Republic of Iraq

Ministry of Higher Education & Scientific Research

University of Technology

Building and Construction Engineering Department

Roads and Bridges Engineering Branch

Mechanistic-Empirical Mixtures Design for Hot Mix Asphaltic Pavement Recycling

A Thesis

Submitted to the Building and Construction Engineering Department/University of Technology in a Partial Fulfillment of the Requirements for the Degree of Master of Science in Highways and Transportation Engineering

By

Yaseen Ata Zuhair

B.Sc. in Building and Construction Engineering, 2015

Supervised by

Prof.	**Assist. Prof.**
Dr. Mohammed Yousif Fattah	**Dr. Zaynab I. Qasim**

October, 2017 **Muharm, 1439**

سورة طه : الاية ١١٤

سورة البقرة : الاية ٣٢

Dedication

TO MY FAMILY

WITH ALL OF

MY LOVE AND

RESPECT

Yaseen Ata Zuhier

ACKNOWLEDGEMENT

Praise be to Allah who gave me the ability and the desire to complete this work in spite of all the obstacles and impediments in the way of its completion.

I would like to express my deep appreciation to my supervisors Prof. Dr. Mohammed Yousif and Assist. Prof. Dr. Zaynab Ibrahim who gave me the opportunity to work on this very interesting and meaningful research project. Their professional support, guidance, useful suggestions, patience and most importantly encouragement during this research project was very important to the successful completion of the thesis in spite of all difficulties.

I would like to thank the Head of Building and Construction Engineering Department and all members of the staff of department in University of Technology for their supporting during the period of this study especially to staff of Asphalt Laboratory for their help in using the various facilities.

Thanks and gratitude are extended to my colleagues, for their valuable assistance throughout this work.

Finally, I am grateful to my dear mother, father, brothers, sister and friends, who have given me the opportunity to complete this study. Their love and support have been a source of strength and motivation.

Yaseen ,2017

CERTIFICATION

We certify that this thesis entitled "**Mechanistic-Empirical Mixtures Design for Hot Mix Asphaltic Pavement Recycling**" submitted by **Yaseen Ata Zuhier** is prepared under our supervision at Roads and Bridges Engineering Branch, Building and Construction Engineering Department, University of Technology in partial fulfillment of the requirements of the degree of Master of Science in Highways and Transportation Engineering.

Signature:	Signature:
Name: Prof. Dr. Mohammed Y. Fattah	Name:Assist.Prof. Dr. Zaynab I. Qasim
(Supervisor)	(Supervisor)
Date : / / 2018	Date: / / 2018

In view of the available recommendations, I foreword this thesis for debate by examining committee.

Signature:

Name: Prof. Dr. Mohammed Y. Fattah

(Head of the Roads and Bridges Engineering Branch)

Date: /   / 2018

EXAMINATION COMMITTEE CERTIFICATE

We certify that we have read this thesis entitled **" Mechanistic-Empirical Mixtures Design for Hot Mix Asphaltic Pavement Recycling"** and as examining committee, examined the student **Yaseen Ata Zuhier** in its content and in what is connected with it, and that in our opinion it meets the standard as a thesis for the degree of Master of Science in Highway and Transportation Engineering.

Signature:
Name: Assist. Prof. Dr. Zainab Ahmed Al-Kaissi
Date: / / 2018
(Chairman)

Signature:
Name: Assist. Prof. Dr. Karim H. Ibrahim Al Helo
Date: / / 2018
(Member)

Signature:
Name: Lecturer Dr. Ali Majeed Al-Dahawi
Date: / / 2018
(Member)

Signature:
Prof. Dr. Mohammed Y. Fattah
Date: / / 2018
(Supervisor)

Signature:
Name: Assist. Prof. Dr. Zaynab I. Qasim
Date: / / 2018
(Supervisor)

Approve by the Dean of Building and Construction Engineering Department

Signature:
Assist. Prof. Dr. Hasan Hamodi Joni
Date: / / 2018

Abstract

As the process of repair and construction of highways expands, the cost of pavement materials increases and there is a lack of resources for quality materials. The recycling process is one of the important solutions to this problem. This process produces a sustainable pavement using aged materials that can be milled from the pavement, and these materials can be blended with original materials for the production of recycled mixtures.

The prime objective of this study is to assess the performance of the recycled asphalt concrete mixtures through investigating the effect of different percentages of the Reclaimed Asphalt Pavement (RAP) on its performance, studying the effect of variables on the asphalt concrete mixtures against stability, retained stability, compressive strength, dynamic modulus, and moisture sensitivity. For these aims and to conduct the experimental part of this study; available local materials were used including two types of asphalt binder: (40 -50) and (60-70) penetration, aggregate with nominal maximum sizes gradation of 12.5 mm for Surface layer and limestone dust used as mineral filler, while the old materials were milled from the field containing RAP materials at four different levels of addition (i.e. 7%, 13%, 19%, and 25%), manufactured to a supplied specification, modified as appropriate for the recycled material.

The Superpave mix design system was adopted with varying volumetric compositions. The Superpave Gyratory Compactor (SGC) was used to compact (20) asphalt concrete cylindrical specimens and the optimum asphalt content of each type of asphalt mixture was determined. The laboratory work involved the manufacture of laboratory SGC mold of diameter (100) mm to compact specimens, from which specimens were used for mechanical testing. The designed mixtures were tested and compared with conventional mixture by using Marshall Stability and flow test, indirect tensile strength test, immersion-

compression test, durability test, and dynamic modulus of asphalt concrete using ultrasonic testing, the testing methodology involved different asphalt contents(Optimum and Optimum +o.5), testing temperatures (25, 60) Cº with different immersion periods of (1, 3 and 7) days.

The results indicate that mixtures with Reclaimed Asphalt Pavement showed better performance than Virgin mixtures, it is found that recycled mixtures with 25% RAP having an increasing in Marshall stability, indirect tensile strength at 25ºC, tensile strength ratio, compressive strength, ultrasonic at 100 gyrations by: 34.47%, 9.35%, 8.42%, 32.75% and 6.74% at optimum asphalt content, respectively; as compared to the virgin mixture. But these results are roughly lower when the optimum asphalt content is increased by 0.5%.

Based on laboratory tests; results were analyzed and models were developed using SPSS version 22 software to predict the stability, retained stability and moisture damage. Analysis of results, calculation of standard error and coefficient of determination show a good correlation with R^2 equals to 98.6, 94.3, and 96.6 percent, respectively.

