6	14.6	72.7	7.4	1.0	2.394	0.15	67.8
B75-G20-6	4.2	14.9	76.3	7.9	0.9	2.398	0.12	65.7
B50-G10-5	6.9	14.9	56.9	5.9	1.1	2.352	0.54	69.9
B50-G10-5.5	5.5	14.9	67.0	7.0	1.0	2.363	0.27	69.7
B50-G10-6	4.9	15.3	72.1	7.7	0.9	2.366	0.18	69.0
B50-G20-5	6.5	14.7	59.2	6.0	1.1	2.339	0.30	69.4
B50-G20-5.5	5.3	14.6	67.7	6.9	1.0	2.353	0.21	68.6
B50-G20-6	4.2	14.7	75.8	7.7	0.9	2.364	0.14	67.7

For this purpose, different combination of aggregates at different rates of bitumen content of 5%, 5.5% and 6% were considered to make specimens with 100 mm diameter at required level of gyration (120 cycles) for considered traffic category. Table 6.4 presents the results of volumetric analysis for asphalt mixtures containing RCA and glass with different bitumen content.

6.4.2.1. Determination of Optimum Bitumen Content for Asphalt Mixtures with Glass

Similar to procedure explained in Section 6.4.1.1, the optimum bitumen content for the asphalt mixtures made with RCA and glass were determined in accordance with Australian standards. To this end and in order to compare the effect of glass addition to RCA-basalt mixtures, three specimens at different bitumen content of 5%, 5.5% and 6% made with 25% RCA and recycled glass at rates of 10% and 20% were prepared and tested for bulk density, maximum density, and subsequently VMA and air voids calculations.

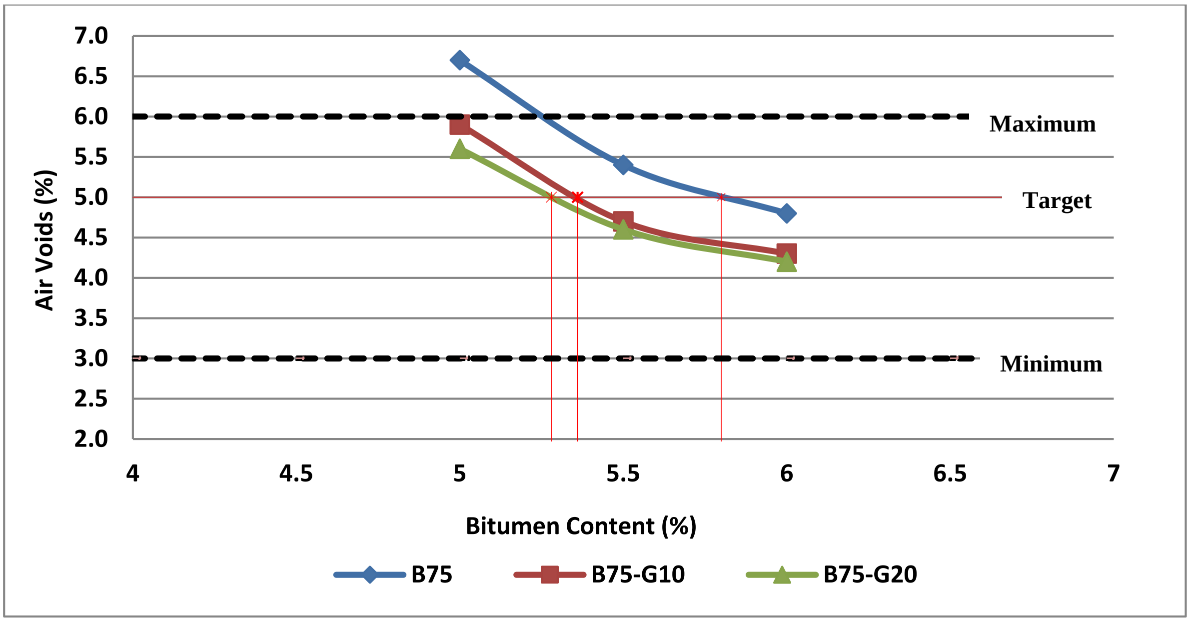

Figure 6.11: Effect of Bitumen Content and Glass Content on Air Voids of Mix II Containing 25% RCA without Glass and Mix IV and Mix V Containing 25% RCA as Coarse Aggregate and Glass as Fine Aggregate

The same bitumen content and glass content was also considered for preparation of samples containing 50% RCA.

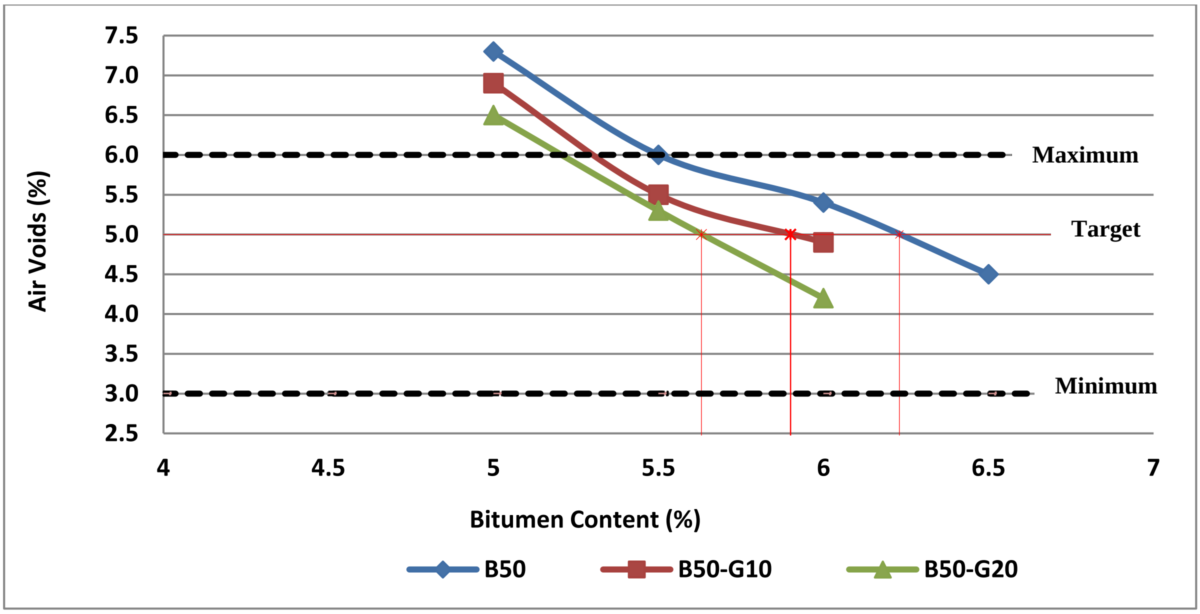

Figure 6.12: Effect of Bitumen Content and Glass Content on Air Voids of Mix III Containing 50% RCA without Glass and Mix VI and Mix VII Containing 50% RCA as Coarse Aggregate and Glass as Fine Aggregate

The effect of bitumen content and glass content on air voids of asphalt mixtures is illustrated in Figures 6.11 and 6.12. As can be clearly observed in these figures, air voids of mixtures containing glass are lower than the mixtures containing RCA without glass due to highly hydrophobic property of glass. In addition, air voids decrease with the increase of bitumen content in all samples.

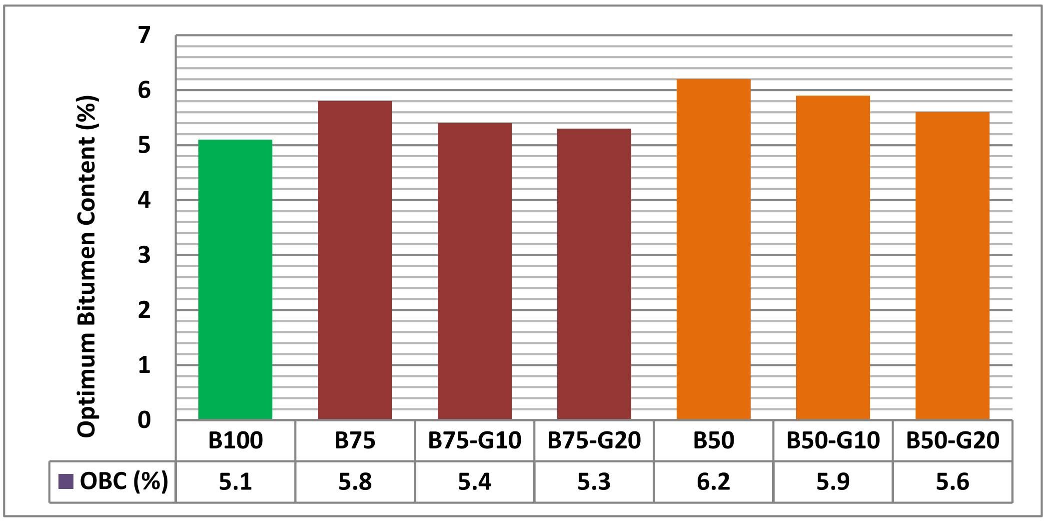

Figure 6.13: Optimum Bitumen Content (OBC) of Asphalt Mixtures

Importantly, the results of volumetric analysis and air voids calculations based on the bulk density test and maximum density test reveal that the optimum bitumen content of asphalt mixtures varies with the amount of recycled glass used, so that mixtures composed of more glass require less bitumen, as presented in Figures 6.11 and 6.12. The results of obtained optimum bitumen content based on all specimens studied in this research work are presented in Figure 6.13. Furthermore, as discussed previously, VMA is another parameter required to be considered in selecting optimum bitumen content. The variation of VMA with bitumen content for mixtures containing 25% and 50% RCA made with recycled glass or without glass are illustrated in Figures 6.14 and 6.15, respectively.

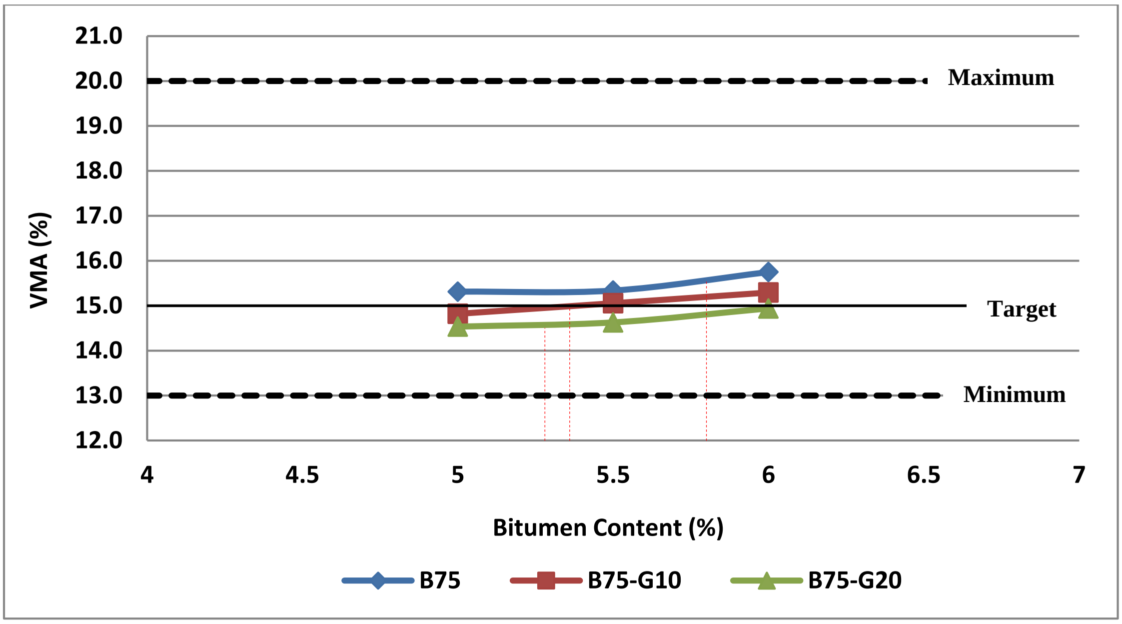

Figure 6.14: Effect of Bitumen Content and Glass Content on VMA of Mix II Containing 25% RCA without Glass and Mix IV and Mix V Containing 25% RCA as Coarse Aggregate and Glass as Fine Aggregate

The test results on different samples showed that VMA of mixtures containing glass is quite lower than the samples made with RCA without glass. It can be due to the lower air voids and lower particle density of combined mineral aggregates in samples made with RCA and glass. However, in all group of Mixes, VMA increases with bitumen content after a minimum point.

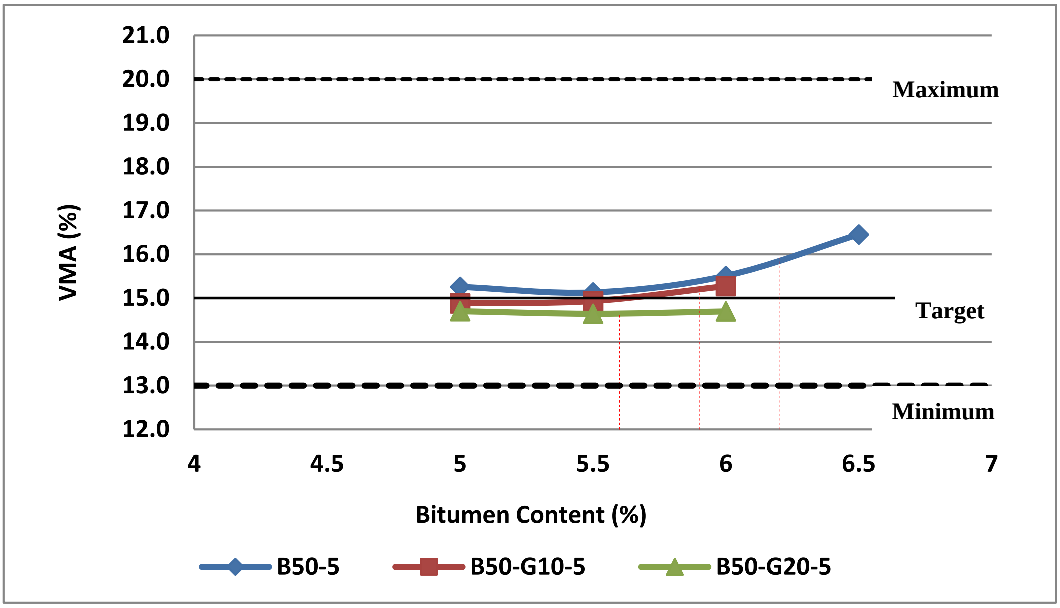

Figure 6.15: Effect of Bitumen Content and Glass Content on VMA of Mix III Containing 50% RCA without Glass and Mix VI and Mix VII Containing 50% RCA as Coarse Aggregate and Glass as Fine Aggregate

As illustrated in Figures 6.14 and 6.15, mixtures without glass and containing 10% glass meet the requirements of 15% (minimum) for VMA at their optimum bitumen content. However, VMA for other mixtures containing 20% glass is still in acceptable range.

6.4.2.2. Determination of Bulk Density and Water Absorption for Asphalt Mixtures with Glass

Bulk density test was conducted on specimens containing glass and RCA to measure the bulk density and water absorption of samples.

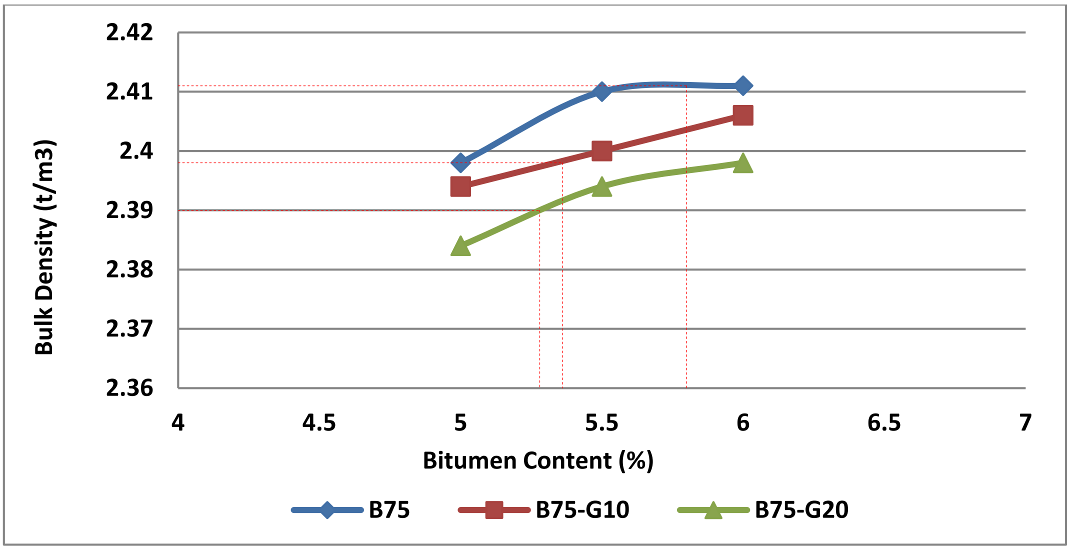

Figure 6.16: Effect of Bitumen Content and Glass Content on Bulk Density of Mix II Containing 25% RCA without Glass and Mix IV and Mix V Containing 25% RCA as Coarse Aggregate and Glass as Fine Aggregate

Based on the results obtained from bulk density test on two groups of asphalt mixtures with 25% and 50% RCA in combination with different rates of recycled glass, it can be noticed that the bulk density decreases as the glass content increases due to lower bulk density of glass in comparison with basalt, as illustrated in Figures 6.16 and 6.17.

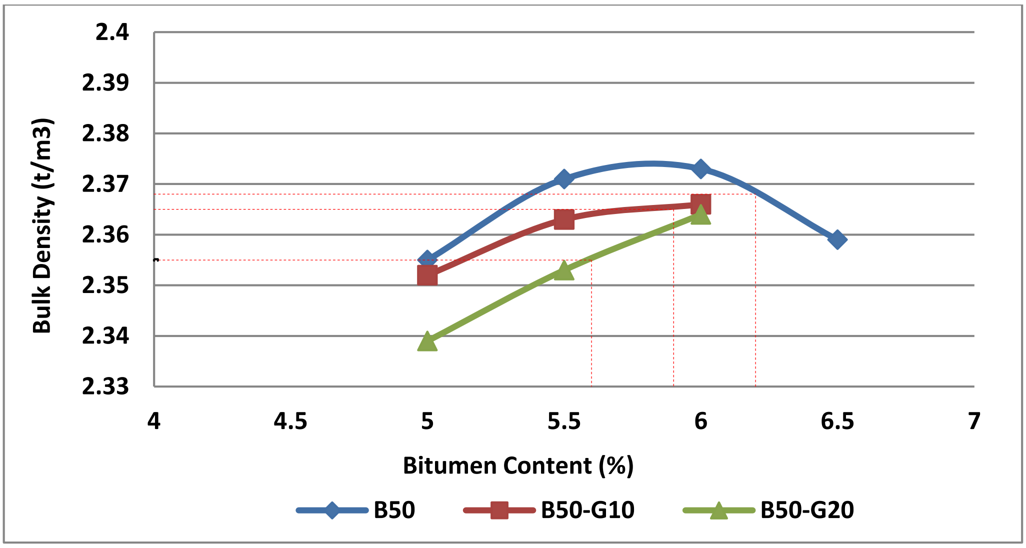

Figure 6.17: Effect of Bitumen Content and Glass Content on Bulk Density of Mix III Containing 50% RCA without Glass and Mix VI and Mix VII Containing 50% RCA as Coarse Aggregate and Glass as Fine Aggregate

In addition, the data obtained from bulk density test indicate that water absorption decreases with respect to the amount of recycled glass, and asphalt mixtures containing glass appear to have low water absorption because of the hydrophobic property of recycled glass. Figure 6.18 illustrates the water absorption of all different specimens.

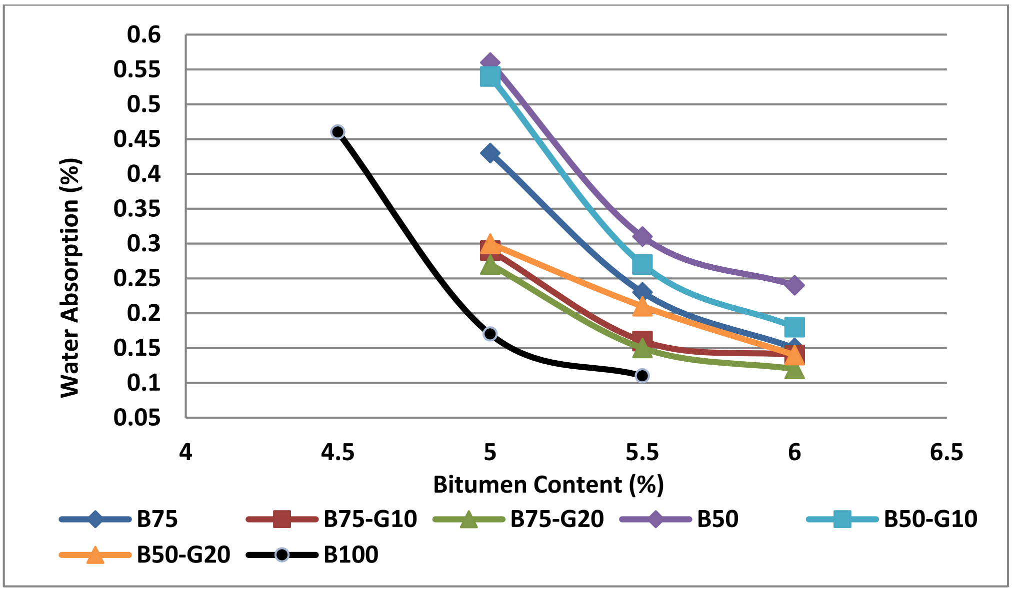

Figure 6.18: Comparison of Water Absorption in Asphalt Mixtures with Different Aggregate Type

6.4.2.3. Determination of Voids Filled with Bitumen (VFB) for Asphalt Mixtures with Glass

Figures 6.19 and 6.20 illustrate the variation of VFB with the bitumen content and glass content. As previously stated, VFB is one of the indicators of asphalt mixture performance in terms of durability, fatigue resistance and susceptibility to rutting.

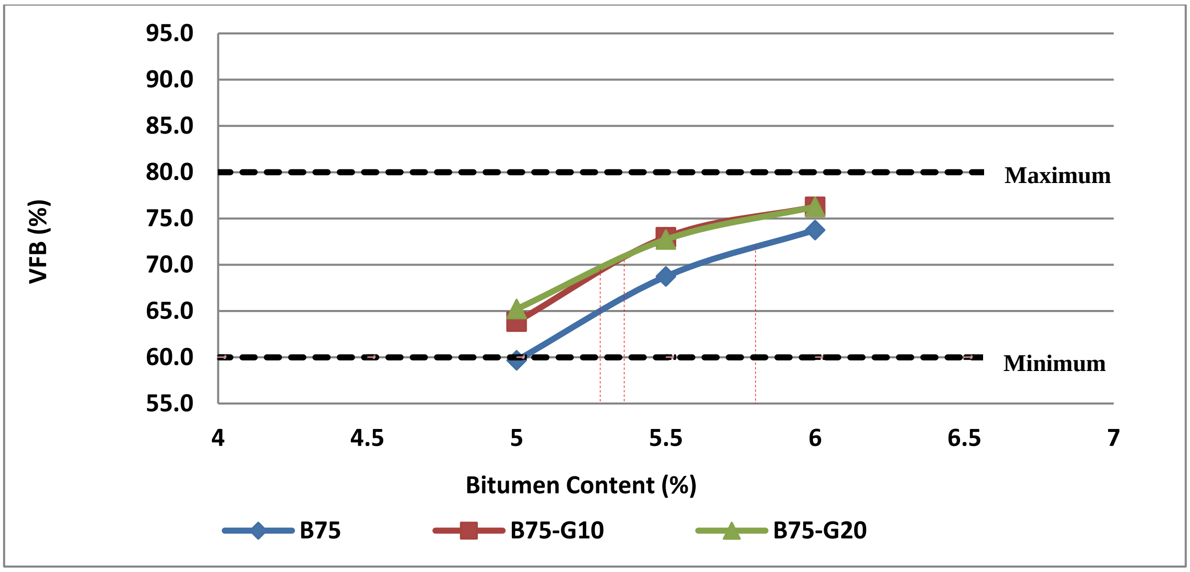

Figure 6.19: Effect of Bitumen Content and Glass Content on VFB of Mix II Containing 25% RCA without Glass and Mix IV and Mix V Containing 25% RCA as Coarse Aggregate and Glass as Fine Aggregate

As can be observed in these figures, the values obtained for asphalt mixtures made with glass and RCA are higher than the samples containing RCA without glass due to lower degree of absorption resulting in increased effective binder.

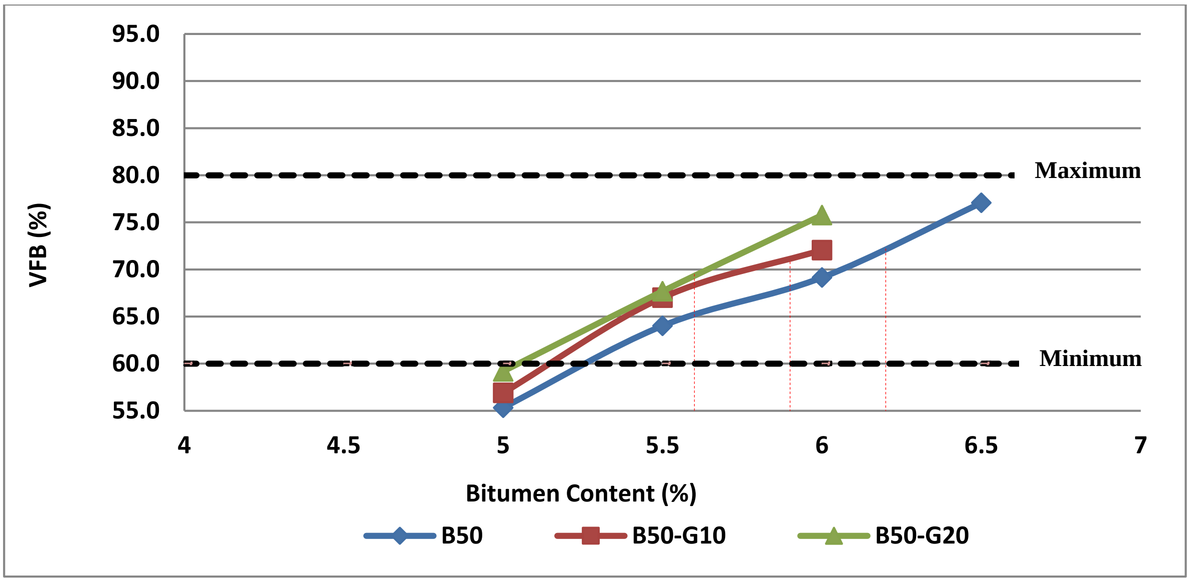

Figure 6.20: Effect of Bitumen Content and Glass Content on VFB of Mix III Containing 50% RCA without Glass and Mix VI and Mix VII Containing 50% RCA as Coarse Aggregate and Glass as Fine Aggregate

6.4.2.4. Determination of Binder Film Index (BFI) for Asphalt Mixtures with Glass

As discussed previously, BFI is an indicator of adequate cohesion and the incorporation of sufficient binder in the asphalt mixture.

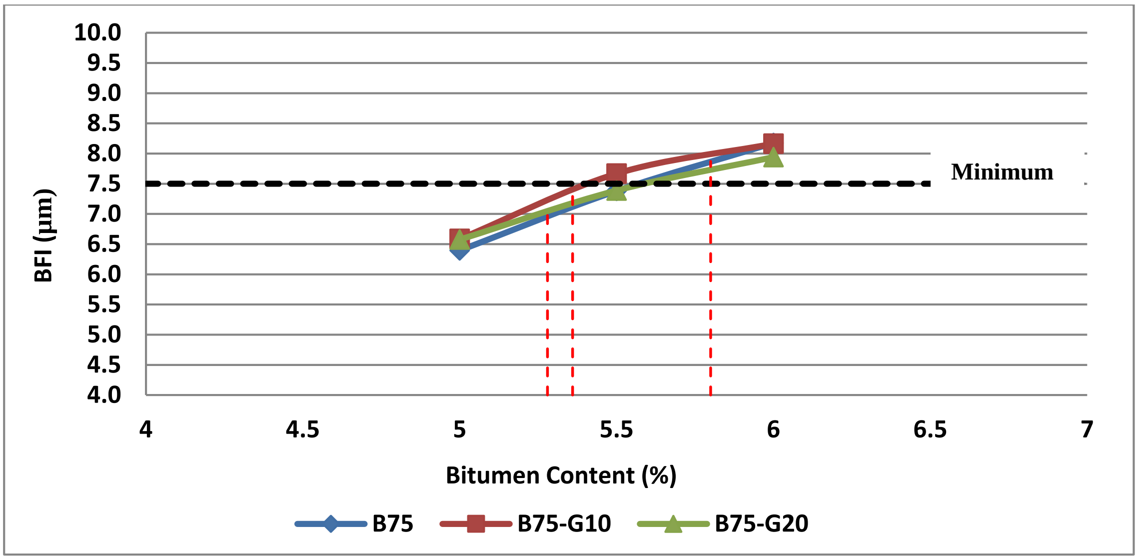

Figure 6.21: Effect of Bitumen Content and Glass Content on BFI of Mix II Containing 25% RCA without Glass and Mix IV and Mix V Containing 25% RCA as Coarse Aggregate and Glass as Fine Aggregate

According to the results obtained, the binder film index of different mixtures were investigated in this research and the variation of BFI with glass and bitumen content in two groups of samples containing 25% and 50% RCA are illustrated in Figures 6.21 and 6.22.

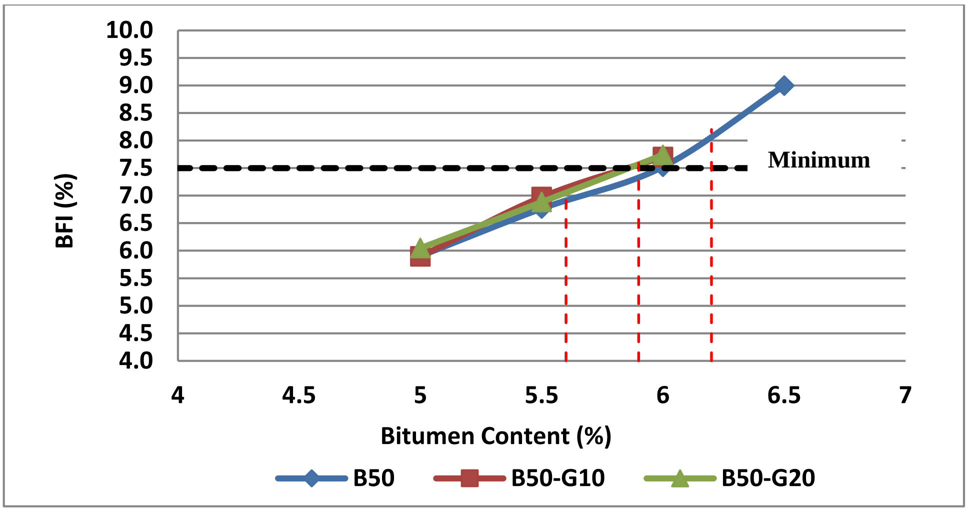

Figure 6.22: Effect of Bitumen Content and Glass Content on BFI of Mix III Containing 50% RCA without Glass and Mix VI and Mix VII Containing 50% RCA as Coarse Aggregate and Glass as Fine Aggregate

As can be observed, both samples containing glass have slightly higher binder film thickness than samples without glass. However, the comparison of samples at their optimum bitumen content shows that BFI for samples containing 20% glass is less than the minimum requirements of 7.5 microns.

6.4.3. Volumetric Analysis of Asphalt Mixtures at Optimum Bitumen Content

The results of volumetric properties evaluation of asphalt mixtures with different combination of aggregates at their optimum content are presented in Table 6.5.

Table 6.5: Volumetric Properties of Asphalt Mixtures at Optimum Bitumen Content

Design Mix	Optimum Bitumen Content (%)	Bulk Density (gr/cm^3)	VMA (%)	VFB (%)	Binder Film Index (µm)	Filler-Binder Ratio
B100	5.1	2.442	15.5	71.5	7.8	1.1
B75	5.8	2.411	15.6	72.0	7.9	0.9
B75-G10	5.4	2.398	14.9	70.5	7.5	1.0
B75-G20	5.3	2.390	14.5	69.5	7.1	1.0
B50	6.2	2.368	15.9	72.2	8.2	0.9
B50-G10	5.9	2.365	15.2	70.8	7.6	0.9
B50-G20	5.6	2.355	14.6	69.5	7.1	1.0
Standard Limit	**-**	**-**	**13% -20%**	**60% - 80%**	**-**	**0.8 – 1.2**
Typical Value	**-**	**-**	**15% (min)**		**7.5µm (min)**	**-**

As shown in Table 6.5, the results of tests on asphalt mixtures indicate that all mixtures made of RCA meet the standard limits for volumetric properties. However, their high bitumen absorption necessitates the study of volumetric properties of asphalt mixtures made with RCA in combination with recycled glass. To this point, as can be observed in Table 8, all asphalt mixtures containing RCA and 10% recycled glass, except for VMA of sample B75-G10, meet the requirements specified by relevant Australian Standards and hence deemed appropriate for considering as asphalt mix design. In addition, results for samples with RCA and 50% of recycled glass, except for binder film index and VMA (which are shown in bold in Table 6.5), meet standards typical values. However, VMA for these samples are within the limits specified by Australian Standards.

In addition, for all samples at different rates of bitumen content, as presented in Tables 6.3 and 6.4, it can be seen that increasing the bitumen content decreases the specimen height. This effect can be due to the extra lubrication supplied by the hot bitumen during compaction permitting the better and quicker compaction for the same compactive effort.

6.5. Summary and Conclusions

The primary test results on asphalt mixtures containing RCA with and without glass indicate that asphalt mixtures containing RCA have lower bulk density, VMA, VFB and BFI than control mixes, whereas the air voids are higher for mixtures containing RCA. Lower bulk density of RCA will result in cost reduction, as asphalt jobs are mostly measured in cubic metre and materials are purchased in tonnes.

In addition, the test results revealed that a RCA increase will increase optimum bitumen content of the mixtures. Therefore, the selection of optimum combination of RCA and other aggregates is required to satisfy the relevant requirements. To this point, utilization of recycled glass with very low water absorption in asphalt mixtures with different combination of RCA was considered to compensate the high bitumen absorption of asphalt mixtures containing RCA.

The results of tests on different asphalt mixtures containing RCA and glass indicated that the bitumen absorption decreases with glass increase in asphalt mixtures. In other words, asphalt mixtures containing glass have lower optimum bitumen content in comparison with asphalt mixtures without glass. So that , as presented in Table 6.5, asphalt mixtures containing 75% RCA with 10% and 20% glass have optimum bitumen content very close to optimum content of control samples (asphalt mixtures without any recycled materials). The test results on asphalt mixtures containing RCA and glass at different rate of bitumen content also showed that air void, VMA and bulk density are lower than the corresponding values for asphalt mixtures containing RCA without glass, whereas introducing glass in the asphalt mixtures increases VFB in asphalt mixtures containing both RCA and glass than asphalt mixtures containing only RCA.

Finally, the results of volumetric properties of all asphalt mixtures at their optimum bitumen content, as shown in Table 6.5, indicates that asphalt mixtures made by combining 75% RCA and 10% glass is the most comparable mixture to control samples in terms of optimum bitumen content and volumetric properties requirements.

Chapter 7

Asphalt Mixtures Preparation

7.1. Introduction

The mixing procedure during the asphalt mixture production influences different properties of asphalt mixtures. Mixing aggregates and bitumen is an important phase and somewhat an art in the asphalt mixtures production affecting different asphalt mixtures characteristics such as mixture homogeneity, bitumen and aggregate adhesion, and film coating index. All these characteristics will subsequently have substantial effect on the asphalt pavements performance. This chapter discusses the sample preparation by two different methods and presents the results of some volumetric tests on samples prepared with these methods. In addition, image processing is conducted on these samples to show the effect of preparation method on internal structure of asphalt mixtures in order to determine the best mixing process for preparation of asphalt mixture samples.

7.2. Asphalt Mixing Method

Almost all asphalt mixtures are currently produced through the mixing the coarse and fine aggregates and filler particles together and then adding bitumen to the aggregates and fillers for further mixing. However, this type of mixing may result in heterogeneity and segregation of the mixture. A new method of mixing has been recently applied for asphalt mixture production in Sweden (Taylor and Khosla, 1983). This method is based on the packing theory and morphology framework, as discussed in Chapter 5, in which the traditional definition of coarse and fine aggregate are changed to Oversized Structure (OS), Primary Structure (PS), Secondary Structure (SS), and filler particles. Based on new definition of aggregate sizes, Primary Structure (PS) is the fraction of aggregate having a substantial role in transmitting the load through the mixture by forming a network within the mixture. In addition, the aggregates smaller than PS and bigger than the filler are referred to as Secondary Structure (SS). This portion of aggregates has direct effect on the stability of the PS. In this new definitions, the aggregates larger than PS float in the mixture. This portion of aggregates which is called Oversized Structure (OS) has little contribution to load carrying capacity of mixture as they are not interconnected to the PS. An adequate percentage of all these groups are necessary to provide an asphalt mixture with optimal performance properties.

The new method of asphalt preparation, called sequential mixing method, differs from the conventional mixing method in terms of the order of mixing (Taylor and Khosla, 1983; Brown and Cooley, 1999; Goodrich, 1988; Brown et al., 1989; Viman et al. 2008; Roberts et al., 2009; Jacobsson, 2009). In this method, aggregates in the PS fraction are firstly mixed with the bitumen. In the next step, the filler fraction is added to the asphalt mixture, and finally, the secondary structure part of aggregates is mixed with the entire mixture.

According to the investigations performed on the asphalt mixtures performance, it has been shown that asphalt mixtures prepared by new mixing method are more homogeneous. In addition, the coarser particles coating is thicker in these asphalt mixtures (Hesami, 2014). These mixtures have also better performance in terms of rutting resistance, skid resistance and noise reduction in comparison with asphalt mixtures prepared by conventional mixing method (Viman, 2011; Viman et al., 2008). In addition, energy consumption is less in asphalt production using the new mixing method as it requires higher bitumen viscosity which can be achieved by the conditioning and mixing temperature by 30 °C. The reduction of temperature helps to increase the bitumen viscosity and to eliminate the risk of draining down of bitumen because of the high amount of bitumen compared to the surface area of the primary structure because the primary structure is the first aggregate fraction which gets mixed with bitumen in of the new mixing method.

Moreover, the high surface area of filler and hence more interaction between the filler particles in conventional mixing method results in clusters formed by the filler grains. These cumulated particles create larger artificial particles which can act as coarser particles. However, these clusters are still relatively small in comparison with the mechanical mixer arms and, therefore, they can hardly break. In addition, during this agglomeration, air bubbles can also be trapped in the mastic resulting in the mastic viscosity change in conventional mixing method. Contrary, the risk of forming clusters and agglomeration is significantly reduced in the new mixing method because bitumen is first mixed with the primary structure and then filler will be added continuously to the bitumen. Furthermore, conventional mixing method results in inhomogeneous mixture, as in this method binder is introduced to all the dry aggregates. Therefore, the smaller particles tend to move to the top of the asphalt mixture upon binder addition, whereas, the coarser aggregates are first coated with bitumen in new mixing method, and filler immediately sticks to the binder upon its introduction. This procedure eliminates the risk of segregation and provides more homogeneous mixture (Jacobsson, 2009; Viman, 2011).

According to the above discussions, the sample preparation has enormous effect on the homogeneity, and the other characteristics of the final asphalt mixture. On the other hand, as this research includes an extensive experimental work to investigate the performance of asphalt mixtures containing different percentages of various materials, to reduce errors in evaluating the asphalt mixtures performance, and to get more reliable and consistent results, the sample preparation should be standardized.

In light of this, in this research, the effects of the mixing method on asphalt mixtures characteristics are evaluated through the comparison of asphalt mixtures produced by conventional and new mixing methods. As the internal structure of asphalt mixtures can effectively help in understanding the role of different mixing procedures in asphalt mixture characteristics and performance, the asphalt mixtures specimens prepared with the two mixing method are analysed through image processing methods in order to study their internal structures. Based on the results of this analysis, the best mixing approach is selected for the preparation of some of the selected samples in this research.

7.3. Evaluation of Different Mixing Methods

To study the effect of mixing method on asphalt mixtures, three sets of asphalt mixtures were prepared at different rates of bitumen using conventional mixing method, sequential mixing method (neglecting packing theory concepts), and Advanced Sequential mixing method (considering the packing theory concepts and new definition for coarse and fine aggregates).

Table 7.1: The Asphalt Mixtures Specifications

Mixing Method	Mix Nominal Size	Aggregate			Filler Type	Bitumen Type
		Type	Coarse Fraction (%)	Fine Fraction (%)		
Conventional	14 mm	Basalt	42	52.5	Portland Cement, Hydrated Lime	C320
Sequential	14 mm	Basalt	42	52.5	Portland Cement, Hydrated Lime	C320
Advanced Sequential	14 mm	Basalt	57	37.5	Portland Cement, Hydrated Lime	C320

The grading target considered for all sets of asphalt mixtures were similar. The specifications of the asphalt mixtures are given in Table 7.1.

7.3.1. Sample Preparation

The asphalt specimens were mixed in the Pavement Laboratory at the Centre for Infrastructure Engineering (CIE) at Western Sydney University to produce cylindrical specimens of diameter 100 ± 2 mm and height of 65 ± 5 mm, in accordance with AS2891.2.1 (2014) and AS2891.2.2 (2014). To prepare samples, both aggregate and filler were heated in an oven to be dried and achieve the required temperature of 180 °C. For conventional mixing method, bitumen was heated in the oven at its conditioning temperature. Then, bitumen and aggregates were mixed using a mechanical mixer for 3 minutes. For the new mixing methods, bitumen was heated at a temperature of about 30 °C less than the conditioning temperature, and then mixed with coarse aggregate, and subsequently with filler and fine aggregates at certain mixing time for each fraction (Figure 7.1).

Aggregate and filler in Conventional Mixing Method

Aggregate and filler in New Mixing Methods (from left to right): Coarse, Filler, Fine

Figure 7.1: Comparison of Aggregate Preparation for Conventional and New Mixing Methods

The mixtures were then conditioned at their own conditioning temperature for about one hour. The mixtures were then placed in preheated moulds for another hour of conditioning before compaction. The prepared samples then compacted with an IPC gyratory compactor with the compaction force of 240 ± 10 kN and the loading angle of 2 ± 0.1 degrees.

7.3.2. Effect of Mixing Procedure on Volumetric Properties of Asphalt Mixtures

To investigate the effect of mixing procedure on volumetric characteristics of asphalt mixtures, the maximum density test and bulk density test were conducted on asphalt mixtures containing different bitumen content of 4.5%, 5%, and 5.5%. The results of these tests on conventional mixtures (B100-C-4.5, B100-C-5, and B100-C-5.5), sequential mixtures (B100-Se-4.5, B100-Se-5, and B100-Se-5.5) and Advanced Sequential mixtures (B100-S-4.5, B100-S-5, and B100-S-5.5) are summarized in Table 7.2.

Table 7.2: Volumetric Properties of Asphalt Mixtures Made with Different Mixing Methods

Sample Name	AV (%)	VMA (%)	VFB (%)	Binder Film Index (μm)	Filler-Binder Ratio	Bulk Density (gr/cm³)	Water Absorption (%)
B100-C-4.5	6.2	15.9	64.6	7.2	1.2	2.356	1.54
B100-Se-4.5	5.9	15.7	66.3	7.3	1.2	2.361	1.46
B100-S-4.5	5.6	15.5	67.6	7.3	1.2	2.367	1.41
B100-C -5	5.1	16.0	72.3	8.2	1.1	2.365	0.96
B100-Se -5	4.8	15.9	74.0	8.3	1.1	2.367	0.85
B100-S -5	4.6	15.8	75.3	8.4	1.1	2.370	0.78
B100-C -5.5	4.4	16.2	77.0	8.8	1.0	2.373	0.70
B100-Se -5.5	4.0	16.1	79.7	9.0	1.0	2.376	0.63
B100-S -5.5	3.3	15.9	83.8	9.4	1.0	2.382	0.60

According to the results obtained, Figure 7.2 compares the optimum bitumen content in asphalt mixtures made with two different mixing methods.

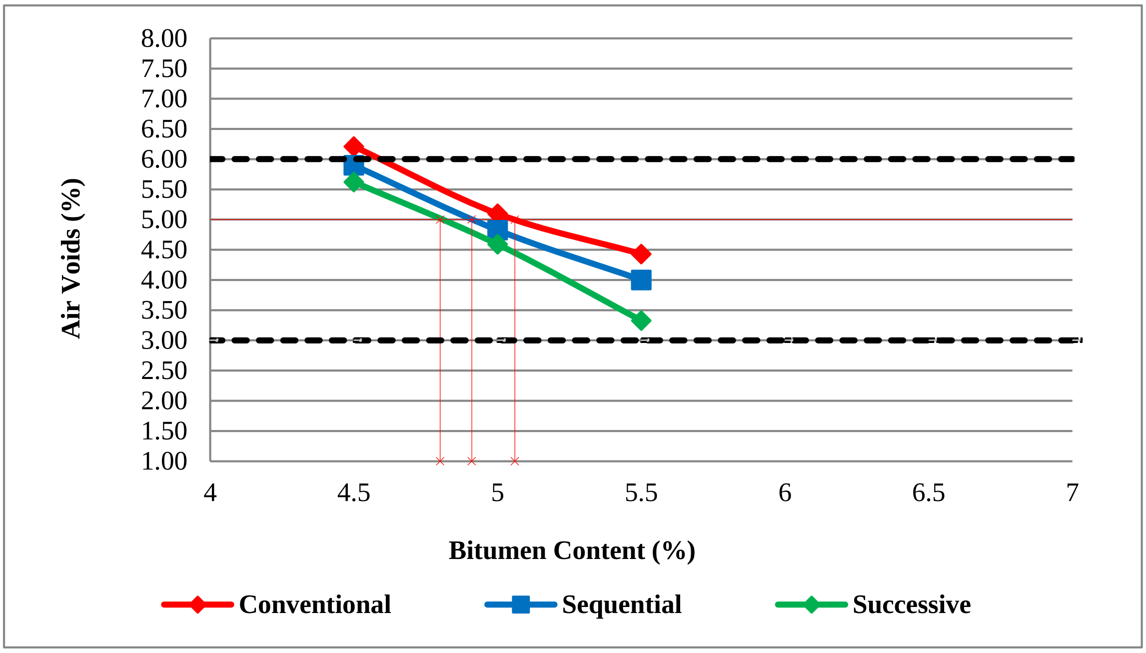

Figure 7.2: Comparison of Optimum Bitumen Content in Asphalt Mixtures Prepared by Different Mixing Methods

As can be seen in this figure, the Advanced Sequential Mixing Method requires less bitumen content (about 0.3%) followed by sequential mixing method (about 0.15%) compared to conventional mixing method. Considering the amount of required bitumen in asphalt mixtures (about 5%) and the high cost and emission associated with bitumen, both new methods of mixing can provide substantial benefits in terms of cost in addition to other advantages discussed in Section 7.2. However, the bitumen saving in the Advanced Sequential Mixing Method, which is part of this research findings, is twice the sequential mixing method, resulting in more sustainable asphalt mixture. Accordingly, this method is named as Advanced Sequential Mixing Method (ASMM).