Finally, Mechanistic-Empirical Pavement Design Guide (MEPDG) Version 6.3 software is utilized to predict the mechanical properties of a flexible pavement structure with a RAP modified hot mix asphalt (HMA) surface layer. Different design runs are conducted by using some hierarchical levels of analysis in the MEPDG software. Runs were done with changing the properties of HMA (RAP content, asphalt type, asphalt content and air voids) for surface layer. The dynamic modules of asphalt mixtures were determined for different asphalt mixtures; it was found that an additional 7% and 19% of RAP increased the dynamic modulus by 7.1 % and 28.7 % from the original mix for asphalt grade (40-50) and (60-70) respectively, over a 20-year period in MEPDG analysis.

LIST OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

NOMENCLATURES

Term	Description
AASHTO	American Association of State Highway and Transportation of officials
AC	Asphalt Content, Percent (By Weight of Total Mix)
Av	Air Void Content of Compacted Mixture, %
E*	Modulus of Elasticity
ESALs	18-Kip Equivalent Single Axle Load Application
FHWA	Federal Highway Administration
ITSR	Indirect Tensile Strength Ratio
Gmb	Bulk Specific Gravity for Compacted Mixture
Gmm	Maximum Theoretical Specific Gravity of Void Less Mixture
HMA	Hot Mix Asphalt Concrete
K-S	Kolmogorov-Smirnov
MEPDG	Mechanistic-Empirical Pavement Design Guide
MnPAVE	Minnesota Pavement Analysis and Design software
Mr	Resilient Modulus
NCHRP	National Cooperative Highway Research Program
Ndes	Design Number of Gyrations
Nini	Initial Number of Gyrations
Nmax	Maximum Number of Gyrations
O.A.C.	Optimum Asphalt Content
Pseff.	Effective Asphalt Binder (%) By Weight of Total Mix
PG	Performance Grade of Asphalt Binder
R	Correlation Coefficient
$\mathbf{R^2}$	Coefficient of Determination
r	Annual Growth Rate
SCRB	State Cooperation for Roads and Bridges
SGC	Superpave Gyratory Compactor
SPSS	Statistical Predictive Software and Solution (V.)
Superpave	Superior Performing Asphalt Pavement
VFA	Voids Filled with Asphalt, %
VMA	Voids in Mineral Aggregate, %

CHAPTER ONE

INTRODUCTION

1.1 General

Recycling Hot Mix Asphalt (RHMA) material results in a reusable mixture of aggregate and asphalt binder known as Reclaimed Asphalt Pavement (RAP) (Kennedy et al. 1998) as shown in Figure (1-1). The Reclaimed Asphalt Pavement (RAP) can save money and save energy when recycling is done; because it is significantly reduced the usage of natural resources (i.e., aggregate and petroleum product), and assisted local governments to meet the global reducing disposal standard.

An increasing number of regional business is being processed every year in the United States (Kelly, 1998), and many are being reused for building pavements. Since the 1930s, the regional action has been used in practice to reduce the high cost of Virgin's aggregates and crude oil and to conserve these depleted resources quickly and minimize the problem of disposing of old mixtures. The first data documented on the use of RAP for the construction of new roads date back to 1915 (Taylor, 1997). However, the actual development and rise of RAP usage occurred in the 1970’s during the oil crisis, when the cost of the asphalt binder (or asphalt) as well as the aggregate shortages where high near the construction sites (Sullivan, 1996). Later, in 1997, with the Kyoto Protocol adaptation by parties and implementation in 2005, recycling received major attention and broader application in the road construction industry (Reyes et al., 2009).

Many practices that were initially developed during that period are still in use today and have become part of routine operations for pavement construction and rehabilitation (West, 2010).

Figure (1.1): Pavement Recycling with Reclaimed Asphalt Pavement (RAP), (Kennedy et al. 1998).

1.2 Problem Statement

The removal of deteriorated roadways leads to increase of waste materials. In addition to the cost of establishing a new asphalt pavement, the recycling considers good solution from the economic and environmental aspects. Recycling reduces the consumption amount of natural resources and helps to perceive energy as well. Due to the current reduction in the natural resource, this will lead the society to search for new sustainable alternatives. In the field of asphalt paving technology, the recycling of pavements can be seen as a sustainable option. The use of RAP has an Environmental benefit by decreasing the amount of waste produced and helps to resolve the disposal problems of highway construction materials.

1.3 Objectives of the Study

The main objective of this study is to explore the ability to design asphalt mixtures with different percentages of RAP up to 25% without sacrificing its properties. Volumetric properties are supported according to Superpave system and Iraqi specifications. The other objectives of this work are explained in the following points:

1- Investigating the physical properties of local reclaimed asphalt pavement materials (RAP) with different percentages.
2- Evaluating the laboratory performance characteristics of HMA mixtures containing different percentages of RAP throughout a designed experimental program using stability, durability, compression strength, ultrasonic testing and moisture susceptibility tests.
3- Investigating the advantages provided by recycling on asphalt pavement properties.
4- Developing statistical models to predict stability, retained stability and indirect tensile strength of local surface asphalt concrete mixture using experimental data obtained from laboratory tests after considering the local material properties and environmental effects.
5- Using Mechanistic-Empirical Design Guide (MEPDG) utilizing Mnpave software version 6.3 to predict the properties of a flexible pavement structure with added a different percentage of RAP to the conventional hot mix asphalt (HMA) surface layer.

1.4 Thesis Structure

The scope of this study is done by the following chapters:

- **Chapter One:** provides a general definition of recycling, problem statement, objectives of study, and scope of the research work.
- **Chapter Two:** presents a review of the literature related to previous experimental work on recycling field; also, it provides information about recycling advantages, techniques, and materials.
- **Chapter Three:** presents an experimental design work, materials, mixture design, and selection of optimum asphalt content.
- **Chapter Four:** includes the results obtained from the experimental work, and discussing the effect of each variable on the performance of asphalt mixture.
- **Chapter Five:** shows the statistical analysis process used in the prediction of stability and retained stability models for the surface layer, which consist of regression technique in addition to moisture sensitivity model utilizing SPSS software version 23.
- **Chapter Six**: consists of the application of Mechanistic-Empirical Approach using MnPAVE software.
- **Chapter Seven:** presents the conclusions and recommendations for further research work.

Appendix A: Criteria of hot mix Asphalt Design

Appendix B: Includes Superpave Mix design results.

CHAPTER TWO
LITERATURE REVIEW

2.1 General

Reclaimed Asphalt Pavement (RAP) is an old, existing asphalt pavement, which is grinded and stored to be used as part of a new pavement. The RAP can be obtained whenever the old current pavement needs to be replaced or whenever part of the pavement needs to be cut to access underground facilities. If the existing old pavement is satisfactorily reclaimed in a smooth and properly stored sense, its aggregation can be used as a valuable source when the total quality is scarce. Besides, the current binder in the RAP can form some of the required binder in the Hot Mix Asphalt (HMA). RAP has been developed for many years. During the 1930's, the Hot In-Place Recycling (HIR) technology was first discovered in the asphalt recycling area (ARRA, 2001). It is essential that the reclaimed materials to be recycled are consistent, as variable materials will cause problems with the control of quality and impede the efficiency of the recycling operation. Suitable sources of consistent material of sufficient quantity for the scheme being considered need to be identified either in existing pavements, from stockpiled planning of known origin or from another suitable source. The assessment of the properties of the existing material proposed for recycling can be made using cores sampled from the carriageway or from samples taken from stockpiles in accordance with current practice. This chapter summarizes the main characteristics of RAP and the scope of recycling asphalt materials.

2.2 Definition and Field Application of Recycling Technique

Related to Federal Highway Administration (FHWA), the amount of recycled pavement that has been milled in every year is 90 million tons and 33% of all recycled RAP is reused in production of hot mix asphalt, (Cosentino et al., 2003). In the Florida Department of Transportation (FDOT) from 1979 to 1994 it produced 22 million ton of RAP (Smith, 1996). However, after the Superpave mix design is applied, the quantity of reclaimed asphalt pavement that has been added to hot mix asphalt designed decreased. The American Asphalt Institute (Asphalt, 1986) has defined the recycling of asphalt as a re-use of materials that have already served or led the original purpose, having processed several processors. Also, Vollor in 1986 considered that asphalt recycling mix is re-used after simple treatment for recycled materials that have already served in the pavement. As explained by Al-Qadi et al., (2007), recycling is reuse of the recycled materials of pavement that is reached to the end of its life service. Recycled paving materials still have value for service where it can be used in hot mix asphalt.

Since 1970, the recycled materials have been used as granular intervention in paving roads or by mixing it with the new materials to be used in the production of new asphalt mixture (Malpass, 2003). Also, the continuous process of construction and maintenance of the old ways and the high cost of new materials highlighted the recycling process as a process which is economically feasible and environmentally acceptable (Ramanujam, 2000). The recycled asphalt mixture can also be used for various other purposes such as construction of the shoulders of roads or dams, or any form of material used as filler (Roberts et al., 1991). RAP materials are oxidized or become aged during the service life and therefore recycled asphalt mix is produced through the addition of two new asphalt and aggregate which are sometimes added in recycling transactions (Doh et al., 2008). Therefore, the use of recycled asphalt mix is beneficial economically and environmentally, as many studies explained it (Perez et al., 2004 and Sarsam, 2007).

2.3 RAP Characteristics to be considered in Mix Design

As mentioned earlier, RAP is the existing asphalt pavement that will be smoothed out and stocked to be used as a part of a new pavement. RAP contains valuable amounts of aggregate and binder. During the years of service, both aggregates and binder subject to changes affecting their characteristics. To ensure that these changes do not adversely affect the performance of the HMA, specific considerations must be taken in to account (McDaniel and Anderson, 2001).

2.3.1 Binder characteristics

The amount of reclaimed asphalt pavement and the age of RAP have the main effects on properties of recycled mixture (Kandhal et al., 1995). The most important thing is to know how much asphalt binder is still in the RAP. The content of the binder for the RAP is important because it can be deducted from the total binder required for the HMA mixture. Once the amount of the binder is known, it is time to consider the changes in the physical and chemical properties of the remnants of the binder in the regional action due to oxidation during the years of service. The old binder is the hardest and it resembles the highest levels of binder grade. Because there is no adequate hardened aged binder to affect the properties of the final mix, it may not be necessary to test the properties of the remaining leaves in the RAP when the lower proportions of the RAP are inserted into the mixture. For mixtures with more than 20 percent of RAP, however, the preservatives of the residue in the RAP should be tested and considered in the process of designing the blend. The recommended process is to extract and retrieve the binder in RAP and conduct performance tests (PG) on it. The extraction method has been explained in the AASHTO T 319 Quantitative Extraction and Recovery of Asphalt Binder from Asphalt Mixtures. AASHTO T 319 is recommended because the extraction process used affects binder properties less than other methods (McDaniel and Anderson, 2001).

2.3.2 Aggregate Characteristics

Beam and Maurer in (1991) stated that there is a difference between the gradations of aggregate that have obtained from the core sample. They also found that the aggregate obtained from milled process is finer than those gotten from the core. Brownie and Hironaka (1979) noted that the change in gradation of aggregate depends on the hardness of aggregate. The large amount of fines caused a failure, and to meet superpave mix design requirements to reduction the amount of RAP can be used in hot mix asphalt and for that (Stroup-Gardner Wagner, 1999) suggested that RAP should be fractioned in fine and coarse aggregate, to keep the large amount of dust fraction out from the mixture. In Superpave HMA design, as a source for binder and aggregation, but the contribution of the binder and the remaining totals is considered separate. Once you have the binder properties in the RAP, it's time to get the assembly properties. Gradation is the most important feature of the aggregate obtained from RAP and it is obtained using the Kansas test method KT-2 (AASHTO: T 27, 2010).

Once the aggregate gradation is obtained, the bulk specific gravity (G_{sb}) of RAP aggregate should be measured. If the history of RAP aggregates exists, the G_{sb} of original aggregates in the RAP can be used in mix design. If the G_{sb} of original aggregates does not exist but the effective specific gravity (G_{se}) records are available, the G_{se} can be replaced by G_{sb}. Replacing G_{se} for G_{sb} will not cause any problem because G_{se} is always greater than G_{sb} and the substitution will overestimate the bulk specific gravity of the blend (the combination of virgin aggregates and RAP). In case that there are no records exist for G_{se} or G_{sb} of the original aggregates in RAP or when higher percentages of RAP are introduced into the mix (causing non-negligible errors if G_{sb} is substitute by G_{se}), a typical value for the asphalt absorption will be assumed and RAP G_{sb} will be calculated using the G_{se}. The assumption for asphalt absorption should be based on experiences obtained during mix designs at similar locations, (Copeland and Opeland, 2011).