Considering the high bitumen absorption of asphalt mixtures containing RCA, ASMM can be useful in decreasing the optimum bitumen content in these asphalt mixtures. To this point, for further investigation and understanding the effect of employing this new method on the asphalt mixtures containing RCA with high bitumen absorption, different asphalt mixtures incorporating 25% RCA were prepared through ASMM method at different bitumen content and the volumetric tests were conducted on them in order to determine their optimum bitumen content.

Table 7.3: Volumetric Properties of Asphalt Mixtures Containing 25% RCA Made with Different Mixing Methods

Sample Name	AV (%)	VMA (%)	VFB (%)	Binder Film Index (μm)	Filler-Binder Ratio	Bulk Density (gr/cm^3)	Water Absorption (%)
B75-C-5	6.5	15.3	64.0	6.7	1.1	2.354	1.93
B75-S-5	5.8	15.3	66.7	7.1	1.1	2.361	1.76
B75-C-5.5	5.3	15.1	71.9	7.5	1.0	2.338	1.37
B75-S-5.5	4.9	15.0	73.1	7.8	1.0	2.347	1.21
B75-C-6	4.8	15.7	73.5	8.4	0.9	2.335	1.07
B75-S-6	3.9	15.5	75.2	8.9	0.9	2.343	0.84

The results of volumetric tests on asphalt mixtures containing RCA prepared in two different mixing methods are presented in Table 7.3. As presented in Table 7.3, it was observed that the binder film thickness in new mixing method is slightly more compared to conventional mixing method, which is because of reduced temperature of binder conditioning resulting in more viscous binder and less absorbed binder.

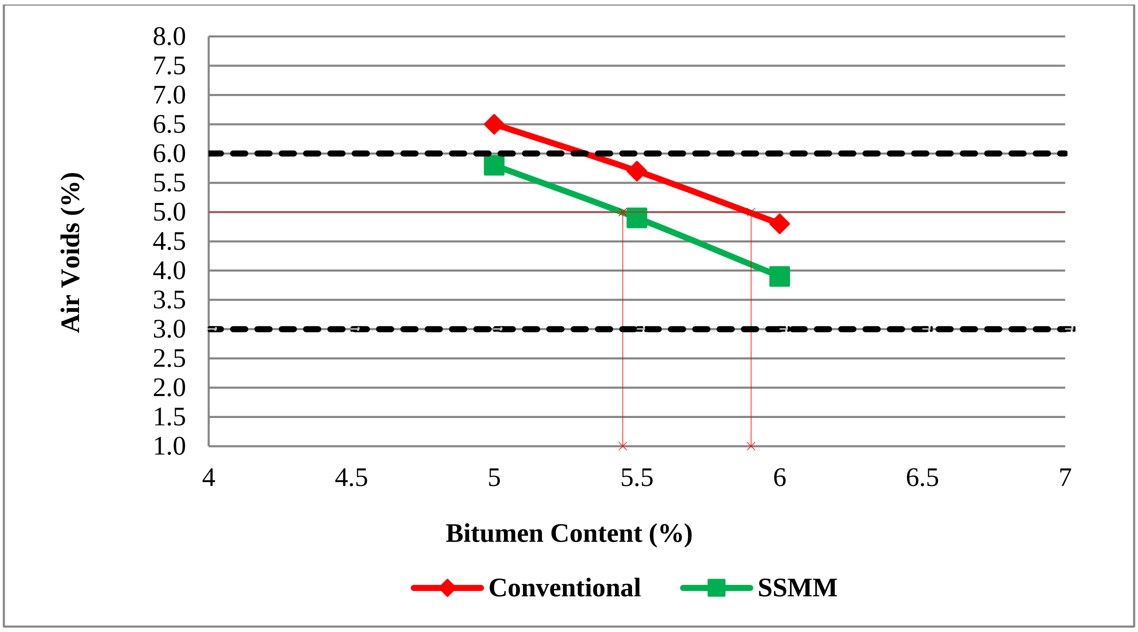

Figure 7.3: Comparison of Optimum Bitumen Content in Asphalt Mixtures Containing 25% RCA Prepared by Conventional and ASMM Mixing Methods

According to the results obtained, Figure 7.3 shows the optimum bitumen content determination in asphalt mixtures containing 25% RCA and made with two different mixing methods. As shown in Figure 7.3, the new mixing method again provides mixtures with less bitumen content compared to conventional method. The results of volumetric properties evaluation of asphalt mixtures prepared with different mixing method at their optimum content are presented in Table 7.4.

Table 7.4: Volumetric Properties of Asphalt Mixtures Made with Different Mixing Method at Optimum Bitumen Content

Design Mix	Optimum Bitumen Content (%)	Bulk Density (gr/cm^3)	VMA (%)	VFB (%)	Binder Film Index (μm)	Filler-Binder Ratio
B100-C	5.1	2.366	16.1	72.8	8.1	1.1
B100-S	4.8	2.368	15.7	72.0	8.3	1.2
B75-C	5.9	2.335	15.6	72.3	7.8	0.9
B75-S	5.5	2.348	15.0	72.5	8.1	1.0
Standard Limit	**-**	**-**	**13% -20%**	**60% - 80%**	**-**	**0.8 – 1.2**
Typical Value	**-**	**-**	**15% (min)**		**7.5μm (min)**	**-**

As shown in Table 7.4, the results of tests on asphalt mixtures prepared with two different mixing methods show that all asphalt mixtures meet the standard limits for volumetric properties. However, mixtures prepared with ASMM method have higher binder film thickness in spite of their less bitumen content.

7.3.3. Effect of Mixing Procedure on the Internal Structure of Asphalt Mixtures

Changing the mixing method in samples with the same materials in terms of type and quantity, may change the internal structure of asphalt mixtures which subsequently results in the changes in the asphalt mixtures behaviour. To show the effect of mixing procedure on the internal structure of the asphalt mixtures, two asphalt samples with the same gradation, materials, and bitumen content but with two different mixing methods of ASMM and conventional were prepared and cut by a cutting machine in order to analyse their internal structure using image processing software of ImageJ. It should be noted that water was used during cutting process in order to prevent the overheating and hence possible damage to the specimens. In both samples, basalt was used as coarse and fine aggregates. Portland cement and hydrated lime with the ratio of 3:2 were used as filler. The nominal size of the asphalt is 14 mm, and 5.0% of weight of samples is bitumen C320. Figure 7.4 illustrates the raw image of the same surface of asphalt specimens made with two different mixing methods.

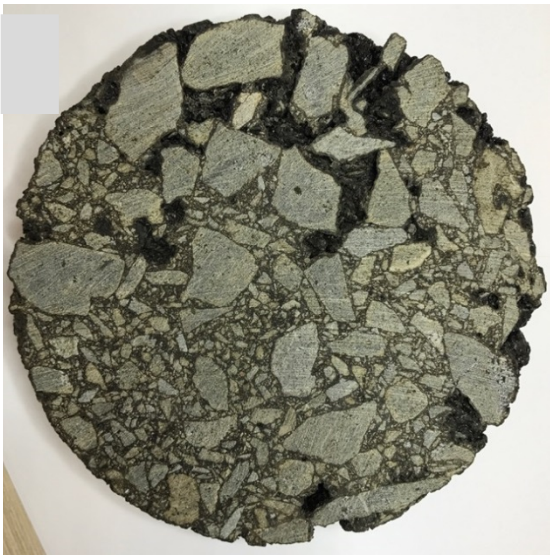

Figure 7.4: Internal Surface of Asphalt Specimen Prepared by Different Mixing Methods Conventional (left) and ASMM (right)

One of the difficulties in image processing of asphalt mixtures is adjustments which refer to enhancing the resolution for recognizing different components of coarse and fine aggregates, air voids, and binder, as these components have similar colour and similar distribution of colour composition. In this study, the images were enhanced utilizing different features of ImageJ such as colour balance, noise reduction, applying threshold, and edge sharpening; the final image was obtained for image processing. Figure 7.5 shows the internal surface of asphalt mixtures with three phases of coarse aggregates (in red), air voids (in yellow) and fine aggregates with bitumen (in gray).

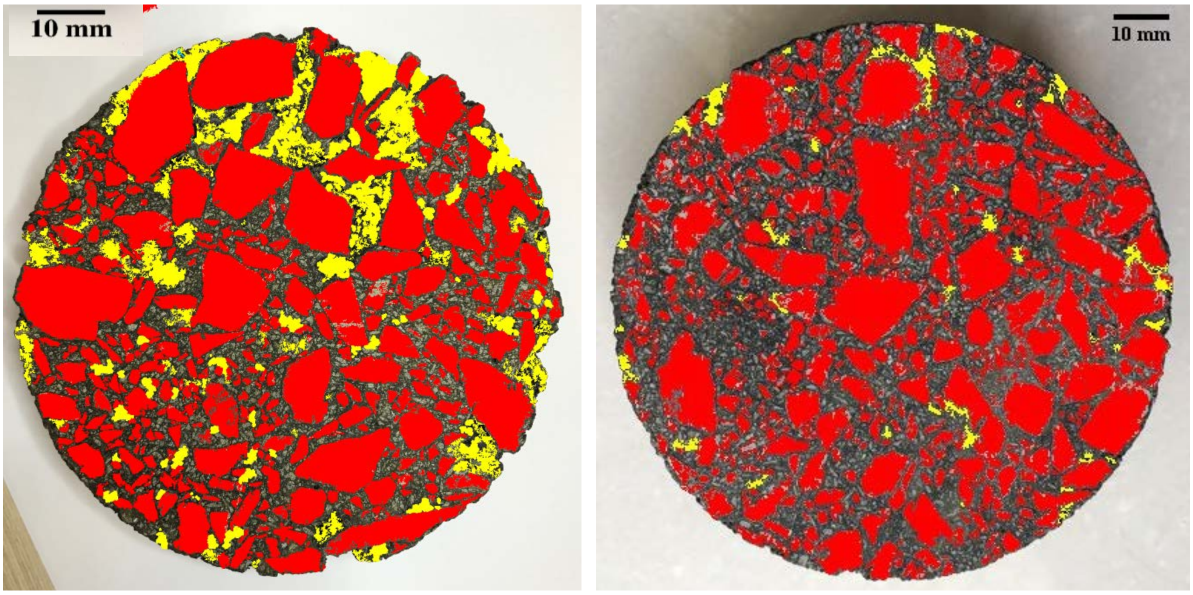

Figure 7.5: Distribution of Different Phases in Internal Surface of Asphalt Specimen Prepared by Different Mixing Methods Conventional (left) and ASMM (right)

From these figures, it can be observed that mixtures prepared through the new mixing method have less air voids and more even distribution of air voids in comparison with mixtures prepared by conventional mixing method. In addition, the distribution of aggregates is more even in mixtures with ASMM mixing method compared to the asphalt mixtures prepared by conventional mixing method. This implies that the new mixing method provides more homogeneity and hence can perform better in transmitting the load due to its better internal structure.

7.4. Summary and Conclusion

In this chapter, the volumetric properties of asphalt mixtures prepared by two different mixing methods and packing theory concepts were evaluated. The advantages of new mixing methods (sequential method and Advanced Sequential Mixing Method-ASMM) were discussed in this chapter. The main advantage of these mixing methods is reduction in temperature by about 30°C. The decrease in temperature decreases the bitumen viscosity which is one of the requirements of this new method as bitumen is firstly mixed with coarse aggregates with lower specific surface area. In addition, the test results on asphalt mixtures revealed that asphalt mixtures produced by ASMM method have the lowest value of air voids and hence requires the least amount of bitumen compared to other mixing methods, which is a significant advantage of applying this new mixing method for preparation of asphalt mixtures, particularly from the economic point of view. As this research investigates the utilization of RCA in asphalt mixtures and due to the high bitumen absorption of RCA, the new mixing method can be useful to reduce the optimum bitumen content in asphalt mixtures containing RCA. To this point, asphalt mixtures with 25% RCA were also prepared by both ASMM and conventional mixing methods. The test results again showed that preparing asphalt mixtures by new mixing method can decrease the optimum bitumen content from 5.9% to 5.5%, while keeping the other volumetric requirements in the standard range.

To demonstrate the internal structure of asphalt specimens prepared with these two different mixing methods, image analysis was performed on mixtures with the same gradation, same bitumen content but with two different mixing methods. The image processing analysis revealed that asphalt mixture prepared by ASMM mixing method demonstrated less air voids and an even distribution of air voids and aggregates, which can have substantial influence on the final performance of asphalt mixtures.

Chapter 8

Evaluation of Resilient Modulus of Asphalt Mixtures

8.1. Introduction

8.2. Prediction of Resilient Modulus Based on Volumetric Parameters

8.3. Determination of Resilient Modulus Based on Laboratory Testing

8.4. Results and Discussion

8.1. Introduction

The stiffness of asphalt mixtures is a fundamental property and plays an important role in determining the performance of asphalt pavement under traffic loading. The resilient modulus (Mr) is a measure of stiffness of materials showing an estimation of materials' modulus of elasticity (E). In fact, modulus of elasticity is defined as the ratio of stress to strain for a load which is applied slowly, whereas, resilient modulus is defined as the ratio of stress to strain for loads which are applied rapidly, similar to loads that pavements actually experience. In addition, the resilient modulus of asphalt mixtures is useful in determination of layer thickness through estimation of relative strength coefficient and calculation of Structural Number (SN). Many variables such as temperature, type of loading, time of loading (i.e. vehicle speed), loading history, and mixture type can affect the M_R value. Many studies (e.g. Yin and Garcia, 2012; Pan et al., 2005; Ji, 2006; Katich, 2003; Demirci, 2010; Clyne et al., 2003) have focused on understanding the effect of certain factors on resilient modulus. It is generally recognized that the volumetric characteristics of mixtures greatly influence their performance. To this point, many predictive equations have been developed by many researchers to link the stiffness modulus and dynamic modulus of asphalt mixtures to their volumetric parameters. Accordingly, this chapter aims to determine the stiffness modulus of asphalt mixtures studied in this research based on these predictive equations as well as through the indirect tensile test conducted on the selected samples identified through this research work.

Accordingly, this chapter consists of three main parts. In the first part, the resilient modulus of asphalt mixtures is estimated by nomographs and empirical equations. Various volumetric properties obtained in Chapter 7 will be used for the calculation of resilient modulus based on these empirical methods. The second part includes the determination of resilient modulus of asphalt mixture through indirect tensile test. Finally, in the last part, the measured modulus of samples through resilient modulus test will be compared with the calculated modulus from different predictive equations, and the accuracy and reliability of different empirical methods will be discussed.

8.2. Prediction of Resilient Modulus Based on Volumetric Parameters

Resilient (Stiffness) modulus of asphalt mixtures can either be determined through laboratory investigation or predicted through the empirical methods based on asphalt components properties. Since the resilient modulus test is difficult and requires expensive machines, empirical methods are generally used for determination of resilient modulus. The following sections discuss the determination of the resilient modulus of asphalt mixtures studied in this research by using current empirical methods.

8.2.1. Estimation of Stiffness of Binder

The stiffness of binder directly affects the stiffness of asphalt mixture. One of the most common models for prediction of binder's stiffness is the one developed by Van Der Poel (1954). In this model, the stiffness of bitumen can be determined from the following equation:

$$\boldsymbol{M_{rb} = 1.157 \times 10^{-7} \times t_w^{-0.368} \times e^{-PI} \times (T_{RB} - T)^5} \qquad (\mathbf{8.1})$$

where M_{rb} is the stiffness of binder, t_w is the time of loading, T_{RB} is the softening point, T is the test temperature, and PI is the penetration index.

Penetration index in the above equation can be obtained from Equation 8.2:

$$\boldsymbol{PI = \frac{20 - 500A}{1 + 50A}} \qquad (\mathbf{8.2})$$

In this equation, A is the temperature susceptibility which can be calculated from the following simplified equation:

$$\boldsymbol{A = \frac{\log(Pen\ at\ T) - \log(800)}{T - T_{RB}}} \qquad (\mathbf{8.3})$$

In the above equation, $Pen\ at\ T$ is the penetration at test temperature(T).

The main parameters considered for estimation of stiffness of the binder used in this study (C320) based on this research results, as well as, the calculated binder's stiffness are summarized in Table 8.1.

Table 8.1: Parameter Identification for Calculation of Stiffness of Binder (C320)

Parameter	Value	Unit
Softening Point ($\mathbf{T_{RB}}$)	52	°C
Test Temperature ($\mathbf{T}$)	25	°C
Time of Loading ($\mathbf{t_w}$)	0.1	s
Penetration at 25°C	51	dmm[1]
Stiffness of Binder ($\mathbf{M_{rb}}$)	7.54	MPa

8.2.2. Estimation of Stiffness of Asphalt Mixture

In empirical methods, the stiffness of asphalt mixture can be obtained by nomographs and predictive equations considering the bitumen stiffness, volume of aggregates and volume of

[1] The penetration is measured in units of 0.1 mm (dmm)

binder. In the following sections, the stiffness of asphalt mixtures is determined through different empirical methods and based on the results of primary tests.

8.2.2.1. Shell Method

Referring to the Shell method (Shell, 1978), the asphalt mixture stiffness can be calculated from binder stiffness and mixture volumetric composition by using predictive equations and nomographs. In this method, the volumetric composition including volume of aggregate (V_g) and volume of binder (V_b) can be obtained from the following equations:

$$\boldsymbol{V_b = VMA - AV} \quad (\mathbf{8.4})$$

$$\boldsymbol{V_g = 100 - VMA} \quad (\mathbf{8.5})$$

Using the results of primary tests on asphalt mixtures and values of air voids (AV) and voids in mineral aggregate (VMA), the volumetric composition of asphalt mixtures can be calculated.

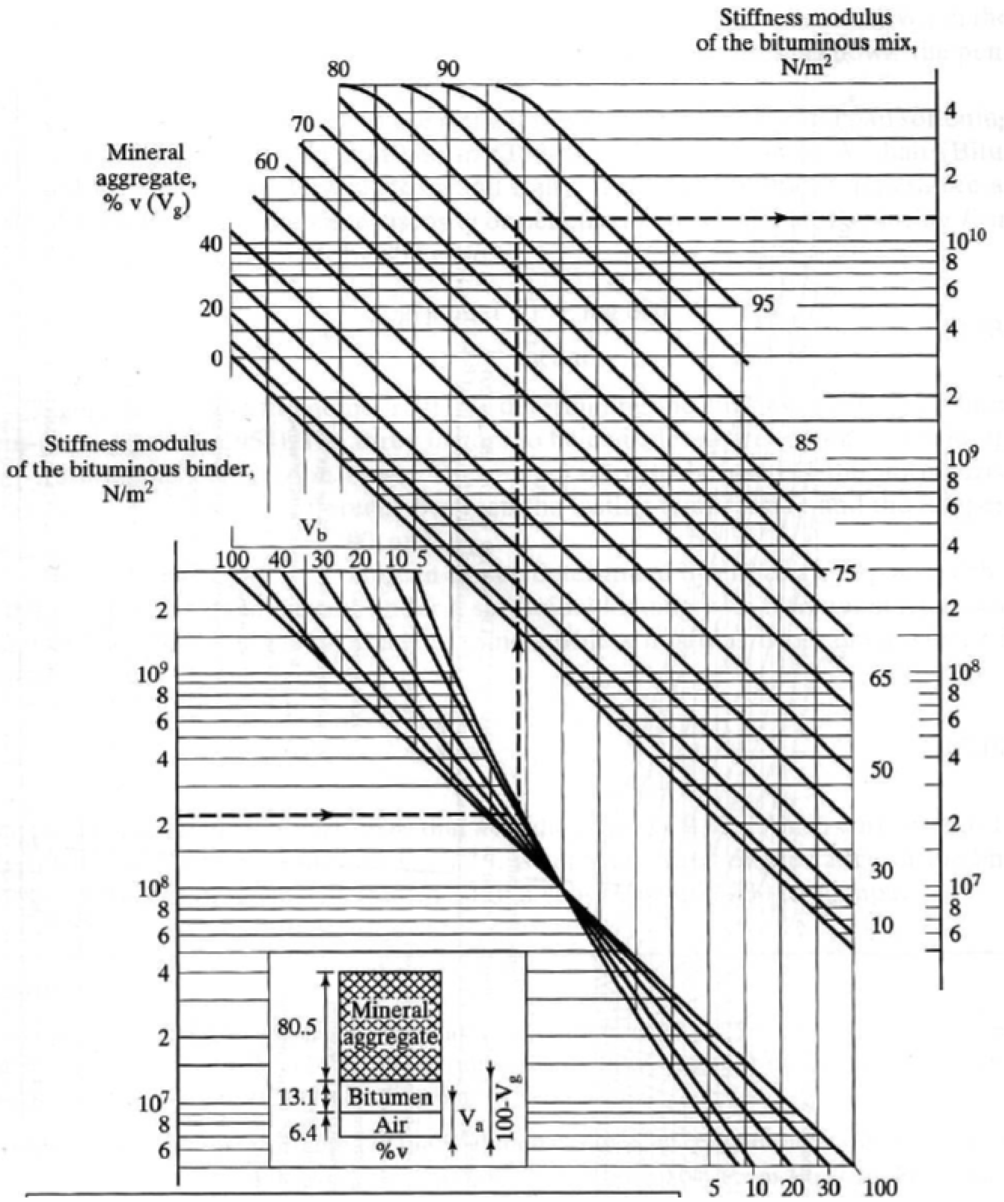

Figure 8.1: Shell Nomograph (Shell, 1978)

The results of this calculation for different asphalt mixtures will be used for determination of the stiffness of asphalt mixtures using the nomograph shown in Figure 8.1.

Table 8.2: Parameter Identification for Estimation of Stiffness of Asphalt Mixture based on Shell Method

Asphalt Mixture	Optimum Bitumen Content (%)	AV (%)	VMA (%)	V_b (%)	V_g (%)	M_r (MPa)
B100	5.1	5.1	16.1	11	84	1,680
B75	5.8	5.0	15.6	11	84	1,795
B75-G10	5.4	5.0	14.9	10	85	1,910
B100-S	4.8	4.9	15.7	11	84	1,800
B75-S	5.5	5.0	15.0	10	85	2,286
B75-G10-S	5.2	4.9	15.4	11	85	2,157

The results of estimation of resilient modulus of asphalt mixtures by Shell method are summarized in Table 8.2.

8.2.2.2. Bonnaure et al. (1977) Method

Bonnaure et al. (1977) developed some predictive equations for determination of the resilient modulus of asphalt mixtures based on the stiffness of binder and volumetric composition of asphalt mixture. The main assumption in this method is that the mixture stiffness is primarily governed by the binder stiffness, as presented in Equations 8.6 and 8.7.

For $5 < \mathrm{M_{rb}} < 1000$ (MPa):

$$\log M_r = \frac{\beta_1 + \beta_3}{2} (\log M_{rb} - 8) + \frac{\beta_4 - \beta_3}{2} |\log M_{rb} - 8| + \beta_2 \tag{8.6}$$

For $1000 < \mathrm{M_{rb}} < 3000$ (MPa):

$$\log M_r = \beta_2 + \beta_4 + 2.0959 (\beta_1 - \beta_2 - \beta_4) \log(M_{rb} - 9) \tag{8.7}$$

In the above equations, the β parameters can be calculated from the following equations:

$$\beta_1 = 10.82 - \frac{1.342(100 - V_g)}{V_g + V_b} \tag{8.8}$$

$$\beta_2 = 8.0 + 0.00568V_g + 0.0002135V_g^2 \tag{8.9}$$

$$\beta_3 = 0.6 \log\left(\frac{1.37V_b^2 - 1}{1.33V_b - 1}\right) \tag{8.10}$$

$$\beta_4 = 0.7582(\beta_1 - \beta_2) \tag{8.11}$$

Table 8.3: Parameter Identification for Stiffness Estimation of Asphalt Mixture based on Bonnaure et al. (1977) Method

Asphalt Mixture	β_1	β_2	β_3	β_4	M_r (MPa)
B100-C	10.592	9.979	0.649	0.465	3,677.7
B75-C	10.600	10.000	0.640	0.454	3,861.0
B75-G10-C	10.610	10.030	0.624	0.440	4,177.1
B100-S	10.598	9.996	0.645	0.457	3,796.1
B75-S	10.608	10.025	0.626	0.442	4,129.6
B75-G10-S	10.603	10.009	0.638	0.450	3,923.5

The results of calculations and estimation of resilient modulus of different asphalt mixtures using Bonnaure et al. (1977) equations are summarized in Table 8.3.

8.2.2.3. Heukelom and Klomp (1964) Method

The resilient modulus can also be calculated using the equations developed by Heukelom and Klomp (1964) based on Van Der Poel (1954) and volumetric composition of asphalt mixtures, as follows:

$$\boldsymbol{M_r = M_{rb} \times \left[\left(1 + \frac{2.5}{n}\right) \times \frac{C'_v}{(1 - C_v)}\right]^n} \qquad (\mathbf{8.12})$$

In the above equation, the relevant parameters can be obtained from Equations (8.13) to (8.15).

$$\boldsymbol{C'_v = \frac{C_v}{\left[0.97 + 0.01\left(100 - \left(V_g - V_b\right)\right)\right]}} \qquad (\mathbf{8.13})$$

$$\boldsymbol{C_v = \frac{V_g}{\left(V_g + V_b\right)}} \qquad (\mathbf{8.14})$$

$$\boldsymbol{n = 0.83 \times \log\left(\frac{40000}{M_{rb}}\right)} \qquad (\mathbf{8.15})$$

The results of calculations and estimation of resilient modulus of different asphalt mixtures using Heukelom and Klomp (1964) equations are summarized in Table 8.4.

Table 8.4: Parameter Identification for Stiffness Estimation of Asphalt Mixture based on Heukelom and Klomp (1964) Method

Asphalt Mixture	C_v	C'_v	n	M_r (MPa)
B100-C	0.884	0.712	3.091	1,889.4
B75-C	0.888	0.721	3.091	2,151.5
B75-G10-C	0.896	0.735	3.091	2,707.7
B100-S	0.886	0.718	3.091	2,030.7
B75-S	0.895	0.733	3.091	2,617.7
B75-G10-S	0.890	0.724	3.091	2,232.8

8.3. Determination of Resilient Modulus Based on Laboratory Testing

As discussed previously, the resilient modulus (Mr) is a measure of stiffness of materials showing an estimation of materials’ modulus of elasticity (E). In fact, modulus of elasticity is defined as the ratio of stress to strain for a load which is applied slowly, whereas, resilient modulus is defined as the ratio of stress to strain for loads which are applied rapidly, similar to loads that pavements actually experience. The resilient modulus may be estimated indirectly through predictive methods, or determined directly from laboratory testing. In laboratory testing, the resilient modulus is determined by applying a repeated deviator stress (σ_d) of fixed magnitude under a certain load and cycle duration and constant cell pressure to a cylindrical specimen and the resilient axial strain is measured at these cycles. The secant modulus[1] increases by the increase in the number of load cycles. However, the modulus becomes almost constant after a certain number of load cycles. It is presumed that the material has the elastic response, in this condition. The resilient modulus is defined as this steady value.

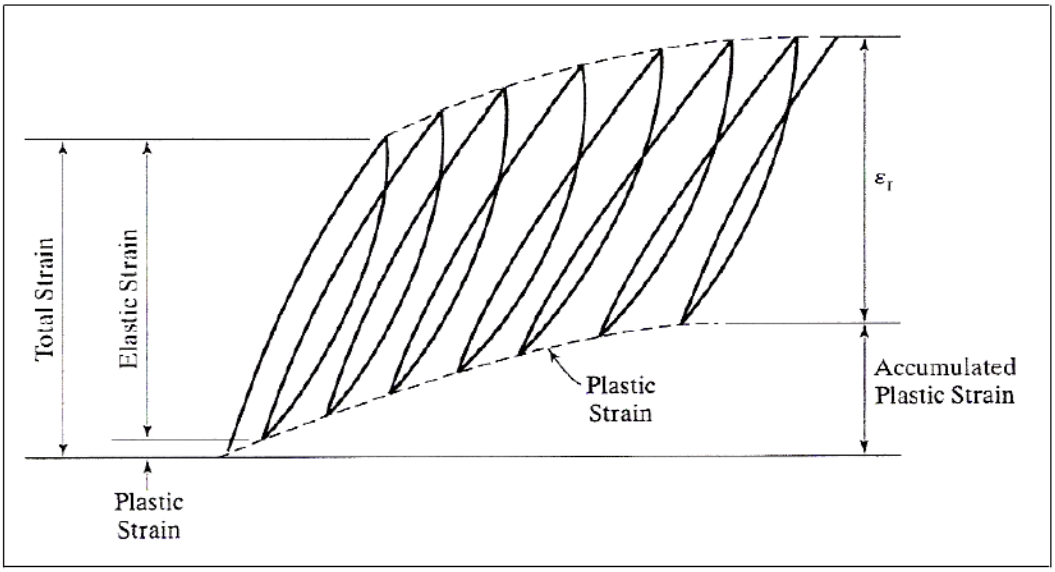

Figure 8.2: Elastic and Plastic Response under Repeated Loads (Huang, 1993)

Asphalt mixtures are viscoelastic materials and hence, experience permanent deformation after each load cycle. In early stages of repeated loading, permanent deformation is considerably high. By the increase in the number of load applications, it can be observed that the plastic strains as a result of load application decreases (Huang, 1993). This will result in the almost complete recovery of the deformation after the load application. Figure 8.2 illustrates the elastic and plastic response of such material under the repeated loads. As shown in this figure, as the number of load repetitions increases, the permanent deformation rate proceeds towards zero. Due to this point, the resilient modulus can be obtained after applying a certain number of loads. Therefore, the resilient modulus can be defined in terms of the recoverable strain under repeated loading, as follows:

$$\boldsymbol{M_r = \frac{\sigma_d}{\varepsilon_r}} \tag{8.16}$$

where σ_d is the deviator stress and ε_r is the recoverable strain.

[1] Secant modulus is the slope of a line from the origin to any point on a stress-strain curve.

In this research, the indirect tensile test is used to measure the resilient modulus of asphalt mixtures. The following sections discuss the test procedure and the results of this test on selected samples.

8.3.1. Specimen Preparation

Several researchers (e.g. Hamzah and Yi, 2008; Kennedy and Prez, 1978; Sondag et al., 2002) have conducted studies on the effect of different materials on the stiffness of asphalt mixtures. However, despite the relevance of the available literature and the detailed studies on resilient modulus, there are no reports that describe the effects of introducing RCA into the asphalt mixture on resilient modulus. Therefore, in this study, resilient modulus test is considered as an advanced test to evaluate the stiffness of some specimens selected based on the results of primary tests to assess the effect of RCA amount as well as the asphalt mixture composition and preparation method on resilient modulus. To this point, the asphalt mixtures were prepared using different combination and preparation method but with the same gradation. Subsequently, the asphalt mixtures were compacted with GyroPac at the same level of compaction to make cylindrical specimens of 100 mm in diameter and 65 mm in height.

Table 8.5: Material Combination for Preparation of Specimens for Indirect Tensile Test

Asphalt Mixture	Coarse Aggregate (%)		Fine Aggregate (%)		OBC[1] (%)	Preparation Method
	Basalt	RCA	Basalt	Glass		
B100-C	100	0	100	0	5.1	Conventional
B75-C	75	25	100	0	5.8	Conventional
B75-G10-C	75	25	90	10	5.4	Conventional
B100-S	100	0	100	0	4.8	ASMM
B75-S	75	25	100	0	5.5	ASMM
B75-G10-S	75	25	90	10	5.2	ASMM

In addition, the optimum bitumen contents as determined in Chapter 7 were used for preparation of cylindrical specimens. Table 8.5 presents the material combination used in these asphalt mixtures.

[1] Optimum Bitumen Content

8.3.2. Test Procedure

Different equipment and test methods are employed to determine the resilient modulus of asphalt mixtures. However, most engineers have selected the indirect tensile test to measure the resilient modulus of asphalt mixtures (Brown et al., 1989). In this research, resilient modulus test is determined through indirect tensile strength test in accordance with AS/NZS 2891.13.1(2013).

In this test, firstly, the diameter and height of specimens are measured. Then, the machine and Linear Variable Differential Transformers (LVDT) will be calibrated to conduct the test. In this study, a triaxial repeated loading machine was used for conducting the indirect tensile test. This machine is an apparatus composed of a triaxial cell and a dynamic actuator which is capable of applying 50 kPa load, deformation and stresses up to 20Hz on specimens with 150 mm and 100 mm diameter. In this testing machine, this system has also the capability to log the data obtained from various sensors. The test device can apply sinusoidal, haversine, square, triangular and other custom waveforms. The indirect tensile test is performed by subjecting the cylindrical specimen of asphalt mixture to a vertical load (Figure 8.3).

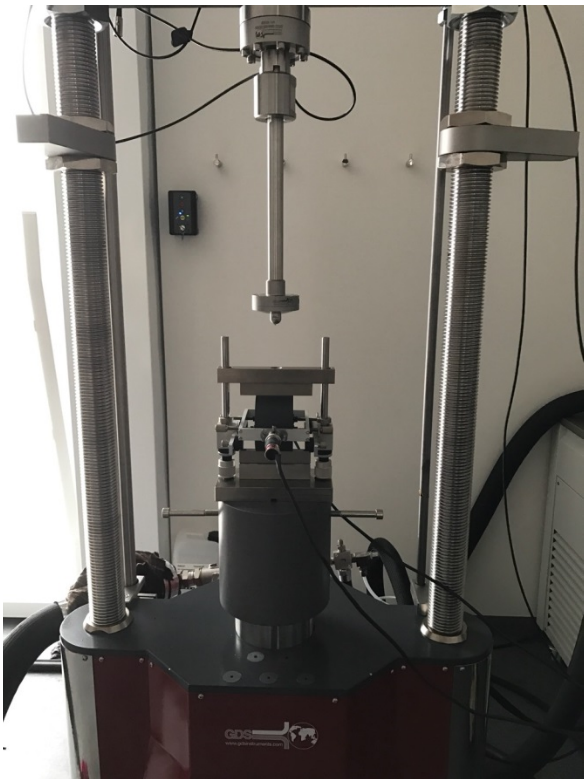

Figure 8.3: Resilient Modulus Test Conduction

During the test, repeated haversine loading is applied to the specimen at the frequency of 0.1 Hz considering 0.1 second of loading and 0.9 sec of rest period (Figure 8.4).

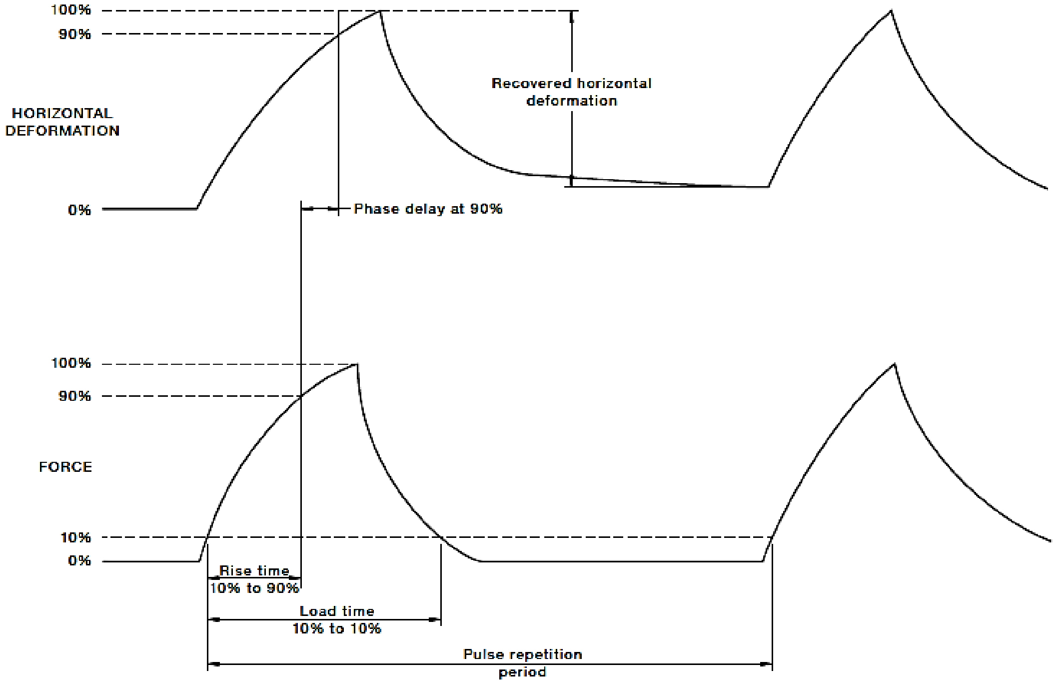

Figure 8.4: Load and Horizontal Deformation Graph in Resilient Modulus Test

In each loading sequence, as shown in Figure 8.5, the total load is the sum of repeated load and contact load. The purpose of contact load is to keep specimen in touch with loading block of the test device.

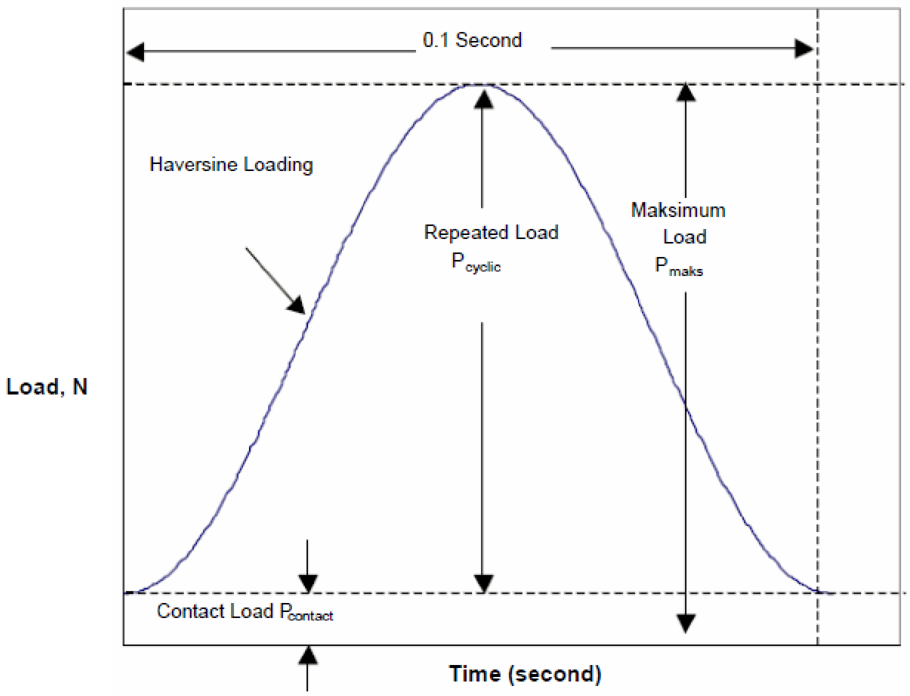

Figure 8.5: Haversine Loading in Resilient Modulus Test (NCHRP, 2004)

Following the preconditioning, five load pulses were applied with a certain rise time to the peak load at a certain pulse repetition period. The recovered horizontal deformation of specimen after application of each load pulse was recorded. The Poisson's ratio considered as 0.4 in accordance with AS 2891.13.1 (2013). The resilient modulus (M_r) in MPa for each specimen for each load pulse during the resilient modulus test can be obtained from the following equation:

$$\boldsymbol{M_r = P \times \frac{(\mu + 0.27)}{H \times h_c}} \qquad (8.17)$$

where P is peak load (N), μ is Poisson ratio, H is the recovered horizontal deformation of specimen after load pulse (mm), and $h_c$is the height of specimen (mm).

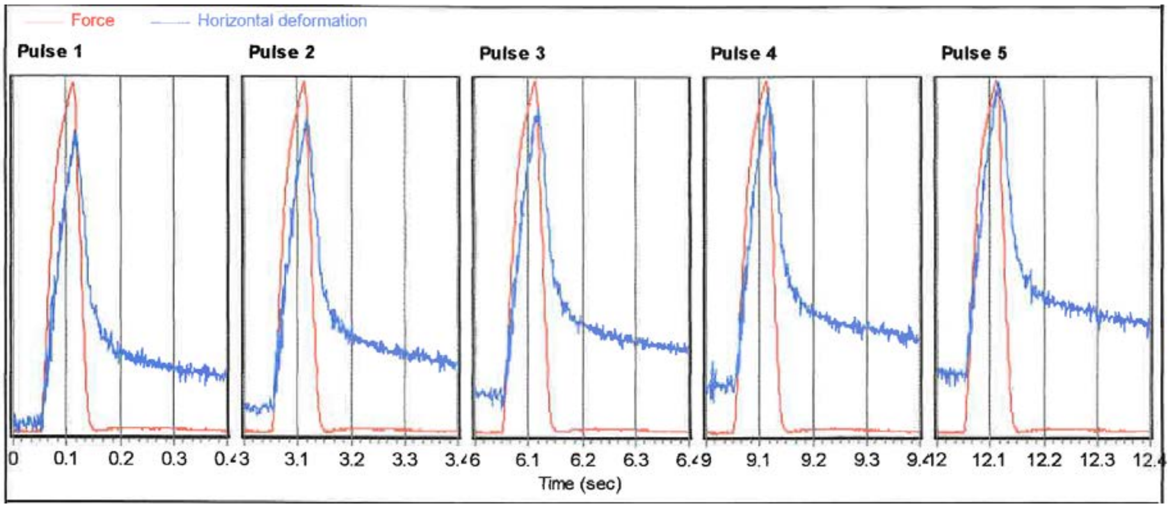

Figure 8.6: Sample Output of Resilient Modulus Test

Using this machine, the test sequence can be monitored through a user friendly program and all test outputs are sent to a desktop computer (Figure 8.6) for analysis.

Table 8.6: Result of Resilient Modulus Test in Accordance with AS 2891.13.1 (2013)

Asphalt Mixture	Ave. Sample Height (mm)	Ave. Sample Diameter (mm)	Peak Load (N)	Recovered Horizontal Strain (με)	M_r (MPa)
B100-C	64.95	99.925	2718.7	50.01	5,613
B75-C	68.225	99.95	3252.7	51.52	6,205
B75-G10-C	68.025	99.975	3302.6	49.08	6,632
B100-S	64.85	99.925	3100.8	51.69	6,196
B75-S	67.425	99.75	3247.9	45.66	7,087
B75-G10-S	68.375	99.925	3098.5	51.18	5,938

The results of resilient modulus test conducted on three specimens from each asphalt mixture type are presented in Table 8.6.

8.4. Results and Discussion

As mentioned previously, in this research, recycled materials are utilized to design an asphalt mixture. It has been well established that predicting the performance of asphalt mixtures containing recycled materials is very difficult given the inconsistency of materials and their complex interaction with natural materials. Therefore, in this research, primary tests were conducted on different mixtures in terms of materials combination as well as the materials amount. Based on the results of primary tests, the most acceptable samples were selected for estimating their resilient modulus through different empirical methods. However, using empirical methods to determine resilient modulus often provide inaccurate values which affect the final structural performance of pavement. Accordingly, in this research, resilient modulus of asphalt mixtures is also determined through the indirect tensile tests for five specimens selected based on the results of primary tests. The estimated resilient moduli of all the selected asphalt mixtures are presented in Table 8.7 for both indirect tensile test and different empirical methods.

Table 8.7: Comparison of Resilient Moduli Obtained from Different Methods

Asphalt Mixture	Shell Method	Bonnaure Method	Heukelom & Klomp Method	Tahmoorian & Samali Method	Indirect Tensile Test
B100-C	1,680	3,677.7	1,889.4	5,590.1	5,613
B75-C	1,795	3,861.0	2,151.5	5,883.3	6,205
B75-G10-C	1,910	4,177.1	2,707.7	6,390.5	6,632
B100-S	1,800	3,796.1	2,030.7	5,779.4	6,196
B75-S	2,286	4,129.6	2,617.7	6,314.2	7,087
B75-G10-S	2,157	3,923.5	2,232.8	5,983.5	5,938

The results of resilient modulus estimation are also illustrated in Figure 8.8 for better comparison of estimation methods.

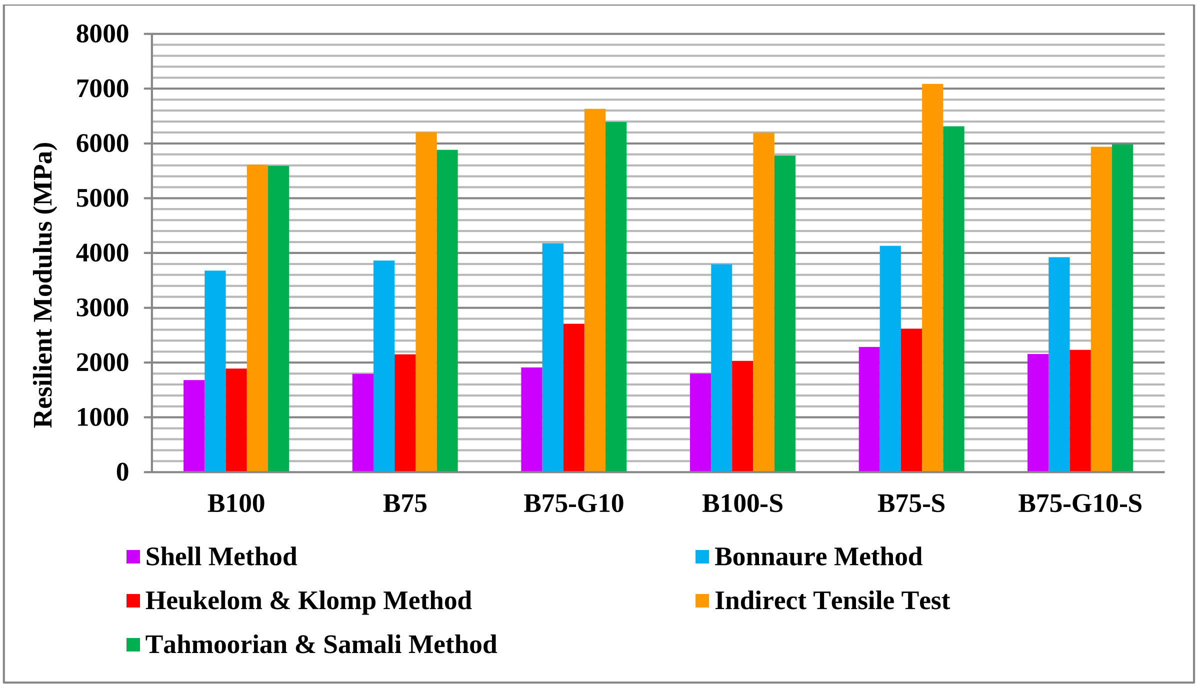

Figure 8.7: Comparison of Resilient Modulus Values obtained from Empirical Methods and Experimental Study

In addition, the results of empirical methods are also compared with the results of experimental study using the error coefficient method based on the following equation:

$$e = 100 \times \left| \frac{E_1 - E_2}{E_1} \right| \tag{8.18}$$

where e is percent error (%), E_1 is resilient modulus obtained from indirect tensile test, and E_2 is estimated resilient modulus from empirical method. The results of comparison based on error coefficient method are presented in Table 8.8.