2.3.3 Physical properties of RAP

The characteristics of the RAP are more dependent on the properties of the component materials and the type of asphalt concrete blend (surface course, binder course, etc.). There can be significant differences between asphalt concrete mixtures in aggregate quality, size, and consistency. Where polishing resistance is not of concern for the aggregates in base course applications that have less quality than aggregates in surface course for that the aggregates in wearing course (surface course) asphalt concrete should possess high strength to polishing (wear and abrasion) to be accepted for the resistance of friction properties. The source of RAP is a well-graded coarse aggregate of a surface layer. Usually, the unit weight of milled or processed RAP depends on the type of aggregate in the reclaimed pavement and the moisture content of the stockpiled material. Although available literature on RAP contains limited data pertaining to unit weight, the unit weight of milled or processed RAP has been found to range from 1940 to 2300 kg/m^3 (120 to 140 lb./ft^3), which is slightly lower than that of natural aggregates, (Kallas, 1984).

The quantity of asphalt in RAP is ranging between 3 to 7 percent by weight. The hardening of new asphalt cement was less than the asphalt cements that adhering to the aggregate. This is mainly due to the exposure of the pavement to oxidation (atmospheric oxygen) through weathering and use. There are many factors that are implemented the degree of stiffness that consist mainly of asphalt cement properties, blending temperature/time (increases with increased high heat exposure), compaction of asphalt concrete (increases if not good compacted), asphalt cement/air void (increases with lower asphalt/high air void) and age in service (increases with age) (Majeed, 2016).

2.3.4 RAP fractionation

Due to the segregation of the RAP stocks and its effect on the dust content asphalt and in the final mixture, the control of the gradation is so hard with the RAP, mainly when the greater proportions of the RAP are added to the mixture. The problem with the RAP segregated is that the finer fraction of the RAP will contain the highest asphalt content, due to the high surface area, which makes the control of air voids, is very difficult.

Fractionated RAP (f- RAP) is that separated into at least two different sizes in order to better control the consistency of the blend of mix and gradation. Typical sizes for coarser and finer fraction are, respectively, +1/2 or +3/8 inches (+12.5 or +9.5 mm) and -1/2 or -3/8 inches, (Copeland and Opeland, 2011), Figure (2-1).

West et al., (2013a) listed the first advantage of the fractionating RAP is that owning stocks of different sizes RAP provides greater flexibility in achieving the design requirements of the mix, However there are disadvantages that arise in the fragmentation of RAP materials:

- Need for more space with many small stockpiles
- More expensive processing option (cost of fractionation unit plus additional RAP cold feed bins), (West et al., 2013b).

Figure (2-1): Samples of fractionated RAP (West et al., 2013b).

2.3.5 Reclaimed Asphalt Pavement Advantages

The use of RAP material has different advantages; few of them are listed below:

- Environmental benefit: the uses of RAP materials have an environmental benefit by reducing the amount of waste materials and preserves natural resources. Chiu et al., (2008) found that the reduction in the amount of asphalt binder required and the amount of energy required to heat the materials produces a 23% reduction in eco-burden
- Economic benefit: Economic benefits include the saving in materials cost through reducing the amount of virgin aggregates and asphalt binders in fresh mixtures, as well as, the reducing in the costs related with transporting virgin materials to plants. Kandhal and Mallick, (1997) stated that the use of 20-50% of RAP material economize up to 34% of the total cost.
- Conservation of energy: the use of RAP material may be save from 25 to 40% of the energy related with extracting and processing of nonrenewable natural resources for pavement construction, maintenance, and rehabilitation activities.

- Providing a rut resistance mixture: according to (McDaniel et al., 2000), it was found that the addition of RAP to the (HMA) mixtures can improve rutting resistance.

2.4 Asphalt Concrete Recycling Methods

There are several ways to use the recycling of asphalt, including the recycling of the hot mix, hot in-place recycling and full depth reclamation.

2.4.1 Hot mix recycling

Santucci (2007) stated that the recycling of hot mix is the most common way to recycle asphalt pavements. It involves combining RAP with a new or "virgin" aggregate, a new asphalt binder, and/or recycling agents at the central hot mix factory to produce a recycled blend. Allowable RAP amount in recycled mixture and guidelines for where the recycled mixture can be used in the structure of the pavement varies by agency. Some agencies routinely allow 15 percent or less of the RAP, while others allow for larger quantities of RAP. Higher RAP concentrations require modifications in the blend design and selection of the binder. Suggested guidelines by AASHTO M 323, which are related to RAP content in a recycled mix are as follows:

- **15% RAP or less:** Performance Grade (PG) binder is the same as that used in a virgin mix.
- **15-25% RAP:** PG binder should be one grade lower on both high and low temperature end, i.e. PG 64-16 rather than PG 70-10.
- **More than 25% RAP:** Test and blend the recovered asphalt from RAP with virgin asphalt as part of the design process to determine the amount of RAP to use.

For the highest levels of RAP, it is critical to address proper physical assessment, mix design, construction and quality control issues. Once the RAP is hauled to a central station, it is processed and stored for future use. The processing process of RAP stockpiling may include crushing and screening. RAP when coming as large amounts from different sources, stockpiles should be separated and identified by source. However, restrictions on space and limited quantities of RAP from some sources often lead to a composite or mixed stock must be properly qualified. Separating the RAP into different sizes to reduce the separation of the RAP particles and allow greater flexibility in modifying the content of the RAP to meet the aggregate final gradation is required. Since the moisture in the RAP can have a significant impact on the amount of RAP used or the quality of the recycled mixture, it is important for the contractor to monitor the moisture of the RAP and use the best management practices to reduce moisture. How to combine a RAP with virgin aggregate and asphalt to produce a largely recycled blend depends on the composition of the hot mix plant. The RAP delivery system for typical batch plant operations is shows in Figures (2-2) and (2-3).

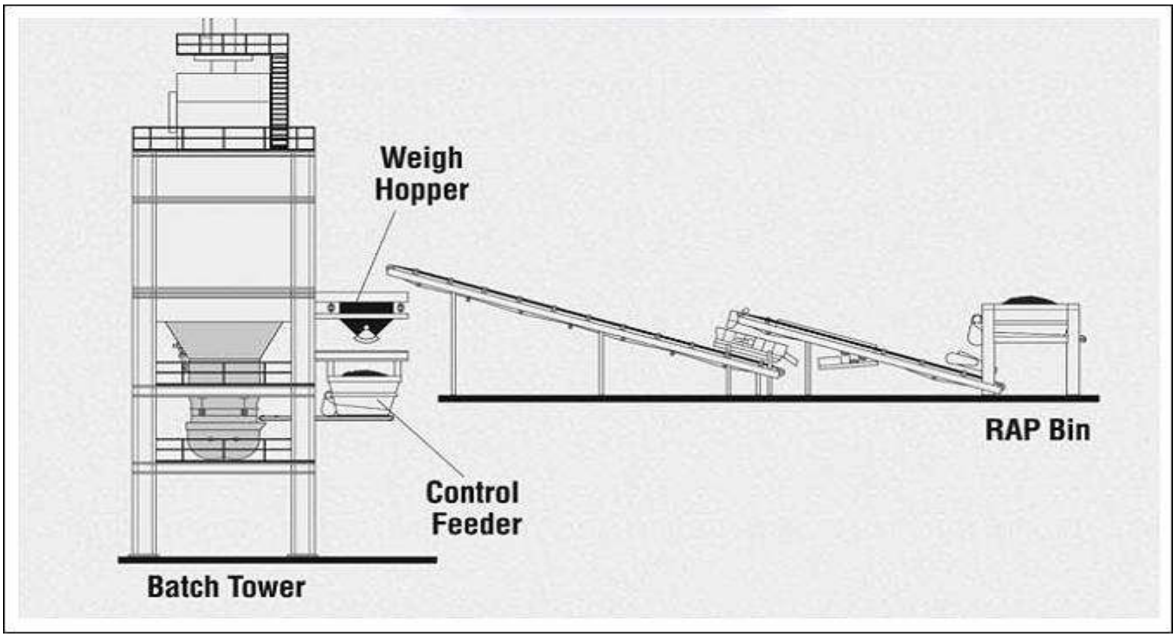

Figure (2-2): RAP delivery system for batch plants, (Santucci, 2007).

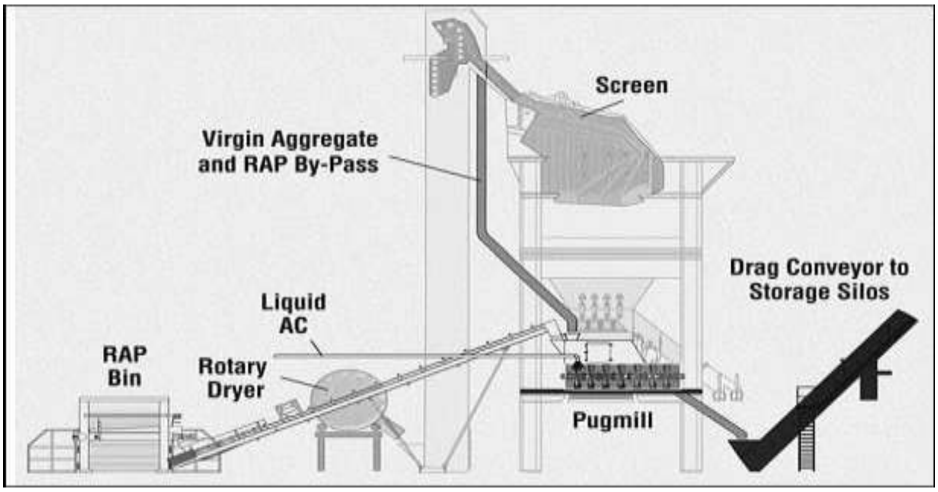

Figure (2-3): Batch plant with pugmill mixer, (Santucci, 2007).

RAP is added to the mixer in the drum plant mix directly. The point of drum mixer at which the RAP is added depends on the type of blender being used (parallel flow, counter flow, or double barrel), and whether or not a separate coater is included in the drum mix operation. Types of drum mixers are illustrated in Figures (2-4) to (2-6).

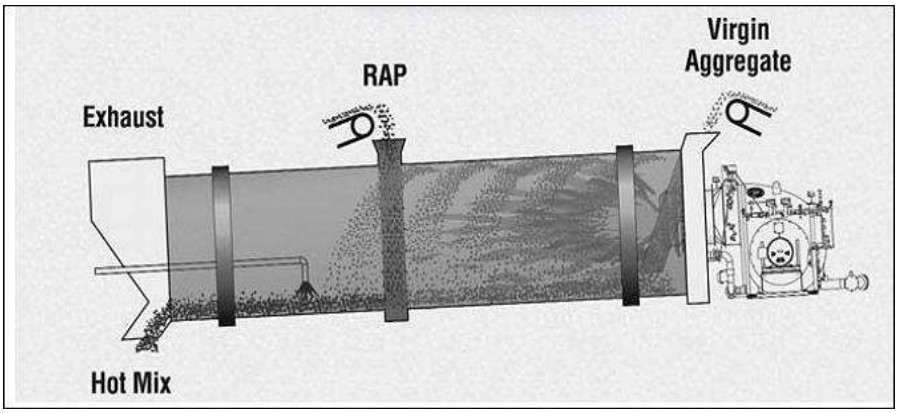

Figure (2-4): Parallel flow drum mixer, (Santucci, 2007).

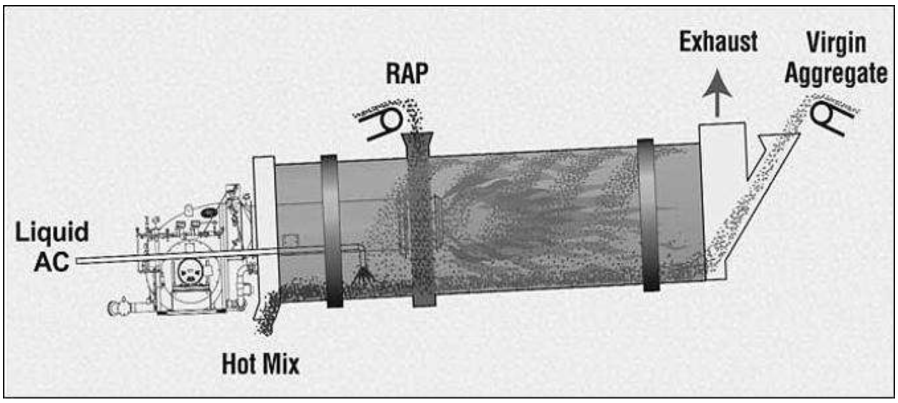

Figure (2-5): Counter flow drum mixer, (Santucci, 2007).