Table 8.8: Error Estimation for Resilient Modulus from Empirical Methods

Asphalt Mixture	Shell Method	Bonnaure Method	Heukelom & Klomp Method	Tahmoorian & Samali Method
B100-C	70.1	34.5	66.3	0.4
B75-C	71.1	37.8	65.3	5.2
B75-G10-C	71.2	37.0	59.2	3.6
B100-S	70.9	38.7	67.2	6.7
B75-S	67.7	41.7	63.1	10.9
B75-G10-S	63.7	33.9	62.4	0.8
Average Error	**69.1**	**37.3**	**63.9**	**4.6**

As presented in Table 8.8, the resilient modulus obtained from empirical methods of Shell (1978), Bonnaure et al. (1977) and Heukelom-Klomp (1964) are much lower compared to the measured values through Indirect Tensile Test and hence too conservative. Among these empirical methods, Bonaure et al. (1977) method gives the best estimation to the measured values of resilient modulus with an average error of about 37%. This shows that in cases where the value of resilient modulus can not be obtained through the laboratory tests, the empirical equations can hardly be used to predict the actual resilient modulus. However, the results of resilient modulus of different samples in Table 8.7 reveals that the empirical methods still can effectively be used for comparison of the stiffness of different samples.

To this point, by modifying the equation proposed by Bonaure et al. (1977) to the following equation, the resilient modulus would be much closer to the corresponding values obtained by indirect tensile test.

$$\boldsymbol{\log M_r = 0.526\ [(\beta_1 + \beta_3)(\log M_{rb} - 8) + (\beta_4 - \beta_3)\ |\log M_{rb} - 8|] + 1.051\beta_2} \qquad (\mathbf{8.19})$$

It should be mentioned that this equation is applicable where stiffness of binder is between 5MPa and 100MPa (i.e. $5\ MPa \leq M_{rb} \leq 100\ MPa$).

The results obtained from this new equation are compared with different methods in Table 8.7. In addition, the estimated error from the modified equation is also presented in Table 8.8.

As can be observed, the average error is substantially lower compared to other empirical methods. The modified equation (8.19) combines the theory and experience and is obtained through the numerical analysis of equation proposed by Bonaure et al. (1977), while is validated with a number of laboratory results.

Chapter 9

Tests on Asphalt Components - Binder

9.1. Introduction

Asphalt pavements currently require high performance design which emphasize on adequate properties for bitumen to withstand the environmental condition and traffic demand. In many cases, bitumen should be modified to support these requirements.

On the other hand, the rapid growthbitumen is construction in population and economy results in a continuously increased consumption and subsequently generation of a large amount of waste materials. Among various waste materials, rubber and plastic, including high density polyethylene (HDPE) constitute some part of the non-biodegradable solid wastes worldwide. Because of the great difficulties in managing these non-biodegradable wastes and also the volume of required bitumen, the idea of using plastic and rubber as bitumen modifier in new asphalt mixtures appears to be an effective and meaningful utilization of these materials. However, as binder plays an important role in the final performance of the asphalt mixture, an understanding of modified binder properties is essential in designing an asphalt mixture.

Therefore, as part of this research, HDPE and crumb rubber have been evaluated as possible binder modifiers. Since compatibility of bitumen with the modifier is the most important factor in modifying the bitumen with polymers, the properties of these waste polymers and modified binders were evaluated by means of advanced thermal analysis. In addition, the structure of HDPE, rubber and modified binder was investigated under scanning electron microscope (SEM). Moreover, Infrared Spectroscopy (FTIR) was performed for all the individual polymers and their blend to characterize the molecular structure of materials. This chapter presents the results of this experimental study to evaluate the properties of polymer modified binder as an alternative for virgin bitumen in asphalt mixture under different percentages and combination with HDPE and rubber. The original bitumen studied in this research corresponds to C320.

9.2. Background

Aggregate and binder are two principal constituents of asphalt mixtures. Although the mechanical and chemical properties of aggregates can vary significantly depending on their source, the overall durability and other performance characteristics of asphalt mixtures are generally limited by the performance of the asphalt binder. Failure of asphalt pavement due to the asphalt binder can be attributed to three main sources including:

- Rutting that occurs at high temperatures as asphalt softens and the elasticity of the binder decreases,
- Fatigue cracking due to the repeated loading and ageing of the pavement,

- Thermal cracking at low temperatures as asphalt becomes brittle (Somayaji, 2001).

Failure of asphalt binders is obviously undesirable and, therefore, in recent years, much research has been performed focusing on the improvement of binder functional properties. The utilization of polymers as modifiers has shown to greatly improve the performance of bitumen; including elasticity increase, cohesion improvement, and temperature susceptibility reduction, which all subsequently lead to the improvement of asphalt mixture performance in terms of flexibility, cohesion, and deformation resistance at high temperatures. However, the costly nature of polymer modifiers has stimulated research into cheaper and more cost effective modifiers produced from recycled materials. For these reasons, and as a relatively effective way of disposal of the non-biodegradable wastes, polymer wastes like plastic and rubber can be reasonable potential materials for consideration as binder modifier.

In light of this, the purpose of the present work is to study the benefits of polymer addition on the bitumen performance. In that sense, as discussed in the following sections, several samples of bitumen, waste polymers and their blends were prepared to characterize the rheological and microstructural properties of the materials and their blends.

9.3. Bitumen

Bitumen is a construction material obtained from crude oil through distillation processes in the petroleum refinery. Bitumen is useful in road construction due to some of its properties such as adhesion, being water proof and durability. The bitumen is principally used in the road construction as a binder where it is mixed with aggregates to produce asphalt mixture. The asphalt mixture must have adequate properties in order to withstand the permanent deformation, fatigue cracking, and thermal cracking which are the main distress modes of flexible pavements. Therefore, it should have proper stiffness and bearing capacity and must be able to spread load evenly over the pavement layers.

A large number of procedures have described different fractions in bitumen. Based on the most common procedure, bitumen is divided into four groups of saturates, aromatics, resins and asphaltenes, namely SARAs. The molecular weight, complexity, heteroatom content, and aromaticity of these fractions increase in the order S<A<R<As (Claudi et al.; 1990, 1991).

The structure of bitumen is mostly regarded as a colloidal system. In this system, the asphaltenes particles are dispersed into the oily dispersion medium called the maltenes which is composed of saturates, aromatics and resins. As illustrated in Figure 9.1, a shell of resins has covered the asphaltene particles. The temperature and equilibrium between the covering part and dissolved part of resins affects the shell thickness (Lesueur et al., 1996; Palade et al., 2000; Redelius, 2006; Lesueur, 2009;).

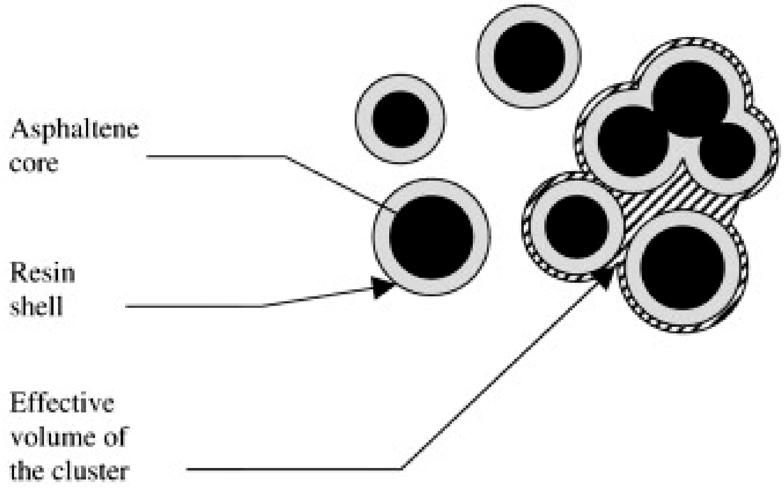

Figure 9.1: A simplified view of the colloidal structure of bitumen (Lesueur, 2009)

There are two different types of bitumen, namely, SOL type and GEL type. SOL type is a well dispersed solution in which sufficient amount of aromatics and resins allow a fully peptized and good dispersion of asphaltenes, whereas, the insufficient quantities of resins and aromatics fraction, in GEL type or the gelatinous type, leads in formation of large agglomerations or even continuous networks of asphaltenes. The stability of the bitumen colloidal structure and hence the type of bitumen can be determined by the index of Colloidal Instability (CI) which is defined as the ratio of the quantity of saturates and asphaltenes to the quantity of aromatics and resins. In other words, the higher value of CI indicates the bitumen which is more regarded as GEL type bitumen, whereas, the lower value of CI shows a more stable colloidal structure and a SOL type bitumen (Airey, 2009). There are other structural models such as conceptual microstructural model and thermodynamic solubility model which are proposed for bitumen structure (Petersen et al., 1994; Redelius, 2006). Irrespective of their differences, all these models aim at establishing a relation between the physical properties of the bitumen and its chemical composition. Referring to available references (e.g. Nellensteyn, 1924; Corbett, 1970; Petersen, 1984; Redelius, 2006; Airey, 2009), the rheological, physical and mechanical properties of bitumen depends on both the chemical composition of bitumen and the physical arrangement of the molecules in the bitumen (generally the structure of bitumen). In general, the chemical properties of bitumen are of importance to bitumen due to their disproportionately large effect on the physical characteristics of bitumen and its performance in flexible pavements. However, the performance of the flexible pavement is largely determined by the rheological characteristics, and to a lesser extent, the chemical properties of bitumen (Read and Whiteoak, 2003). In light of this, in this study, emphasis is placed on the binders' rheological properties. One of the most critical rheological properties of the bitumen which makes it suitable for its application in flexible pavements, is its thermoplastic and viscoelastic behaviour. Bitumen softens gradually on heating due to its thermoplastic property, and creeps under prolonged loading at elevated temperatures, while deforming elastically at ambient temperature.

9.3.1. Bitumen Classification

In 1987, as part of the Superior Performing Asphalt Pavement Program (Superpave), an asphalt binder specification system was developed for evaluation of asphalt performance properties and classification of binders based on specified maximum and minimum service temperature In this specification, the physical properties of an asphalt binder was matched with a performance grade (PG) based on climatic and environmental condition.

Upper Specification Temperature, °C	Lower Specification Temperature, °C						
PG 82	-10	-16	-22	-28	-34		
PG 76	-10	-16	-22	-28	-34		
PG 70	-10	-16	-22	-28	-34	-40	
PG 64	-10	-16	-22	-28	-34	-40	
PG 58		-16	-22	-28	-34	-40	
PG 52	-10	-16	-22	-28	-34	-40	-46
PG 46					-34	-40	-46

Figure 9.2: Performance Grades for Commercially Available Binders (after Asphalt Institute, 2008)

In this PG system, two numbers are assigned to each asphalt grade. For example for asphalt grade of PG 67-22, the first number shows the maximum temperature (in degree Celsius) at which the binder can still resist permanent deformation adequately. This number is an average 7-day maximum pavement service temperature. The second number is the minimum temperature at which the binder can perform properly to resist thermal cracking, and hence it is the minimum pavement service temperature. As presented in Figure 9.2, the binders with the maximum temperature ranging from 46 °C to 82°C and the minimum temperature ranging from -10 °C to -46°C (both in increments of six degrees) are commercially available.

As shown in Figure 9.2, the diagonal line which connects the asphalt grades of PG 82-10 to PG 46-46 is the border between the asphalt grades which can be produced at refineries and those produced only by modification (shaded areas). Useful Temperature Range (UTR) is a measure of the difference between upper and lower service temperatures. The binders produced at refineries without modification have a UTR of not more than 86 °C, whereas the modified binder often has a UTR of more than 92°C. Therefore, UTR can be used as an indicator showing the degree of required modification and the cost needed for modification. As UTR increases, this cost increases accordingly (Asphalt Institute, 2008).

9.4. Polymer Modifiers

The binder characteristics strongly influence the mechanical properties of asphalt mixture. To this point, binder should have a certain mechanical and rheological requirements as follows in order to fulfil the pavement criteria:

- For homogeneous coating of aggregates, the bitumen should be fluid enough at mixing and construction temperatures of about 160°C.
- To resist permanent deformation, the bitumen should be stiff enough at high temperatures (about 60°C depending on the local climate)
- To resist the cracking, the bitumen should be soft enough at lower temperature that pavement experiences (approximately down to -20°C depending on the local climate).

Accordingly, it can be concluded that obtaining bitumen to work well under all aforementioned conditions can be difficult. To overcome this problem, many researchers have tried to develop asphalt pavement performance by improving the asphalt binder behaviour through using different modifiers. Accordingly, as discussed in Chapter 2, in recent years considerable research has focused on modifying the binders' characteristics. There are a large variety of materials which are often used for modifying the binder. According to a comprehensive survey by Bahia et al. (2001), 55 modifiers have been identified that can be classified into 17 groups in terms of the nature of modifier and the effect of modifier on the pavement failures. Among all these modifiers, polymers are widely known to be easy to use and cost effective. Referring to available literature (e.g. Yeh et al., 2010; Afroz Sultana and Prasad, 2015; Huang et al., 2007, Casey et al., 2008), polymer addition may result in both a more flexible binder at low in-service temperature and enhanced properties at high in-service temperature, which significantly prevent the pavement from being deformed. They also improve the adhesive bonding to aggregate particles (Ghuzlan et al, 2013).

Polymers can exist in two different morphologies while in a solid phase:

- Amorphous, in which molecules are randomly oriented within the polymer when the material is cooled in a relaxed state. The cooled state of amorphous materials is highly similar to their molten state. The only difference between these two states is the molecules distance. These polymers can easily be altered in shape and generally exist in a rubbery state.
- Semicrystalline, which is an arrangement of ordered molecules with some amorphous regions. As the semi-crystalline polymer cools, a portion of the molecular chains forms crystals by folding up into densely packed regions. The polymer is classified as semi-crystalline, if more than 35% of the polymer chain forms these crystals. These polymers are stiff and exist in a glassy state.

The degree of crystallinity in a polymer is affected by different factors such as polymer type, additives, and cooling rate. The morphology and degree of crystallinity significantly influence the polymers properties. Polymers with high degree of crystallinity have a higher glass transition temperature and higher modulus, toughness, stiffness, tensile strength, hardness. In addition, they have more resistance to solvents but are less resistant to impact strength (Farrkkawa et al., 2006).

Today, there are a large variety of polymers which are often used for modifying the binder. These polymers can be mainly classified into the following categories:

- Elastomers such as rubber, which can be stretched and then recover their shape when the stretching force is released. Elastomers contribute to the elastic component of binder. The addition of elastomers to the binder results in an increase in the binder stiffness at high temperature and loading, and subsequently will improve the resistance to permanent deformation. However, elastomers will not substantially improve the ability of asphalt mixtures in thermal resistance.
- Plastomers such as polyethylene can form tough, rigid, three dimensional networks within the bitumen resulting in increase of the initial strength of the bitumen, and subsequently improving the ability of asphalt concrete to resist the heavy loads. Plastomers have less elasticity compared to elastomers as they don't provide an increase in the plastic component of binder while they increase the binder's stiffness at high temperature and loading (Lavin, 2003). Therefore, plastomers can improve

the rutting resistance but they lack the improvements in fatigue resistance, cracking resistance and low temperature performance (Vlachovicova et al., 2007) because of increased intermediate and low temperature stiffness. This makes them inferior to elastomers.

Although modifying the binders will result in the enhancement of binder's properties, but using virgin additives as modifier will increase road construction cost. Therefore, in recent years, many investigations have been conducted on modifying binders using waste materials as additives. Among these waste materials, application of waste plastics and rubber in certain amount as binder modifier can substantially enhance the stability, strength, fatigue life and generally the asphalt performance on one hand (Verma, 2008), and on the other hand it would be an ideal solution for reducing the environmental pollution associated with these non-biodegradable wastes. In light of this, according to the characteristics of elastomers and plastomers, this chapter analyses the effect of using HDPE and crumb rubber as binder modifier on the compatibility and thermal stability of the bitumen. The tests used to evaluate the binder properties were thermal analysis by thermogravimetric analysis (TGA) and differential scanning calorimetry (DSC), microstructure analysis by scanning electron microscope (SEM). In addition, FTIR was used to study the molecular structure of individual polymers and binder.

9.5. Materials

9.5.1. Bitumen

Bitumen of C320 was used as base material for this research which was kindly supplied by Boral Ltd. This bitumen corresponds to the most common bitumen used in Australia.

Table 9.1: Characteristics of the Original Bitumen

Characteristics	Unit	Methods	Value
Softening point	°C	AS 2341.18	52
Penetration at 25°C	dmm	As 2341.12	min 40
Flashpoint	°C	AS 2341.14	min 250
Viscosity at 60°C	Pa.s	AS 2341.2	320
Viscosity at 135°C	Pa.s	As 2341.2	0.5
Specific Gravity	Kg/m^3	AS 2341.7	1.03

C320 is classified and manufactured in accordance with AS 2008 (2013) and is suitable for medium to heavy traffic loading as well as for heavy duty pavements and hot climate seals. The typical characteristics of Bitumen C320 are presented in Table 9.1.

As emphasized earlier and shown in Figure 9.3, the complex structure of bitumen is composed of unsaturated structures which is categroized into two main chemical groups, namely, asphaltenes (which are insoluble in n-heptanes) and maltenes.

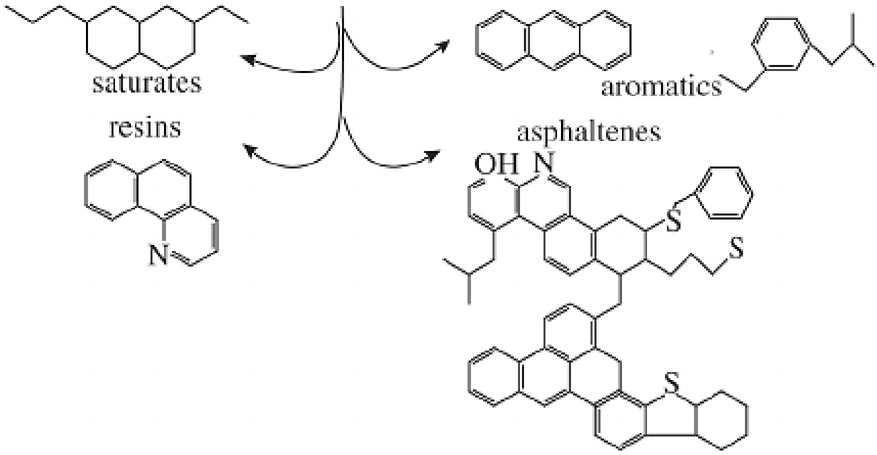

Figure 9.3: Main Compounds in Representative Structures of the Four Bitumen Fractions (Lucena et al., 2004)

The maltenes are further split into saturates, aromatics, and resins. The proportion of bitumen fractions and the molecular weight of each fraction is presented in Table 9.2.

Table 9.2: Proportion and Molecular Weight of Bitumen Chemical Fractions (Read and Whiteoak, 2003; Lesueur, 2009; Nellensteyn, 1924; Airey, 2009; Paliukaite et al., 2015)

Fraction	Proportion of the Overall Bitumen	Molecular Weight	Description
Asphaltenes	5 – 25%	600 – 3000	Substantial effect on bitumen rheological properties
Resins	15-25%	500 – 1300	Dispersing agent for asphaltenes; the proportion of resins to asphaltenes controls the structural characteristics of the bitumen
Aromatics	40 – 65%	300 - 800	Major dispersion medium for asphaltenes
Saturates	5 – 20%	300-600	Non-polar viscous oil

The rheological properties of bitumen are highly affected by the asphaltene content due to its physical parameters such as glass transition and bitumen viscosity (Corbett, 1970; Halstead, 1987). An increase of the asphaltenes content will generally result in harder bitumen with a lower penetration, higher softening point, and higher viscosity (Scholz, 1995).

9.5.2. High Density Polyethylene (HDPE)

Among plastics, polyethylene (PE) forms the largest portion followed by polyethylene therephtalate (PET). To this point, this study focused on polyethylene and particularly High Density Polyethylene (HDPE). The HDPE used in this research were obtained from plastic recycling plant. As shown in Figure 9.4 (a), the HDPE is in the granular form with the particle size of 2.36 mm. HDPE, like other plastics, is polymers consisting of very large molecules made up of smaller units called monomer which are joined together in a chain by a process called polymerization. Polyethylene is semi-crystalline material with a wide range of properties and appropriate resistance to chemicals and fatigue. A molecule of polyethylene has a very simple structure, composed of a long chain of carbon atoms with two hydrogen atoms attached to them, as shown in Figure 9.4(b). Sometimes other elements such as oxygen, nitrogen, chlorine or fluorine attach to these polymer molecules. They are light weight molecules with low moisture absorption rates and good resistance to organic solvents.

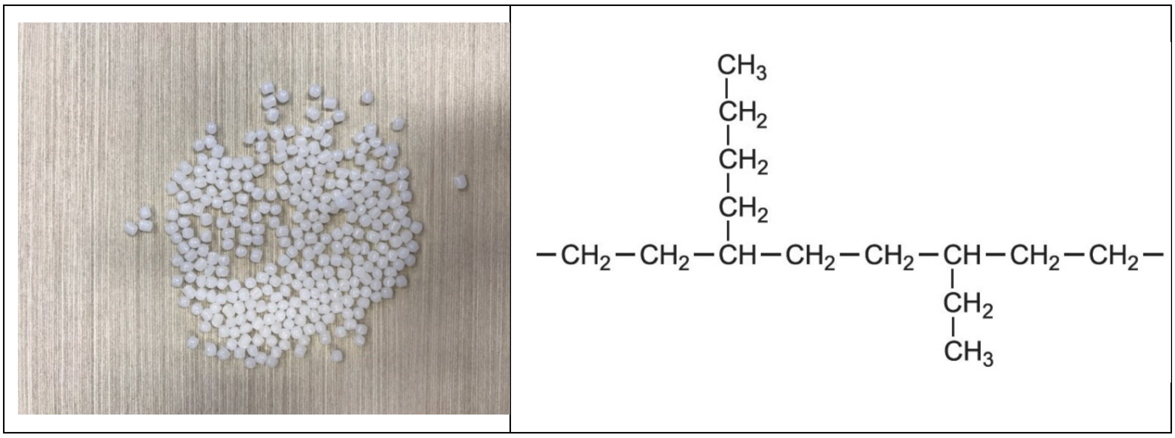

Figure 9.4: Analysed Material a) HDPE, b) Main Compounds in HDPE

HDPE is one type of thermoplastics. As most of thermoplastics can soften at temperature ranging from 130°C to 140°C with no gas emission in the temperature range of 130°C to 180°C, they can be a potential option for blending with bitumen in asphalt production because the heating temperature for bitumen ranges from 155°C to 165°C during the whole process of asphalt pavement construction (Gawande et al., 2012). Table 9.3 presents the information regarding thermal behaviour of polyethylenes, which emphasizes its suitability as binder modifier.

Table 9.3: Thermal Characteristics of Polyethylene (Gawande et al., 2012)

Characteristics	Unit	Methods	Reported Products
Solubility in Water	-	Nil	-

Softening Temperature	°C	100 - 120	No gas
Decomposition Temperature	°C	270 - 350	CH_4 , C_2H_6
Ignition Temperature Range	°C	> 700	CO, CO_2

In order to find the relevant properties of HDPE, some tests were conducted on this material and the results of these tests are presented in Table 9.4.

Table 9.4: Characteristics of High Density Polyethylene (HDPE)

Characteristics	Unit	Methods	Value
Density	g/cm^3	AS 1141.5	0.963
Size	mm	AS 1141.11.1	2.36
Water Absorption	%	AS 1141.5	0.0

9.5.3. Crumb Rubber

Crumb rubber used in this research was obtained from a tyre recycling plant which processes the car tyres into crumb rubber through the ambient grinding method. The crumb rubber was provided in granules form, as shown in Figure 9.5 (a).

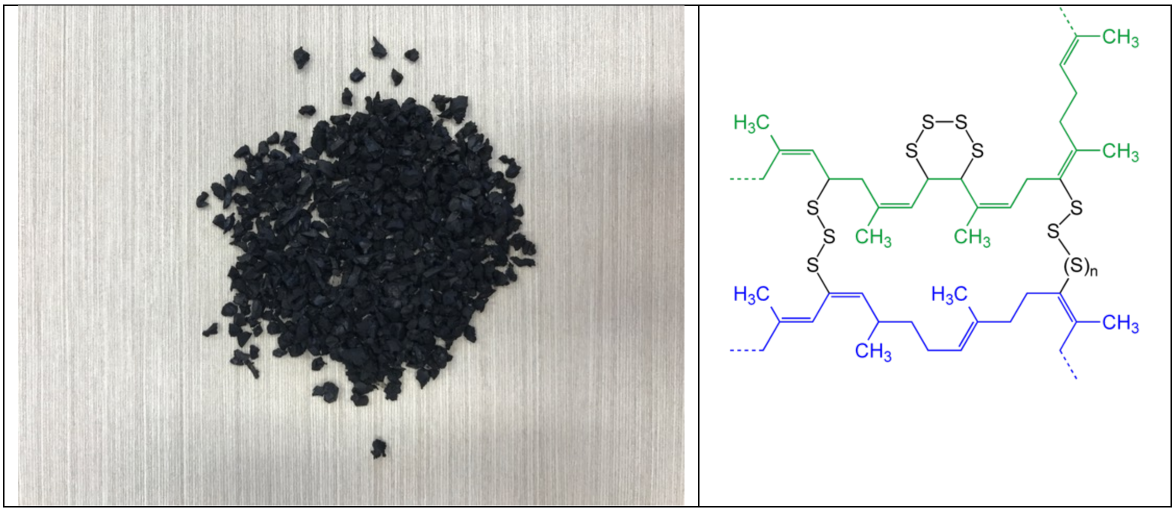

Figure 9.5: Analysed Material a) Crumb Rubber, b) Main Compounds in Rubber

The particle distribution test was performed on crumb rubber. The result of sieve analysis is presented in Table 9.5. The ground tyre rubber has a particle size average between 8 and 50 mesh (2.36 mm and 0.300 mm).

Table 9.5: Particle Size Distribution of Crumb Rubber

Sieve No.	Sieve Size (mm)	Mass Retained (%)
4	4.75	0.0
8	2.36	25.7
16	1.18	67.7

30	0.600	6.2
50	0.300	0.4
100	0.150	0.0
200	0.075	0.0

In this study, the particle size of 2.36 and 1.18 mm were chosen due to its presence in the ground tyre rubber in higher amounts.

Table 9.6: Characteristics of Crumb Rubber

Characteristics	Unit	Methods	Value
Density	g/cm^3	AS 1141.5	0.982
Size	mm	AS 1141.11.1	1.18-2.36
Water Absorption	%	AS 1141.5	0.1

The properties of crumb rubber are presented in Table 9.6, which are obtained from conducting relevant tests on crumb rubber. It should be noted that tyre rubber is typically a composition of three polymers including polyisoprene (natural rubber), polybutadiene and polystyrene-butadiene (Quek et al., 2012). The main compounds in rubber are shown in Figure 9.5(b).

9.6. Methodology

In this research, both individual materials (i.e. bitumen, HDPE, and crumb rubber) and their blend in a certain percentage were analysed based on their calorimetric curve, thermal transition, distribution in bitumen after modification, and their overall quality. The analysis of the materials and blend were performed by means of Thermogravimetric Analysis (TGA), Differential Scanning Calorimeter (DSC) and Scanning Electron Microscope (SEM). For analysing the performance of original bitumen and modified binders, Fourier Transform Infrared (FTIR) Spectrometry was used. These tests were performed at the Advanced Materials Characterisation Facility (AMCF) of the Western Sydney University.

9.6.1. Sample Preparation

For performing analysis on individual materials, the samples were prepared based on the requirements of the equipment. In the case of modified binders, the modifiers were incorporated into the bitumen as per the following percentages by weight of bitumen:

- HDPE: 2% (as provided by recycling plant approximately retained on sieve No. 8)
- Crumb rubber: 8% (material retained on sieves No. 8 and No.16)

The formulations of samples under study are summarized in Table 9.7.

Table 9.7: Formulation of Composite Samples

Composite Type	Bitumen	HDPE	Crumb Rubber
B100-H0-R0	100	0	0
B0-H100-R0	0	100	0
B0-H0-R100	0	0	100
B90-H2-R8	90	2	8

Modified binders were prepared using a high shear mixer (Figure 9.6). For this purpose, the respective amount of modifiers and bitumen were mixed thoroughly using rotation speed of 4000 rpm. The system was kept at 185ºC during mixing.

Figure 9.6: Preparation of the Blend of Polymers Using High Shear Mixer

In order to study the thermal behaviour of individual polymers and their blend, in all cases, a small amount of material (5 to 10 mg) was placed in a measuring pan. To prevent pressure build up during the test, it was advised to have a small opening in the small pan. After this preparation, the samples were placed in DSC or TGA equipment.

9.6.2. Analysis Method

It is expected that the addition of polymers influences the microstructure of binder. In theory, the addition of polymers containing hard segments provides higher strength, whereas the soft segment polymers improve toughness and low temperature cracking. Since the binder modification depends on the compatibility of bitumen and polymer as modifier, this chapter covers the study of the individual polymers to identify some of their physical and chemical properties, their thermal behaviour, and their microstructure. The tests to characterize the analysed materials were performed in the Advanced Materials Characterisation Facility (AMCF) at Western Sydney University as mentioned earlier. These tests included thermal analysis, structural characterization and microstructure analysis and the main features of testing procedure are represented in the following sections.

9.6.2.1. Thermal Analysis

Thermal analysis corresponds to a group of techniques used to measure the physical and chemical properties of materials as a function of temperature. The measurements can be performed in different atmospheres including inert atmosphere (nitrogen, argon, helium) or an oxidative atmosphere (air, oxygen). The gas pressure can also selectively vary in thermal analysis. In this research, the thermal behaviour of materials and their blend were investigated through differential scanning calorimetry (DSC) and thermal gravimetric analysis (TGA).

- **Differential Scanning Calorimetry (DSC)**

Parameters such as glass transition temperature (T_g), melting point, and the degree of crystallization which can be correlated with the pavements thermal susceptibility, were monitored by DSC. It should be noted that the glass transition temperature is more important in polymer applications compared to the melting point, because it corresponds to the polymer behaviour under ambient conditions. If the glass transition temperature of a polymer is much higher than the ambient room temperature, the polymer would be a brittle glassy polymer which is stiff and has low resistance to impact, whereas, a polymer with a glass transition temperature of much less than the room temperature is commonly termed a rubber or elastomer. These types of polymers are soft and stretchable. The materials with glass transition temperature of relatively close to the ambient temperature act like plastic materials. These materials are tough and strong and they have proper resistance to impact.

In this research, DSC analysis was conducted in accordance with ASTM E473-85 in a NETZCH DSC 204 F1 to obtain the thermal critical points of materials (Figure 9.7). The test specimen weighing about 5 mg were heated up and subjected to different temperature range, depending on polymer type, in an aluminium crucible under an air flow of 100 mL/min at a rate of 10 °C per minute.

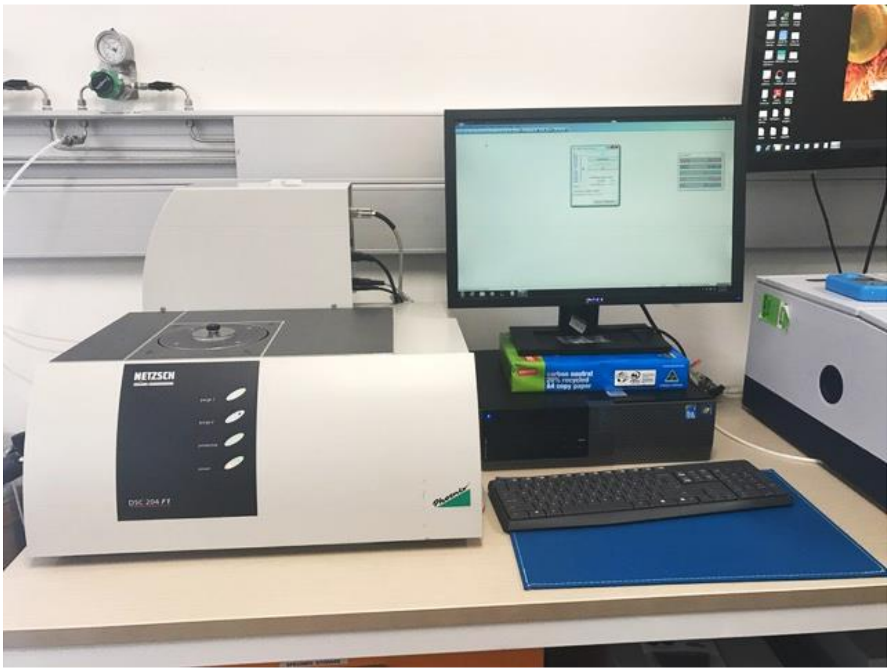

Figure 9.7: Conducting DSC Analysis on Polymers and Their Blend

For bitumen, DSC experiments were conducted on about 5 mg samples placed in aluminium crucibles with perforated covers. Before conducting DSC, the bitumen sample was homogenized at 130°C for about one hour, and then placed in the DSC equipment. In DSC running, first, the samples were cooled to -100°C at a heating rate of -10°C/min. The samples maintained at the low temperature for about 15 minutes to ensure a stabilized initial reading. Then they heated up to 200°C at a heating rate of 10°C/min. The DSC thermograph recorded during this heating scan is named as the first scan. After completion of the first scan, the sample was maintained at 200°C for 5 minutes to eliminate the thermal history and then quickly cooled from 200°C to its starting temperature (-100°C) at a cooling rate of -10°C/min and again held for about 15 minutes before being reheated to 200°C at a heating rate of 10°C/min. The DSC thermograph recorded during this second heating scan is named as the second scan. The same procedure was repeated to provide the third heating scan.

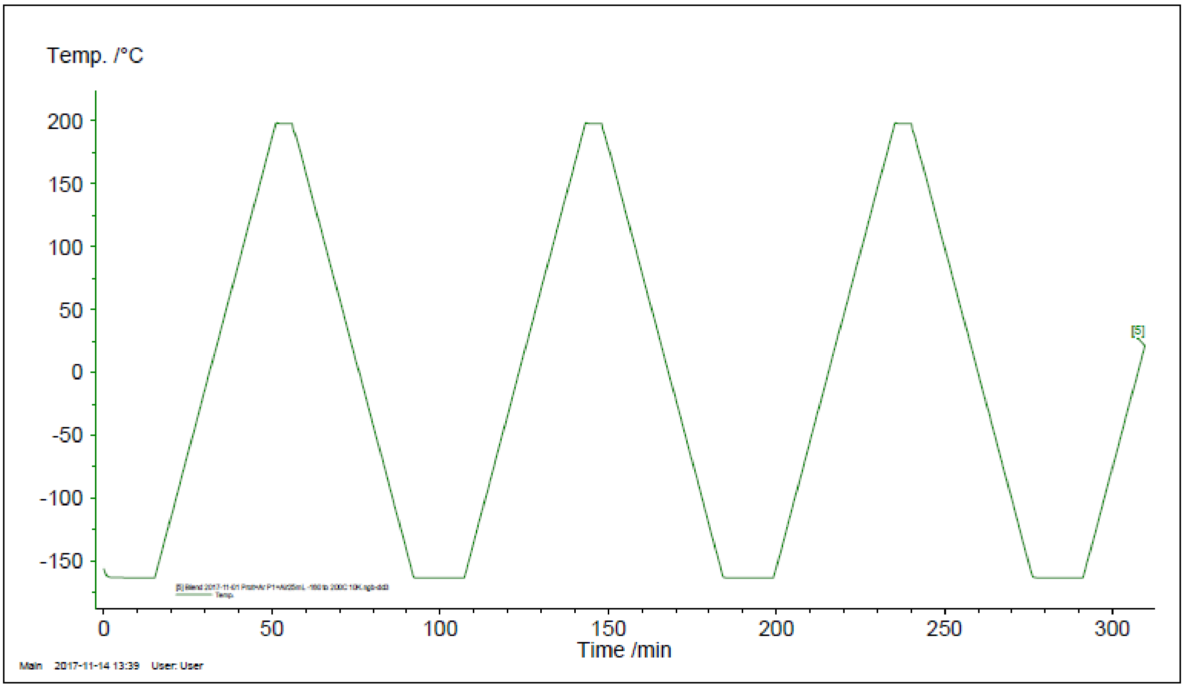

Figure 9.8: Schematic of DSC Procedure

For rubber, similar to bitumen, three cycles of cooling and heating were considered as the method of the experiment with the same heating rate of 10°C/min, cooling rate of -50°C/min and temperature range of -100°C to 200°C. The thermal history is depicted schematically in Figure 9.8.

For HDPE, the DSC procedure was the same as bitumen and rubber with an exception of the temperature range which was considered from -160°C to 200°C.

Glass transition and melting point were measured from DSC curves. The percentage of crystallized fraction (CF) was determined from the following equation through dividing the observed melting enthalpy (ΔH_{obs}) by the melting enthalpy of 100% crystalline material (ΔH_o).

$$\boldsymbol{CF = \frac{(\Delta H_{obs} \times 100)}{\Delta H_o}} \qquad (\mathbf{9.1})$$

The values of ΔH_o depends on the material type and can be found in the literature. For example, a value of 200 J/g was used by Claudy et al. (1998) and Lesueur (2009). The values of 180 J/g and 121 J/g was used by Michon et al. (1999) and Lu and Redelius (2007), respectively.

- **Thermogravimetry Analysis (TGA)**

Thermogravimetry analysis (TGA) was performed to study the kinetics and to investigate the degradation process of materials at high temperatures. In this research, the thermal decomposition of materials was studied using 5 mg samples in an aluminium crucible under an air flow of 100 mL/min from 30 °C to 600°C and at heating rate of 10 °C/min. The TGA curves and its differential (DTG) were analyzed in a NETZSCH STA-449C thermogravimetric analyser (Figure 9.9). The onset temperature of the mass loss effect (T_O) and temperature of peak rate of mass loss (T_p) were determined from TGA thermographs.

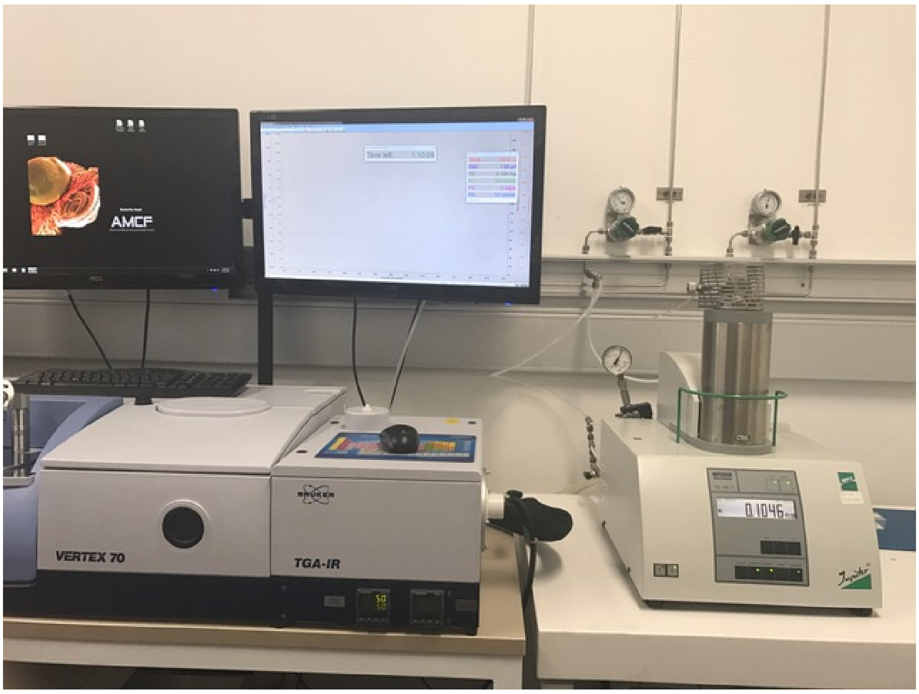

Figure 9.9: Conducting TGA Analysis on Polymers and Their Blend

9.6.2.2. Structural Characterization

In addition to TGA, the molecular structure of the materials was also monitored using FTIR. In this technique, the structural characterization after modification was investigated by infrared spectroscopy using NETZSCH VERTEX-70 spectrometer (Figure 9.10).

Figure 9.10: Conducting FTIR on Polymers and Their Blend

The spectra of the materials were analysed in the range of 600 to 4000 cm-1.

9.6.2.3. Microstructure Characterization

The microstructure of the materials was investigated under Scanning Electron Microscope (SEM). Scanning electron microscopy analysis was done in 6510LV SEM employing between 10 and 20 kW (Figure 9.11).

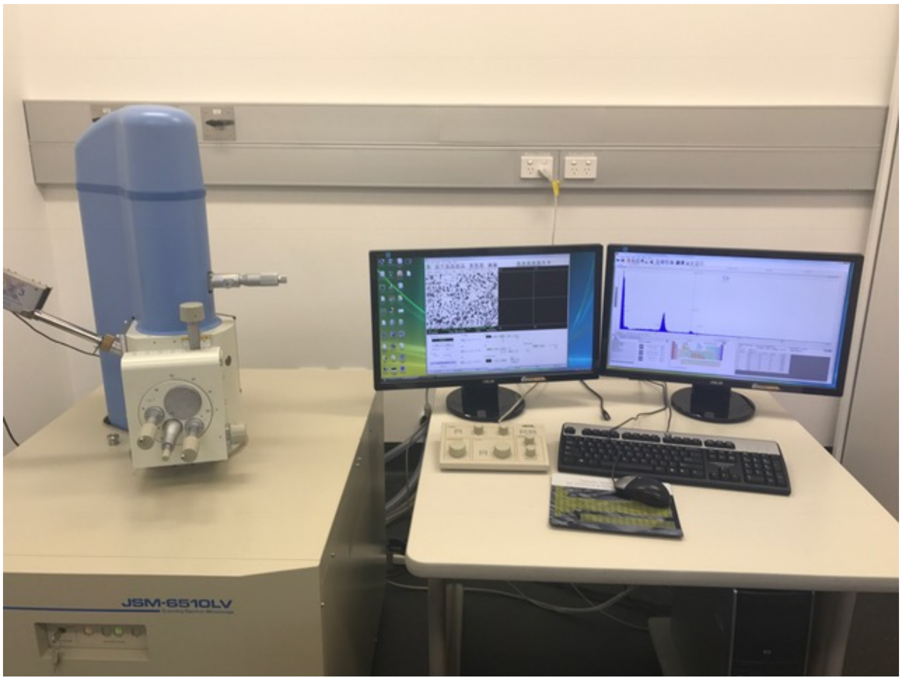

Figure 9.11: Conducting SEM on Polymers and Their Blend

The specimens in this study were examined with magnifications of 100 – 1000 and the results of this study with best magnification are presented in the following sections.

9.7. Results and Discussion

9.7.1. Thermal Analysis by DSC

Heating the polymers results in a number of phase changes such as the glass transition (T_g), crystallization transition (T_c) and melting point (T_m). DSC analysis is a useful technique to identify the location of these thermal parameters. In the DSC curves, the sharp peaks are related to the polymer melting and the areas under these peaks provides the heats of fusion (ΔH). Furthermore, the smaller inconsistencies at the lower temperature are most likely related to the glass transition. The polymer morphology substantially influences the polymers properties on a certain range of heating at a specific rate.

In this research, DSC technique is employed to investigate the influence of modifying the binder by polymers on the transition temperatures and the crystallization process. Accordingly, the DSC curves were examined to evaluate the physical characteristics of individual materials and their blend. It should be noted that for DSC runs, the complete set of heating-cooling process cycle were repeated three times for each polymer and polymer modified binders, where the first run is usually carried out to remove all impurities and moisture from the sample. In addition, for the precise evaluation of transitions, a wide temperature range is considered for DSC analysis. The following sections discuss the phase transitions observed for bitumen, rubber, and HDPE through DSC analysis.

9.7.1.1. DSC Analysis of Bitumen

In DSC analysis, the thermal parameters for bitumen depend on the refined petroleum source as well as the petroleum refining process. Figure 9.12 presents the DSC thermograph of neat bitumen and the corresponding first-derivative curve. The effects identified in the thermograph, as mentioned previously (e.g. Claudy et al., 1993; Bosselet et al., 1983; Letoffe et al., 1995), are described below.

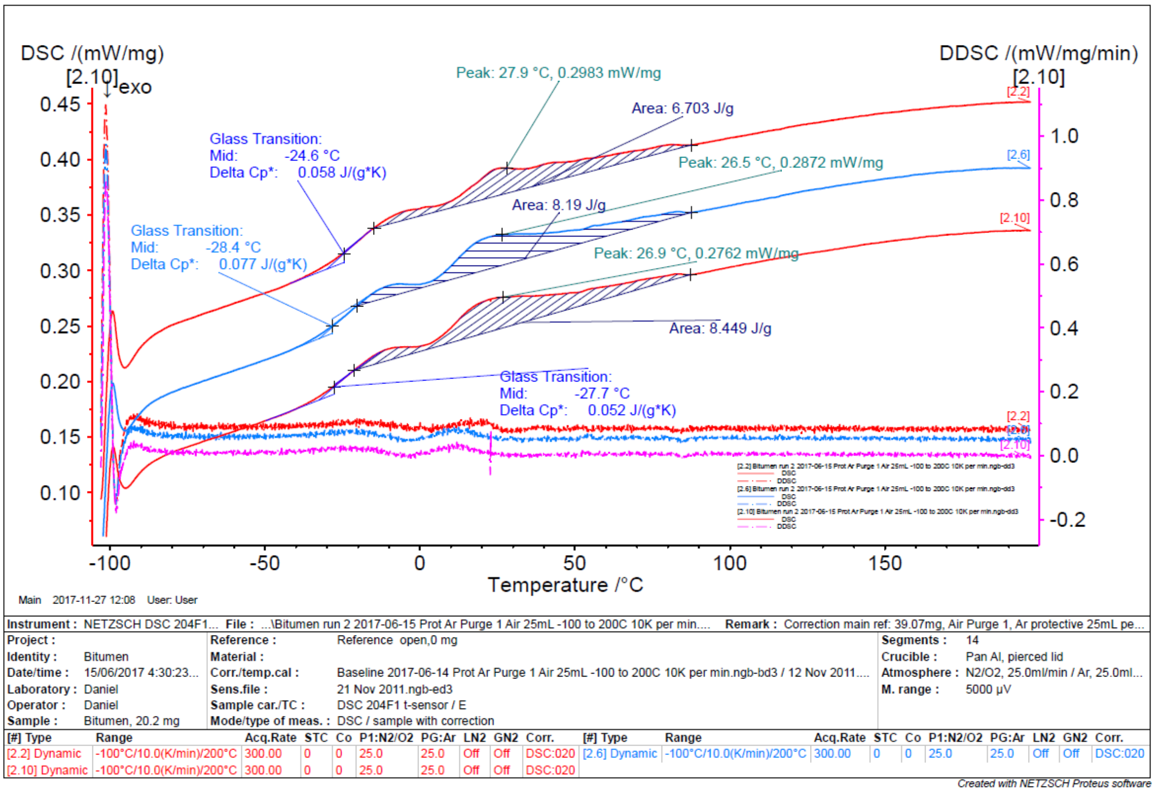

Figure 9.12: DSC and DDSC Thermographs of Bitumen

An increase in the heat capacity for neat bitumen can be observed in the DSC curve by an abrupt change in the slope of the curve placed in the low temperature region (around -30°C) corresponding to the glass transition temperature (T_g) of the bitumen. The glass transition temperature (T_g) is a material's temperature at which all molecular transitional motion is frozen, therefore, the material becomes rigid and brittle at or below this temperature. The glass transition temperature of polymers is one of the most important parameters as it is related to the average molecular weight of polymers and hence provides information about their composition. Moreover, it demonstrates the viscoelastic behaviour of polymers at low temperatures (Jimenez-Mateos et al., 1996). Therefore, the glass transition temperature of neat bitumen is believed to be closely related to the low temperature performance of asphalts. As illustrated in Figure 9.12, the middle point of the temperature range where the transition occurs is considered as the glass transition temperature. In addition, as shown in Figure 9.12, T_{gonset} of bitumen is at about -40°C. Referring to Harrison (1992), the T_{gonset} temperature is more closely related to the glass transition temperature of the saturate fraction that has the lowest T_g. The main T_g reflects the characteristics of the glass transition temperatures of the majority of components.