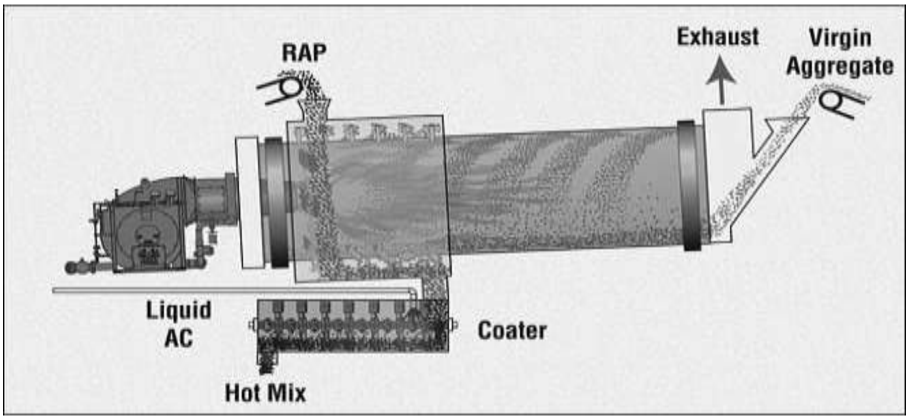

Figure (2-6): Double RAP dryer with coater, (Santucci, 2007).

When the recycled mixture is produced, either sends it to the storage silo for the future delivery to a job or it is immediately transported, placed, and compacted with conventional hot mix equipment at the project site. No special techniques are required to handle the recycled mixture. However, the paving crew should be aware that the recycled mix may be delivered at a slightly lower temperature than a virgin mix to prevent overheating the mix at the plant.

2.4.2 Hot in-place Recycling (HIR)

The hot recycling in place was described by Kandhal and Malick (1997) as an on-site, in-place method that rehabilitates deteriorated asphalt pavements and thereby minimizes the use of new materials. Terrell and Lee (1997) stated that in the hot in-place recycling process, correcting asphalt pavement distress occurs by softening the current surface with heat, and mechanically removing the surface pavement, blending it with the asphalt binder, and possibly adding the virgin aggregate, and replacing the recycled materials on the pavement without being removed from the original of pavement site. The main purpose of the hot recycling in place is to correct the surface distresses that are not caused by structural insufficiency, such as raveling, cracks, ruts and holes, and shoves and bumps. It may be performed as a single pass or multipath operation. In a single-pass operation the virgin materials are mixed with the restored reclaimed asphalt pavement (RAP) material in a single-pass, whereas in the multi-step process, a new wearing course is added after re-compacting the RAP materials as shown in Figure (2-7), (Kandhal and Malick, 1997). In certain cases, hot in-place recycling should not be adopted; these cases are described by Ramanujam, (2000):

- Any of the lower courses are not stable.
- If excessive hardening of binder has taken place.
- Surface maintenance problems are associated with base or drainage problem in the base or subgrade.
- The variation in asphalt surface thickness is excessive.
- The pavement structure is weak and cannot bear the weight of the mixing train and equipment.
- The pavement is excessively wet.

This approach requires several pieces of equipment such as pre-heaters, heaters, mixers, pavers, and rollers. The combined equipment is often referred to as a "train". Treatment depths range from ¾ to 3 inches (19 to 75 mm) depending on the HIR Recycling process used. The most common HIR processes are surface recycling, remixing, and repaving, (Santucci, 2007).

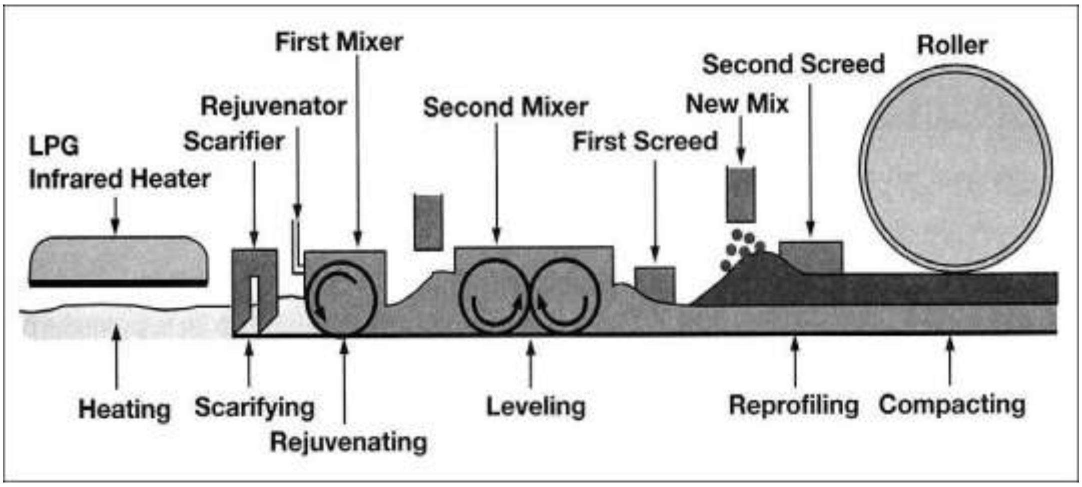

Figure (2-7): Hot in-place repaving process and equipment, (Santucci, 2007).

There are some advantages for HIR over conventional Hot Mix Recycling process; these are (Ramanujam, 2000):

- Reduce construction time, with resurfacing being completed in one process and reducing delays of traffic.
- Easy to improve the asphalt blend exists either by adding a new mix, aggregate, binder or rejuvenators.
- Improvement in ride-ability due to hot bonding between joints.
- Maintaining of existing surface levels e.g. curbs and manholes need not be raised.
- Treatment of only heavily trafficked lane in a four lane road (which is not possible for conventional overlays).

2.4.3 Full depth reclamation

It is a method of recycling where a predetermined amount of the underlying base or subbase material is blended with the entire thickness of the distressed asphalt pavement to produce an upgraded, uniform base material. Depths of Treatment can vary from 4 to 12 inches (100*250 mm) depending on the existing pavement layers thickness, (Santucci, 2007).

2.5 Mix Design Consideration with High Percentages of RAP

One of the advantages of Superpave is the flexibility of the blend design that allows adding various additives, such as RAP, to the HMA blend as long as the specified gradation can be achieved. There are two ways to determine the percentages of the RAP in the mixture. The first method includes deciding on the expected contribution of the RAP to the overall mixture based on the weight of the RAP (as a percentage of the total mix by weight). The second method includes deciding on the expected contribution from the binder to the total binder in the mixture (as a percentage of the total required binder by weight) while satisfying the volumetric properties requirements.

To make up for the aged and hardened binder in RAP, a softer virgin binder needs to be add to the mix, especially when higher than 15 percent RAP is being added to the mix. In order to find the right binder PG grade for high RAP mixtures, a blending chart or blending equation is frequently used. The blending charts or equations can be used if the virgin binder PG grade is already chosen and the percentage of RAP in the mix is to be determined, or if the percentage of RAP to be added to the mix is known and binder PG grade for the virgin binder is to be determined. Procedures for using a blending chart are provided in the appendix of AASHTO M 323 and the recycled HMA should meet all test procedures and criteria as required for the virgin HMA (Al-Qadi et al., 2007 and Brown et al., 2009).

Many agencies limited the maximum amount of RAP in HMA and these are:

- The UK Specification for Highway Works – Clause 902, allows RAP material to be used in the production of asphaltic wearing course, binder course and road base with the maximum amount of 10% in wearing course and 50% in all other layers. Additional performance requirements are required when the recycled content exceeds 25% by mass (Widyatmoko, 2008).
- The Virginia Department of Transportation (Virginia DOT) recently increased the threshold of allowable RAP for Superpave mixtures from 20% to 30%. Virginia AC mixtures produced in other parts of the United States may contain up to 30% or more RAP (Apeagyei et al., 2011).
- The Spanish General Technical Specifications for Highway Rehabilitation, which define and specify the design requirements of recycled mixtures with RAP percentages between 10% to 50% (Valdés et al., 2011).
- The Washington State Department of Transportation (WSDOT) in the United States allows using RAP content reaching 20% of the total aggregate mass "(WSDOT: M 41–10, 2002)". However, the M 41–10 standard allows for the inclusion of a higher RAP percentage in "Hot Mix Asphalt (HMA)" production. A separate mix design is outlined in the M

41–10 standard that specifically accounts for the inclusion of a higher content of RAP can be used (> 20%) in the HMA, (Tabakovic, 2013).

- In the Irish context, the National Roads Authority (NRA) showed in 2005 that RAP can be used in the manufacture of bituminous base (National Roads Authority NRA, 2005). The maximum content of RAP used by the NRA 20% for a coated macadam base. This content Matches the British Standard (British Standard Institution BSI, 2005); (Tabaković et al., 2010).
- Italian Specification (MIT, 2002; ANAS, 2003; Autostrade, 2004) indicted that the maximum amount of RAP is (30%) for surface course, (40%) for binder course and (50%) for base course (Anita et al., 2016).

2.6 Performance of HMA Mixtures Containing RAP

Various works with various laboratory tests were conducted to evaluate the performance of recycled pavement; this section will discusses these works in details.

2.6.1 Properties of recycled mixtures for Marshall test

Tabaković et al. (2010) studied the physical properties of the recycled asphalt pavement and its effect on the mechanical properties of the mixture. RAP was given to the bituminous mixtures at 10%, 20% and 30%. Control samples that did not contain RAP were also used at all stages of the test, and a total of 104 samples were tested. The results of the Marshall tests are shown in Table (2-1). These results showed that the percent of optimum binder content added to the blend decreases with each corresponding increase in the RAP ratio. This is a function of the pre-existing binder within the RAP, and the calculated reduction in the aggregate surface area of this mixture with increased RAP content as shown in Table (2-2).

Table (2-1): Optimum Binder Content for Selected RAP Percentages, (Tabaković et al., 2010).

RAP content (%)	Optimum binder content (%)
0	4.70
10	4.20
20	4.17
30	4.00

Table (2-2): Mix Surface Area Factor, (Tabaković et al., 2010).

Mix aggregate surface area (mm^2\kg)				
	0% RAP	10%RAP	20%RAP	30%RAP
Total	6.315	6.202	6.101	5.8 49

Sarsam (2007) prepared cylindrical and beam samples of asphalt concrete with various asphalt ratios in the lab, using the dense and gap aggregate gradations. Several tests, including a Marshall test, were performed on these samples. The samples were then subjected to accelerated aging using the Superpave procedure, and another set of cylindrical and beam samples was constructed from the aged asphalt concrete and exposed to the same tests. The effects of aging and recycling on the properties of different asphalt concrete mixture have been analyzed. It was concluded that aging caused a decrease in Marshall stability and an increase in Marshall flow. Recycling causes an increase in Marshall Stability especially for gap graded mixes, and the values of flow are due to their original pre-aging values. Table (2-3) shows the change in the stability and flow percentage of the Marshall after the recycling of each dense and gap gradations.

Celauro et al. (2010) examined specific laboratory study aimed to combine highly mechanical recycled asphalt mixture for surface and structural layers, for this purpose, mixtures with close-graded and (0%, 40%, and 50%) RAP content was investigated.

Table (2-3): change in the stability and flow percentage of the marshall after the recycling, (Sarsam, 2007).

	Gradation	Dense			Gap		
Change%	Asphalt%	4	5	6	4	5	6
	Marshall stability	+2.3	-40.5	+36.6	+877.6	+99	+295
	Marshall flow	-32.7	-15.1	-20.8	+39.4	+45	0

From the result of Marshall test, all mixtures that have considered in study not only comply with the minimum levels of Marshall stability and stiffness as required, but also with the higher performances pursued by research. As far as the Marshall Air voids are concerned, too low values for only one type of mixtures (thin surface mix), corresponding with the higher bitumen contents (5.8% - 6.2%). A decrease in Marshall stability has been observed with an increase in binder content. This may be due to the fact that, in order to increase resistance to damage caused by water, as well as with regard to ageing of the mixtures, a prior decision was chosen to have thicker binder film on aggregate, consequently binder content used was higher than the optimum value for these properties. The same downward trend can be observed with the content of the bitumen for the Marshall stiffness. Conversely, when considering the results as a function of RAP content, a growing trend of Marshall's stability, hardness, can be observed.