Table 9.8: Obtained Transition Temperatures from DSC Thermographs for Bitumen

Cycle	T_g (°C)	T_{onset} (°C)	T_{end} (°C)	T_m (°C)	T_c (°C)	ΔH_m (J/g)	CF (%)
1st Heating	-24.6	-37	-11	27.9	-	6.703	3.72
2nd Heating	-28.4	-44	-16	26.5	-	8.190	4.55
3rd Heating	-27.7	-43	-18	25.7	-	8.449	4.69

At temperatures above T_g, an exothermal peak and a broad endothermal peak from about -20°C to 85°C is observed. The big exothermal peak next to the glass transition is most likely the result of crystallization of small paraffin molecules and melting of the crystallites formed during heating or cooling are known as the main reason to produce endothermal peaks in this region (Harrison et al., 1992). The melting and crystallization temperatures are presented in Table 9.8, based on the heating and cooling scans observed in Figure 9.13.

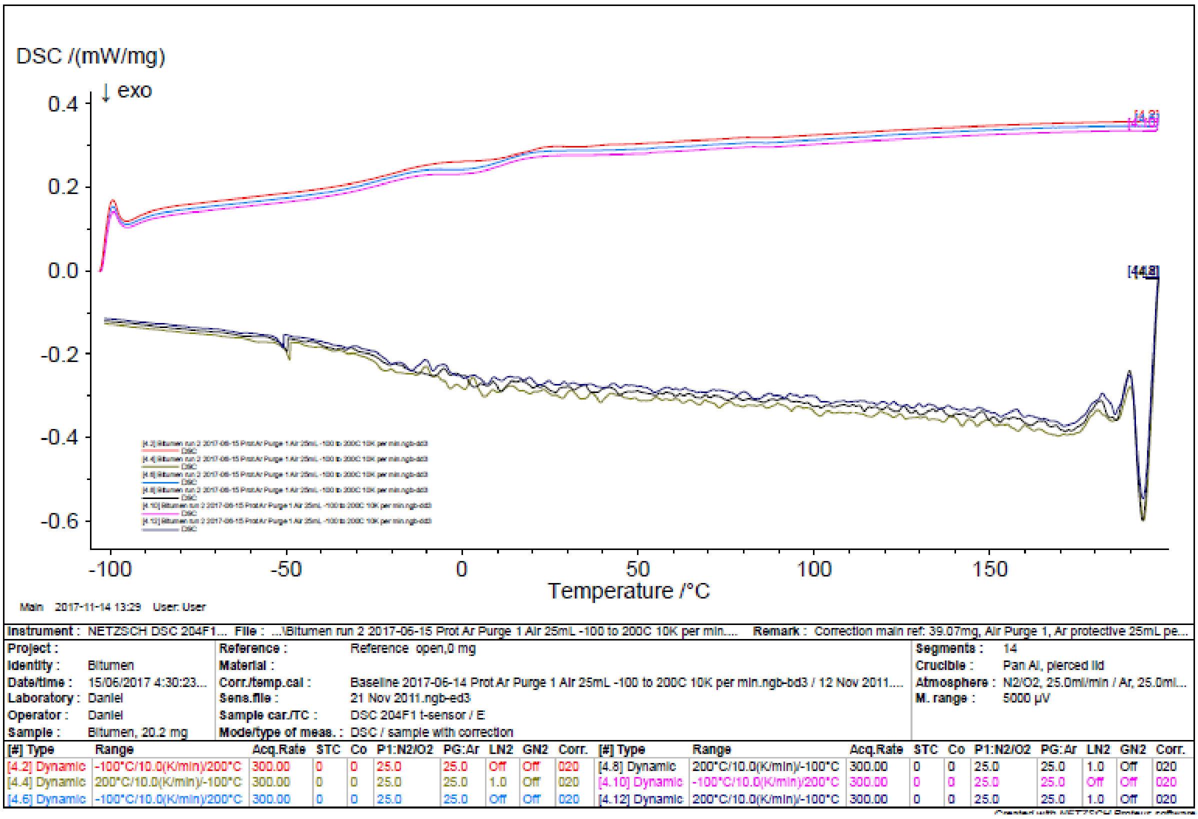

Figure 9.13: Heating and Cooling Cycles of Bitumen

It should be noted that the exothermic effect just above the T_g in DSC thermographs is negligible, as it has been associated in previous studies (Letoffe et al., 1995; Claudy et al., 1993) with the crystallization of certain molecules which are not crystallized during cooling.

In addition, referring to the available literature (Masegosa et al., 2012) the dissolution of the crystallized fractions (CF) is the main reason of the enthalpy changes and can be calculated from the area under the peak to a reference enthalpy of dissolution. As illustrated in Figure 9.12, in order to calculate this parameter, a straight baseline between the end of the glass transition and the end of the endothermic effects is drawn. In this research, the reference enthalpy value of 180 J/g is used for the estimation of the amount of crystallized fraction of bitumen based on previous investigations (Claudy et al., 1991; Jimenez-Mateos et al., 1996; Michon et al., 1999; Claudy et al., 1998; Lesueur et al., 2000; Lucena et al., 2004). The calculation of the crystallisable fraction content shows a value of about 4%, which is considered small. The presence of wax content in bitumen is commonly responsible for the extent of crystallisable fractions, which is the main reason for the problem of pavement exudation and inappropriate thermal susceptibility (Lucena et al., 2004).

9.7.1.2. DSC Analysis of Rubber

To achieve DSC curve for rubber, similar to bitumen, three cycles of cooling and heating were considered as the method of the experiment with the same heating rate of 10°C/min, cooling rate of -10°C/min and temperature range of -100°C to 200°C.

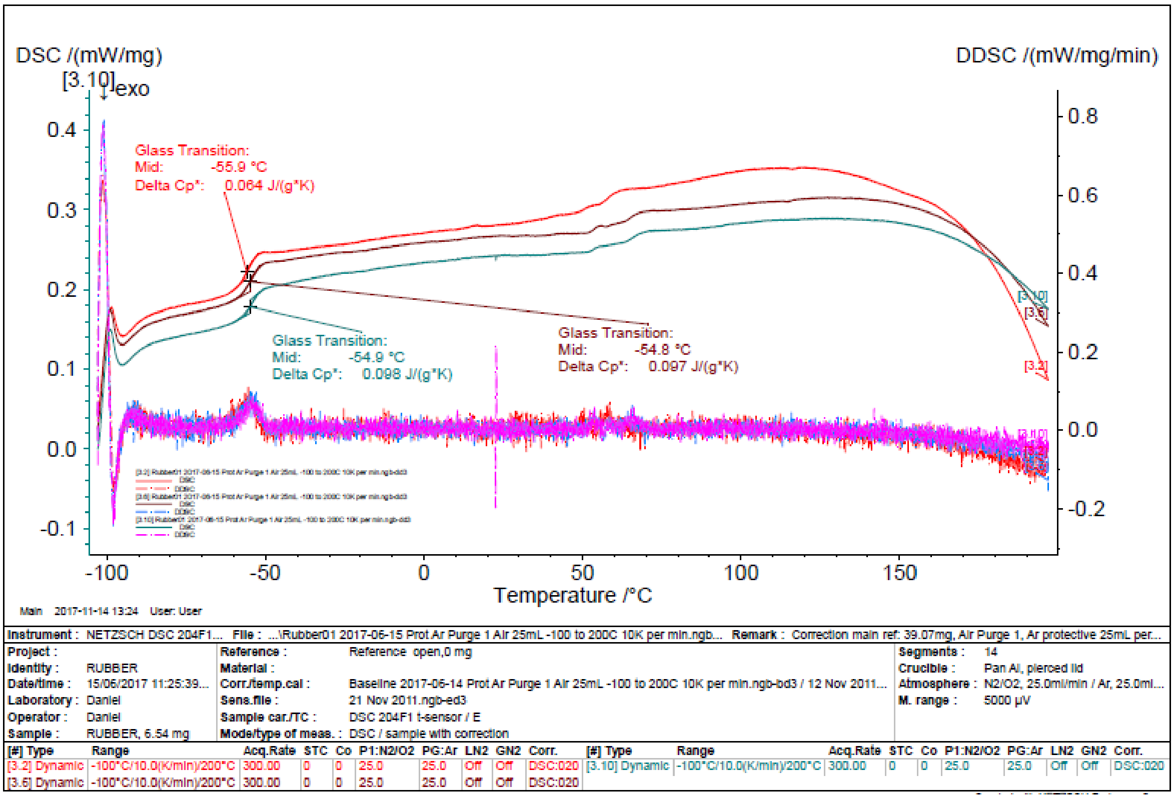

Figure 9.14: DSC and DDSC Thermographs of Rubber

The DSC thermograph of rubber (Figure 9.14) presented a glass transition temperature (T_g) at -55° C. However, due to amorphous nature of rubber, DSC curve does not present a well-defined melting temperature.

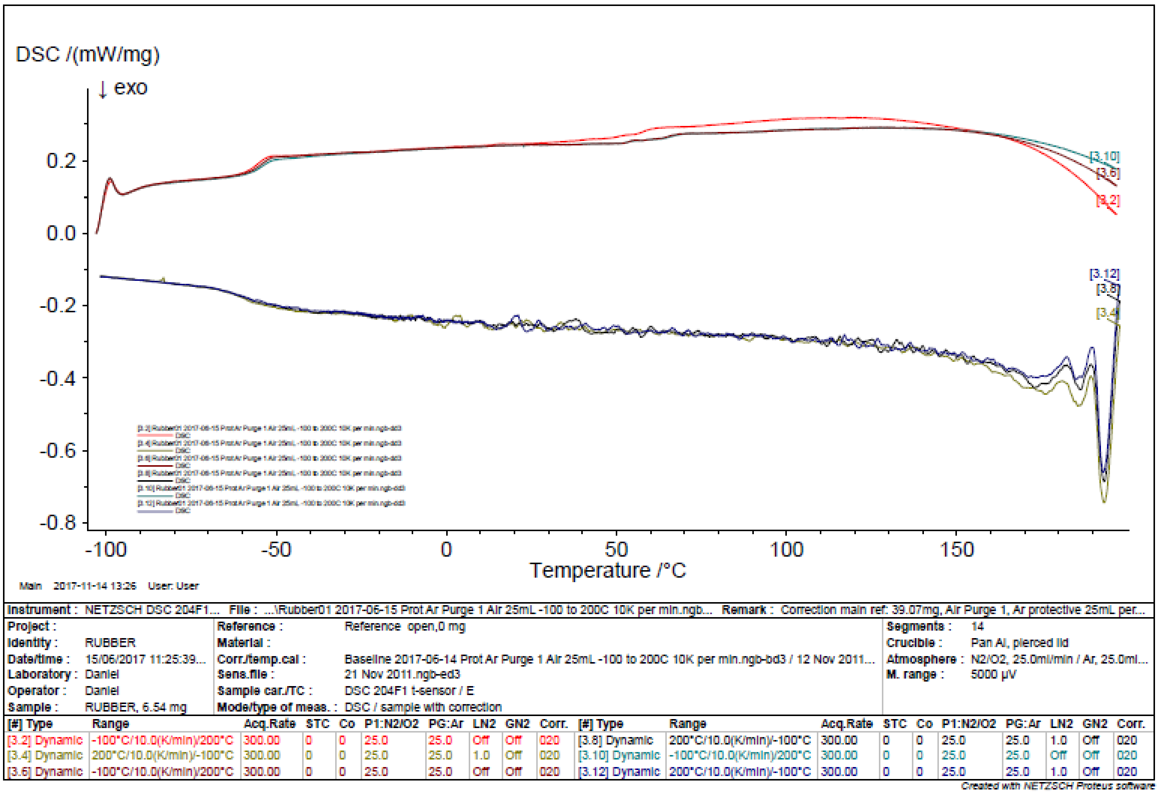

Figure 9.15: Heating and Cooling Cycles of Rubber

The glass transition temperature for different cycles of heating and cooling (Figure 9.15) of rubber are presented in Table 9.9.

Table 9.9: Obtained Glass Transition Temperatures from DSC Thermographs for Rubber

Cycle	T_g (°C)	T_{onset} (°C)	T_{end} (°C)	T_m (°C)	ΔH_m (J/g)
1st Heating	-55.9	-60.5	-52.6	-	-
2nd Heating	-54.8	-61.9	-51.2	-	-
3rd Heating	-54.9	-60.7	-50.4	-	-

9.7.1.3. DSC Analysis of HDPE

For HDPE DSC analysis as shown in Figure 9.16, it can be observed that HDPE started to lose its solid form at around -100°C corresponding to the glass transition temperature (Tg) of HDPE. As the temperature increases, a strong endothermic peak average value at 134°C can be observed, which is most likely related to the melting of crystalline domains of HDPE.

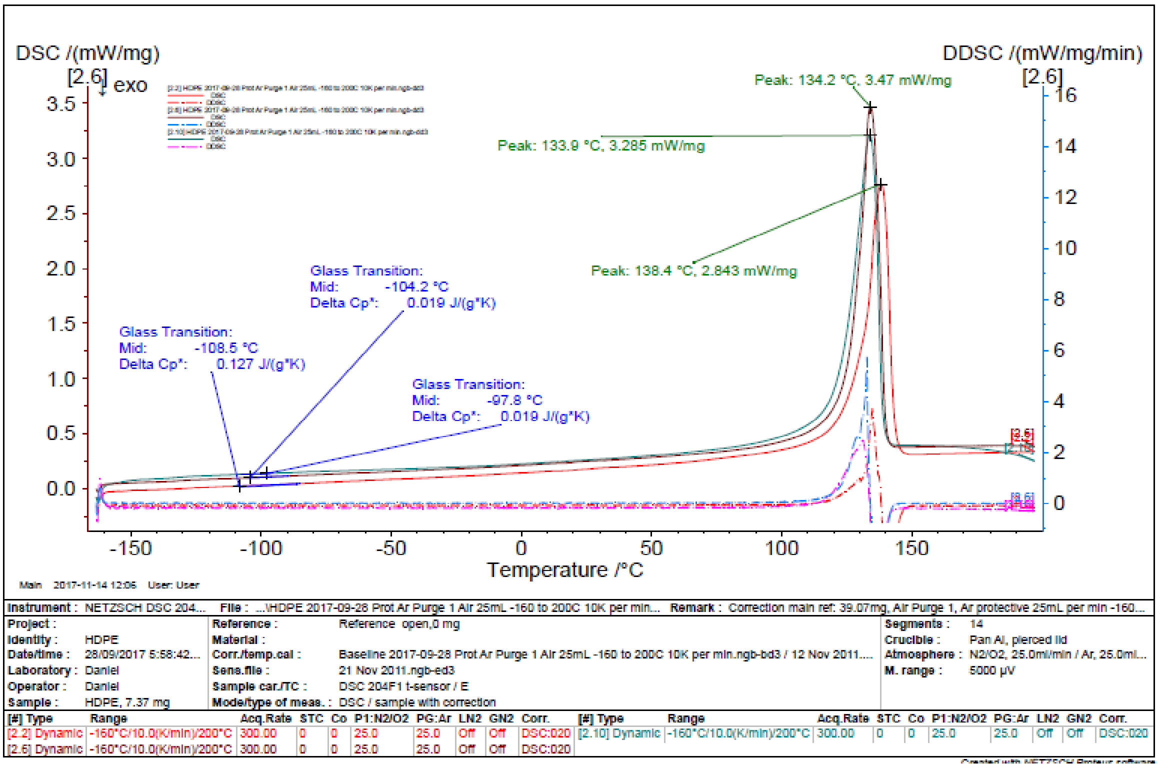

Figure 9.16: DSC and DDSC Thermographs of HDPE for Heating Cycles

The DSC curve of HDPE in second heating cycle is illustrated in Figure 9.17. As can be observed, the energy consumption for melting of crystalline domain of HDPE was 221.1 J/g that occurred between the beginning and end of melting point.

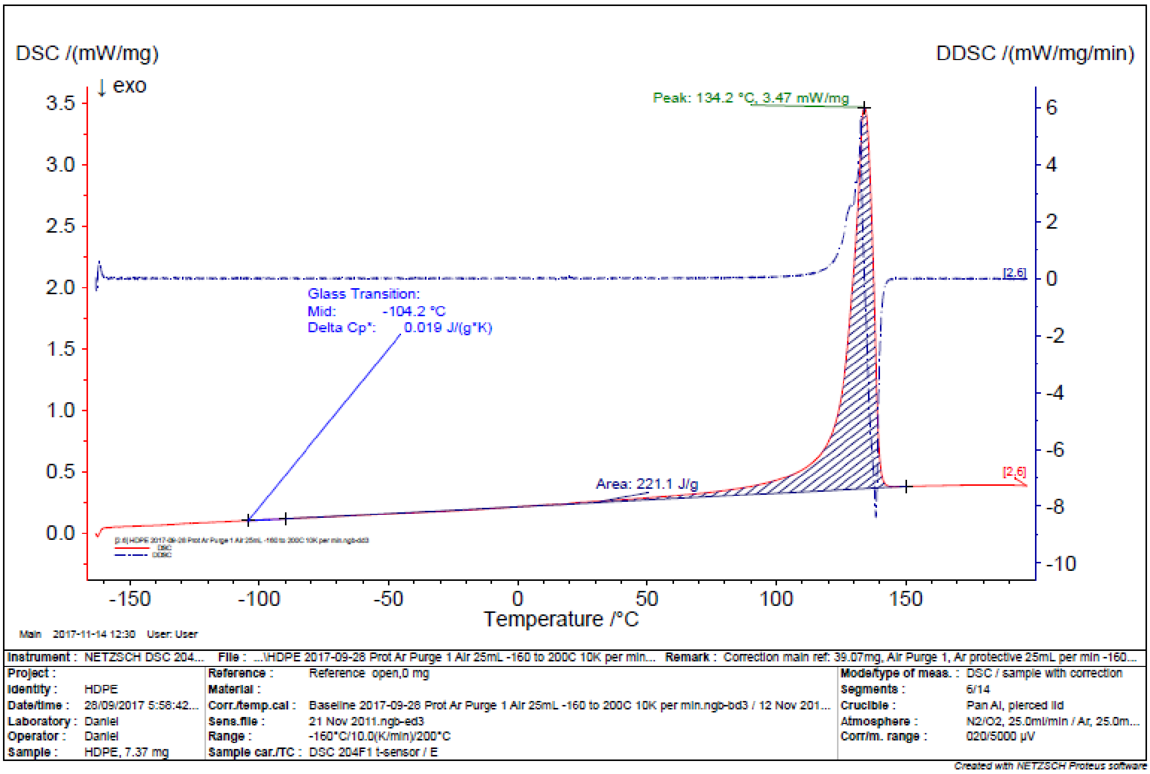

Figure 9.17: DSC and DDSC Thermographs of HDPE for Second Heating Cycle

The cooling cycle involves the rate of cooling temperature of -10°C /min. As shown in Figure 9.18, the peak point for HDPE becomes totally solid at around 115 °C which means that the crystallization temperature (Tc) of HDPE, an exothermic peak, is 115°C.

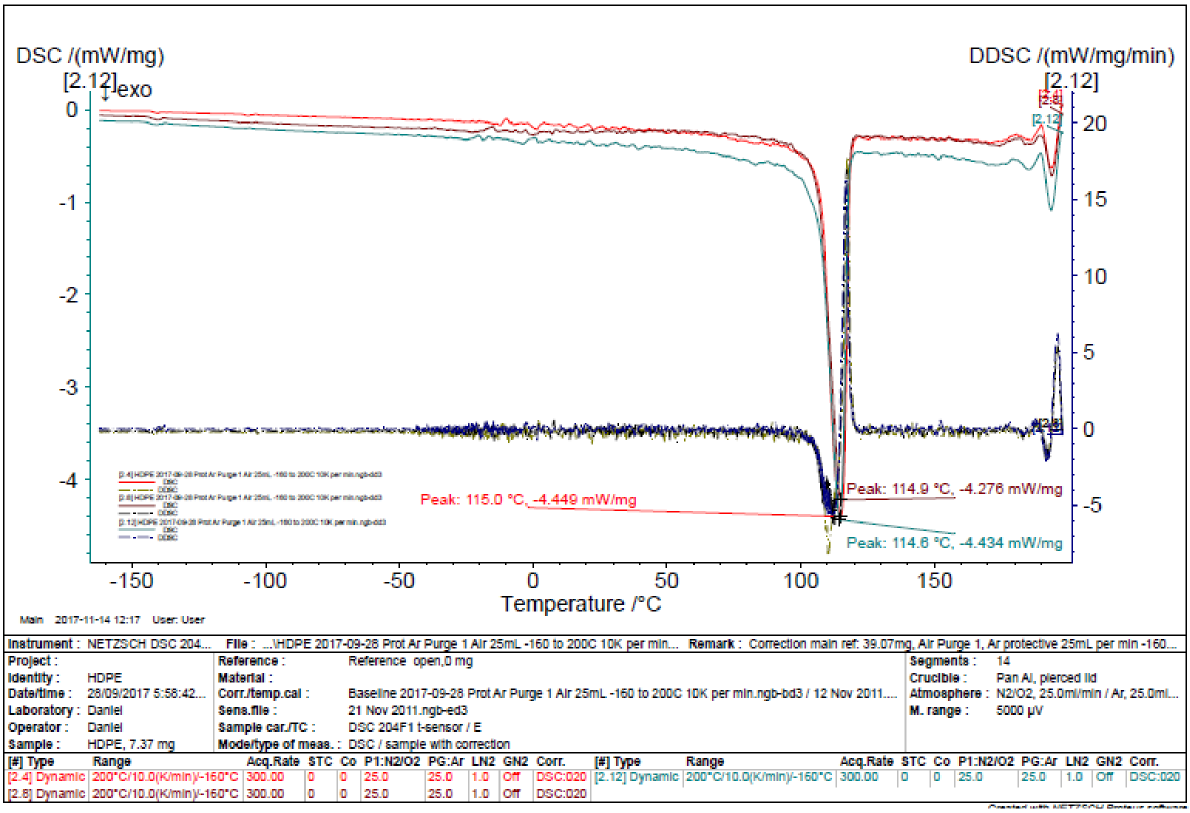

Figure 9.18: DSC and DDSC Thermographs of HDPE for Cooling Cycles

From Figure 9.19 for second cooling cycle of HDPE, the energy released from the sample can be obtained from the calculation of the area under the cooling curve which equals to -189.1 J/g.

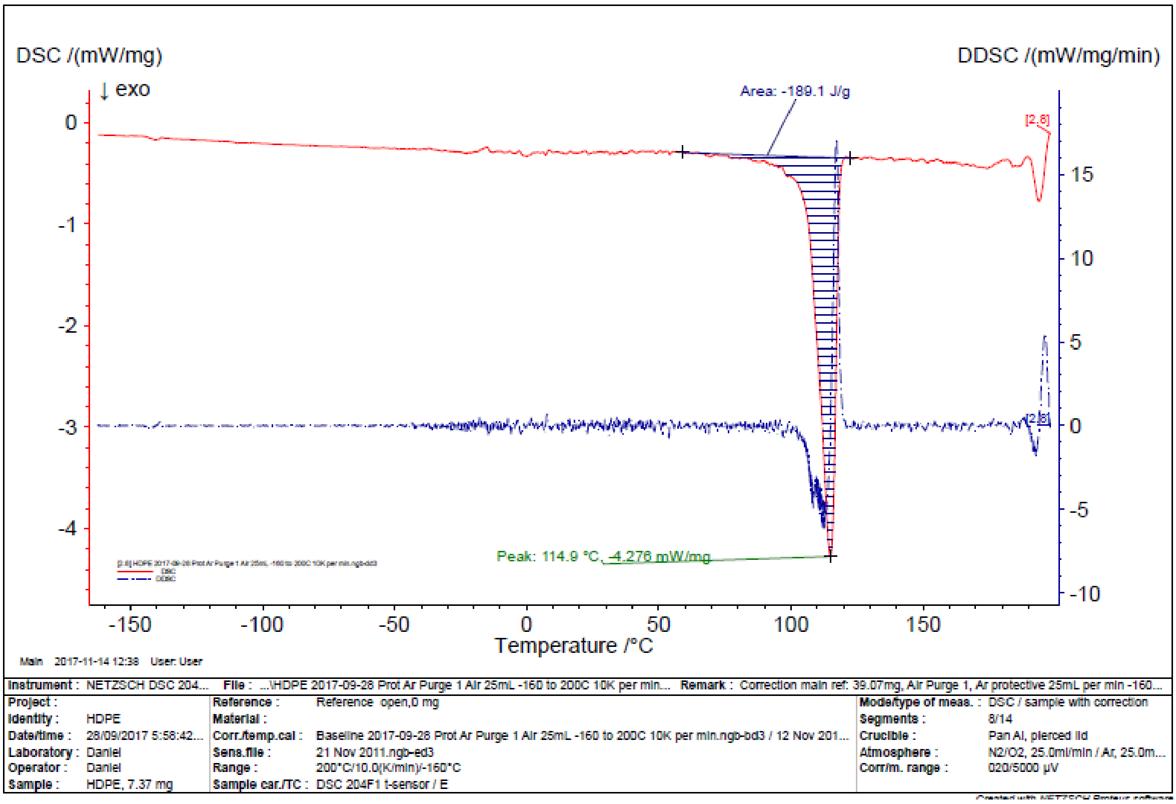

Figure 9.19: DSC and DDSC Thermographs of HDPE for Second Cooling Cycle

The transition temperatures obtained from DSC analysis on HDPE at different heating and cooling cycles are presented in Table 9.10. To estimate the amount of crystallized fraction of HDPE, the value of ΔH_o of 287.3 J/g has been found from literature for HDPE (Banat and Fares, 2015; Mirabella and Bafna, 2002).

Table 9.10: Obtained Transition Temperatures from DSC Thermographs for HDPE

Cycle	T_g (°C)	T_{onset} (°C)	T_{end} (°C)	T_m (°C)	T_c (°C)	ΔH_m (J/g)	CF (%)
1st Heating	-108.5	-127	-99	138.4	-	205.9	71.67
2nd Heating	-104.2	-119	-96	134.2	-	221.1	76.96
3rd Heating	-97.8	-113	-91	133.9	-	231.7	80.65
1st Cooling	-	-	-	-	115.0	-	-
2nd Cooling	-	-	-	-	114.9	-	-
3rd Cooling	-	-	-	-	114.6	-	-

It should be noted that fully crystalline polymers do not exhibit glass transition temperature and their structure will not change until the melting point. However, according to DSC thermographs, HDPE is considered as semi-crystalline polymer.

In addition, as HDPE has higher molecular weight than bitumen, the melting and crystallization temperature of HDPE is higher to provide more energy for reaching to these points. In addition, the melting and crystallization temperature of HDPE are close to each other which can be confirmed from literature survey (e.g. Ashraf, 2014).

9.7.1.4. DSC Analysis of Blend

The bitumen demonstrates considerably different rheological properties below and above T_g so that bitumen behaves like a glassy material below T_g whereas it act like a rubbery material above T_g. Accordingly, the determination of glass transition temperature of bitumen can provide information about its performance at different temperatures depending on its final application. Therefore, in this research, the thermal properties of blend of bitumen, HDPE, and rubber were investigated by DSC and the corresponding thermograph is shown in Figure 9.20.

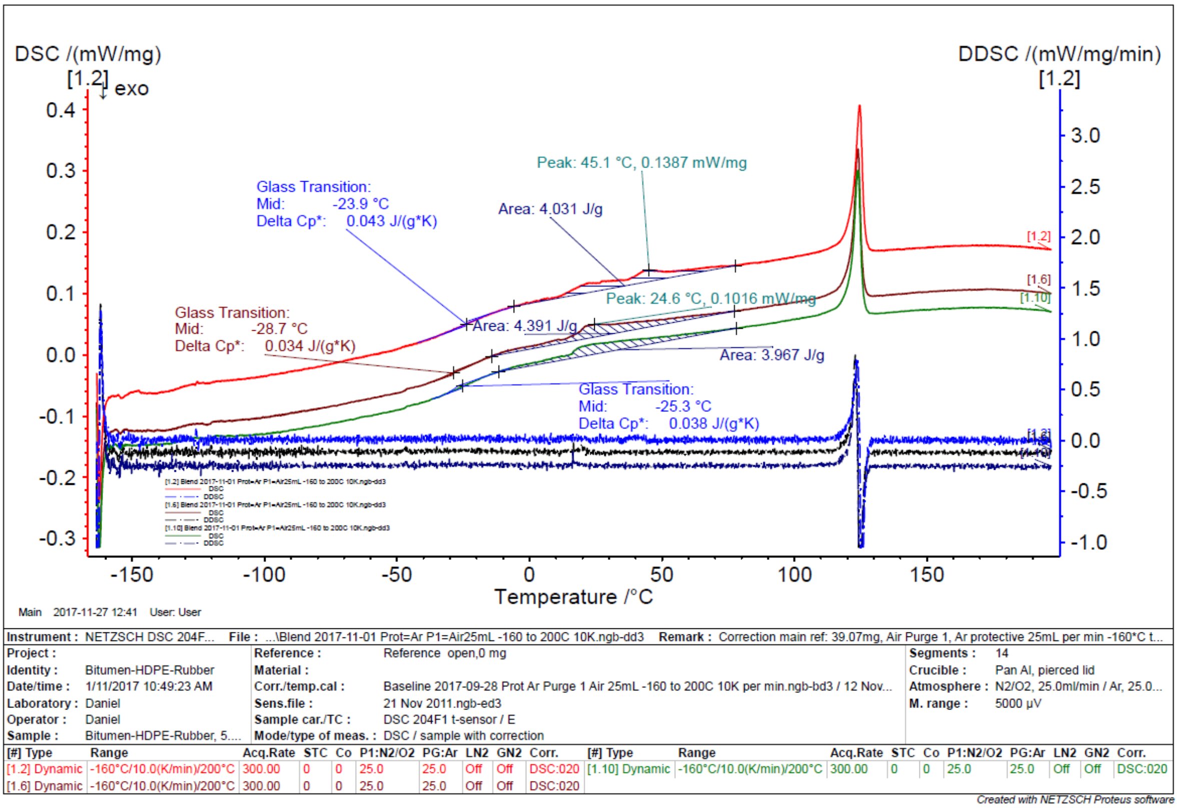

Figure 9.20: DSC and DDSC Thermographs of Blend of Polymers for Heating Cycles

In addition, the DSC curves of second heat cycle corresponding to neat bitumen together with DSC curves for the blends of bitumen, HDPE and rubber are shown in Figure 9.21.

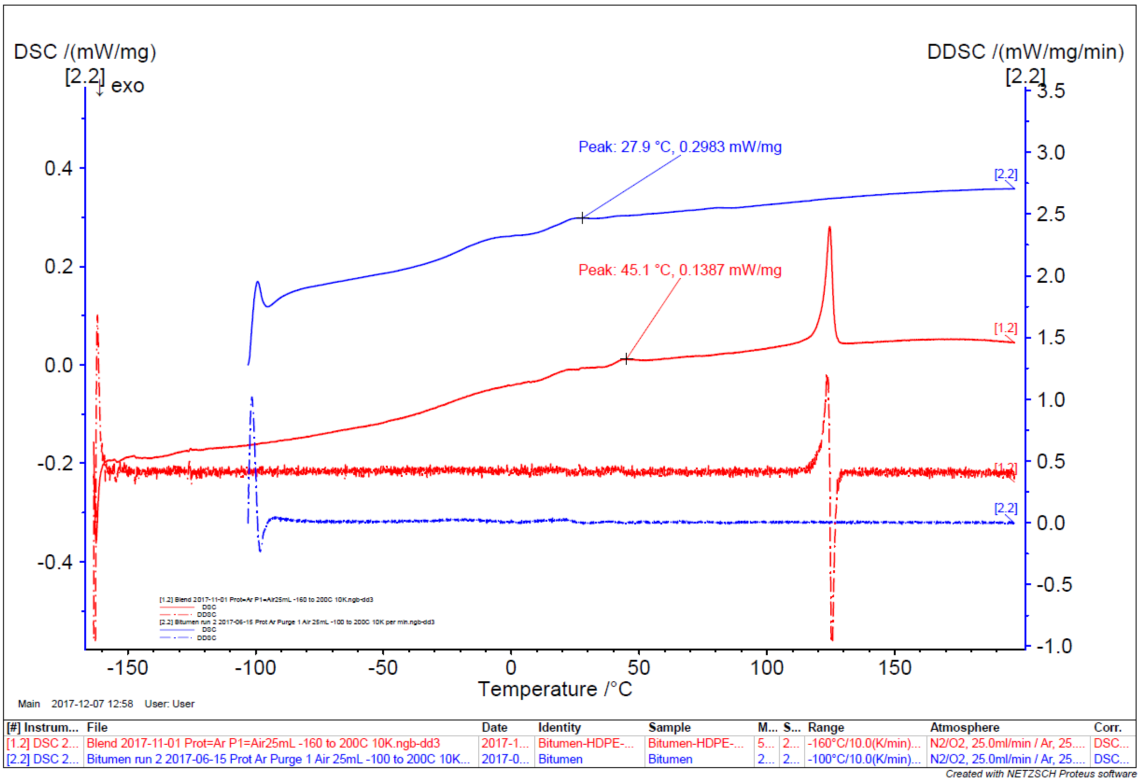

Figure 9.21: DSC Curves for Bitumen and Blend

As can be seen, the T_m values obtained for the blend containing 2% HDPE and 8% rubber is higher than the corresponding value obtained for neat bitumen. Therefore, the addition of HDPE and rubber into the blends provokes an increase in the T_m value of bitumen. The changes in transition temperatures of blend can be attributed to a certain level of miscibility between the additives (i.e. HDPE and rubber) and bitumen.

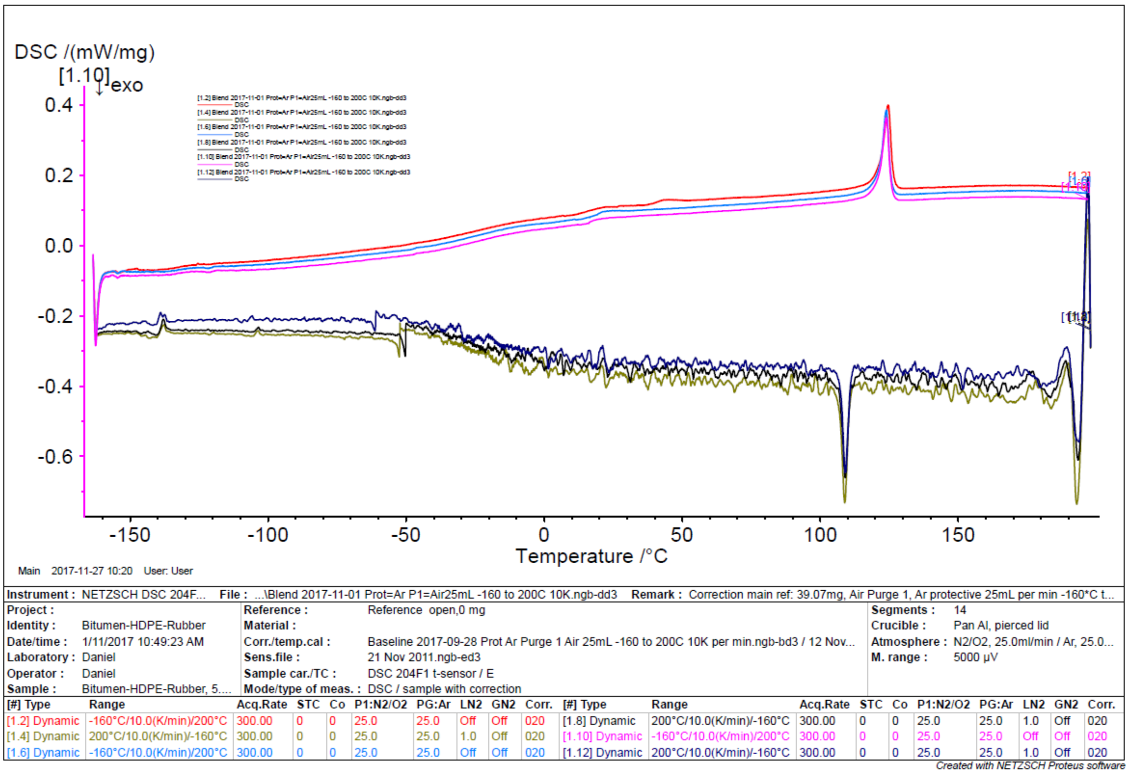

Figure 9.22: Heating and Cooling Cycles of Blend

The transition temperatures obtained from DSC analysis on blend of bitumen, HDPE and rubber at different heating and cooling cycles (Figure 9.22) are presented in Table 9.11.

Table 9.11: Obtained Transition Temperatures from DSC Thermographs for Blend of Polymers

Cycle	T_g (°C)	T_{onset} (°C)	T_{end} (°C)	T_m (°C)	T_c (°C)	ΔH_m (J/g)	CF (%)
1st Heating	-23.9	-42.1	-7.2	45.1	-	4.031	70.15
2nd Heating	-28.7	-38.9	-11.4	24.6	-	4.391	76.42
3rd Heating	-25.3	-36.7	-12.3	-	-	3.967	69.03
1st Cooling	-	-	-	-	108.7	-	
2nd Cooling	-	-	-	-	109.2	-	
3rd Cooling	-	-	-	-	108.3	-	

It should be noted that in case of blends, the percentage of crystallized fraction (CF) was determined from the following equation in which w is the weight fraction of HDPE into the blend.

$$CF = \frac{(\Delta H_{obs} \times 100)}{\Delta H_o \times w} \quad (9.2)$$

9.7.2. Thermal Analysis by TGA

The thermal stability of polymers is one of the important characteristics that should be considered for fitting their performance to the proper final application. Thermogravimetrical analysis is an adequate technique for the evaluation of the thermal stability of materials. Hence, in this research, the thermal stability of three polymers and their blend was studied by TGA in air and the main features of the curves including the onset temperatures of the mass loss effects (T_O) and the peak temperatures (T_p) were calculated from the TGA and DTG curves, respectively, as discussed in the following sections.

9.7.2.1. TGA Analysis of Bitumen

For bitumen, the thermogravimetric experiment results for 5mg samples under air atmosphere over the temperature range of 30°C to 590°C using a total purge gas flow of 100 mL/min and a heating rate of 10 °C.min-1 show that the onset temperature of the main mass loss effect (T_O) is 370°C, as shown in Figure 9.23.

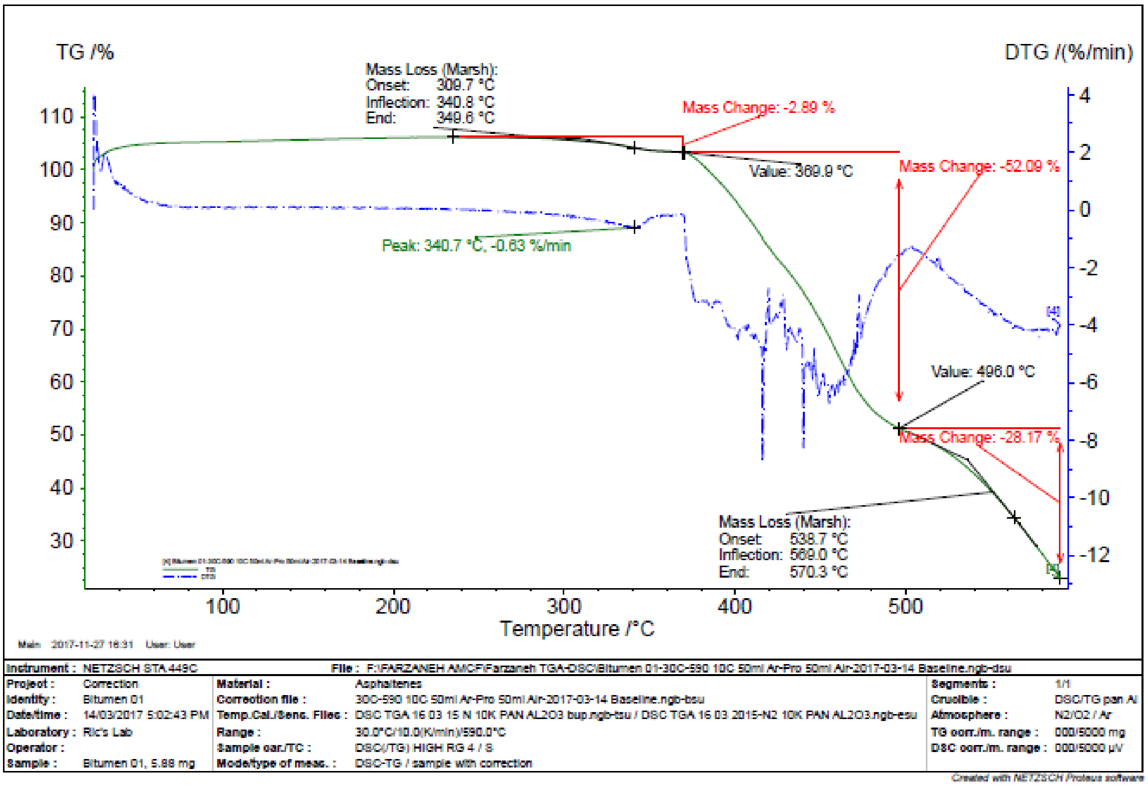

Figure 9.23: TGA, DTG and D2TG thermographs of Neat Bitumen

Referring to Jimenez-Mateos et al. (1996), the decomposition of bitumen occurs in at least three steps, considering three temperature ranges, as shown in Figure 9.23. In the temperature range of T<350°C, the decomposition of saturates and aromatics results in mass loss of bitumen. Over the temperature range of 350°C< T <500°C, resins and aromatics as well as asphaltenes are the main decomposed fractions, and at high temperatures of T >500 °C, the substantial mass change in bitumen occurs as a result of decomposition of asphaltenes. However, resins and aromatics are still decomposed in this range of temperature.

Table 9.12: Thermogravimetric Characteristics in Air for Bitumen

Effects	T_0 (°C)	T_p (°C)	Δw (wt%)
1st effect	309.7	340.7	2.89
2nd effect	369.9	-	52.09
3rd effect	538.7	569.0	28.17

Table 9.12 presents the thermogravimetric characteristics of bitumen considering these three temperature ranges.

9.7.2.2. TGA Analysis of Rubber

When selecting materials for modifying the binder, it is important that the modifier begins to degrade at a temperature above the bitumen modification temperature or the asphalt production temperature. Otherwise, it will lose its initial properties by the time the modification process is finished. In this research, TGA is used for determination of the degradation temperature of the waste materials which are used for modifying the binder (i.e. rubber and HDPE). Figure 9.24 shows the result of TGA analysis on rubber.

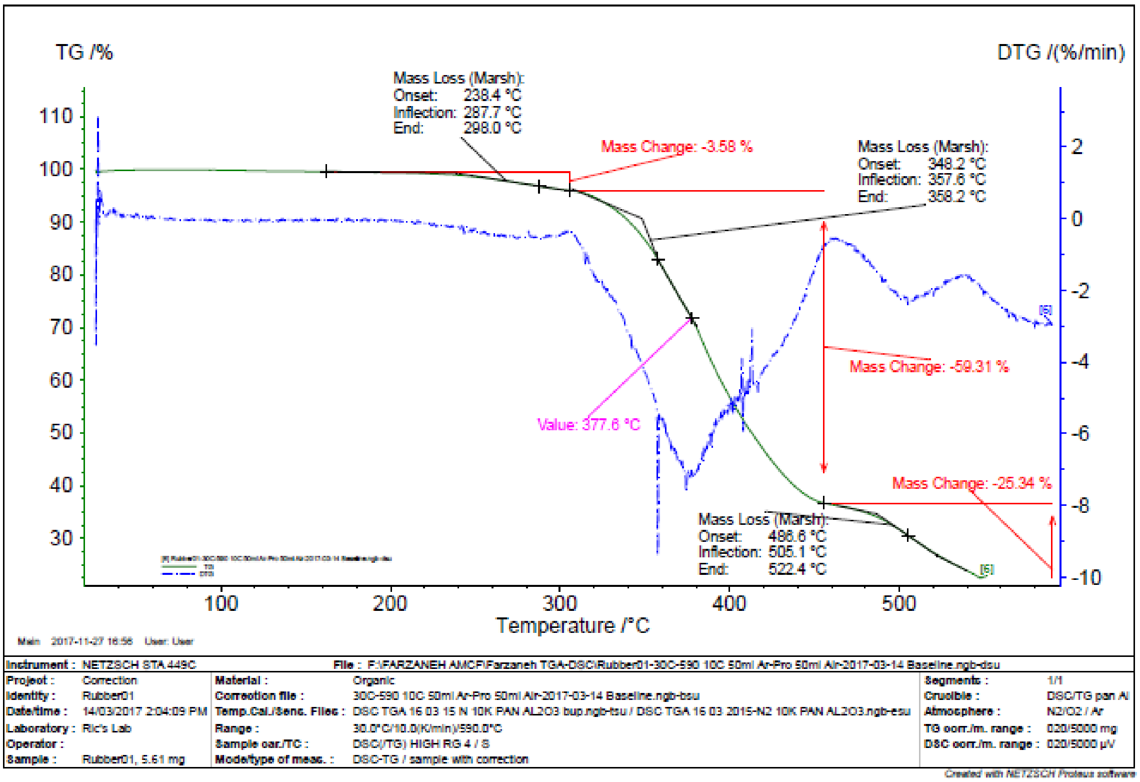

Figure 9.24: TGA, DTG and D2TG thermographs of Rubber

As can be seen, the onset temperature of degradation for rubber is 238°C and the peak temperature of mass loss is 378°C which can be observed as a peak in the first-derivative curve.

9.7.2.3. TGA Analysis of HDPE

Similar to other polymers, TGA of the HDPE samples was done on approximately 5 mg samples over the range of room temperature to 590°C under air with 100 mL/min flow rate at a heating rate of 10 °C.min-1. The onset degradation temperature and peak temperature are determined from the derivative TGA curves for HDPE, as shown in Figure 9.25.

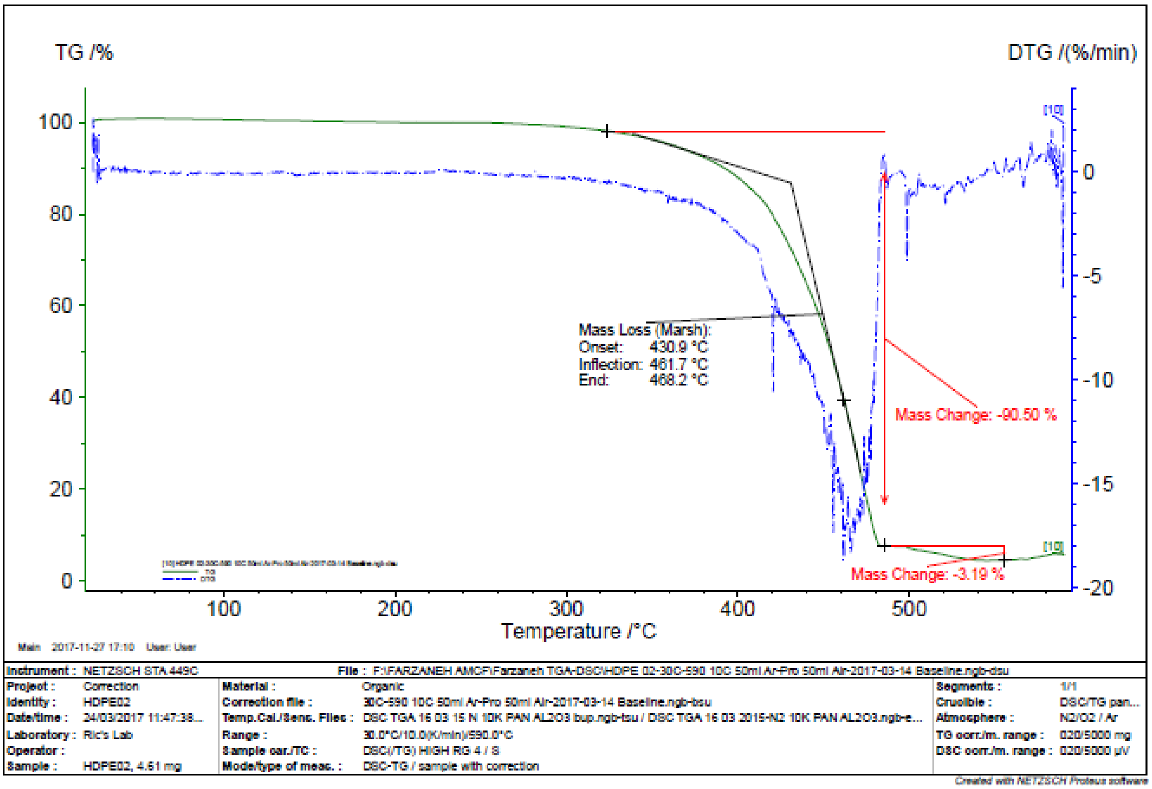

Figure 9.25: TGA, DTG and D2TG thermographs of HDPE

In this figure, it can be observed that HDPE remains thermally stable up to a temperature of 430°C. After this temperature, HDPE starts to degrade dramatically followed by a substantial step with maximum mass loss rates placing at 462°C in the DTG curve. This degradation involves a mass loss of about 91% in HDPE due to the thermal cracking of hydrocarbon chains and the production of oxygenated hydrocarbons including CO, CO_2, and H_2O (Purohit and Orzel, 1998). The degradation ends approximately around 490°C.

9.7.3. Structural Characterization by FTIR

The influence of adding rubber and HDPE to bitumen was investigated by FTIR spectrometry. The FTIR spectra for all individual polymers and their blend are illustrated in Figure 9.26.

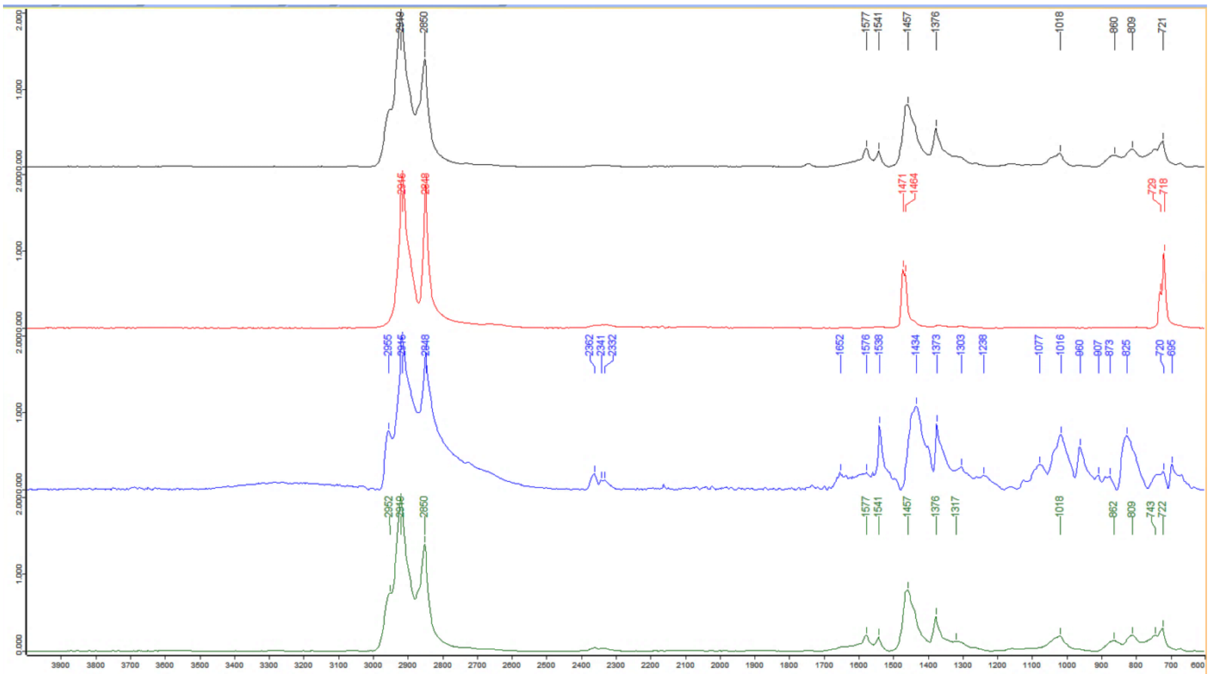

Figure 9.26: IR Spectra for Bitumen, HDPE, Rubber and Their Blend

The main characteristic bands of bitumen are also presented in Table 9.13, which are in agreement with results of available literature (Aguiar-Moya et al., 2013).

Table 9.13: FTIR Characteristics of Bitumen

Wave Lengths (cm^{-1})	Type	Description
2850 to 2950	CH2 bands	associated to saturated hydrocarbons
1541 to 1577	C=C bands	associated to aromatic hydrocarbons
1457	C=C bands	associated to aromatic hydrocarbons
1317 to 1376	C-N bands	associated to aromatic amines
1018	R-O-Ar	correspond to alkyl-aryl-ethers

The spectrum bands do not show substantial increase due to the modification by this certain amount of additives. It can be concluded that addition of rubber and HDPE at the studied percentage and with considered size of particles do not make significant changes in the content of aromatic hydrocarbons and aromatic amines.

9.7.4. Microstructure Analysis by SEM

The analysis of the microstructure of polymers was performed using Scanning Electron Microscopy (SEM). The results of the microscopy as well as the Energy Dispersive Spectroscopy (EDS) analysis on the individual polymers are given in Figures 9.27 to 9.28.

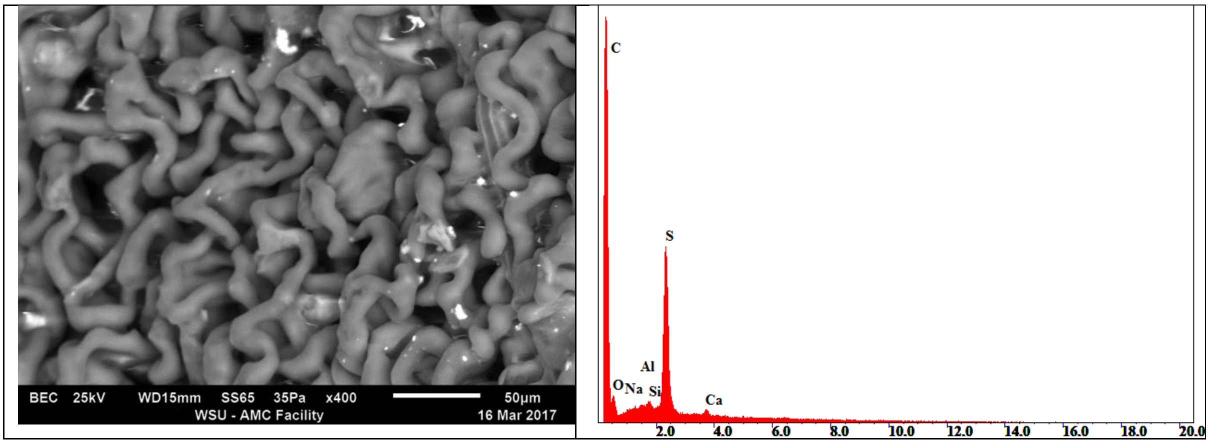

Figure 9.27: EDS Analysis and SEM Image of Bitumen at 400 Magnification

As can be observed in Figure 9.27, the surface of bitumen appears as networks of highly-entangled strings.

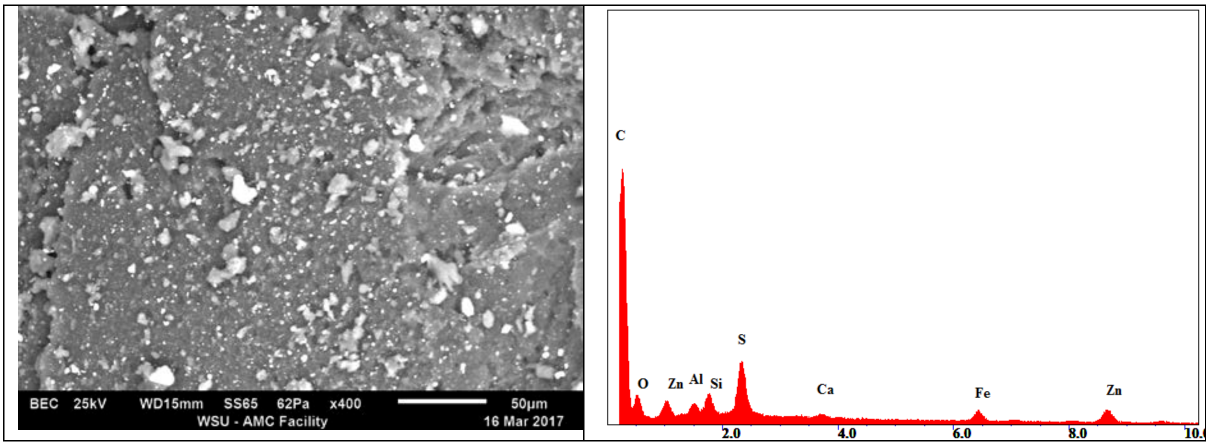

Figure 9.28: EDS Analysis and SEM Image of Rubber at 400 Magnification

Figure 9.28 shows the coarse texture of rubber. The irregular shape and rough texture of rubber can be attributed to its processing method which is the ambient procedure.

Furthermore, the microstructure of HDPE is shown in Figure 9.29. It should be noted that HDPE has a higher viscosity compared to bitumen. The materials with high viscosity do not separate easily and, therefore, they present in the form of dispersed phase, as can be clearly observed in Figure 9.29.

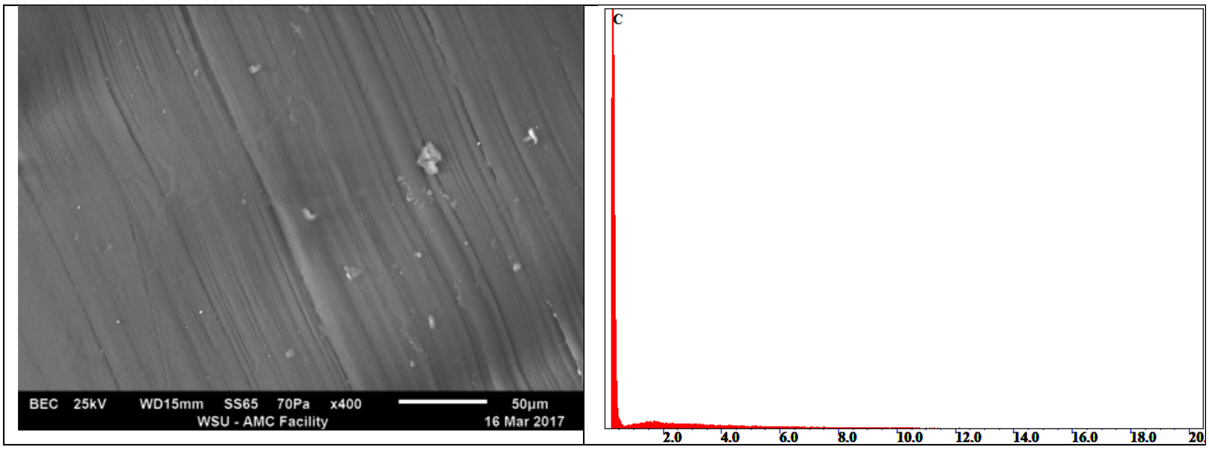

Figure 9.29: EDS Analysis and SEM Image of HDPE at 400 Magnification

In addition, the SEM image of the blend of polymers and the result of EDS analysis for different areas in the blend are shown in Figure 9.30.

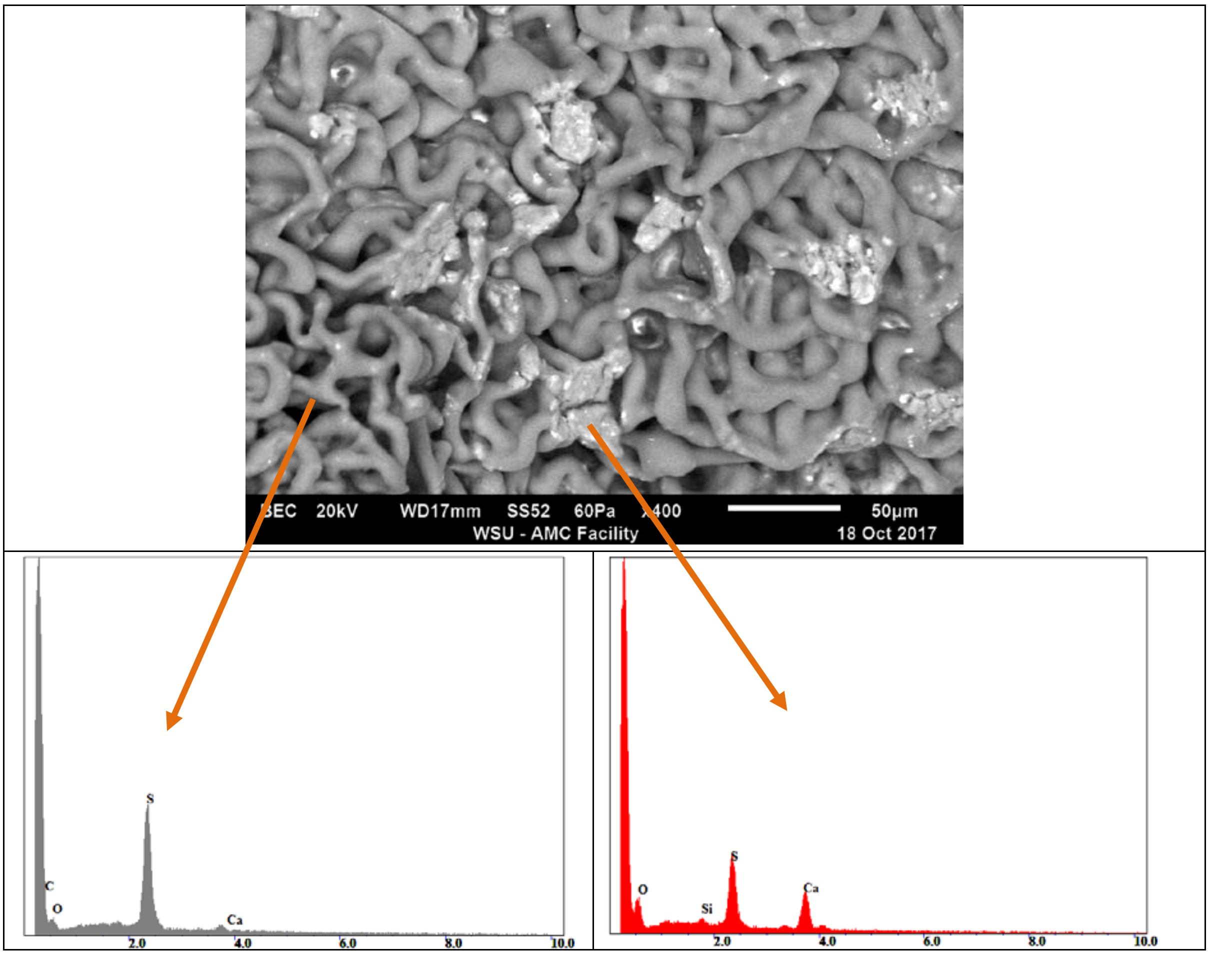

Figure 9.30: EDS Analysis and SEM Image of Blend at 400 Magnification

The EDS results show the differences in the way that the modifiers are incorporated into the bitumen. Note that in this case, there are particles that are not properly incorporated into the bitumen. This is an indicator that the material should be incorporated to the bitumen in smaller particles than the ones used.

9.8. Summary and Conclusions

Today, pure bitumen doesn't provide suitable performance for pavements due to the current traffic. Therefore, attempts have been made to maximize the effectiveness of asphalt binders selected for construction projects based on a standard asphalt binder classification system. Binder modification technique is employed as an alternative to minimize the pavement failures due to poor performance of asphalt binders, as well as to increase the PG grade of the asphalt binder (Somayaji, 2001). Based on these research studies, the utilization of polymers as modifier improves some of the bitumen's properties such as elasticity, cohesion, and temperature susceptibility, which subsequently lead to the improvement of asphalt mixture performance.

For these reasons, and as a quite effective way of disposal of non-biodegradable wastes, plastic wastes and rubbers can be reasonable potential materials for consideration as binder modifier.

In modification of bitumen with additives, having knowledge about the effects of modifiers on thermal stability is of high importance resulting in manufacturing more thermally stable binders. Accordingly, in this research the thermal behaviour of modifiers, bitumen and their composite were studied using TGA, DSC, FTIR, and SEM facilities. Table 9.14 presents the thermal parameters, T_g and T_m, enthalpy of fusion, ΔH_m and the percentage of crystallinity, CF (%) obtained from the first cycle of DSC analysis on neat bitumen, HDPE, rubber and their blends.

Table 9.14: Thermal Parameters Obtained from DSC Analysis on Individual Polymers and Their Blend

Material	T_g (°C)	T_{onset} (°C)	T_m (°C)	T_c (°C)	ΔH_m (J/g)	CF (%)
Neat Bitumen	-24.6	-37	27.9	-	6.703	3.72
HDPE	-108.5	-127	138.4	115.0	205.9	71.67
Rubber	-55.9	-60.5	-	-	-	-
Bitumen+HDPE+Rubber	-23.9	-42.1	45.1	108.7	4.031	70.15

As can be observed in Table 9.14, the glass transition temperature as well as the crystallized fraction of samples can easily be determined from the DSC curves. This information can be useful in understanding the characteristics and the composition of polymers. In addition, many researchers have proposed different equations for estimation of the glass transition temperature of the blends of materials based on the composition and the glass transition of the materials in the blend. All these equations are basically representing the relation between the glass transition temperature of a mixture and those of its components using the following mathematical form but with minor variations:

$$\boldsymbol{T_{gm}^{eq} = \frac{\varphi_1 T_{g1} + k\varphi_2 T_{g2}}{\varphi_1 + k\varphi_2}} \qquad (9.3)$$

where T_{gm}^{eq} is the glass transition temperature of the blend, T_{g1} and T_{g2} are the glass transition temperature of the materials in the blend, φ_1 and φ_2 are the weight fractions of the materials in the blend, and k is a parameter depending on the considered physical model in the equation. Considering k equals 1, the glass transition temperature of the blend can be estimated in the simplest way. The glass transition obtained from this expression for the blend of bitumen with 2% HDPE and 8% rubber is -28.8°C. The calculated Tg value is higher than the measured one (about 4.9°C) which could be due to the simplification by the assumption of k. Thus, using these equations, it may be possible to achieve the formulation for the desired modified binder considering the composition of components and the results of DSC analysis on each component.

Furthermore, in this research, TGA Analysis was used in determining the degradation temperature of the waste materials. A modifier that begins to degrade at a temperature below the bitumen modification temperature or the asphalt production temperature is not adequate since it will have lost its initial properties by the time the modification process is finished. In the case of analysed waste materials, all degrade at temperatures above 200° C and, therefore, should be adequate for bitumen modification. The main features of TGA curves for individual polymers and their blends are summarized in Table 9.15.

Table 9.15: Thermal Parameters Obtained from TGA Analysis on Individual Polymers and Their Blend

Material	**T_0 (°C)**			**T_p (°C)**			**Δw (wt%)**		
	1st peak	**2nd peak**	**3rd peak**	**1st peak**	**2nd peak**	**3rd peak**	**1st peak**	**2nd peak**	**3rd peak**
Neat Bitumen	309.7	369.9	538.7	340.7	-	569.0	2.89	52.09	28.17
HDPE	317.1	-	-	-	461.7	-	-	90.50	-
Rubber	238.4	348.2	486.6	287.7	377.6	505.1	3.58	59.31	25.34

From these results, it seems that HDPE, followed by bitumen, have higher thermal stability than crumb rubber. Unfortunately, the TGA curve for blend did not give clear results which could be due to improper incorporation of polymers in bitumen. In this regard, the size of the modifier particles is of extreme importance. This was observed in the present study in the case of the crumb rubber materials. It is suggested to use smaller particle sizes of the modifier to ensure that the material incorporates more homogeneously into the bitumen.

Chapter 10

Structural Design of Pavement

10.1. Introduction

The main goal of pavement design is the selection of the most economical composition and thickness in order to provide an adequate level of service for the expected traffic without significant failures. To this point, it is required to collect sufficient knowledge about the pavement materials, the traffic loading, the local environment and other influencing parameters to achieve this goal.

This research work investigates the effect of the utilization of certain waste materials on the performance of asphalt mixture. Throughout this study, emphasis is placed on the parameters that are used to evaluate mixtures for their resistance to permanent deformation. Permanent deformation is one of the most important failures in asphalt pavements. The material selection and the mixture design are two important factors that play an important role in minimizing the rutting. To meet this objective, a laboratory investigation at different levels was conducted on several asphalt mixtures with different materials, varying composition, and different preparation method. Based on the results of this experimental work, five samples were selected as the most acceptable mixtures in terms of volumetric and mechanical properties. An essential part of the asphalt mixture design process is the assessment of the asphalt mixture performance in the pavement structure. This will be achieved through the structural design of pavement and evaluation of the performance of different layers of flexible pavement. In light of this, the purpose of this chapter is to illustrate the result of the structural design of a flexible pavement for a section of the M2 motorway in Sydney using different asphalt mixtures proposed in this research and subsequently compare them with the corresponding values in a pavement with conventional asphalt mixture.

It can be noted that all analyses have been carried out using the weigh-in-motion (WIM) survey data collected throughout Australia in 2010 and employing the software CIRCLY 6.0 to facilitate the design of flexible pavements.

10.2. Permanent Deformation Mechanisms in Flexible Pavements

Permanent deformation in the form of rutting accounts for a significant portion of maintenance and associated costs in highways and secondary roads. Rutting represents a continuous accumulation of irrecoverable deformations in all or some of the pavement layers from each traffic load application as a result of lateral distortion and densification, and is observed as a longitudinal depression along the wheel path. Rutting progression can result in cracking and finally the complete failure of pavement.

Although the rutting can be the total sum of accumulated irrecoverable deformations in pavement layers, today the accumulation of permanent deformation in the asphalt surface course and particularly in the upper part of asphalt surface course (the top 75 to 100 mm of the asphalt surface course) is mostly taken into account as the main reason of rutting (Garba, 2002). Therefore, it is required to pay further attention to the materials selection for asphalt mixture and mix design. It has been mentioned that the shear resistance of asphalt mixture to repeated loads plays a significant role in the prevention of rutting. The following equation can be used to determine the shear strength of asphalt mixture:

$$\boldsymbol{\tau = c + \sigma \tan \varphi} \qquad (\mathbf{10.1})$$

where τ is shear strength, σ is normal stress, c is cohesion and φ is the angle of internal friction. Based on Equation (10.1), the shear strength depends on the cohesion and the angle of internal friction. In asphalt mixtures, the cohesion is obtained from the viscosity of binder and the proportion of filler, and is highly dependent on temperature so that at elevated temperatures, the cohesion would be less because the bitumen becomes less viscous. In contrast, the angle of internal friction is independent of temperature, and it is highly affected by the aggregate interlocking. Thus, the aggregates with rough texture, more angularity and good gradation provide higher value for friction angle. To this point, it can be concluded that the mechanical interlock of the aggregates play an important role in shear resistance of asphalt mixtures. However, the bitumen content can also affect the shear strength of asphalt mixtures as it changes the aggregates mechanical interlock because the aggregate particles would be further apart by the increase in bitumen content (Mattos et al., 2016).

10.3. Flexible Pavement Design Methods

In the past, the empirical methods were mainly used for designing the pavements. These methods were not able to consider the pavement distresses fully and clearly. In recent years, the more logical and practicable method referred to as mechanistic–empirical method have been developed. This method generally considers two procedures for limiting the permanent deformation. The first procedure limits the vertical strain (compressive strain) on top of the subgrade, whereas the other procedure limits the total sum of accumulated irrecoverable deformation on the pavement surface resulting from the irrecoverable deformation of each individual layer in pavement structure. However, the increased tyre pressure result in occurrence of the rutting mostly in the asphalt surface course compared to the subgrade, and hence makes the first procedure as an improper approach for investigation of permanent deformation in pavement structural design.

The second procedure is based on the consideration of the permanent deformation behaviour of each layer. The characterization materials used in the pavement layers as well as the amount of stresses induced to the selected points of each layer are involved in this procedure. The total rutting will be obtained by summing up the irrecoverable deformation of each individual layer.

The Australian guidelines provide two distinct design procedures for the structural design of flexible pavements called Empirical Design and Mechanistic Design. Empirical design is applicable in designing the granular pavements covered with sprayed seal or asphalt layer of less than 40 mm thickness (thin bituminous surfacing) in which only one type of damage representing the overall pavement deterioration in terms of the level of roughness and permanent deformation is considered. However, pavements containing more than one bound layers (pavements with asphalt course or cemented layer), up to three types of damage are considered based on the mechanistic design, including the rutting on the surface, asphalt fatigue damage, , and the cemented material fatigue damage. The design procedure considered in this research is the mechanistic design which is applicable to the pavements with moderate to heavy traffic loading and provides the minimum requirements for the layer thicknesses.

10.4. Principles of Pavement Model Based on Mechanistic Procedure

The structural design has been carried out on a multi-layered pavement model subject to normal traffic loading in which the critical responses are assessed for asphalt, cemented material and subgrade. Figure 10.1 shows the critical location of the strains and the idealized loading situation. As illustrated in this figure, in this model, it is assumed that the pavement is constructed of three layers including an asphalt layer, unbound granular base layer, and cemented subbase layer placed on subgrade layer. All these layers (except unbound granular base layer) contribute to the prediction of permanent deformation in the pavement model. Referring to Australian Guide Part 2, the granular layer is not included in the mechanistic procedure due to the lack of suitable and reliable model for prediction of the rutting development in a granular material under traffic. Moreover, it is supposed that all pavement materials are homogeneous and elastic. The asphalt and cemented material are assumed to be isotropic, while the granular materials and subgrade are considered to be anisotropic.

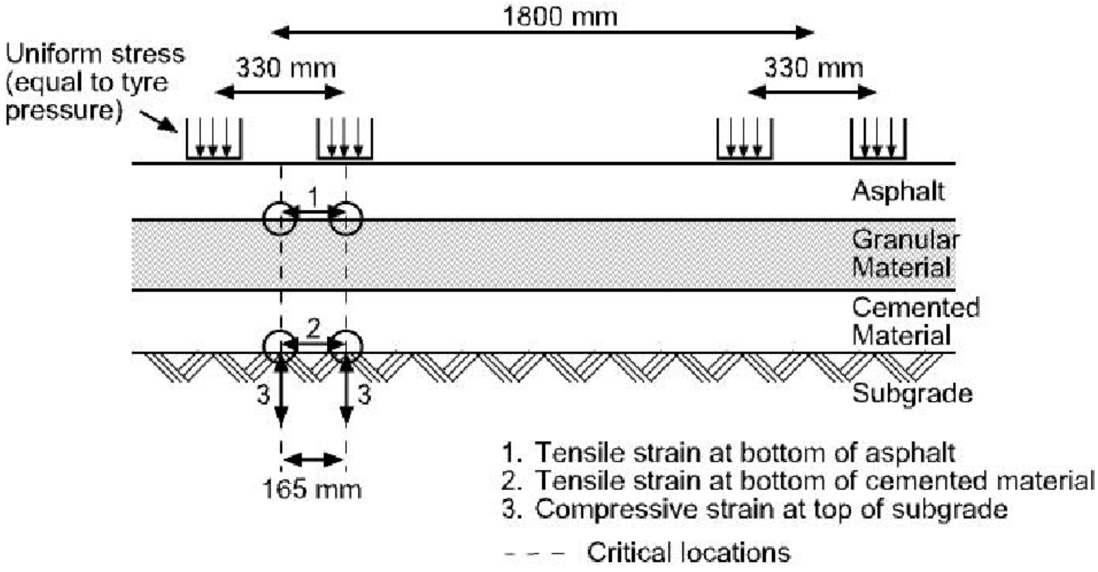

Figure 10.1: Schematic Diagram of Pavement Model Based on Mechanistic Procedure (Austroads, 2012)

In this model, the single axle with dual tyres (SADT) applying a load of 80 kN is considered as the standard axle loading. To simplify the design, the standard axle loading has been considered as four circular areas with uniform vertical stress distribution of 750 kPa, which are separated by centre to centre distance of 330 mm, 1470 mm, and 330 mm (Figure 10.1). Thus, assuming an equal area for circular areas, a 20 kN vertical load is applied to each circular area with the radius of about 92.1 mm for highway traffic.

10.5. Selection of Design Input Parameters and Configurations for the Pavement Model

The fundamental philosophy underlying this research is evaluating the properties of asphalt mixtures incorporating waste materials. Based on the results of the laboratory investigation on over 100 samples with different composition and prepared by different mixing methods, it was found that five samples have the most acceptable and similar properties to the conventional asphalt mixtures in terms of volumetric and mechanical characteristics. Therefore, this chapter aimed at the structural analysis of flexible pavements composed of these new asphalt mixtures as the asphalt surface layer for estimation of the allowable loading of pavement and comparison of pavements of different composition. The mechanistic structural design process of the pavement mainly includes the selection of pavement configuration, determination of input parameters, analysis of the response of pavement to the applied loads, and finally the allowable traffic loading for pavement distresses.

In addition to suitable geometry of the pavement, another important input parameter affecting all aspects of the pavement is the design traffic, as a road pavement should be strong enough to withstand the load from the vehicles as well as the cumulative effects of the load from the passage of all vehicles. Vehicular traffic consists of different vehicle types. Based on the Austroads vehicle classification system, vehicles are categorized into two main categories of light vehicles and heavy vehicles consisting of 12 classes. As the light vehicles have a small effect on the deterioration of pavements, the heavy vehicles will only be considered in pavement design. Referring to Austroads (2012), the heavy vehicle axle group (HVAG) types are classified into six main groups of single axle with single tyres (SAST), tandem axle with single tyres (TAST), single axle with dual tyres (SADT) , tandem axle with dual tyres (TADT), triaxle with dual tyres (TRDT) and Quad-axle with dual tyres (QADT).

The pavement design task is to select the suitable pavement configurations in order to provide acceptable service for the cumulative traffic expected over the pavement design period. The number of axle groups, the proportion of each axle group type, and frequency distribution of the axle group loads are taken into account when determining the cumulative loading on a pavement.

Table 10.1: Characteristics of traffic at selected WIM site (Austroads, 2012)

<table>
<tr><th colspan="2">Characteristic</th><th>Value</th></tr>
<tr><td colspan="2">Lanes in selected direction</td><td>2</td></tr>
<tr><td colspan="2">Lane surveyed</td><td>2</td></tr>
<tr><td colspan="2">Lane Average daily traffic (ADT)</td><td>15,356</td></tr>
<tr><td colspan="2">Percentage of Heavy Vehicles (HV)</td><td>4.9</td></tr>
<tr><td colspan="2">Average number of axle groups per HV, N_{HVAG}</td><td>2.56</td></tr>
<tr><td rowspan="6">Percentage Distribution of Axle Group Types</td><td>SAST</td><td>37.6</td></tr>
<tr><td>SADT</td><td>23.1</td></tr>
<tr><td>TAST</td><td>1.4</td></tr>
<tr><td>TADT</td><td>25.5</td></tr>
<tr><td>TRDT</td><td>12.4</td></tr>
<tr><td>QADT</td><td>0.0</td></tr>
<tr><td colspan="2">Equivalent Standard Axle (ESA) per HVAG</td><td>0.704</td></tr>
<tr><td rowspan="3">Standard Axle Repetition (SAR) per ESA</td><td>Asphalt Fatigue (SAR5/ESA)</td><td>1.14</td></tr>
<tr><td>Rutting and Shape Loss (SAR7/ESA)</td><td>1.65</td></tr>
<tr><td>Cemented Material Fatigue (SAR12/ESA)</td><td>7.22</td></tr>
</table>

In this research, a section of M2 Motorway in Sydney near North Rocks (opposite the Barclay Road) heading East is considered to be designed with the proposed asphalt mixtures. The characteristics of traffic at selected site were obtained from available WIM data, as given in Table 10.1. Australian Guideline Part 2 (2012) provides the procedure for estimations of design traffic using information in Table 10.1 along with other assumptions as presented in Table 10.2.

Table 10.2: Parameter Identification for Determination of Design Traffic in Pavement Model

Parameter	Value	Description
Design Period	20 years	For flexible pavement
Direction Factor (DF)	0.5	-
Lane Distribution Factor (LDF)	1	Urban location with two lanes each direction
Annual growth rate	4%	-

Australian Guideline Part 2 (2012) provides the procedure for estimations of design traffic. The Equation (10.2) is used to determine the Design Traffic (N_{DT}) in terms of cumulative heavy axle groups (HVAG) in the design lane over the design period:

$$\boldsymbol{N_{DT} = 365 \times AADT \times DF \times \% \frac{HV}{100} \times LDF \times CGF \times N_{HVAG}} \qquad (\mathbf{10.2})$$

where AADT is Annual Average Daily Traffic in the first year in vehicles per day, DF is the proportion of the two-way AADT travelling in the direction of the design lane, HV is the average percentage of heavy vehicles, LDF is the proportion of heavy vehicles in design lane, CGF is the cumulative growth factor, and N_{HVAG} is the average number of axle groups per heavy vehicle.

In flexible pavement design, the pavement configuration is assessed for its ability to withstand the design traffic. For this purpose, the degree of damage resulting from the Standard Axle to the pavement configuration is determined. The Standard Axle is a SADT which causes 80 kN axle load to the pavement. Referring to Austroads (2012), the design traffic is formally stated as the total number of Standard Axle Repetition (SAR) during the design period leading to the same degree of damage as the cumulative traffic. Therefore, it is required to calculate the design number of Equivalent Standard Axles of traffic loading (DESA) and the design number of Standard Axle Repetitions (DSARm) for each relevant damage type using Equations (10.3) and (10.4).

$$\boldsymbol{DESA = \frac{ESA}{HVAG} \times N_{DT}} \qquad (\mathbf{10.3})$$

$$\boldsymbol{DSARm = \frac{SARm}{ESA} \times DESA} \qquad (\mathbf{10.4})$$

where DESA is the design traffic loading in ESA, $\frac{SARm}{ESA}$ is the average number of Standard Axle Repetitions per Equivalent Standard Axle for load damage exponent of m, DSARm is the design number of Standard Axle Repetitions (SAR) for damage type, and m is the load damage exponent for the damage type.

Table 10.3: Result of Design Traffic Calculation for the Pavement Model

Parameter	Value
Cumulative Growth Factor (CGF)	29.8
Design Traffic (N_{DT}) in terms of Cumulative Number of HVAG	10.5×10^6 HVAG
Design Traffic (DESA) in terms of the Number of ESA	7.4×10^6 ESA
Design Traffic in terms of SAR for Asphalt Fatigue (DSAR5)	8.4×10^6 SAR5
Design Traffic in terms of SAR for Cemented Materials Fatigue (DSAR12)	5.3×10^7 SAR12
Design Traffic in terms of SAR for Subgrade Rutting (DSAR7)	1.2×10^7 SAR7

Based on this procedure and using all information presented thus far, the results of all calculations for design traffic for the section of M2 Motorway in Sydney near North Rocks are summarized in Table 10.3.

10.6. Structural Analysis by CIRCLY 6.0

The main aim of structural analysis is the determination of the critical strains or stresses resulting from the traffic loading in the pavement. There are many factors such as pavement configuration, subgrade support including California Bearing Ratio (CBR) and elastic parameters, the properties and quality of base and subbase layer, etc., which have significant effects on analysing the flexible pavement.

Table 10.4: Assumed Configuration for Trial Pavement in Model

Layer No.	Material Type	Thickness
1	Size 14 mm – Wearing Course	40
2	Size 20 mm - Intermediate	80
3	Size 20 mm - Base	75
4	Cemented Subbase	170
5	Subgrade	Semi-infinite

In this research, CIRCLY 6.0 is employed for the structural analysis of pavements with the asphalt properties obtained from the experimental results of this research. This software can be used for the design of flexible pavements and models the pavement as a series of layers considering the elastic properties of materials in each layer. The main assumptions made in the pavement model to the input data for CIRCLY software are summarized in Tables 10.4 and 10.5.

Table 10.5: Assumed Properties for Pavement Layers in Model

Parameter	Value	Description
Subgrade CBR	5	Silty-clay with excellent to good drainage
Vertical Modulus of Subgrade	50 MPa	Anisotropic
Degree of Anisotropy of Subgrade	2	-
Poisson's ratio of Subgrade	0.40	Cohesive materials
Modulus of Cemented Subbase (Pre-cracking)	2000 MPa	Isotropic
Poisson's ratio of Cemented Subbase (Pre-cracking)	0.20	-
Modulus of Cemented Subbase (Post-cracking)	500 MPa	Anisotropic
Poisson's ratio of Cemented Subbase (Post-cracking)	0.35	-
Degree of Anisotropy of Cemented Subbase (Post-cracking)	2	-
Vertical Modulus of Unbound Granular Base	210 MPa	Anisotropic
Poisson's ratio of Granular Base	0.35	-
Degree of Anisotropy of Granular Base	2	-
Modulus of Size 20 mm Asphalt (intermediate/base course)	5700/5500 MPa	Isotropic
Poisson's ratio of Size 20 mm Asphalt (intermediate/base course)	0.4	-

In addition, since the selected samples (B100-C, B75-C, B75-G10-C, B100-S, B75-S, and B75-G10-S) are considered as wearing course (size 14 mm asphalt) in this structural analysis, different pavements with the same base and subbase layer properties but with different wearing course are studied in this chapter. It should be mentioned that the properties of asphalt mixture considered in this model for wearing course of different pavements are the values obtained from the laboratory investigation performed in this research. Moreover the asphalt modulus has been correlated for the in-service temperature (WMAPT) of 28°C in the considered region, using the following equation (Equation 10.5), and as summarized in Table 10.6:

$$\frac{\boldsymbol{Modulus\ at\ WMAPT}}{\boldsymbol{Modulus\ at\ Test\ Temperature\ (T)}} = \boldsymbol{e^{-0.08(WMAPT-T)}} \qquad \textbf{(10.5)}$$

Table 10.6: Properties for Wearing Course of Different Considered Pavements

Parameter	Modulus (MPa)	Poisson's ratio	Binder Volume (%)
Pavement 1 (B100-C as Wearing Course)	4420	0.4	11
Pavement 2 (B75-C as Wearing Course)	4880	0.4	11
Pavement 3 (B75-G10-C as Wearing Course)	5220	0.4	10
Pavement 4 (B100-S as Wearing Course)	4870	0.4	11
Pavement 5 (B75-S as Wearing Course)	5580	0.4	10
Pavement 6 (B75-G10-S as Wearing Course)	4670	0.4	11

In addition, the pavement model discussed in this section assumes the following features:

1. There is no increase in load magnitudes for some or all axle groups during the design period.
2. All tyres referred to in this research are conventional tyres with the design tyre-pavement contact stress of 750 kPa.
3. Linear elastic model is used to calculate the response to load.

Furthermore, Equations (10.6) to (10.8) are used for determination of permanent deformation allowable loading, cemented material fatigue allowable loading and asphalt fatigue allowable loading, respectively.

$$\boldsymbol{N = \left(\frac{9300}{\mu\varepsilon}\right)^{7}} \qquad (\mathbf{10.6})$$

$$\boldsymbol{N = RF\left[\frac{113000/_{E^{0.804}} + 191}{\mu\varepsilon}\right]^{12}} \qquad (\mathbf{10.7})$$

$$\boldsymbol{N = RF\left[\frac{6918(0.856V_b + 1.08)}{E^{0.36}\mu\varepsilon}\right]^{5}} \qquad (\mathbf{10.8})$$

where is E is modulus of elasticity, RF is reliability factor, $\mu\varepsilon$ is strain, and V_b is volume of binder.

Reliability factor (RF) in above equations for asphalt and cemented materials are determined based on the desired and target project reliability considered in the model. The assumed values for these parameters are given in Table 10.7.

Table 10.7: Assumed Values for Reliability Parameters in Pavement Model

Parameter	Value
Desired Project Reliability	95%
Reliability factor for Cemented Materials Fatigue	1
Reliability factor for Asphalt Fatigue	1

Different pavement combinations with various types of wearing course were modelled using CIRCLY 6.0 and the results of mechanistic modelling for the specified pavements including the strain and cumulative damage factor (CDF) are presented in Table 10.8.

Table 10.8: Properties for Wearing Course of Different Considered Pavements

Parameter		Pre-Cracking Phase		Post-Cracking Phase	
		Critical Strain	Cumulative Damage Factor	Critical Strain	Cumulative Damage Factor
Pavement 1	B100-C Size 14 / Wearing Course	7.64E-06	8.44E-31	-	-
	Asphalt Size 20/ Intermediate Course	1.87E-05	6.11E-05	-	-
	Asphalt Size 20/ Base Course	4.66E-05	3.48E-03	-	-
	Cemented Material	1.01E-04	1.10E+00	-	-
	Subgrade	2.32E-04	7.33E-05	-	-
Pavement 2	B75-C Size 14 / Wearing Course	6.42E-06	8.44E-31	1.50E-05	8.44E-31
	Asphalt Size 20/ Intermediate Course	1.93E-05	7.12E-05	3.52E-05	1.42E-03
	Asphalt Size 20/ Base Course	4.67E-05	3.50E-03	1.04E-04	1.89E-01
	Cemented Material	1.00E-04	9.96E-01	n/a	n/a
	Subgrade	2.30E-04	6.84E-05	3.67E-04	1.83E-03
Pavement 3	B75-G10-C Size 14 / Wearing Course	4.81E-06	8.44E-31	1.30E-05	8.44E-31
	Asphalt Size 20/ Intermediate Course	1.97E-05	7.90E-05	3.56E-05	1.52E-03
	Asphalt Size 20/ Base Course	4.67E-05	3.51E-03	1.03E-04	1.85E-01
	Cemented Material	9.96E-05	9.28E-01	n/a	n/a
	Subgrade	2.28E-04	6.53E-05	3.64E-04	1.71E-03
Pavement 4	B100-S Size 14 / Wearing Course	6.44E-06	8.44E-31	1.51E-05	8.44E-31
	Asphalt Size 20/ Intermediate Course	1.93E-05	7.10E-05	3.52E-05	1.42E-03
	Asphalt Size 20/ Base Course	4.67E-05	3.50E-03	1.04E-04	1.89E-01
	Cemented Material	1.00E-04	9.97E-01	n/a	n/a
	Subgrade	2.30E-04	6.85E-05	3.67E-04	1.83E-03
Pavement 5	B75-S Size 14 / Wearing Course	4.23E-06	4.78E-08	3.46E-06	8.44E-31
	Asphalt Size 20/ Intermediate Course	2.04E-05	9.39E-05	3.64E-05	1.69E-03
	Asphalt Size 20/ Base Course	4.68E-05	3.52E-03	1.03E-04	1.80E-01
	Cemented Material	9.90E-05	8.64E-01	n/a	n/a
	Subgrade	2.27E-04	6.23E-05	3.60E-04	1.59E-03
Pavement 6	B75-G10-S Size 14/Wearing Course	6.96E-06	8.44E-31	1.59E-05	8.44E-31
	Asphalt Size 20/ Intermediate Course	1.91E-05	6.66E-05	3.49E-05	1.36E-03
	Asphalt Size 20/ Base Course	4.67E-05	3.49E-03	1.04E-04	1.92E-01
	Cemented Material	1.01E-04	9.98E-01	n/a	n/a
	Subgrade	2.31E-04	7.05E-05	3.69E-04	1.91E-03

CDF is an indication of total damage of pavement. When CDF reaches 1, it is presumed that the pavement has reached its design life. The pavement with CDF of more than 1 is unacceptable and must be modified. As can be observed in Table 10.8, for different asphalt comprising 190 mm of asphalt and 170 mm cemented material with different wearing course, the asphalt with wearing course of 100% basalt gives a CDF of more than one. Therefore, in next trial, the wearing course in Pavement 1 (100% Basalt) has been increased to 45 mm to overcome this deficiency.

In addition, as can be seen in Table 10.8, the pavement is designed for post cracking phase since the thickness of asphalt layer is more than 175 mm. For taking into account post-cracking phase in calculation of allowable loading of layers, Australian standard considers the following equations:

$$\boldsymbol{N_A = N_C + \left(1 - \frac{N_C}{N_{1stA}}\right) \times N_{2ndA}} \tag{10.9}$$

$$\boldsymbol{N_S = N_C + \left(1 - \frac{N_C}{N_{1stS}}\right) \times N_{2ndS}} \tag{10.10}$$

where N_A is total allowable loading to asphalt fatigue, N_{1stA} is the allowable number of load repetitions to asphalt fatigue prior to cemented material fatigue, N_C is allowable number of load repetitions to cemented material fatigue, N_{2ndA} is the allowable number of load repetitions to asphalt fatigue after cemented material fatigue, N_S is the total allowable loading to unacceptable permanent deformation, N_{1stS} is the allowable number of load repetitions to unacceptable permanent deformation during the pre-cracking phase and N_{2ndS} is the allowable number of load repetitions to unacceptable permanent deformation during the post-cracking phase. The allowable numbers of load repetitions for different layers of modified pavements is presented in Table 10.9.

Table 10.9: Total Allowable Loading in Pavement Model

Pavement	Thickness (mm)	Total Allowable Loading to	
		Asphalt Fatigue (ESA)	Unacceptable Permanent Deformation
Pavement 1 (B100-C)	195	2.42E+12	4.57E+09
Pavement 2 (B75-C)	190	5.40E+11	4.07E+09
Pavement 3 (B75-G10-C)	190	6.38E+11	4.31E+09
Pavement 4 (B100-S)	190	5.24E+11	4.07E+09
Pavement 5 (B75-S)	190	4.21E+14	4.66E+09
Pavement 6 (B75-G10-S)	190	4.38E+11	3.92E+09

As can be observed in Table 10.9, total allowable loading for asphalt fatigue and rutting is higher in Pavement 5 with 25% RCA and prepared in ASMM mixing method compared to conventional asphalt mixture although this pavement has 5 mm less thickness.

10.7. Results and Discussion

Structural analysis of pavements is crucial to understanding their performance over the design period. In this study, the structural design of pavement was performed through mechanistic design based on Australian standard for different pavements with different wearing courses made of most acceptable asphalt mixtures proposed based on the laboratory investigation conducted in this research. To investigate the performance of the pavements with respect to rutting resistance as well as their resistance to fatigue cracking, software CIRCLY 6.0 and the Australian Standard procedure were used. The results of this investigation indicates that asphalt mixtures proposed in this research, particularly the mixture prepared with ASMM mixing method containing 25% RCA substantially increase the allowable loading to asphalt fatigue and rutting in comparison to conventional asphalt mixtures.

Chapter 11

Life Cycle Assessment of Asphalt Mixtures

11.1. Introduction

11.2. LCA Framework

11.3. LCA Models

11.4. Asphalt Pavements LCA

11.5. Description of Asphalt Pavements LCA Model

11.6. Cost Analysis

11.7. Case Study

11.8. Results and Discussion

11.1. Introduction

Application of recycled materials has considerable advantages in reduction of virgin materials consumption and greenhouse gas emissions. In spite of these encouraging benefits, the utilization of recycled materials, based on limited factors such as emissions reduction or material consumption reduction, may not be advisable to key decision makers since the recycled materials promotion requires a thorough evaluation of this technology as well as the consideration of all critical environmental factors.

Life cycle assessment (LCA) is a methodical tool that considers the economical and environmental factors for a given technology to evaluate the potential impacts of the technology. LCA identifies and quantifies all economical and environmental impacts of a product or technology by considering different stages including the extraction of raw materials, manufacturing stage, using the product and the final disposal of the product. LCA result is a useful tool which gives a better understanding of the systems and is effective in minimizing the energy and resources consumption as well as the emissions to the environment through finding an alternative product, process or activity. LCA can be conducted by two separate qualitative and quantitative methods (Jensen et al., 1997). The qualitative LCA methods such as red flag method (RFM) and material, energy, and toxicity (MET) matrix assess the life cycle of a product by comparing its quality with the Data Quality Objectives (DQO) defined based on the LCA goal and scope and using a qualitative scale matrix rather than a systematic computational procedures. Therefore, completion of LCA through qualitative methods requires the knowledge of environmental experts. In contrast, the environmental impacts are evaluated through the mathematical processing of the data in quantitative methods. There are several quantitative methods of LCA which all are based on International Standards Organization (ISO) standards 14040-43 and hence they involve different phases including classification, characterization, normalization and weighting. The quantitative methods generally calculate a single score for different impact categories which shows the level of adverse/positive environmental impact of a product, and hence make more reproducible and reliable judgement.

Therefore, the objective of this chapter is to conduct a quantitative life cycle assessment of asphalt mixture incorporating recycled materials as compared to a conventional asphalt mixture.

11.2. LCA Framework

An international standard for conducting LCA is developed by the International Standards Organization (ISO) which includes four different phases as follows:

1) **Goal and Scope Definition:** The system, its boundaries and the functional unit selection are described in this phase. In addition, the analysis period and all the processes included in the life cycle are identified in this step. In this study, the functional unit selected in this study is the production of one metric tonne of HMA.
2) **Life Cycle Inventory (LCI):** This phase quantifies the inputs and outputs for all analysed processes. In this study, the resources (materials and energy) consumption and the quantities of emissions and waste associated with the production of HMA is estimated for LCI.
3) **Life Cycle Impact Assessment (LCIA):** This phase determines the environmental impact of a given product life cycle in terms of selected environmental impact categories such as ozone depletion, global warming potential, fossil fuel depletion, impact on human health, etc.
4) **Life Cycle Interpretation:** This phase compiles the results of LCIA phase and compares the scores obtained for all environmental impact categories. The results of both environmental and economical analysis are considered in the interpretation phase of this study.

In general, as illustrated in Figure 11.1, the LCA results can be useful in identifying the opportunities for improvement of the performance of a process or product over its design life in terms of environmental factors.

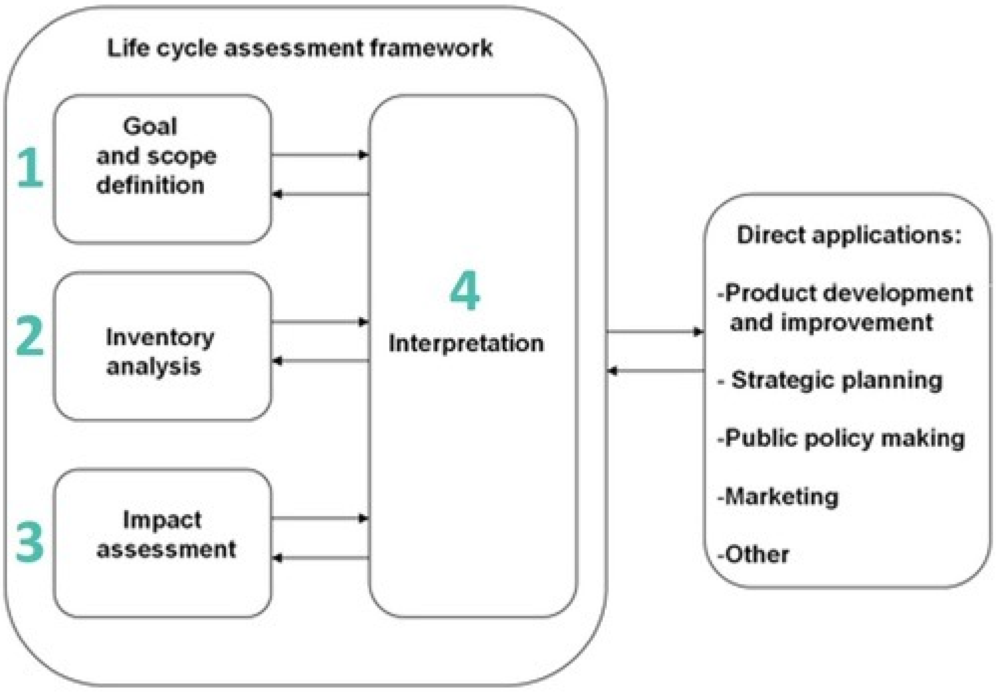

Figure 11.1: Life Cycle Assessment Framework (ISO14040 2006)

11. 3. LCA Models

LCA is usually performed by employing relevant softwares. Today, there are a variety of LCA software tools, some of which are summarized as follows:

- **U.S. Life Cycle Inventory Database** is an online database created by National Renewable Energy Laboratory (NREL). This database is useful in determination of the energy and materials input and output for the production of a component or assembly processes.
- **SimaPro Life Cycle Assessment Software (SimaPro 7)** is a professional software for assessment of the environmental impact of a product. This software models and analyzes the life cycle of a product or service based on ISO 14040.
- **Buildings for Environmental and Economic Sustainability (BEES)** which is introduced by the National Institute of Standard and Technology and is a Windows TM-based software program. This software is useful in selecting sustainable alternatives for materials and construction while providing a balance in terms of economical and environmental factors (Gloria et al., 2007).
- **BRE Environmental Profiles** provides a consistent approach for applying LCA to construction materials.

- **Building Life Cycle Cost (BLCC)** is a software useful in economic analysis of construction systems aiming at reducing the long-term operating costs.
- **Boustead Model** is a comprehensive database including data on the required material and energy as well as the emissions to the environment associated with a product or service.
- **Eco-Indicator 99** is a method of impact assessment focusing on damage analysis of a system.
- **ECO-it** is a computer program for environmental impact analysis of common materials such as metals, paper and cardboard, plastic and the input and output associated with the production and transport of materials. It can be used to calculate the environmental impact of different stages in manufacturing a product.
- **Ecoscan 3.0** is a software used for the environmental impact and cost analysis of a product.
- **Environmental Priority Strategies (EPS)** is a software for life cycle analysis and the environmental impact assessment of products aiming at developing sustainable products.
- **GaBi Software System and Database** is a software for life cycle analysis of products, technologies and processes. It considers the environmental criteria, cost and social aspects in evaluation of the impact of a process or product manufacturing. This software is introduced by PE Europe GmbH and IKP University of Stuttgart and contains Ecoinvent data and comprehensive GaBi databases covering the worldwide data.
- **Global Emission Model for Integrated Systems (GEMIS)** is a program for life cycle analysis of systems. The GEMIS database contains data on the required energy and resources consumed for producing electricity, raw materials as well as transport.
- **IDEMAT** is a program which is useful in selecting the materials in the design process through enabling the user to compare the information about the materials and processes.
- **LCAiT** is a program for the environmental impact analysis of processes and products including a database with characterization factors and weighting factors.
- **Life Cycle Inventory of Asphalt Pavements spreadsheet model** is a spreadsheet useful in developing the life cycle inventory for asphalt pavements. This spreadsheet enables the user to input alternative data for a project. However, this spreadsheet has

some confidentiality restrictions and cannot be used widely in the industry as it is only distributed by and inside EAPA.

- **Life Cycle Assessment of Road Construction Models** is a program for developing the life cycle inventory for the assessment of utilization of industrial by-products in roads and earth works through comparing them with the similar projects using only natural aggregates. This program focuses on atmospheric emissions and energy consumption.
- **Pavement Life-cycle Assessment Tool for Environmental and Economic Effects (PaLATE)** is a Microsoft Excel Model for environmental impact analysis and cost analysis of projects associated with the design, construction and maintenance of pavements. PaLATE is developed by University of California Berkeley.
- **Road-RES model** is a model for environmental impact assessment of the road construction as well as the disposal of bottom ash from municipal solid waste incinerators. This model is developed with C++ programming.
- **TEAM™** is a powerful software for evaluation of the environmental impact and cost of products and technologies over their life cycle containing a comprehensive database with worldwide coverage.
- **Umberto** is a software for life cycle analysis using a visual approach to illustrate the relationship between inputs and outputs of producing a product.
- **OpenLCA** is a software for the life cycle analysis which is effective in identifying the opportunities for resources consumption and emissions reduction for a project or process. OpenLCA is a free open source software.

In this study, the software used to conduct the analysis is GaBi 6.0. The GaBi software system is a modular system composed of plans, processes, and flows with a clear and transparent structure. GaBi's visualization approach enables modelling the construction and manufacturing processes in an easy way. In GaBi, individual modules of life cycle inventory (LCI), life cycle impact assessment (LCIA) and normalizing and weighting are only combined for the balance calculation since data on these modules are separated from each other providing more manageable modelling. All these features make GaBi software a leading tool for life cycle assessment with solutions to optimize the product manufacturing and processes in terms of economic and environmental criteria.

11. 4. Asphalt Pavements LCA

Asphalt mixture is composed of aggregate (85%-90%), filler (4%-6%), and binder (4%-6%), and occasionally some additives. asphalt is used in the construction of roads. The life cycle of asphalt covers all activities covering the extraction of raw materials, processing of recycled materials, the pavement demolition and reuse or recycling of the materials. This work investigated the application of some certain waste materials in asphalt mixture by quantifying the environmental benefits and savings associated with producing asphalt mixtures with a major percentage of RCA and using a new mixing method.

11.4.1. Goal and scope

Based on ISO 14044 requirements, a LCA study should have a clear goal intending the reasons for conducting the study. In addition, the scope as well as the product or service to be assessed and the functional unit should be defined in this phase. In general, the main points which should be considered in this phase are:

- Purpose of the study and the reason for performing the analysis
- Scope of the study
- Functional units and the product systems to be assessed
- System boundary

This study aimed at finding out how much the construction of the asphalt surface course through the innovative technology for asphalt production can contribute to energy consumption and emissions generation reduction and natural resources (aggregate and binder) conservation. In this study, different innovative options for asphalt mixture design are compared with the conventional asphalt mixtures of the same size. Therefore, this study is a comparative LCA study in accordance with ISO 14040 series. To achieve these aims, all processes needed to produce an asphalt mixture are analyzed through different phases of LCA and explained in the following section.

The scope of life cycle analysis on asphalt mixture varies and generally can be classified into four groups of cradle-to-gate, gate-to-grave, cradle-to-grave, and grave-to-cradle, as illustrated in Figure 11-2.

The cradle-to-gate phase covers all processes from raw material extraction, the materials processing, transportation to the asphalt plants, asphalt mixing and production. The gate-to-grave phase covers asphalt installation and construction, operational phase, asphalt layer maintenance, demolition of road, and disposal of demolished materials to the landfill. The cradle-to-grave phase covers all processes mentioned in two phases of cradle-to-gate and gate-to-grave. The final phase which is the grave-to-cradle includes the strategies for the reuse and recycling of the demolished materials.

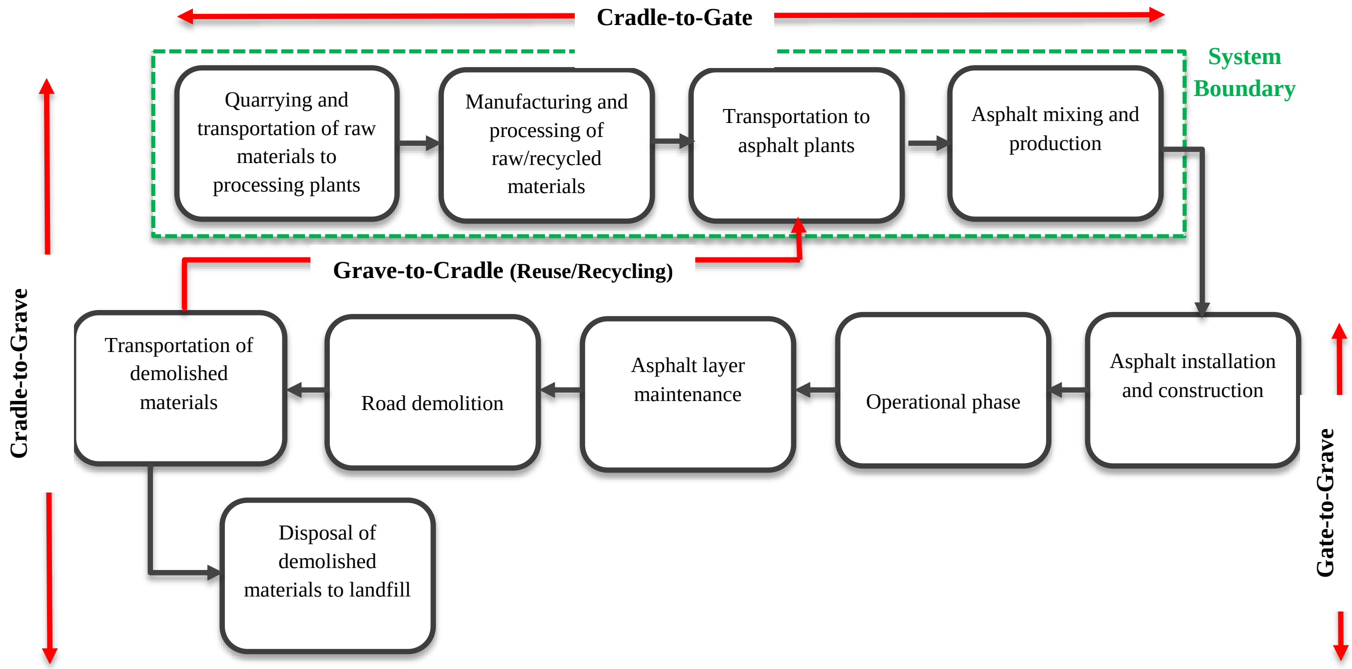

Figure 11.2: Life Cycle Phases of Asphalt Pavement

This study presents an inclusive review of the cradle-to-gate environmental impacts of asphalt mixture production in Australia. For the purpose of this analysis, different types of asphalt mixtures including conventional asphalt mixture (made of virgin aggregates) and innovative asphalt mixture (made of virgin and recycled aggregates) are assumed with their optimum bitumen content, 39.7% coarse aggregate, 49.7% fine aggregate, 5.2% filler. In summary, the scope of this study includes:

1) Determining and quantifying raw materials consumed for asphalt production. This involves the extraction, processing, manufacturing, and transportation of materials. The review excludes the environmental impacts associated with the manufacture of mining machinery.

2) Identifying and quantifying the corresponding emissions generated during the extraction, processing, manufacturing, and transportation of the materials.

3) The data sources used are specific to Australia.

In addition, prior to developing life cycle inventory (LCI), we need to identify the processes occurring in analyzed life cycle and to define the system boundary which determines the limits. The functional unit is the measure of performance delivered by the system. The functional unit is a reference which all inputs and outputs are related to it.

To define system boundary, it should be noted that in any LCA, each product consists of subsequent processes required for production, transportation, use and disposal of the product. As product systems have normally complex interconnections, it is usually impossible to separate a single life cycle of a product from life cycles of other products. Accordingly, the boundaries of the system must be defined in order to separate the system under study from the environment and other products and systems.

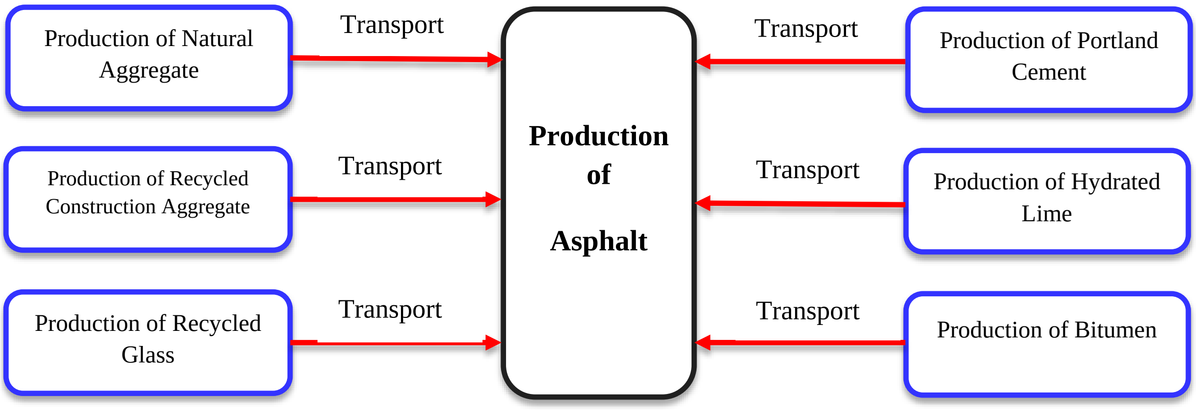

Figure 11.3: Processes for Asphalt Mixture Production included in the study

In the present study, the production of one metric tonne of HMA is defined to describe functional unit. The flowchart for unit processes in asphalt production is illustrated in Figure 11.3 in which system boundaries are defined. The unit processes in asphalt pavement project will be further discussed in the following sections.

11.4.1.1. Production of Natural Aggregates

Natural aggregates can be obtained from quarrying (excavating aggregate and stones from rock deposits through the use of explosives, automatic drills, and heavy machinery) or excavation from deposits in river valleys. The excavated materials are then transported to crushing equipment for crushing and screening into a range of sizes. It should be noted that the sand and gravel particles excavated from the river valleys are more rounded and of more similar size, and hence need less crushing and sieving. However, the road industry prefers crushed rocks from quarries because of their higher friction and bonding with the bitumen due to their angular shape. In general, the processes for production of natural aggregate include blasting and excavation, primary crushing, secondary crushing, and tertiary crushing, screening, transfer by conveyor, and storage operations.

11.4.1.2. Production of Recycled Construction Aggregates

In general, recycled construction aggregates from construction and demolition wastes are produced through different stages including separation of coarse materials, crushing, and ferrous elements separation, screening, and removal of impurities (i.e. wood, plastic, paper, etc.). Crushing stage involves primary and secondary crushing and screening in order to obtain different sizes of recycled aggregates. In addition, impurities can be removed by air separation technique or washing separation in which air method is more common due to economic reasons.

11.4.1.3. Production of Recycled Glass

After collection of waste glass, contaminants are removed from the glass. The final step in production of recycled glass as an alternative aggregate in asphalt is crushing the clean waste glass in required sizes for utilization. However, the production of recycled glass for other purposes (glass cullet and making new glass products) requires other stages such as colour sorting and more cleaning to remove the contaminants to a certain level of impurity content.

11.4.1.4. Production of Bitumen

Bitumen production obtained from petroleum requires high energy consumption for crude oil extraction, transporting the crude oil to refinery, refining (i.e. heating, distillation, cooling and final processing), and transportation to storage centers (Zapata and Gambatese, 2005). In some countries, such as Australia, bitumen is imported as a liquid product and then redistilled to recover the bitumen portion. It must then be restored (at temperature) and transported (at temperature) to the asphalt plant.

11.4.1.5. Production of Filler (Portland Cement and Hydrated Lime)

Portland cement and hydrated lime are man-made powders produced by the chemical transformation of raw materials into different types of products, by-products and wastes through different stages requiring high energy consumption.

11.4.1.6. Production of Hot Mix Asphalt

In asphalt production, aggregates should be dried and heated before mixing with bitumen. Bitumen should also be heated to a certain temperature. Generally, manufacturing processes considered in the asphalt mixture production included heating and drying aggregates and filler, heating bitumen, mixing aggregates and bitumen, and conditioning the mixture. During the asphalt mixture production, energy is consumed for bitumen heating, aggregates and fillers drying, mixing the ingredients, and conditioning the mixture. Producing hot mix asphalt requires providing a high temperature (approximately more than 150°C). in the production of HMA in asphalt plant, the substantial amount of emission occurs in the rotary drum dryer as this stage produces carbon monoxide, particle matters (PM), and many different organic compounds.

11.4.2. Life Cycle Inventory (LCI)

The inputs and outputs of a product throughout its life cycle are quantified in LCI phase of LCA. In this study, identification of data related to energy, resources and emission of each process are obtained from reviewing a wide range of published reports and database. In order to model the unit processes, LCI data were used from a commercial LCA software, GaBi 6.0. The GaBi software database is one of the most comprehensive databases containing broad range of data including Ecoinvent and other several databases. The GaBi software is based on process-based assessment approach. In this approach, the energy input of manufacturing used materials for all main processes is considered which emphasizes the necessity of setting system boundaries at early stages of life cycle analysis.

11.4.3. Life Cycle Impact Assessment (LCIA)

The amount of energy, emissions and wastes measured in units of MJ, grams and tonnes cannot express clearly the effect of such activities on environment and human health. Accordingly, a method is required to interpret the results of LCA in a recognized and clear way for better decision making. LCIA enables the compilation and interpretation of the inventory results.

11. 5. Description of Asphalt Pavements LCA Model

In this study, considering the unit processes in asphalt mixture production, an asphalt pavement LCA model has been created based on process parameters, asphalt mixture parameters, energy and emissions inventory, and impact assessment categories. The following sections describe the LCA model as well as the procedure for the analysis of unit processes considered in asphalt production project.

11.5.1. Model Structure

As mentioned previously, the pavement LCA model described in this study is intended to reflect the processes for the production of one tonne of selected asphalt mixtures proposed in this research with different materials, varying composition, and different preparation methods. The unit processes in this study are modelled in GaBi 6.0 based on the available resources, as illustrated in Figure 11.4.

In order to obtain more realistic results in this comparative LCA model, it is assumed that asphalt mixtures are produced in Boral Asphalt Plant located in Enfield, NSW, Australia which is one of the largest asphalt plants in Australia. Accordingly, the process parameters and, particularly, the transport distances are considered as close as possible to real situation.

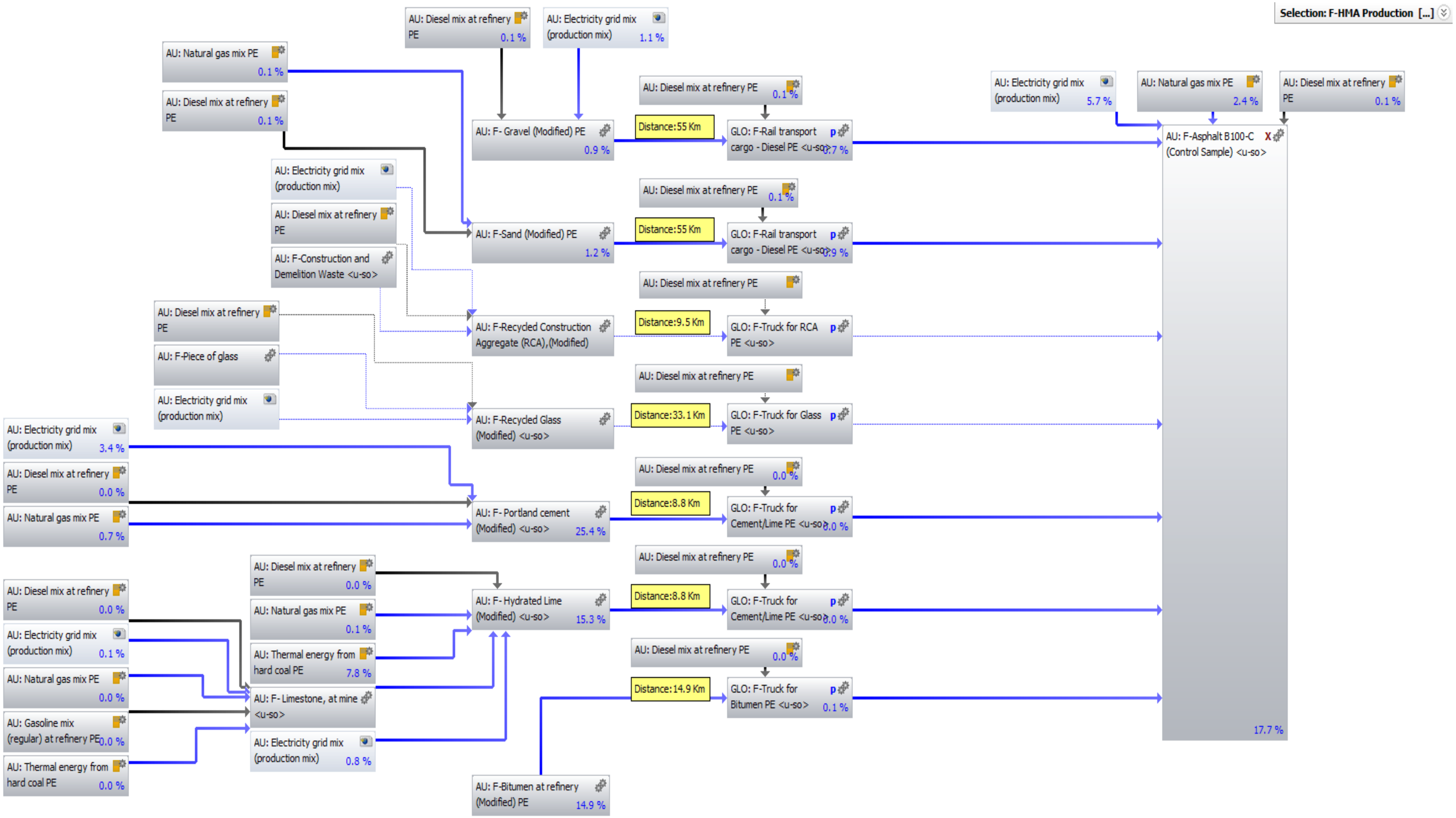

Figure 11.4: Structure of Asphalt Pavement LCA Model

11.5.2. Process Parameters

The process parameters include data on energy consumption for materials production as well as materials transportation, and transport distances.

The main materials used in different asphalt mixtures proposed in this research and their production process was briefly discussed in Section 11.4.1. A summary of these major materials and the energy consumption for production per unit of materials are presented in Table 11.1. The data provided in this table are obtained from GaBi database and various external sources in case the data were not available in GaBi LCI database.

Table 11.1: Energy Consumption per Unit of Material Production

Materials Production	Unit	Value	Data Source
Production of gravel	**MJ/tonne gravel**	**55.0**	**Calculation**
Electricity in mining and processing	kWh/tonne gravel	3	RMIT University (2012)
Diesel for production	MJ/tonne gravel	44.2	RMIT University (2012)
Production of sand	**MJ/tonne sand**	**43.0**	**Calculation**
Natural gas in mining and processing	kWh/tonne sand	18.0	AUPLCI (2010)
Diesel for production	MJ/tonne sand	25.0	AUPLCI (2010)
Production of bitumen	**MJ/tonne bitumen**	**2579.5**	**Calculation**
Natural gas for production	MJ/tonne bitumen	1908.7	GaBi 6.0
Coal combustion	MJ/tonne bitumen	339	GaBi 6.0
Lignite in production and processing	MJ/tonne bitumen	208	GaBi 6.0
Other energy sources (solar, hydro, etc.)	MJ/tonne bitumen	51.8	GaBi 6.0
Production of cement	**MJ/tonne cement**	**4430.0**	**Calculation**
Electricity in cement plant	kWh/tonne cement	114	RMIT University (2012)
Natural gas for clinker production	MJ/tonne cement	2780	RMIT University (2012)
Diesel for equipment operation	MJ/tonne cement	100	RMIT University (2012)
Waste (oil, tyre, carbon)	MJ/tonne cement	1140	RMIT University (2012)
Production of hydrated lime	**MJ/tonne hydrated lime**	**4626.2**	**Calculation**
Electricity for production	MJ/tonne hydrated lime	145.2	Huang (2007)
Diesel for equipment operation	kg/tonne hydrated lime	0.67	Huang (2007)
Natural gas for production	kg/tonne hydrated lime	12.0	Huang (2007)
Coal combustion	MJ/tonne hydrated lime	3968.2	Huang (2007)
Production of RCA	**MJ/tonne RCA**	**35.7**	**Calculation**
Electricity for production	kWh/tonne RCA	1.33	AUPLCI (2010)
Diesel for production	Litre/tonne RCA	0.86	AUPLCI (2010)
Production of glass aggregate	**MJ/tonne glass**	**34.3**	**Calculation**
Electricity in production	kWh/tonne glass	2.04	Carre et al. (2009)
Diesel for production	Litre/tonne glass	0.75	Huang (2007)

Energy consumption for production of HMA varies significantly depending on available techniques in asphalt plant. In ASMM mixing method, as discussed in previous chapters, the temperature for bitumen conditioning as well as mixing temperature is about 30°C lower than conventional mixing method. Considering the specific heat of bitumen as 2.093 kJ/kg/°C (Harder, 2008), it can be estimated that the ASMM method results in 2.12% less energy required for increasing the materials temperature to the mixing temperature. It should be noted that this estimated value does not include the energy saving in providing the mixing temperature and maintaining the temperature during the mixing process. Based on calculations, approximately 50% of energy is consumed for increasing the materials temperature to the mixing temperature. Considering this fact, Table 11.2 provides the data regarding energy consumption per unit of asphalt mixture production using conventional mixing method and ASMM mixing method.

Table 11.2: Energy Consumption per Unit of Asphalt Mixture Production

Energy Consumption	Asphalt Production		Unit	Data Source
	Conventional Mixing	ASMM Mixing		
Electricity	6	5.94	kwh/tonne asphalt	Grant and Peters, 2009
Diesel	8.49	8.22	MJ/tonne asphalt	Grant and Peters, 2009
Natural gas	300	296.82	MJ/tonne asphalt	Grant and Peters, 2009)
Total Energy Consumption	**330.1**	**326.4**	**MJ/tonne asphalt**	**Calculation**

Energy consumption for transportation depends on the mileage and vehicles for transport. Assuming that asphalt mixtures are produced in Boral asphalt plant, the transport distances have been selected as close as possible to real condition. In Boral Asphalt Plant, natural aggregates of different grading are produced from quarry site located in Emu Plains, NSW, Australia.

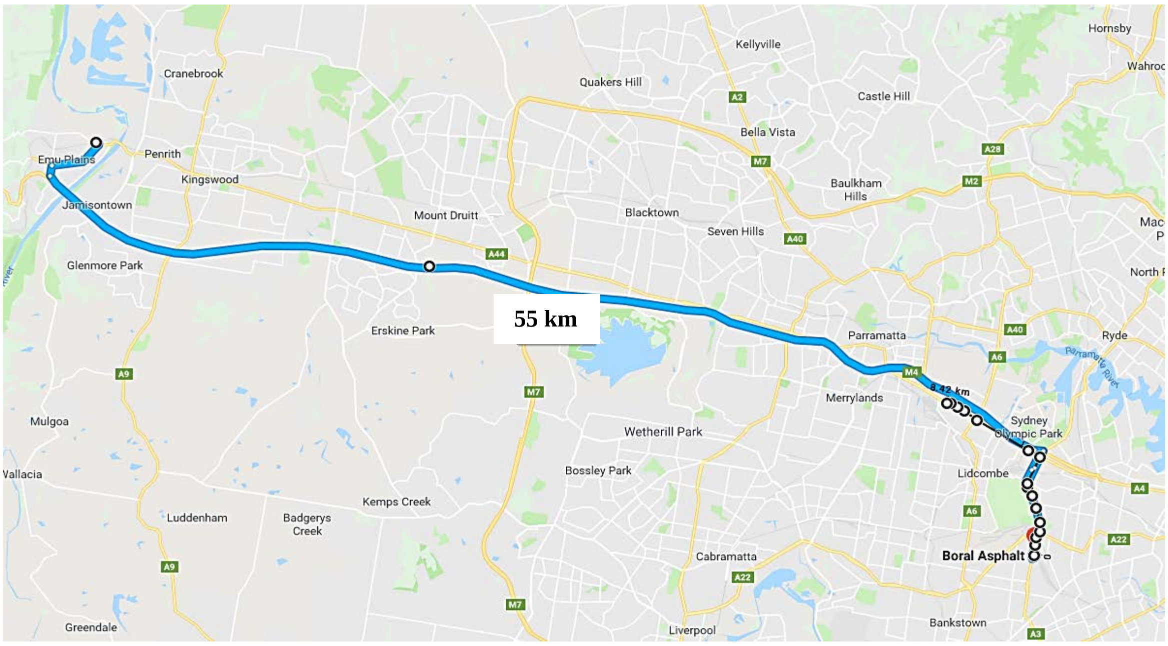

Figure 11.5: Transport Distances for Asphalt Mixture Production in LCA Model

The aggregates are transported to the plant by railway locomotives. In addition, in this study, it is assumed that recycled aggregates, fillers and bitumen are provided from the nearest suppliers to the asphalt plant using trucks. Figure 11.5 illustrates the comparison of distance for supplying the virgin and recycled aggregates to Boral Asphalt Plant. It should be mentioned that energy consumption for transport vehicles are obtained from GaBi LCI data supplied by PE International GmbH.

11.5.3. Asphalt Mixture Parameters

The asphalt mixture parameters include data on mixture composition and materials tonnage. The material quantities have been calculated based on the asphalt mixture composition for different selected asphalt mixtures proposed by this research. The major materials considered in production of one tonne of asphalt mixture are presented in Table 11.3.

Table 11.3: Composition of One Tonne of Asphalt Mixture

Asphalt Mixture	Coarse Aggregate (kg)		Fine Aggregate (kg)		Filler (kg)		Bitumen (kg)
	Gravel	RCA	Sand	Glass	Cement	Hydrated Lime	
B100-C	398.6	0	498.2	0	31.3	20.9	51
B75-C	296.7	98.9	494.6	0	31.1	20.7	58
B75-G10-C	298.0	99.3	470.2	26.5	31.2	20.8	54
B100-S	399.8	0	499.8	0	31.4	21.0	48
B75-S	297.7	99.2	496.1	0	31.2	20.8	55
B75-G10-S	298.6	99.5	471.2	26.5	31.3	20.9	52

11.5.4. Energy and Emission Inventory

The emission inventory includes LCI data for energy production, vehicle and equipment engine operations, and emission from different unit processes in asphalt production.

Table 11.4: Emission Inventory Data for Asphalt Production

Material	CO	NOx	SO_2	$PM_{2.5}$	PM_{10}	VOC	Particulate
Sand/Gravel	0.00085	0.00267	1.155E-05	0.0290	0.142	0.00033	0.524
Cement/Lime	1.086	3.265	0.246	0.378	0.442	0.0044	0.806
RCA	0	0	0	0.0527	0.294	3.94E-05	0.887
Bitumen	0.331	0.956	1.07	0.0267	1.46E-05	0.155	0.06988
Recycled glass	0	0	0	0.0527	0.294	3.94E-05	0.887
Asphalt Mixture	0.153	0.0157	0.00557	0.0308	0.0525	0.0168	0.0839

In this study, inventory data for energy production and vehicle engine operations are obtained from GaBi LCI data, whereas emission inventory for other processes are achieved through the calculation based on the energy consumption of process and available references (e.g. Department of Environment and Climate Change NSW, Optimised Operations Pty Ltd, AusLCI Draft Best Practice Guidelines and Australian Industry Data) regarding the emissions during the process itself. A summary of emission inventory for material production is presented in Table 11.4. The emission inventory data given in this table does not include the emission during energy production, as they are considered during LCA using GaBi based on energy input data for each process.

11.5.5. Life Cycle Impact Assessment (LCIA)

The quantity of chemicals released through the production of asphalt mixtures will be determined by LCA. However, as mentioned previously, these quantities will not clearly show their impact on human health and environment. Accordingly, LCIA translates the emissions and resource extractions into the environmental impact factors. In this study, the ReCipe model at midpoint level is considered as the LCIA method. The midpoint level considers 18 indicators focusing on single environmental problems, whereas endpoint level includes 3 indicators showing the environmental impacts on three factors of human health, biodiversity, and resource scarcity. It is possible to convert midpoint level to endpoint level in ReCipe structure. A list of impact categories considered in this study are presented in Table 11.5.

Table 11.5: List of Impact Categories Considered in This Study

Impact Category	Unit
Climate change	kg CO_2-Equiv
Ozone depletion	kg R11-Equiv.
Ecotoxicity for aquatic fresh water	CTUe
Human Toxicity- cancer effects	CTUh
Human Toxicity- non cancer effects	CTUh
Particulate Matter/Respiratory effects	kg $PM_{2.5}$-Equiv.
Ionizing Radiation – Human Health Effects	U235-Equiv.
Photochemical Ozone Formation	kg NMVOC- Equiv.
Acidification	Mole of H^+ -Equiv.
Eutrophication- terrestrial	Mole of N -Equiv.
Eutrophication- aquatic	kg P -Equiv.
Resource Depletion- Water	kg
Resource Depletion- mineral, fossil and renewable	kg Sb-Equiv.

According to ISO 14040, LCIA consists of some mandatory elements (i.e. classification and characterization) as well as optional elements (i.e. normalization and weighting). In this study, normalization is considered as part of LCIA. Weighting, however, is not included in this study, as weighting is not developed at the midpoint level by the ReCipe authors.

11.5.6. Result of LCA Model

A full summary of all impact categories for conventional asphalt mixture (B100-C) are given in Figures 11.6 to 11.18.

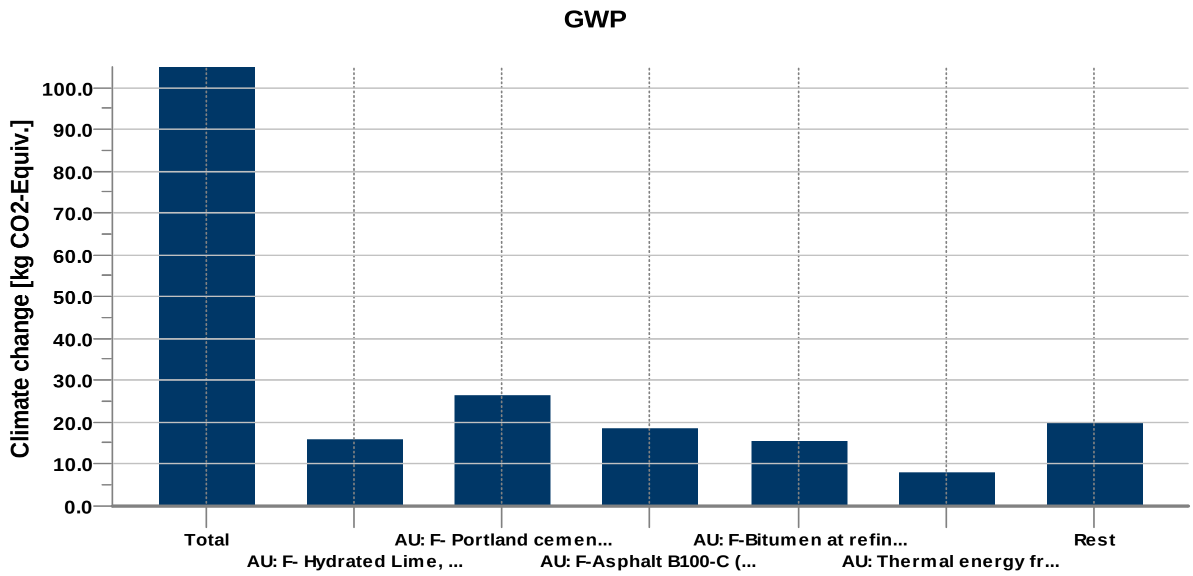

Figure 11.6: Global Warming Potential (GWP) for Different Phases of Asphalt Mixture Production

As can be observed in Figures 11.6 and 11.7, Portland cement production has the main contribution to global warming, whereas gravel and sand production can be the main contributors in ozone depletion.

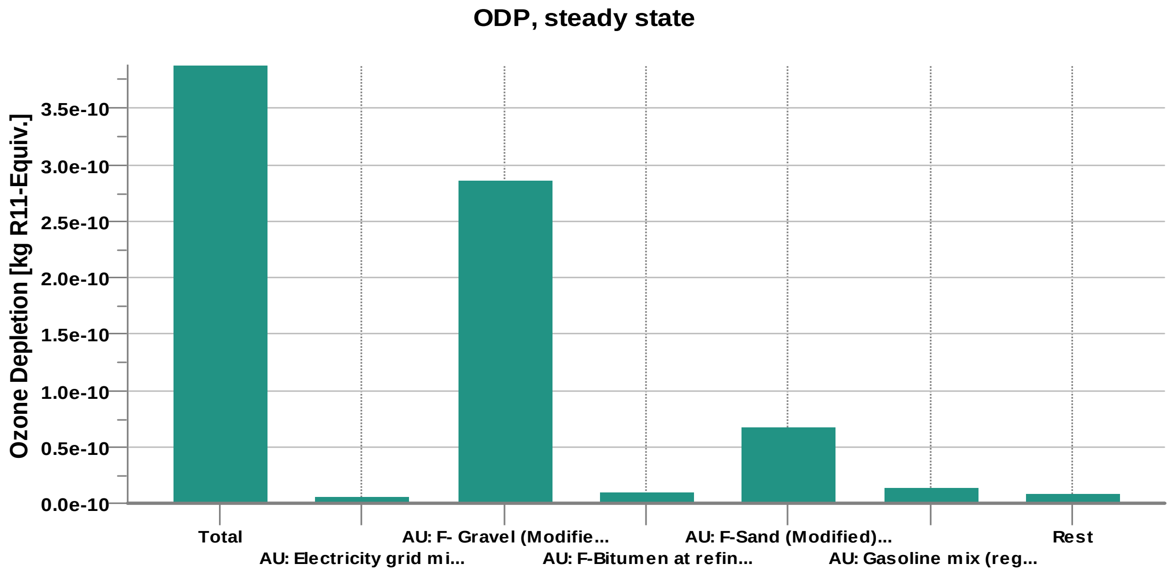

Figure 11.7: Ozone Depletion Potential (ODP) for Different Phases of Asphalt Mixture Production

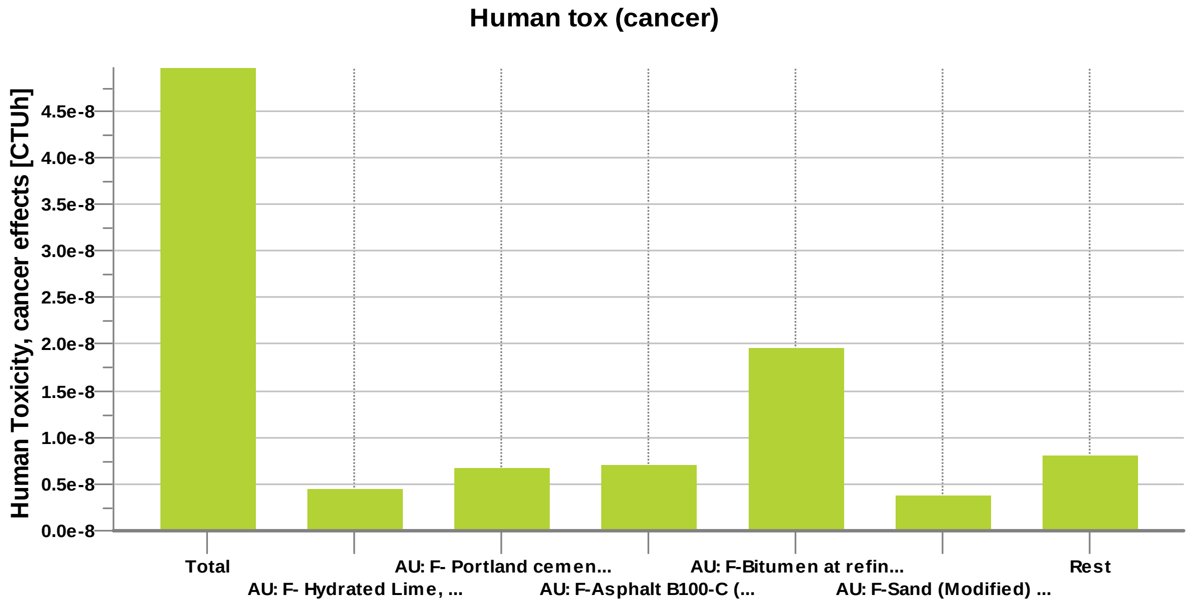

Figure 11.8: Human Toxicity (Cancer Effect) for Different Phases of Asphalt Mixture Production

In terms of human toxicity, as presented in Figures 11.8 and 11.9, bitumen production highly impacts human toxicity for cancer effects, whereas Portland cement production is the main contributor in human toxicity for non-cancer effects.

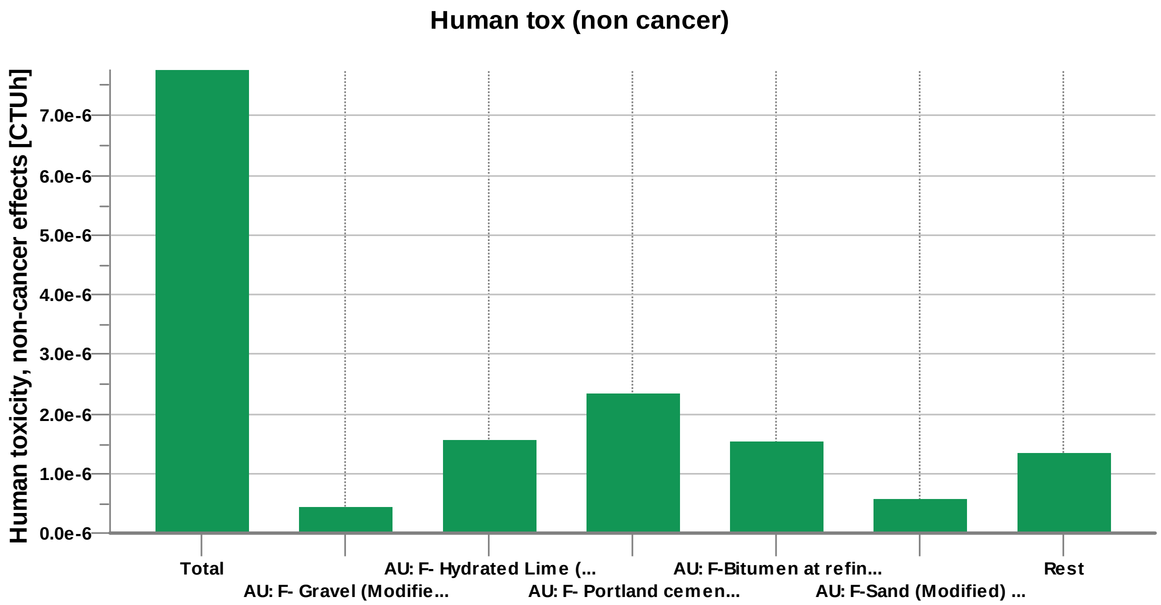

Figure 11.9: Human Toxicity (Non-Cancer Effect) for Different Phases of Asphalt Mixture Production

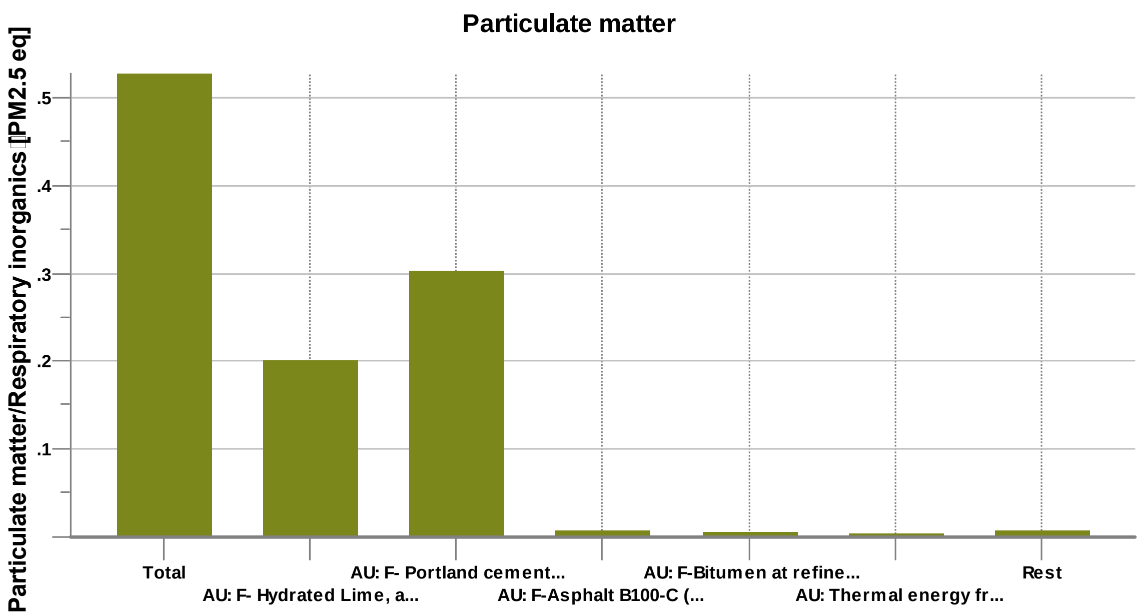

Figure 11.10: Particulate Matter/Respiratory Inorganic for Different Phases of Asphalt Mixture Production

As can be observed in Figures 11.10 and 11.11, Portland cement followed by hydrated lime production are the main contributors in particulate matter production, whereas gravel and sand production significantly affect the human health through ionizing radiation impact.

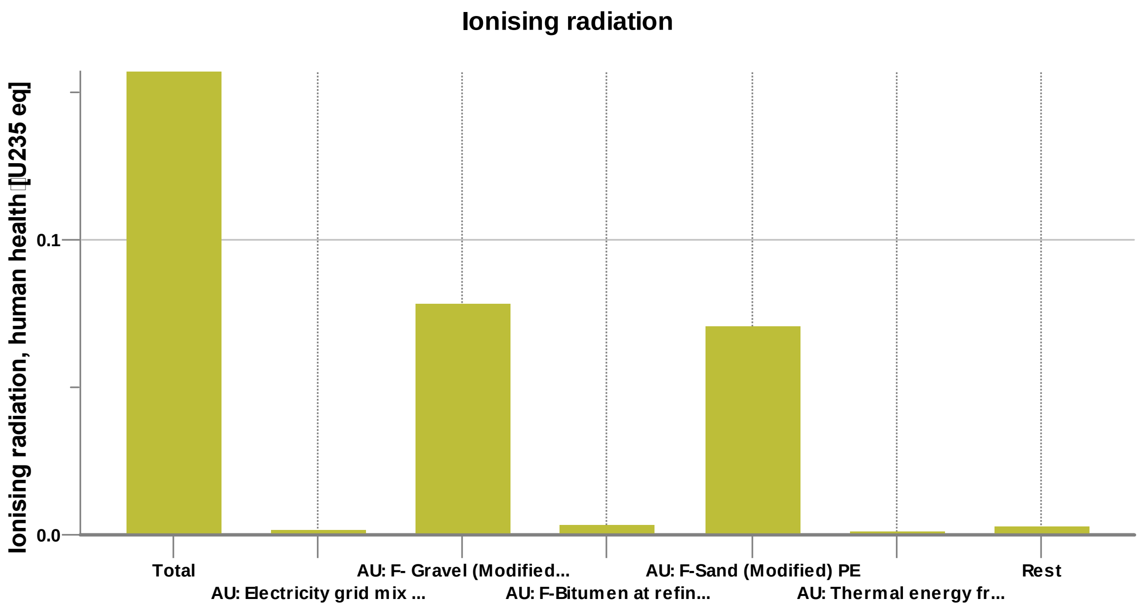

Figure 11.11: Ionizing Radiation (Human Health) for Different Phases of Asphalt Mixture Production

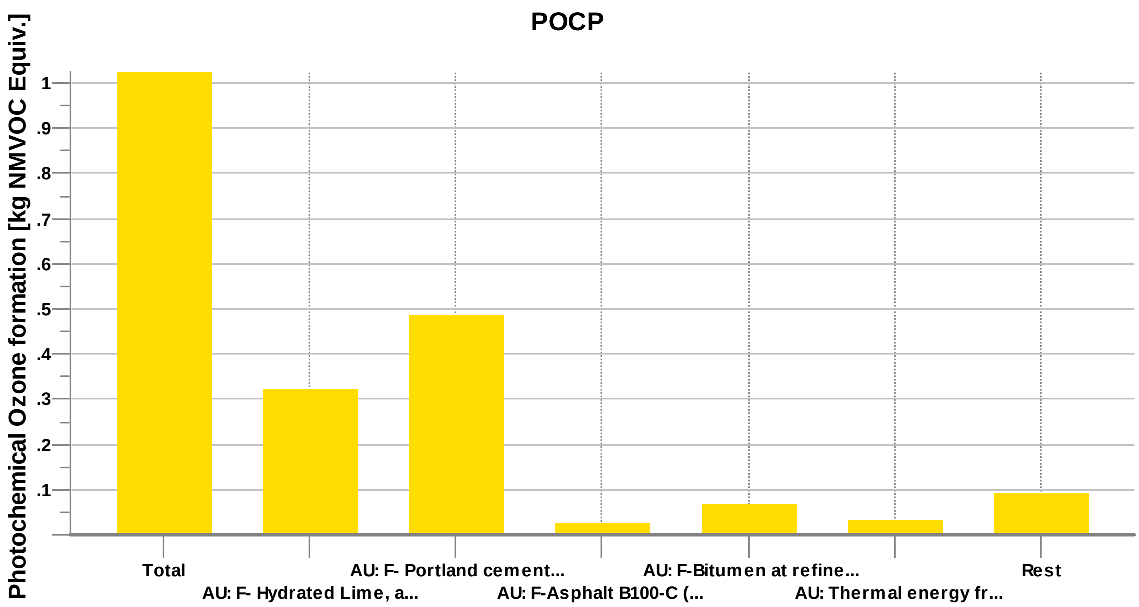

Figure 11.12: Photochemical Ozone Formation Potential for Different Phases of Asphalt Mixture Production

Portland cement followed by hydrated lime production is also the main contributors in photochemical ozone depletion and acidification potential (Figures 11.12 and 11.13).

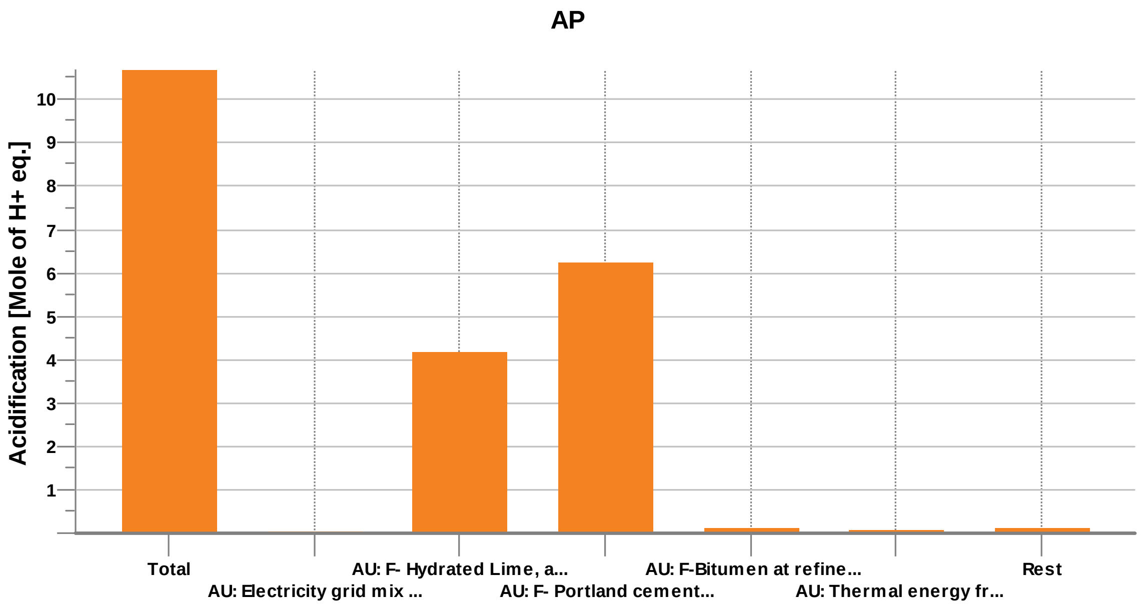

Figure 11.13: Acidification Potential for Different Phases of Asphalt Mixture Production

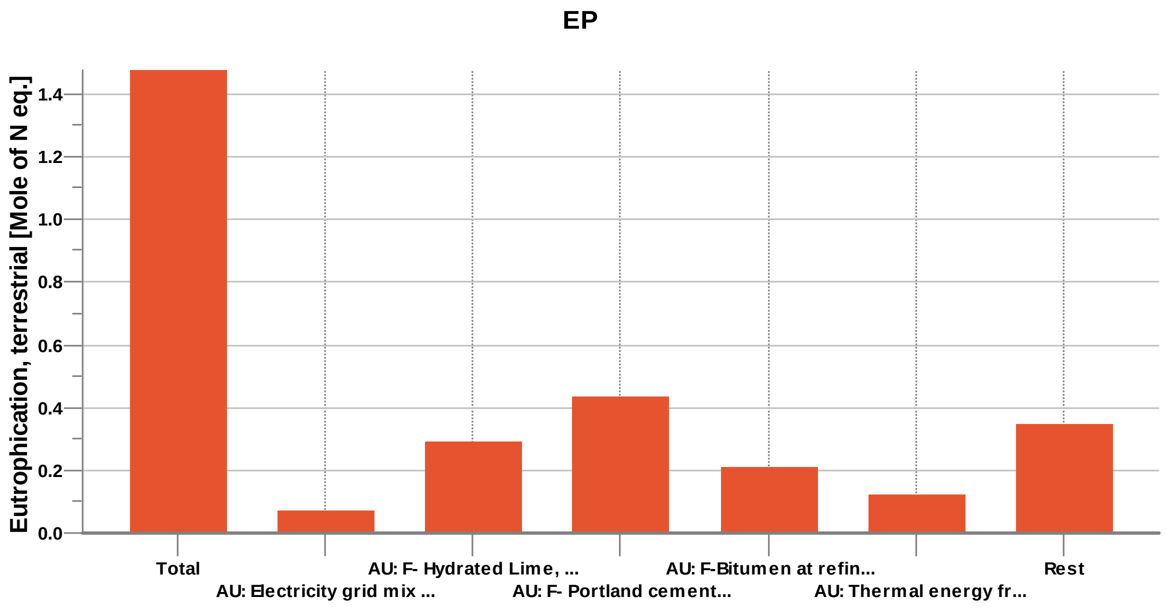

Figure 11.14: Impact of Different Phases of Asphalt Mixture Production on Eutrophication (Terrestrial)

Based on LCA results, Portland cement significantly attributes to terrestrial eutrophication, whereas bitumen production followed by sand production is the main contributor to aquatic eutrophication (Figures 11.14 and 11.15).

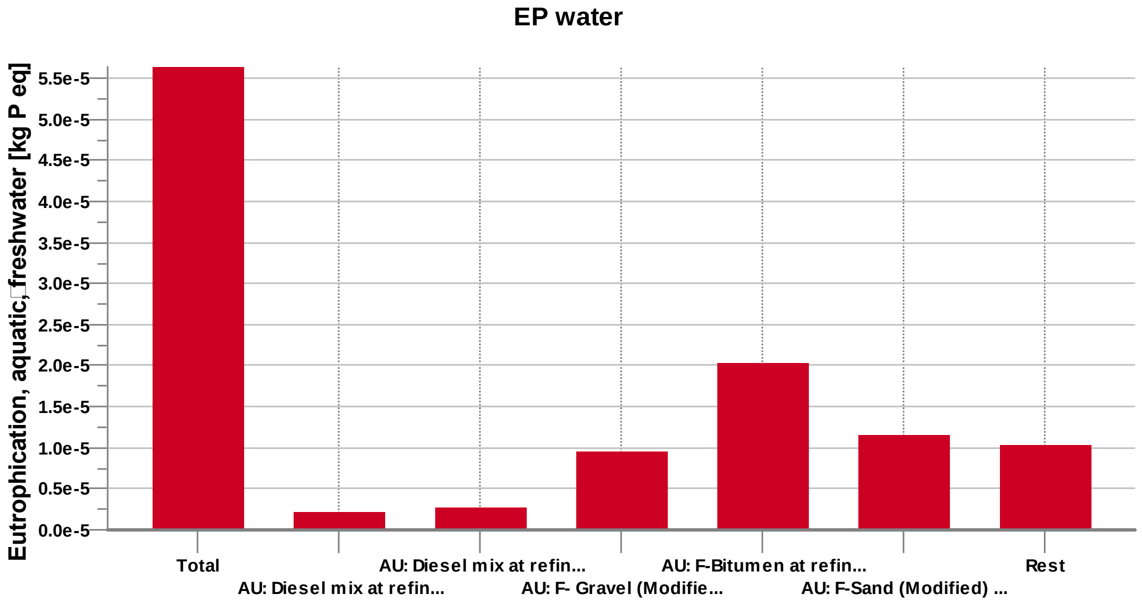

Figure 11.15: Impact of Different Phases of Asphalt Mixture Production on Eutrophication (Aquatic, Freshwater)

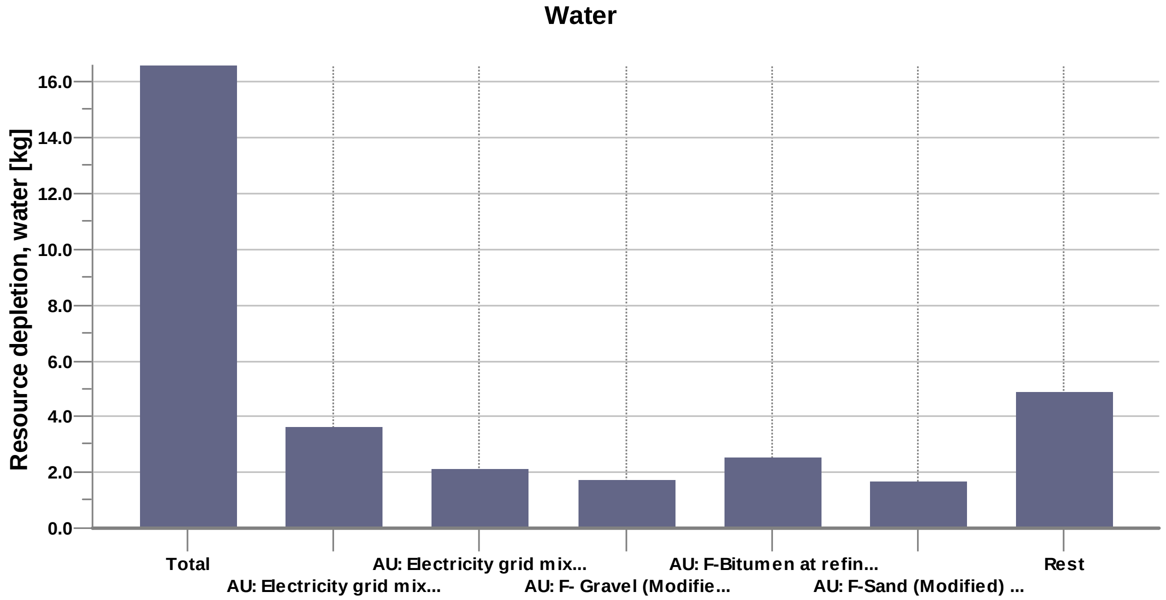

Figure 11.16: Impact of Different Phases of Asphalt Mixture Production on Resource Depletion (Water)

The results of LCA also show that bitumen production highly contributes to resources depletion and ecotoxicity of fresh waters (Figures 11.16 to 11.18).

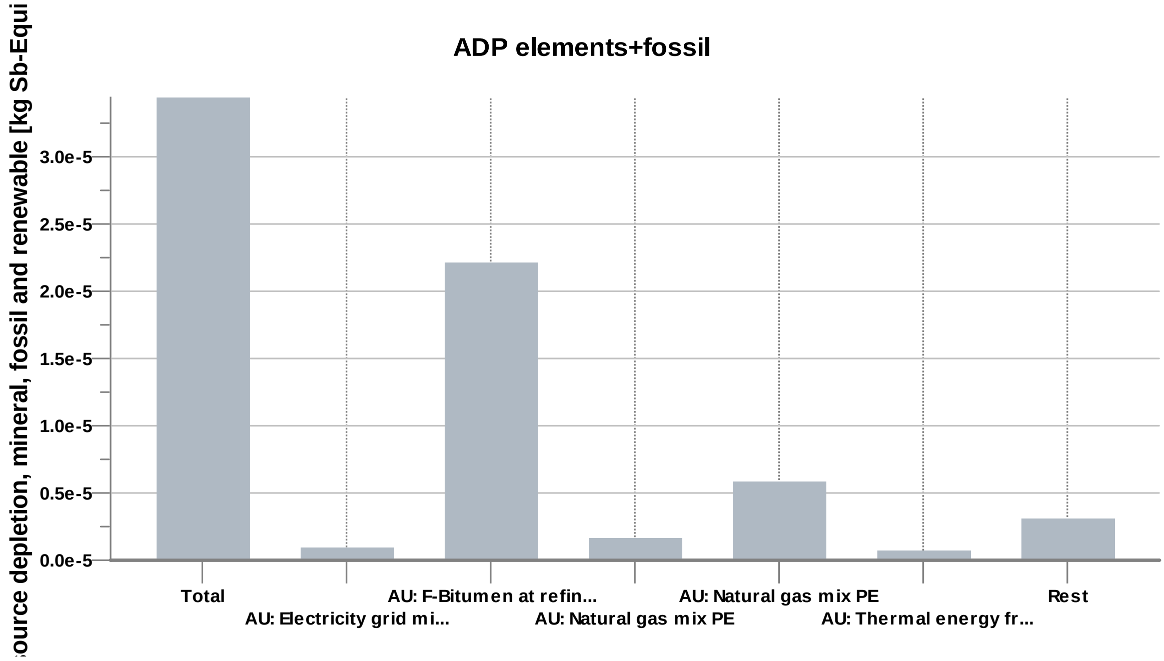

Figure 11.17: Impact of Different Phases of Asphalt Mixture Production on Resource Depletion (Minerals and Fossil Fuels)

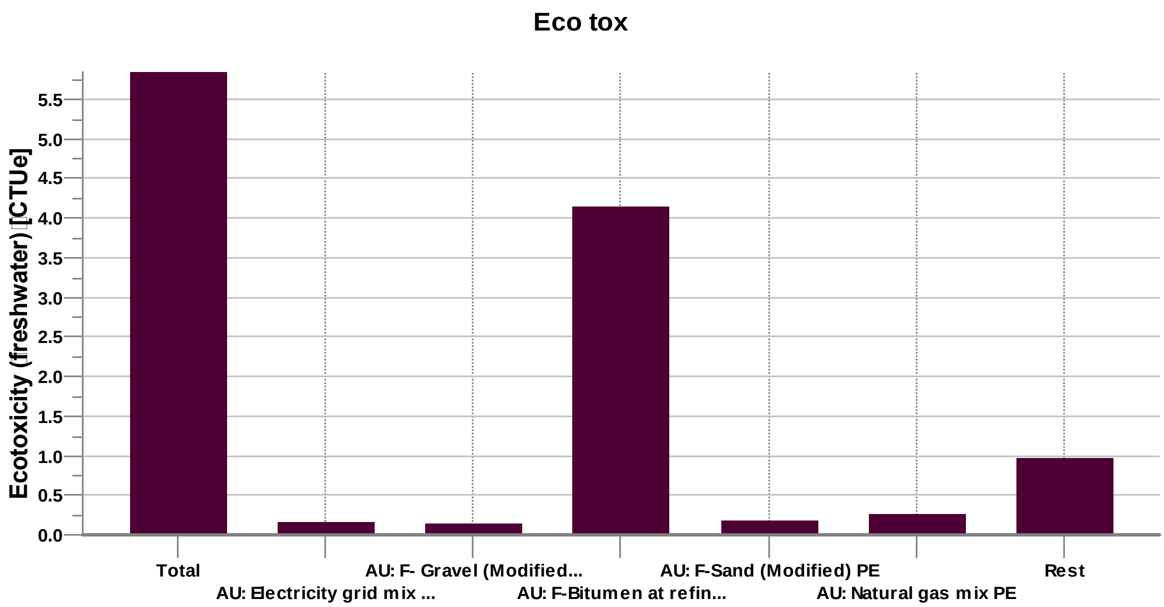

Figure 11.18: Impact of Different Phases of Asphalt Mixture Production on Ecotoxicity (Fresh Water)

In addition, the selected asphalt mixtures based on the experimental research work are compared with each other in terms of different impact categories considered in this study. This comparison is illustrated in Figures 11.19 to 11.31.

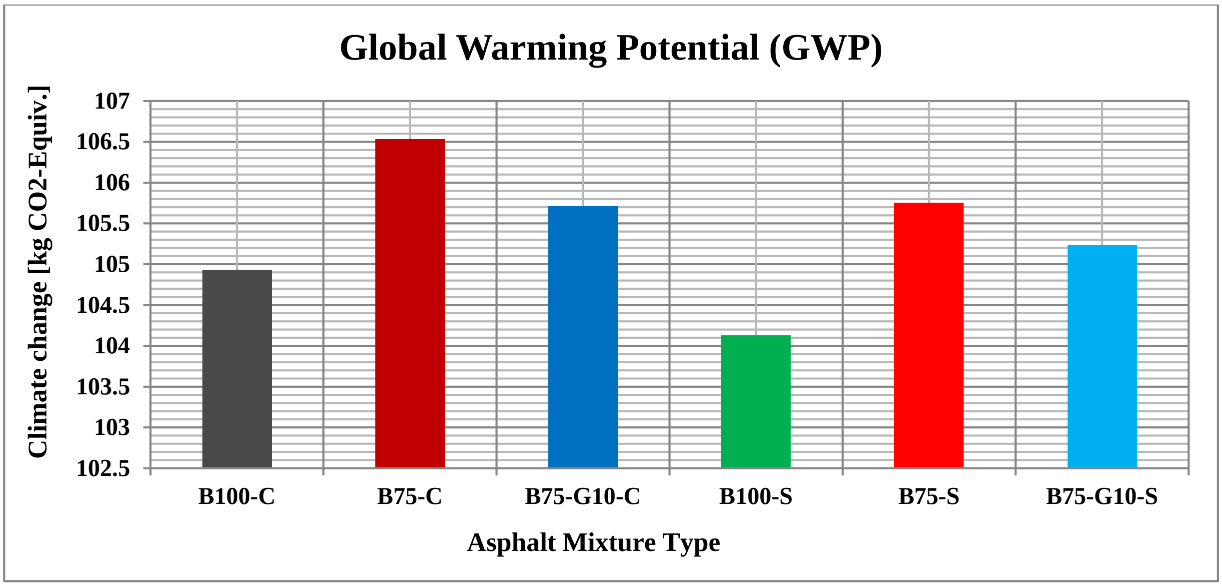

Figure 11.19: Impact of Different Types of Asphalt Mixture on Global Warming

As can be observed in Figure 11.19, the asphalt mixtures produced through the ASMM mixing method (S) have significantly less adverse effect on climate change in comparison with their corresponding asphalt mixtures produced by conventional mixing method (C).

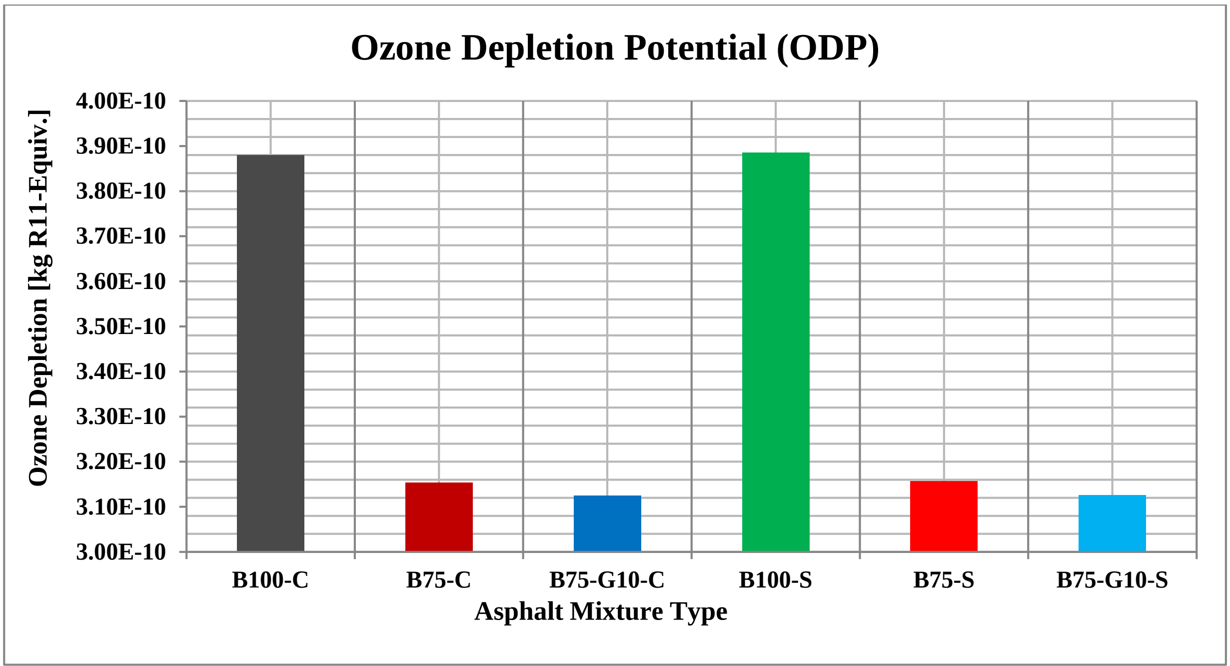

Figure 11.20: Impact of Different Types of Asphalt Mixture on Ozone Depletion

Figure 11.20 clearly demonstrates that asphalt mixtures incorporating waste materials have minor impact on ozone depletion compared to asphalt mixtures produced just by virgin materials.

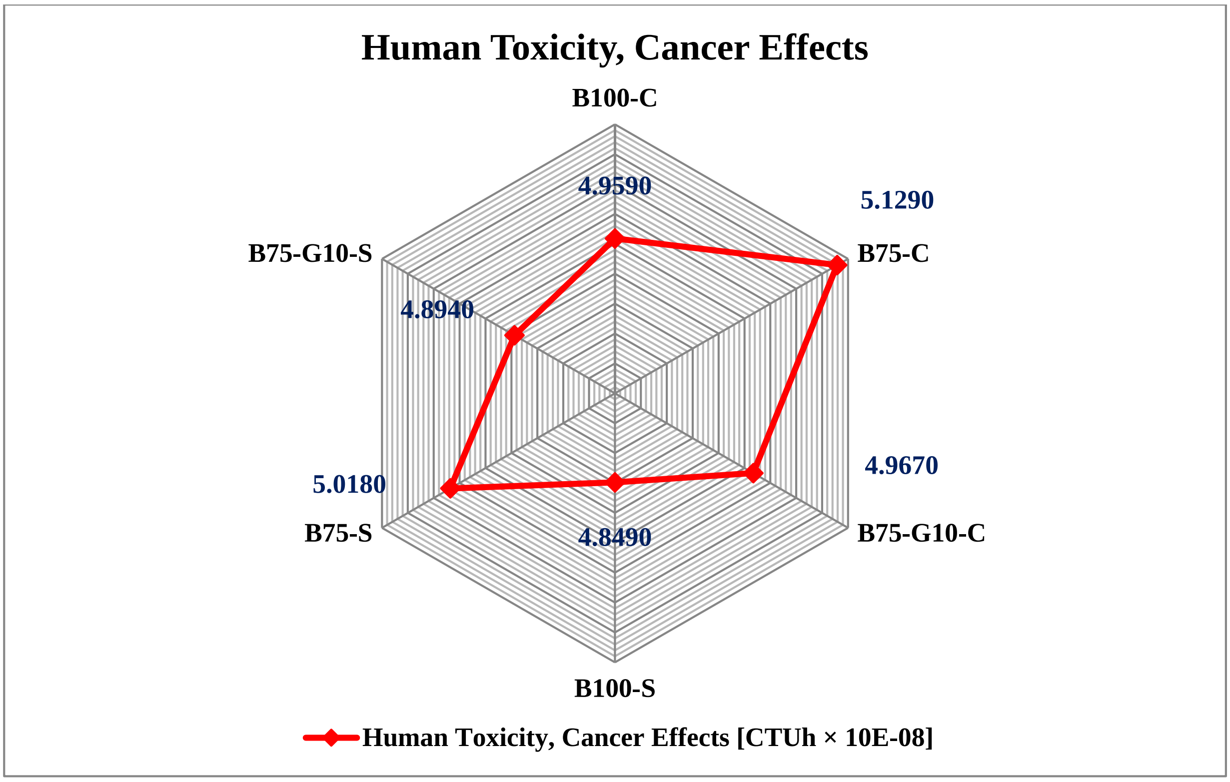

Figure 11.21: Impact of Different Types of Asphalt Mixture on Human Toxicity (Cancer Effects)

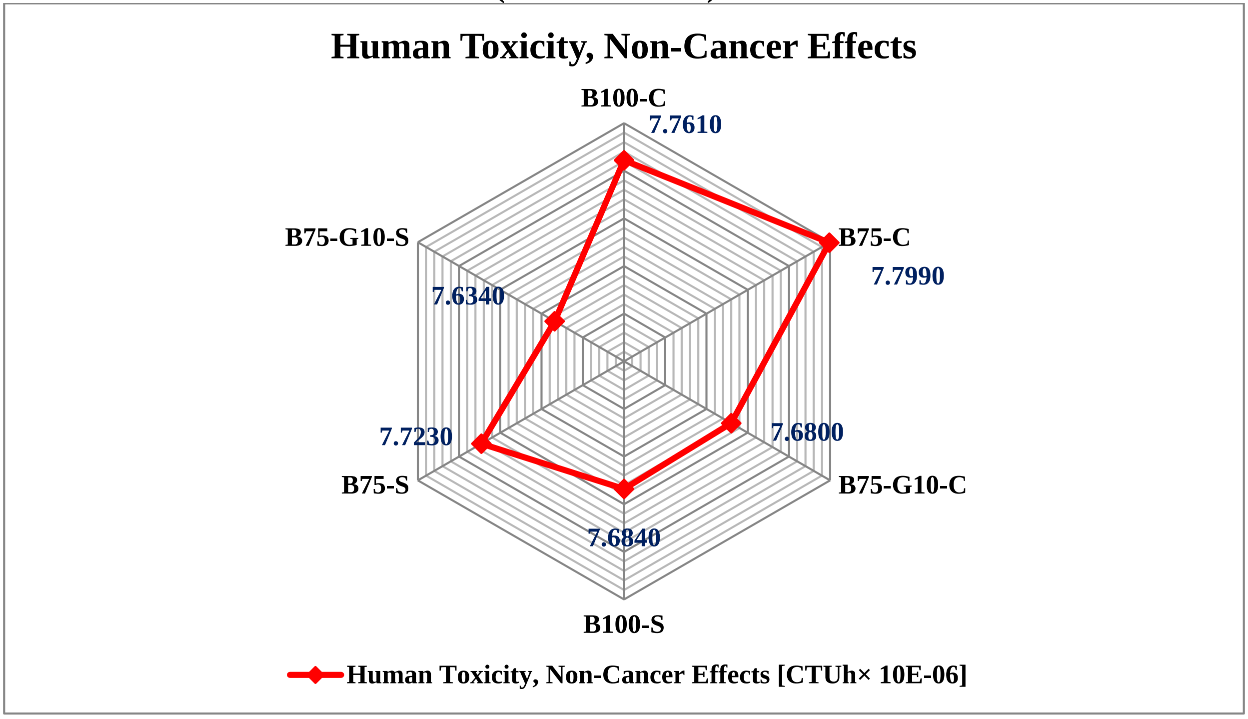

Figure 11.22: Impact of Different Types of Asphalt Mixture on Human Toxicity (Non-Cancer Effects)

As can be observed in Figures 11.21 to 11.22, asphalt mixtures produced through the new mixing method have less adverse impact in terms of human toxicity and no effect on human health (Figure 11.23).

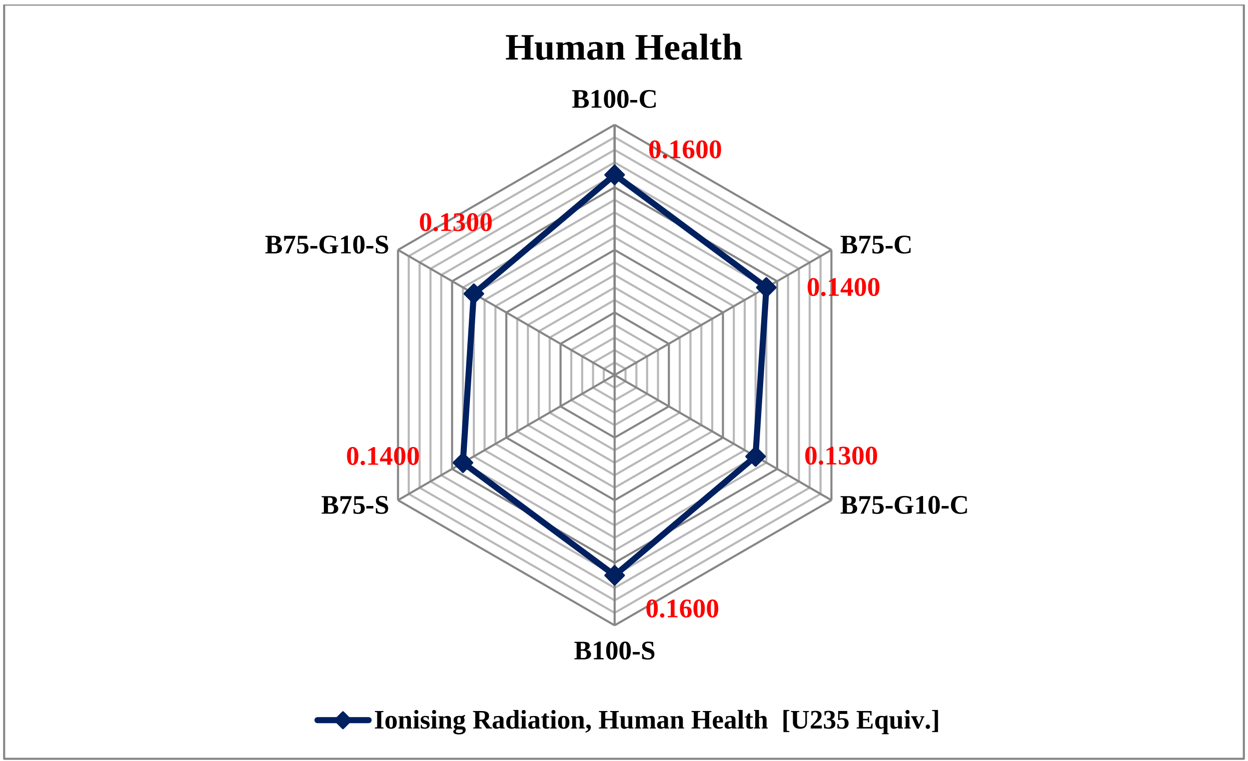

Figure 11.23: Impact of Different Types of Asphalt Mixture on Human Health

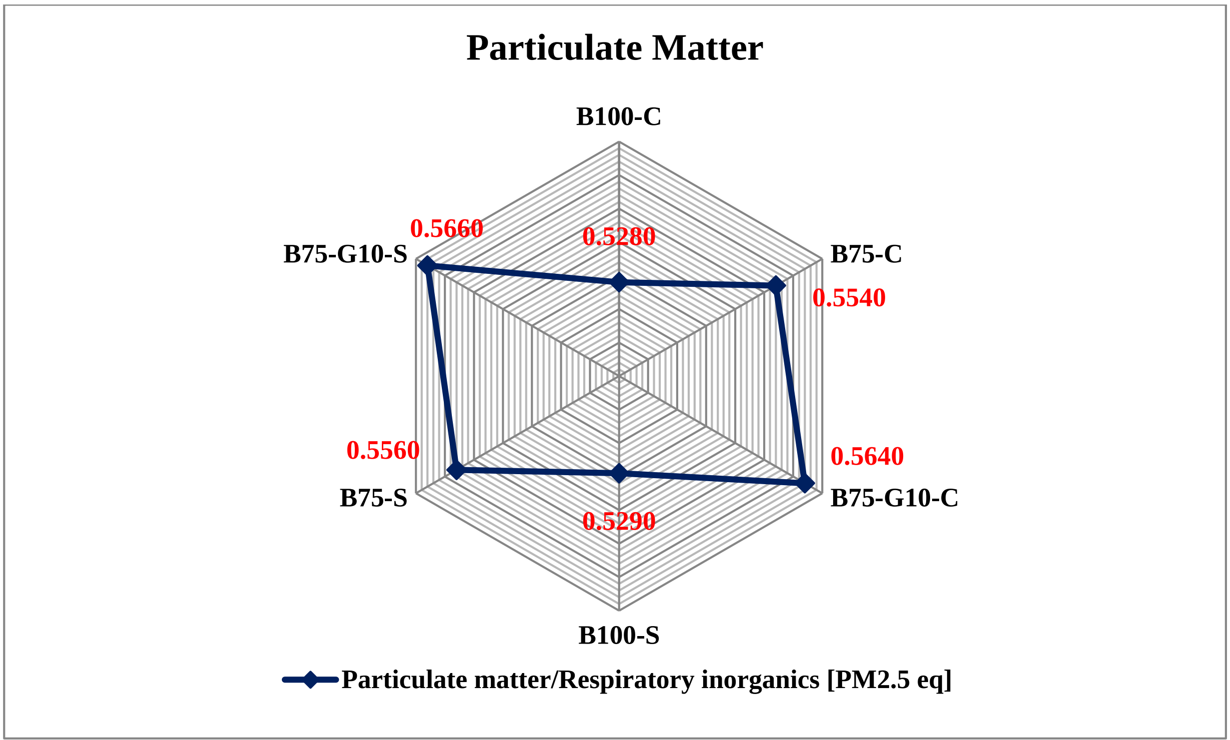

Figure 11.24: Impact of Different Types of Asphalt Mixture on Particulate Matter

Ionizing radiation received by body can cause damage to internal organs, tissues of the body and acute health effects. The LCA results indicate that the ionizing radiation decrease by the increase in the amount of recycled materials in asphalt mixtures so that the asphalt mixtures containing 25% RCA and 10% glass have the least ionizing radiation, as shown in Figure 11.24.

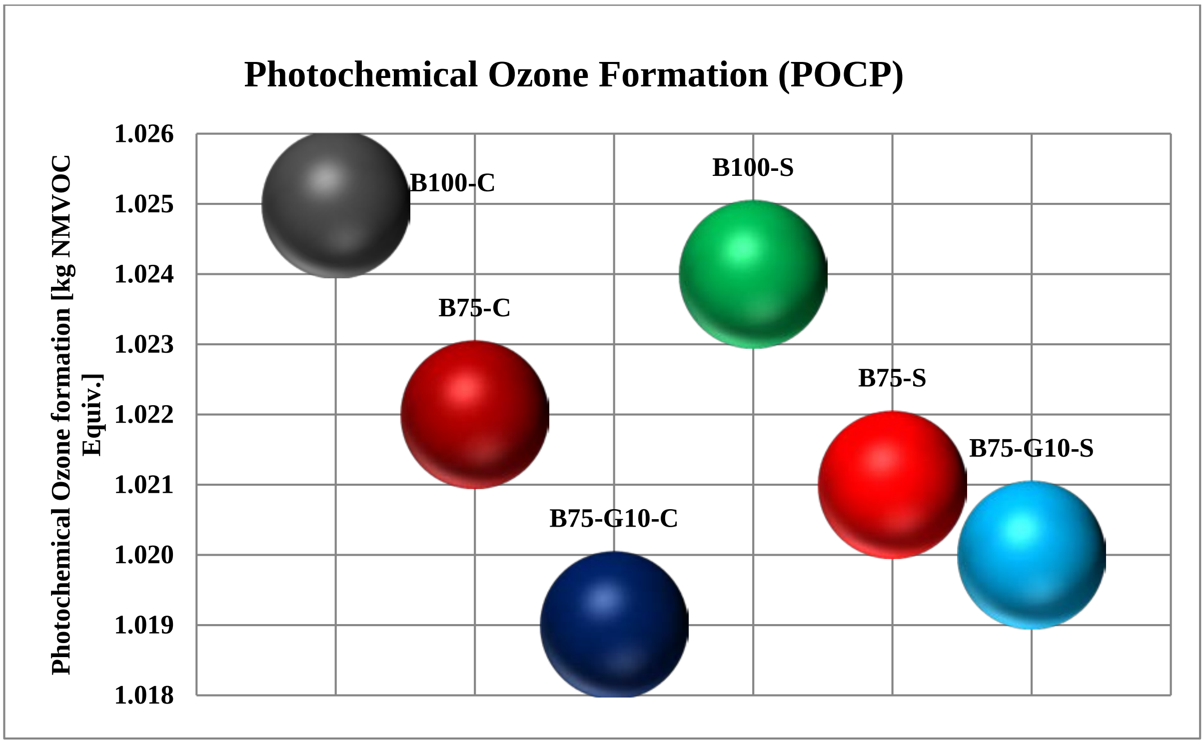

Figure 11.25: Impact of Different Types of Asphalt Mixture on Photochemical Ozone Formation

In addition, the photochemical ozone formation occurring through the reaction of nitrogen oxides (NOx), volatile organic compounds (VOCs) and carbon monoxide (CO) in the atmosphere in the presence of sunlight cause different health concerns such as asthma and other respiratory diseases as well as crop damage. The LCA results reveal that asphalt mixtures containing recycled materials again have less adverse impact in terms of photochemical ozone formation, as can be observed in Figure 11.25.

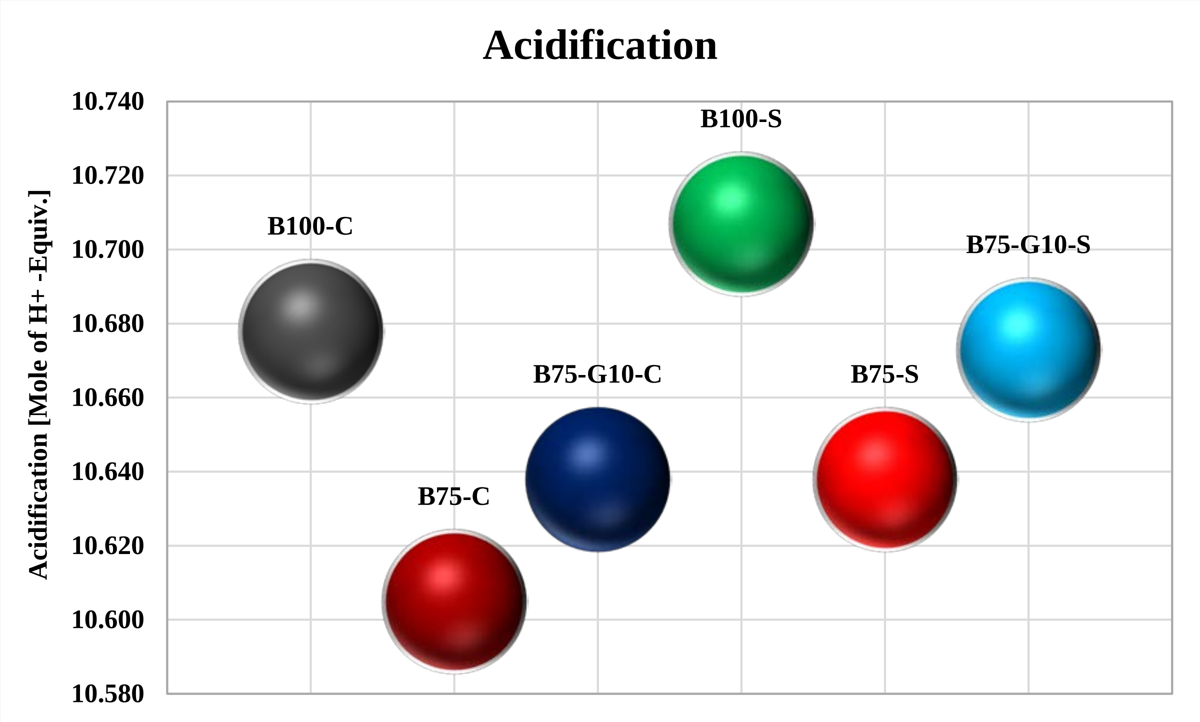

Figure 11.26: Impact of Different Types of Asphalt Mixture on Acidification

The acidification is closely linked to the greenhouse gas emissions and increase in absorption of carbon dioxide (CO_2) from the atmosphere by ocean resulting in decrease in pH of the oceans. This phenomenon has direct chemical adverse effects on ocean waters, and subsequently the food chain. As shown in Figure 11.26, asphalt mixtures containing recycled materials have less impact on ocean acidification.

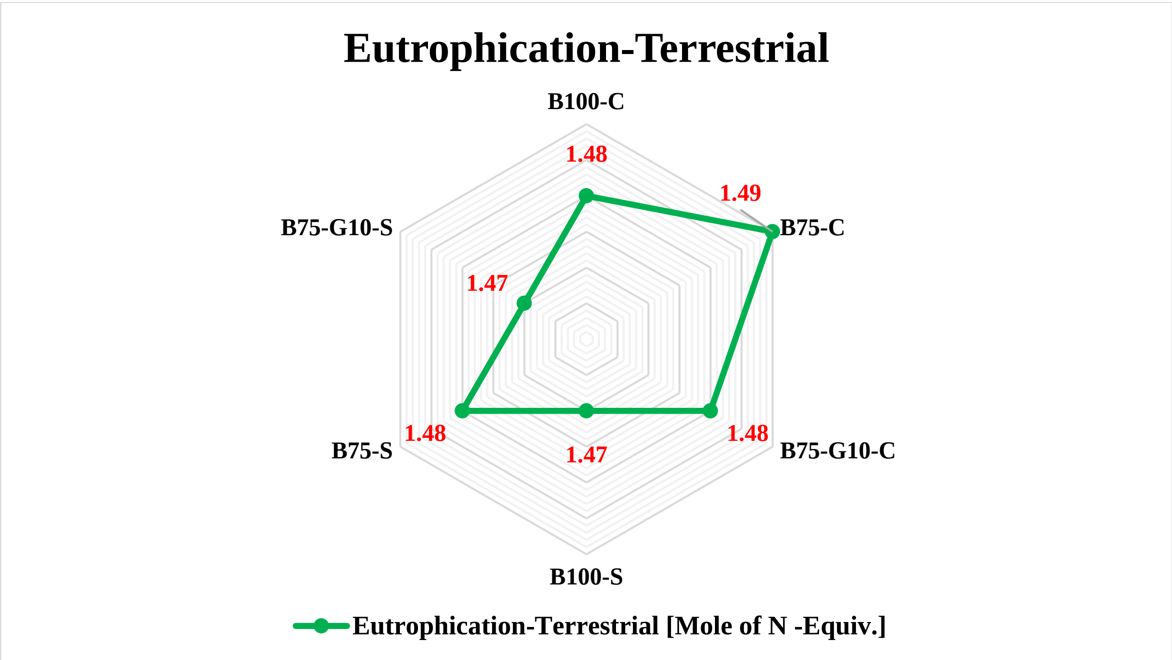

Figure 11.27: Impact of Different Types of Asphalt Mixture on Eutrophication (Terrestrial)

Eutrophication is the enrichment of an ecosystem with nutrients such as chemical compounds containing nitrogen, phosphorus. Eutrophication can cause some changes to the aquatic ecosystem including water quality deterioration due to the increased production of algae and aquatic plants, depletion of fish species or can result in terrestrial ecosystem damages such as changes to habitat structure and the vegetation types.

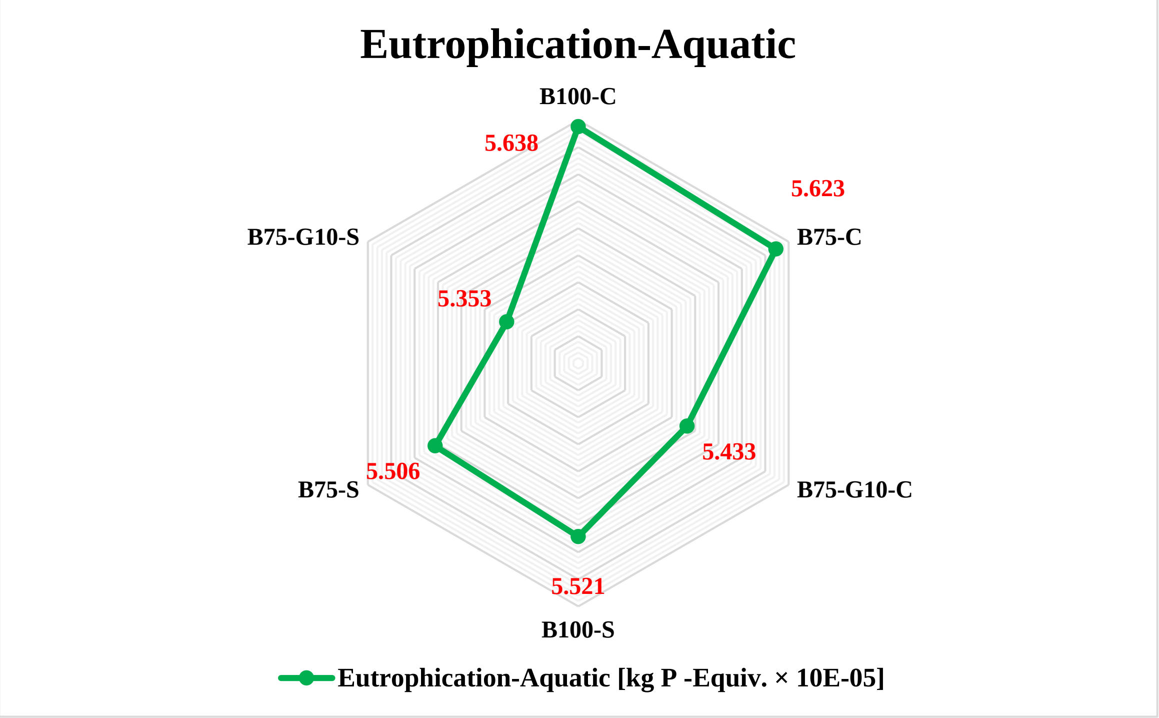

Figure 11.28: Impact of Different Types of Asphalt Mixture on Eutrophication (Aquatic)

As can be observed in Figures 11.27 and 11.28, the asphalt mixture containing RCA and glass produced through the ASMM mixing method is the most environmental friendly mixture in terms of eutrophication.

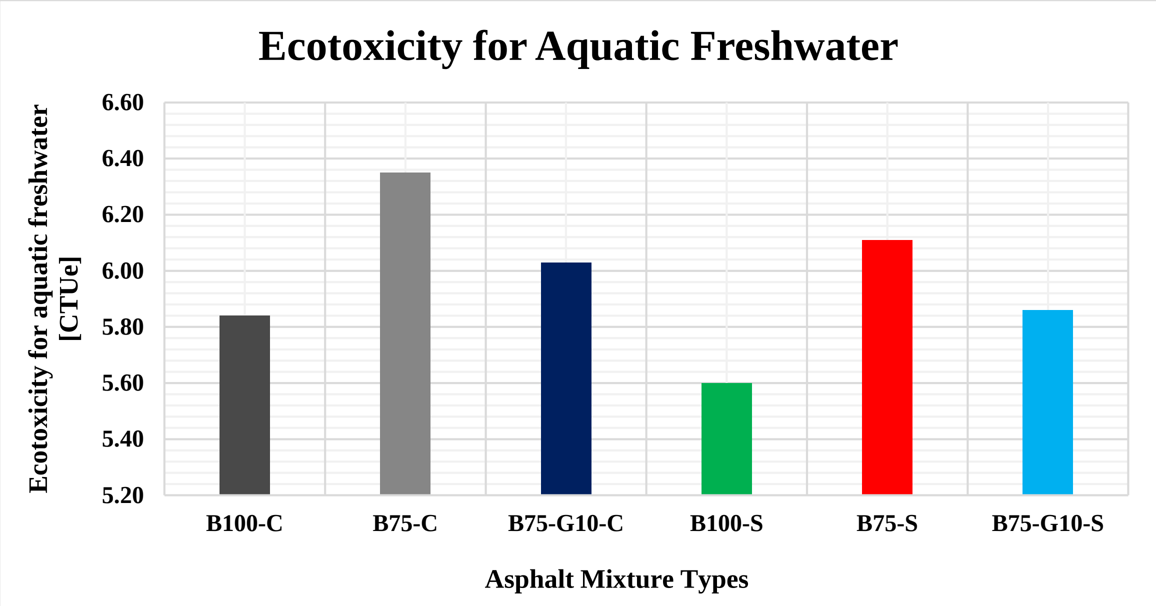

Figure 11.29: Impact of Different Types of Asphalt Mixture on Ecotoxicity (Aquatic Freshwater)

As can be observed in Figure 11.29, the asphalt mixtures produced by ASMM mixing method has less adverse impact in terms of aquatic ecotoxicity and hence less negative effect on the aquatic ecosystem and the organisms living in it.

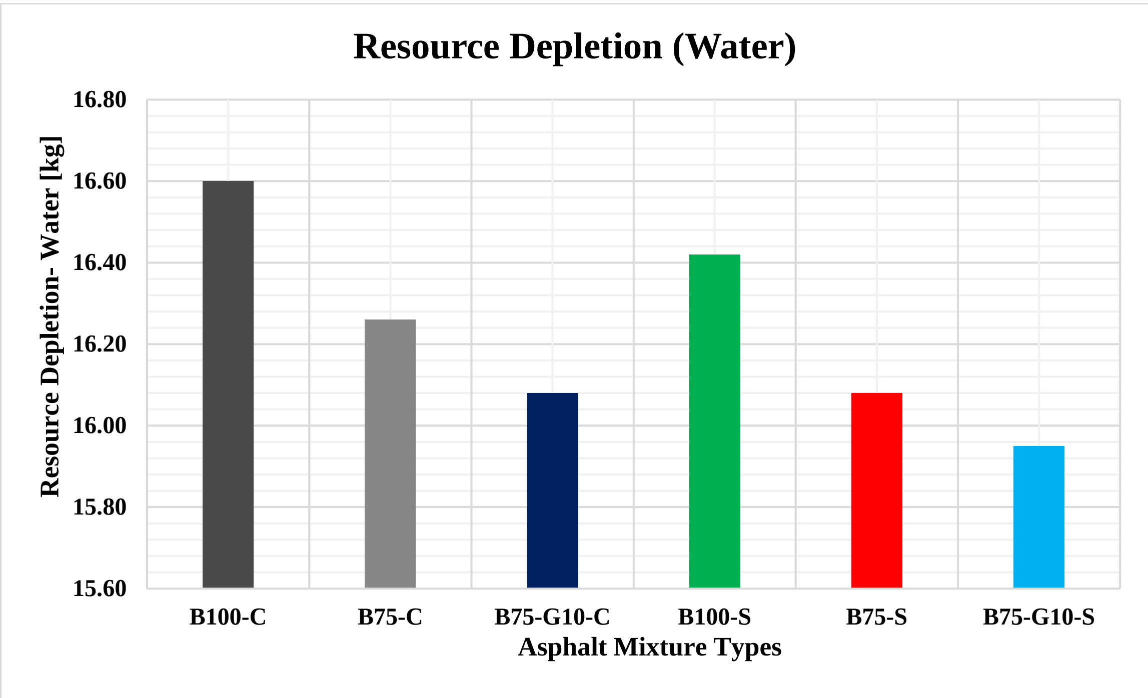

Figure 11.30: Impact of Different Types of Asphalt Mixture on Resource Depletion (Water)

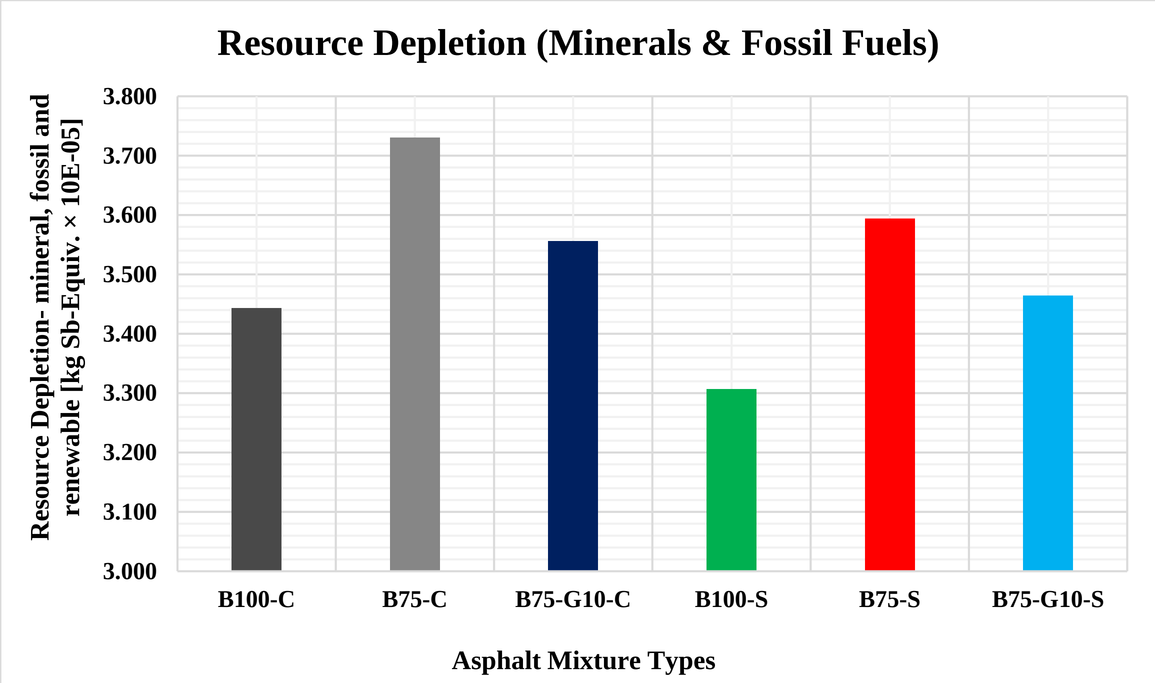

Figure 11.31: Impact of Different Types of Asphalt Mixture on Resource Depletion (Minerals and Fossil Fuels)

As shown in Figures 11.30 and 11.31, asphalt mixtures produced by ASMM mixing methods help in saving natural resources in comparison with their corresponding asphalt mixtures produced through the conventional method.

11.6. Cost Analysis

Referring to the report by Macromonitor (2013) regarding the cost analysis of infrastructure construction in Victoria, the single biggest cost component in an infrastructure construction is materials. This report also predicts that the infrastructure construction costs may increase by 3.6% per year (on average) over the next 10 years in Victoria, with materials cost being the biggest contributor to these future cost increases, as shown in Figure 11.32.

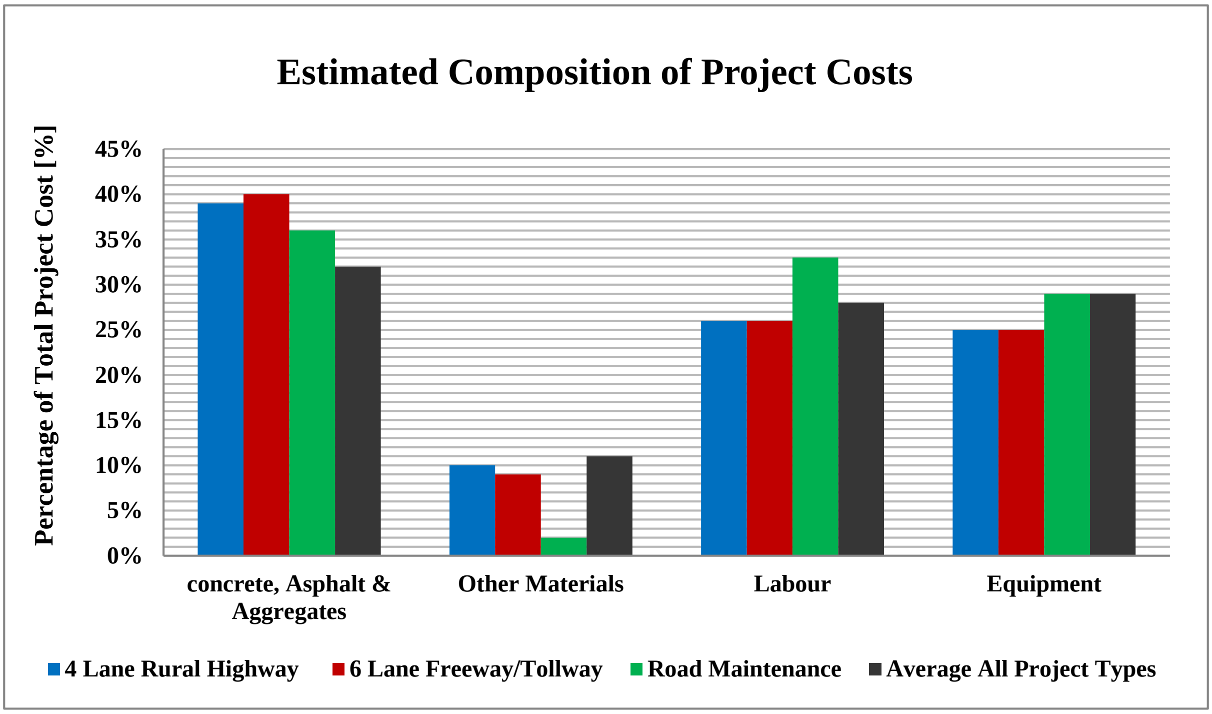

Figure 11.32: Estimated Composition of Project Costs

The Victorian example is an indicative of the infrastructure cost throughout the world meaning that the cost of providing public infrastructure for future demands will increase since construction of public infrastructure requires efficient supply of construction materials. In this regard, the location plays an important role in the construction materials supply efficiency since about 20% to 25% of the total cost of materials allocates to transportation. This demonstrates the significant impact of the transportation costs on total construction cost. For example, according to a report from Access Economics (2006), in Melbourne with many quarries which are located in the metropolitan area providing the average transport distance of 30 km from quarry to asphalt plant, the cost for materials delivery is 70% less than in Sydney with one remaining metropolitan quarry and hence the average transportation distance of 60 km. Therefore, it is essential to recognize the importance of construction materials which are locally supplied. In addition, the identification of new and innovative resources of construction materials (like recycled aggregates) is of high importance in this regard. Accordingly, cost analysis of different selected asphalt mixtures is considered as part of this research and the following section provides more details on asphalt mixture cost analysis.

11.6.1. Cost Analysis on Selected Asphalt Mixtures

The primary objective of this section is to investigate the benefits of using recycled materials and the new mixing method in asphalt mixtures in order to examine their influence on unit cost of asphalt mixture.

Table 11.6: Unit Cost of Materials Used in Asphalt Mixture Production

Material	Unit	Unit Price
Gravel	$/tonne	50
Sand	$/tonne	45
Portland Cement	$/tonne	0.165
Hydrated Lime	$/tonne	0.275
Bitumen	$/tonne	1025
RCA	$/tonne	20
Recycled Glass	$/tonne	20
Water	$/cubic meter	2.276
Diesel	$/litre	1.55
Electricity	$/kwh	0.29
Natural Gas	$/MJ	0.025

Considering this, based on the composition of different asphalt mixtures proposed in this research, the unit cost of asphalt mixture is calculated by multiplying the unit cost of material and energy (Table 11.6) by the total amount of energy and material in one tonne of asphalt mixture. Table 11.7 compares the results of cost analysis for asphalt mixtures made with recycled materials or through the ASMM mixing method with conventional asphalt mixtures containing only virgin materials and made with the conventional mixing method.

Table 11.7: Cost of Different Selected Asphalt Mixture per Tonne

Asphalt Mixture	Unit	Unit Price (AUD)	Initial Cost Comparison
B100-C	$/tonne	115.2	-
B75-C	$/tonne	119.0	3.30
B75-G10-C	$/tonne	114.4	-0.69
B100-S	$/tonne	112.1	-2.69
B75-S	$/tonne	115.9	0.61
B75-G10-S	$/tonne	112.4	-2.43

As can be observed in Table 11.7, the asphalt mixtures produced with ASMM mixing methods (excluding B75-S) have the maximum cost saving compared to conventional asphalt mixture (B100-C).

11.7. Case Study

To demonstrate the effect of asphalt mixture type on different aspects of environment, human health, and cost in a clearer way, the LCA framework has been employed for the same case study as the one considered in Chapter 10 for structural design in NSW, Australia. This section discusses the result of a comparative LCA and cost analysis on this case study to compare the application of different selected asphalt mixtures in road construction for their cost and environmental impact.

11.7.1. Case Study Background

The case study in this chapter focuses on the cost, energy and emissions regarding production of asphalt mixture (wearing course) required for a 10 km pavement section located at M2 Motorway in Sydney near North Rocks (opposite the Barclay Road), as shown in Figure 11.32. M2 Motorway is a six-lane motorway with 21 kilometer length linking the lower north shore and the north-west regions of Sydney.

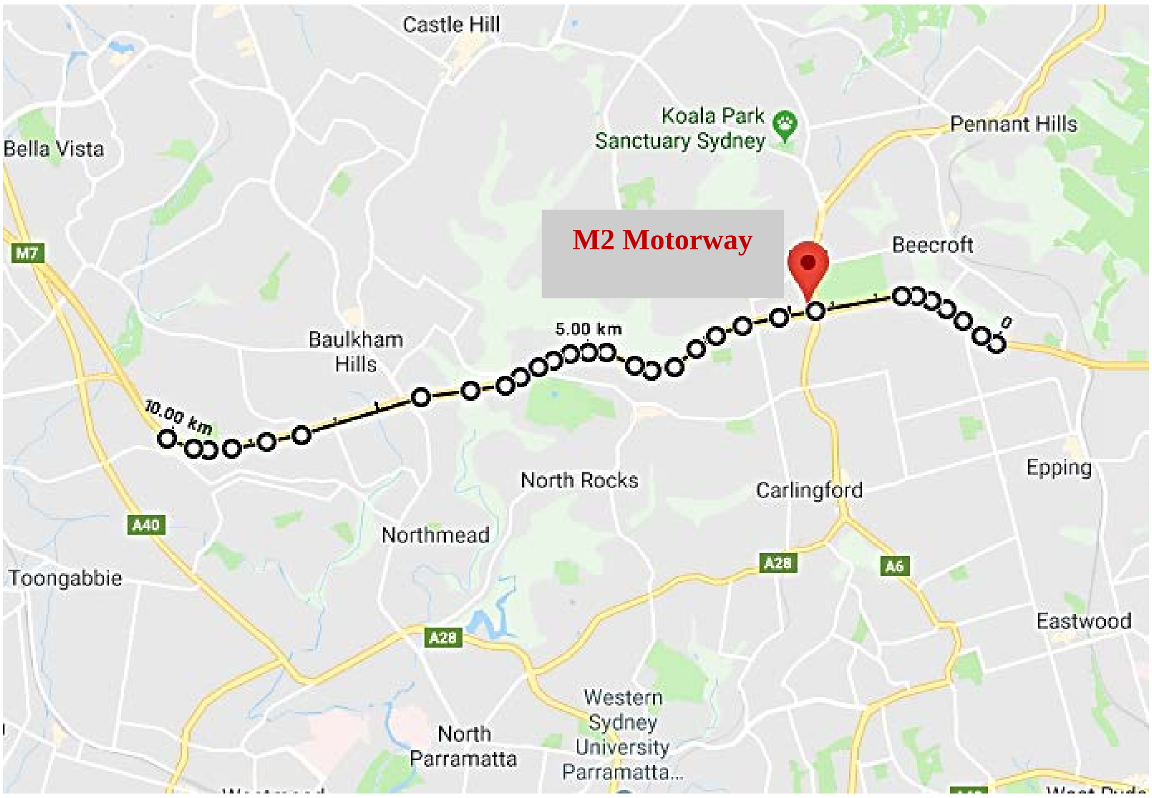

Figure 11.32: M2 Motorway Section Considered as Case Study

The goal of this section is to implement a comparative LCA and cost analysis in a realistic condition for different scenarios with different asphalt mixtures proposed by this research as wearing course. Based on the results of structural design (Chapter 10), the thickness of 50 mm has been assumed for the wearing course thickness.

11.7.2. Asphalt Tonnage

Materials composition together with pavement dimension considered in this case study will be used to determine the asphalt tonnage for LCI calculations. Data including the pavement surface area and asphalt layer thickness are presented in Table 11.8.

Table 11.8: Pavement Parameters Considered in This Case Study

Pavement Parameters	Value	Unit
Pavement length	10	km
Pavement width	21	m
Asphalt layer thickness (wearing course)	50	mm
Swelling factor	1.3	-

In order to calculate the quantity of asphalt mixture required in this case study, the parameter of Swelling Factor is defined which is the ratio of materials volume in loose state to their volume in compacted state. Table 11.9 gives the quantity of asphalt mixture required as wearing course for this case study based on different asphalt mixtures proposed in this research.

Table 11.9: Required Asphalt Mixture for Constructing the Wearing Course of the Pavement Considered in This Case Study

Asphalt Mixture Type	Density (gr/cm^3)	Tonnage (tonne)	Required Tonnage Comparison
Pavement 1 (B100-C as Wearing Course)	2.442	3333.33	-
Pavement 2 (B75-C as Wearing Course)	2.411	3291.02	-1.27
Pavement 3 (B75-G10-C as Wearing Course)	2.398	3273.27	-1.80
Pavement 4 (B100-S as Wearing Course)	2.445	3337.43	0.12
Pavement 5 (B75-S as Wearing Course)	2.418	3300.57	-0.98
Pavement 6 (B75-G10-S as Wearing Course)	2.408	3286.92	-1.39

As presented in Table 11.9, the amount of asphalt required for the same section of road is different depending on type of asphalt mixtures resulting from different density of asphalt mixtures. The amount of saving in required asphalt mixture is given in the last column of this table. This comparison shows that the utilization of recycled materials in asphalt mixture production even by neglecting the unit price of asphalt mixtures can still result in decreasing the required asphalt mixture for pavement construction.

As presented in Table 11.10, the results of cost estimation for the considered case study indicates that cost of asphalt mixtures containing 25% RCA is higher than the conventional asphalt mixture because of the high bitumen consumption of this type of asphalt mixture and the high unit cost of bitumen.

Table 11.10: Cost Estimation for Producing the Asphalt Mixture Required for Wearing Course of the Pavement Considered in This Case Study

Asphalt Mixture Type	Unit Cost ($/tonne)	Total Cost (AUD)	Cost Saving (AUD)
Pavement 1 (B100-C as Wearing Course)	115.2	3,839,996.2	-
Pavement 2 (B75-C as Wearing Course)	119.0	3,916,307.9	76,311.7
Pavement 3 (B75-G10-C as Wearing Course)	114.4	3,744,620.9	-95,375.3
Pavement 4 (B100-S as Wearing Course)	112.1	3,741,253.4	-98,742.7
Pavement 5 (B75-S as Wearing Course)	115.9	3,825,360.6	-14,635.5
Pavement 6 (B75-G10-S as Wearing Course)	112.4	3,694,498.1	-145,498.1

In addition, the replacement of 10% glass in asphalt mixtures together with 25% RCA results in a significant cost saving in asphalt mixture production. The highest cost saving would occur for asphalt mixtures containing RCA and glass produced through the ASMM mixing method (B75-G10-S). In other words, considering B75-G10-S as wearing course in a project of 10 km long will result in AUD145,500 cost saving, as shown in Table 11.10.

11.8. Results and Discussion

Roads are essential component in the infrastructure development and can be considered as assets to the society. A substantial quantity of materials and resources (aggregates, bitumen and additives) are required for roads construction and maintenance. In addition, production and processing of these materials consume considerable amounts of energy and resources. To conserve these resources, the efficient ways should be employed in order to minimize the waste generation, emissions, resources consumption and cost. To this point, it is required to use some environmental tools which are useful in determining and improving the efficient use of resources. LCA is a technique which is used in the evaluation of the environmental impacts associated with a product or a technology over its life cycle. In this chapter, life cycle assessment and cost analysis have been conducted on selected asphalt mixtures base on this experimental research work. The results of LCA model on different asphalt mixtures proposed in this study indicate that using recycled materials, and particularly together with ASMM mixing method, will result in significant cost savings, energy and water consumption, gas emission, and waste generation, confirming that utilization of recycled materials along with new mixing technologies can contribute to more sustainable asphalt mixtures.

Chapter 12

Conclusions

12.1. Summary

Over the past decades, growing population has created high demand for new road pavements. Due to considerable usage of various natural aggregates for constructing roads, these materials have started to deplete gradually. At the same time, waste materials are increasingly generated with the rapid growth in economy and continuously increased consumption. The growing quantities of waste materials, scarcity of natural resources and shortage of landfill spaces represent the importance of finding innovative ways of reusing and recycling waste materials. In this regard, utilization of waste materials in different pavements layers including the asphalt surface layer remains an attractive route for wastes because in most cases it is more economical than using the virgin materials. In addition, there is important sustainability benefits associated with the use of waste materials in pavement industry as it helps the environment by reducing resource extraction and the use of virgin materials, thereby reducing energy and water use, reducing harmful gas emissions and helping reduce waste to landfills. However, the selection of waste materials to be used for pavement construction, particularly asphalt surface layer, is of high importance as the application of waste should not adversely influence the structural and functional aspects of the pavements. Therefore, the ability to determine the best materials and the optimum content of materials becomes a key issue in the design of asphalt mixtures.

For conducting this research, a comprehensive study has been carried out on different aspects and methods associated with asphalt mixture design and incorporation of waste materials in asphalt mixtures. Among the waste materials, construction and demolition wastes, including Recycled Construction Aggregate (RCA) was selected to be used as part of coarse aggregates in asphalt mixtures as they constitute a major part of the municipal solid wastes in Australia. In addition, recycled glass was also selected as part of fine aggregate to compensate RCA for some of its shortcomings.

Furthermore, this study involves a detailed thermal study of crumb rubber and recycled HDPE using differential scanning calorimetry (DSC) and Thermogravimetric Analysis (TGA) equipment as well as their structural and microstructural analysis using infrared spectroscopy (FTIR) and scanning electron microscope (SEM) for utilization of these waste materials as binder modifier in asphalt mixtures.

This research also covers a statistical study on recycled construction aggregates produced in Sydney by collecting RCA samples over a twelve month period, classifying them into different groups, and conducting different physical tests on these samples to characterize their properties.

To achieve the goals of this research, an experimental program was set up to investigate the properties of asphalt mixtures considering various parameters and compositions. Moreover, different mixing methods were examined in this research for production of asphalt mixtures and the effects of these methods on asphalt mixtures characteristics.

An image processing software (ImageJ), a structural design software (CIRCLY 6.0) and a commercial life cycle assessment software (GaBi 6.0) were employed for the investigation of asphalt mixtures in terms of their performance response and their environmental and economical aspects. In general, this research covers several important parts as follows:

- Evaluation of physical and mechanical properties of RCA as part of coarse aggregate in asphalt mixture
- Conducting a statistical study on RCA by collecting RCA samples at different dates over one year and classifying them into different groups of sedimentary, metamorphic and igneous rocks
- Evaluation of physical and mechanical properties of recycled glass as part of fine aggregate in asphalt mixture
- Evaluation of structural and thermal properties of crumb rubber and recycled HDPE for utilization as binder modifier in asphalt mixture
- Investigation of the effects of RCA on volumetric parameters of asphalt mixtures
- Studying the utilization of recycled glass to optimize bitumen absorption of hot mix asphalt containing RCA
- Studying different approaches for determination of aggregate gradation to conclude the optimum gradation for designing the asphalt mixture in this research based on the packing theory and morphology framework
- Investigation of the effect of mixing method on volumetric properties and performance of asphalt mixtures
- Image processing analysis by using ImageJ software to characterize the aggregate and voids distribution through the asphalt mixtures produced by different mixing method
- Evaluation of the stiffness and resilient modulus of asphalt mixtures containing RCA and asphalt mixtures containing the combination of RCA and glass, produced by conventional and ASMM mixing method

- Investigation of the permanent deformation and fatigue behaviour of asphalt mixtures containing certain amount of RCA and recycled glass and produced by different mixing method by using CIRCLY 6.0
- Life cycle analysis of selected asphalt mixtures based on the experimental work results and their comparison with conventional asphalt mixtures by using GaBi 6.0
- Cost analysis of different asphalt mixtures proposed in this research and comparing them with conventional asphalt mixtures

All these parts can estimate different properties and characteristics of asphalt mixtures while they enable the reader to explore the role of various parameters and conditions on asphalt mixture.

12.2. Conclusions

As mentioned previously, asphalt mixtures containing different waste materials (i.e. RCA and glass) at various content and composition were made and studied in this research to estimate the asphalt mixtures performance and to determine the optimum content of selected waste materials in asphalt mixtures.

Since the aggregate properties are identified as the second most important parameter after gradation for the performance of asphalt mixtures (Oduroh et al., 2000), therefore, in this research, attempts were made to assess the properties of RCA for use in asphalt mixture as coarse aggregate, and recycled glass as fine aggregate and a comprehensive set of preliminary tests on RCA, glass and basalt as well as different mixes of these aggregates were conducted to evaluate their basic mechanical and physical properties. It was argued that information on these fundamental properties were paramount in designing durable and sustainable asphalt mixtures. To this end, different aggregates and aggregate mixes were investigated in this research to assess their suitability as coarse and fine aggregates in asphalt. In addition,

Based on these preliminary tests, it was concluded that:

- RCA has lower value of flaky and misshapen particles in comparison with RAP and basalt. This implies that asphalt mixtures containing a certain amount of RCA can have better workability, deformation resistance and compaction properties.
- RCA exhibits comparatively more absorption and wet/dry strength variation than conventional aggregate, while the results of other tests show that RCA still meets the requirements for aggregate in asphalt mixtures. Cracks and adhering mortar and

cement paste can be the main reasons for the high water absorption of RCA which needs to be compensated for during mix design.

- The results of water absorption and particle density test on different mixes of coarse aggregates revealed that RCA increase will increase water absorption of the mixture. Therefore, the selection of optimum combination of RCA and other aggregates is required to satisfy the relevant standards' requirements.
- Regression analysis applied to the results of water absorption tests on different combination of RCA and basalt, indicates that mixing of almost 8% of RCA with natural aggregates will provide the standard water absorption limit of 2%.
- Since, according to Austroads (2014), the aggregates with water absorption of between 2% and 4% of their mass should be carefully examined by other tests, this standard limit will allow further investigation of the application of up to 50% of RCA in mixtures because based on the water absorption results, it was observed that the combination of 25% RCA and 75% basalt would provide water absorption of 2.93%, and the combination of 50% RCA and 50% basalt would provide water absorption of 3.71%, which are still in the range that requires further research for their water absorption properties.
- The results of water absorption and particle density on recycled glass shows that recycled glass has negligible water absorption and hence can be an adequate material for combination with RCA to compensate for high water absorption of RCA.

Based on the results obtained from the aggregate specification tests on unbound RCA, glass and virgin aggregate, it was concluded that the selection of optimal combination of RCA and other aggregates is required to satisfy the relevant standards' requirements while taking advantage of other strong points of RCA. To this point, different asphalt mixtures incorporating substitutions of coarse virgin aggregate with 25% and 50% RCA were prepared and evaluated through primary tests for their volumetric properties. The tests on asphalt mixtures containing RCA indicated that:

- Asphalt mixtures containing RCA have lower bulk density, VMA, VFB and BFI than control mixes, whereas the air voids are higher for mixtures containing RCA. Lower bulk density of RCA will result in cost reduction, as asphalt jobs are mostly measured in cubic metre and materials are purchased in tonnes.
- The results of tests on different asphalt mixtures containing different percentages of RCA revealed that an increase in RCA will increase optimum bitumen content of

the mixtures. Therefore, the selection of optimum combination of RCA and other aggregates is required to satisfy the relevant requirements.

- The optimum bitumen contents were found to be 5.1%, 5.8% and 6.2% of C320 bitumen for control samples, mixtures containing 25% RCA and mixtures made with 50% RCA, respectively.
- The results show that asphalt specimens containing up to 22% RCA provide properties which are within the acceptable limits recommended by Australian Standards. Higher substitution of virgin aggregates with RCA beyond 22% will lead to a failure of the specification criteria and higher bitumen absorption.

The result of primary tests on asphalt mixtures containing RCA showed that Asphalt mixtures made with RCA have the problem of high bitumen absorption. Therefore, in the next step, the effects of glass on the bitumen absorption and volumetric properties of asphalt mixtures containing 25% and 50% RCA were studied. Three glass contents of 0%, 10%, and 20% in terms of the total weight of fine aggregates were used in the mixture designs for preparing specimens containing 0%, 25% and 50% RCA. Different types of tests including volumetric analysis tests were carried out on asphalt mixtures in accordance with Australian standards. The test results reveal that:

- Utilization of recycled glass with very low water absorption in asphalt mixtures with different combination of RCA was observed to reduce the bitumen absorption of these asphalt mixtures.
- The results of tests on different asphalt mixtures containing RCA and glass indicate that the bitumen absorption decreases with glass increase in asphalt mixtures. In other words, asphalt mixtures containing glass have lower optimum bitumen content in comparison with asphalt mixtures without glass. So that asphalt mixtures containing 75% RCA with 10% and 20% glass have optimum bitumen content very close to optimum content of control samples (asphalt mixtures without any recycled materials).
- The results of tests on asphalt mixtures containing RCA and glass at different rate of bitumen content reveals that air void, VMA and bulk density are lower than the corresponding values for asphalt mixtures containing RCA without glass, whereas introducing glass in the asphalt mixtures increases VFB in asphalt mixtures containing both RCA and glass than asphalt mixtures containing only RCA.
- The results of volumetric properties of all asphalt mixtures at their optimum bitumen content indicates that asphalt mixtures made by combining 75% RCA and 10% glass

is the most comparable mixture to control samples in terms of optimum bitumen content and volumetric properties requirements.

In addition, as the mixing procedure of asphalt mixtures can influence different characteristics of asphalt mixtures, and subsequently the final performance of asphalt mixture, different samples prepared by different mixing methods (i.e. conventional, sequential and ASMM mixing method). Furthermore, due to the importance of aggregate gradation in asphalt mixtures particularly the mixtures prepared by new mixing methods, different gradation approaches were studied and a framework for designing asphalt mixtures that involve proper aggregate gradation by considering aggregate interlock and aggregate packing was developed in order to design a mixture that meets all required criteria and provides acceptable performance. The results of this study and tests for evaluation of volumetric properties of asphalt mixtures prepared by new mixing methods and considering the gradation framework indicated that:

- Asphalt mixtures produced by new mixing methods, particularly ASMM method, have lower air voids and require less bitumen content. This advantage can be particularly useful in compensating for the high bitumen absorption of the asphalt mixture containing RCA.
- The new mixing methods involve the reduction in temperature by about 30°C for bitumen conditioning and mixing which results in energy saving and reduction of fume emissions.
- The results of image processing conducted on these samples to show the influence of preparation method on internal structure of asphalt mixtures by using ImageJ software revealed that mixtures prepared through the ASMM mixing method have less air voids and an even distribution of aggregates and air voids, which can have substantial effect on the final performance of asphalt mixtures.

Based on the result of primary tests, the most acceptable samples were selected for estimating their resilient modulus through different empirical methods and the indirect tensile test. The study of resilient modulus of asphalt mixtures showed that:

- The resilient moduli obtained from empirical methods are lower compared to the measured values through Indirect Tensile Test. This shows that in cases where the measured resilient modulus value is not available in the design phase of pavements, the empirical equations can hardly be used to predict the actual resilient modulus.

- The results of resilient modulus of different samples reveal that the empirical methods still can effectively be used for comparison of the stiffness of different samples.
- The results of indirect tensile test showed that samples containing RCA have higher resilient modulus in comparison with control sample due to the particle shape and lower value of flaky and misshapen particles.
- Preparation of asphalt mixtures through ASMM mixing method can significantly increase the resilient modulus and hence improve the performance of asphalt pavement under traffic loading.
- The stiffness of mixtures containing RCA and prepared through the ASMM mixing method is quite higher compared to the other asphalt mixtures.

An integral part of the asphalt mixture design process is the assessment of how well the asphalt mixture will perform in the pavement structure. In this research, the structural design of pavement was conducted through mechanistic design based on the relevant Australian standard. The analysis was performed for different pavements with different wearing courses made of most acceptable asphalt mixtures determined through laboratory investigation conducted in this research and by employing the software CIRCLY 6.0 for a section of the M2 motorway in Sydney and the weigh-in-motion (WIM) survey data collected throughout Australia in 2010.

- The result of this investigation indicated that asphalt mixtures proposed in this research; particularly the mixtures prepared with ASMM mixing method containing 25% RCA substantially increase the allowable loading against asphalt fatigue and rutting in comparison to conventional asphalt mixtures.
- The asphalt mixtures containing RCA and prepared with ASMM mixing method decrease the layer thickness due to their higher resilient modulus.
- The increase in the allowable loading and decrease in the layer thickness both result in significant savings in cost and resource consumption.

Although the application of recycled materials has considerable advantages in reduction of virgin materials consumption and emissions to air, In spite of these encouraging benefits, the utilization of recycled materials, based on limited factors such as emissions reduction or material consumption reduction, may not be advisable to key decision makers since the recycled materials promotion requires a thorough picture of this technology as well as the consideration of all critical environmental factors. Accordingly, a quantitative and comparative life cycle assessment of asphalt mixture incorporating recycled materials was conducted in this research to evaluate both potential environmental and economic impacts of different asphalt mixtures over their entire life cycle by employing GaBi 6.0 software. The result of LCA model on different asphalt mixtures proposed in this study indicates that:

- Using recycled materials and ASMM mixing method significantly decreases the energy and resource consumption, gas emission, and waste generation.
- The positive influence of utilization of recycled material and new mixing method was clearly identified in the result of LCA model for thirteen different impact categories (i.e. ozone depletion, global warming, resource depletion, ecotoxicity, human toxicity, ionizing radiation, particulate matter, acidification, photochemical ozone formation and eutrophication) considered in this study, revealing that utilization of recycled materials along with new mixing technologies can contribute to more sustainable asphalt mixtures.
- Using recycled materials and particularly together with ASMM mixing method will result in cost savings of about 3.8%.

In addition, as binder plays a vital role in the final performance of the asphalt mixture, an understanding of modified binder properties is essential in designing an asphalt mixture. Therefore, as part of this research, High Density Polyethylene (HDPE) and crumb rubber were evaluated as possible binder modifiers by means of advanced thermal analysis including differential scanning calorimetry (DSC) and thermogravimetric analysis (TGA) and using scanning electron microscope (SEM) and infrared spectroscopy (FTIR). This investigation showed that:

- From DSC, it was shown that the melting point (T_m) values obtained for blends containing 2% of HDPE and 8% rubber is higher than the corresponding value obtained for neat bitumen. Therefore, the addition of HDPE and rubber into the blends provokes an increase in the T_m values of bitumen. The changes in transition temperatures of blend can be attributed to a certain level of miscibility between the additives (i.e. HDPE and rubber) and bitumen.
- The TGA analysis revealed that the analysed waste materials all degrade at temperatures above 200°C and, therefore, should be adequate for bitumen modification.
- From the results, it was observed clearly that HDPE followed by bitumen have higher thermal stability than crumb rubber.

This research work and the experimental results further confirmed the importance of developing a suitable mix design containing optimum content of waste materials before deploying this technology as an alternative option for asphalt production. It also highlighted the significant positive impact that the mixing procedure can have on the final performance of asphalt mixture.

12.3. Recommendations for Future Investigations

Although many laboratory and field investigations have been already performed on the performance of asphalt mixtures made with recycled materials such as RCA, recycled glass, plastic and crumb rubber, more studies are still required to deal with the challenges of this sustainable approach for further use. In this regard, a set of recommendations are provided for researching the engineering properties and other aspects of this technology, as follows:

- This research concluded that the utilization of up to 25% RCA and 10% glass provides asphalt mixtures with improved performance and comparable bitumen absorption with conventional asphalt mixtures. Further studies are required on the moisture susceptibility of asphalt mixtures incorporating RCA.
- The resilient moduli of asphalt mixtures containing RCA as well as combination of RCA and glass were investigated through indirect tensile tests and empirical methods. The evaluation of dynamic modulus of these asphalt mixtures and developing master curves in terms of the variations of the load and environmental condition is another subject for further investigation.
- The incorporation of waste glass in asphalt mixtures affects the bitumen absorption of mixtures, and therefore can compensate for the high bitumen absorbing RCA. This research investigated the effect of glass on asphalt mixtures containing RCA considering three different percentages. However, it is recommended to investigate the effect of glass size, colours of glass and the glass content in asphalt mixtures.
- Further studies are also suggested on the performance of asphalt mixtures incorporating RCA and glass in terms of moisture susceptibility.
- The performance of asphalt mixtures containing glass and RCA, in terms of the dynamic modulus is suggested as another possible subject for further study.
- Further investigation is required on the fatigue behaviour and also ageing of asphalt mixtures containing RCA and combination of RCA and glass.

- This research studied the thermal characteristics of modified binder composed of 2% HDPE and 8% crumb rubber from recycled vehicular tyres. Investigation of thermal characteristics of other blends with different composition and size of materials can be another topic for further investigation.
- Evaluation of volumetric and mechanistic properties of asphalt mixtures containing plastic (HDPE) and crumb rubber using wet process under different combination and percentages is required for further investigation in order to determine the optimum content of crumb rubber and HDPE as binder modifier.
- Evaluation of volumetric and mechanistic properties of asphalt mixtures containing plastic (HDPE) and crumb rubber using dry process under different combination and percentages is required for further investigation in order to determine the optimum content of crumb rubber and HDPE as part of aggregate.
- Asphalt mixtures containing recycled construction aggregates (RCA) have the problem of high bitumen absorption. This shortcoming of RCA can be compensated through different approaches. The evaluation of the properties of asphalt mixtures containing RCA coated with plastic is another subject for further investigation in order to characterize the effects of coating RCA aggregates with different types of plastic (HDPE, LDPE, PET, and PP) on the bitumen absorption and volumetric properties of asphalt mixtures.
- The performance of an asphalt mixture is considerably influenced by the physical, mechanical, and chemical properties of the filler. The application of recycled and secondary filler materials in the asphalt mixtures is an efficient, environmental friendly and cost effective way to improve the asphalt mixtures properties and solve the real world problems of industry. Investigation of the effects of using different types of fillers on the performance of asphalt mixtures is another area requiring further investigation.
- Evaluation of volumetric and mechanistic properties of asphalt mixtures containing different types of plastic (HDPE, LDPE, PET, and PP) using the wet process under different size and percentages is another subject requiring for further investigation in order to determine the best type and the optimum content of plastic as binder modifier.
- Evaluation of volumetric and mechanistic properties of asphalt mixtures containing different types of plastic (HDPE, LDPE, PET, and PP) using the dry process under

different size and percentages is required for further investigation to determine the optimum content and the best type of plastic as part of aggregate.

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