Hussain and Yanjun in (2012) presented an experimental study to evaluate the effect of various types and percentages of RAP on the properties of asphalt mixtures. Four mixtures, which were the combination of two different virgin aggregates (Limestone and Quartzite) and two different RAP sources were studied in the research. A wide range of 0 to 100% RAP blends of mixtures was designed by Marshall method. Virgin aggregate was blended with RAP material such that all specimens tested have the same gradation approximately. Table (2-4) shows the results that all the mixtures fulfilled stability criteria and also satisfied the VMA and VFA requirements. The mixtures with high content of RAP did not meet the minimum flow criteria (more than 2 mm). Mixtures with RAP content up to 30% satisfied the flow criteria of (2-4) mm. Generally the addition of RAP material improved the properties of the mixtures which show that recycling is a viable option for HMA design.

Table (2-4): Marshall properties for virgin and recycled mixtures, (Hussain and Yanjun, 2012).

RAP(%)	Air Voids(%)	VFA(%)	VMA(%)	Stability(KN)	Flow(mm)	Unit weight (Kg\m^3)
Control Mix						
0	4	71.5	14.15	9.89	2.63	2376
Recycled Mix						
10	4.06	74.75	16.08	9.59	2.04	2358
20	3.93	72.84	14.47	10.92	2.56	2353
30	4.24	70.70	14.47	18.40	3.07	2353
45	4.87	67.66	15.02	11.73	2.10	2348
60	3.84	74.74	15.20	14.98	1.80	2348
100	3.69	77.47	16.38	21.19	0.91	2333

2.6.2 Durability test based on retained stability

In Iraq several local studies have been implemented, Qasim et al., (2016) had studied the durability for mixtures with RAP contents (5, 10 and 15) immersed in water bath at 25ºC for (1, 2, 4, 7 and 14) days. It was concluded that the durability with mixture contianing RAP is higher than control mix when increased immersion days as shown in Figure (2-8).

Majeed (2016) studied the durability index based on retained stability for mixtures containing different RAP percentage (20, 30, 40, and 50). It was concluded that the durability index is decreased with the increase of RAP content as shown in Table (2-5).

Table (2-5) Relation between DI and RAP contents (Majeed 2016).

RAP %	DI for RAP heated to 110 c
0	10.4
20	8.6
30	7.6
40	6.4
50	5.7

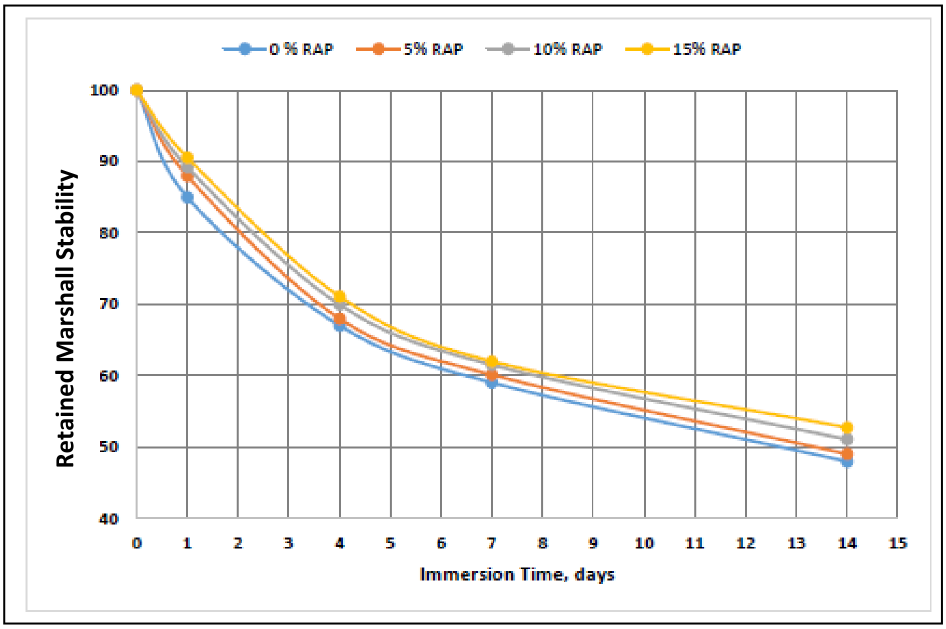

Figure (2-8): Durability curves for RAP mixture, using marshall stability criterion at immersion temperature 25ºC (Qasim et al., 2016).

2.6.3 Indirect tensile strength (ITS) test for recycled mixtures

Shu et al. in (2008) conducted ITS test to determine strain and tensile strength of HMA. One source of aggregate (limestone) and one type of binder (PG 64 - 22), containing 0%, 10%, 20%, and 30% of RAP was used to prepare the mixtures. Value of indirect tensile strength (ITS) was higher for mixtures containing higher percentages of RAP, lower strain at peak-load than the control mixtures (0% RAP). These phenomena can be attributed to the aged, stiffened, and brittle asphalt binder in RAP due to the aging process. The test results showed that incorporation of RAP increased the strength of HMA mixtures. However, due to the increase in the brittleness (decreased failure strain), the fatigue life of HMA mixtures` may still be compromised.

Watson et al. (2008) evaluated the moisture susceptibility of mixtures blended with various RAP contents from 0% to 30% following a version of AASHTO T283 modified by the Georgia Department of Transportation. The tensile strength of both dry and saturated specimens increased as the percentage of RAP increased; however, Tensile Strength Ratio (TSR) did not increase significantly with the addition of RAP.

Al-Qadi et al. (2009) confirmed that RAP content increases the tensile strength of blended mixtures and therefore the resistance to moisture susceptibility. Partial blending between RAP binder and virgin binder may cause double coating on RAP particles, which improves the stripping resistance of the particles. However, selective absorption of binder that creates a bond and improves the stripping resistance does not occur immediately for virgin aggregate during the mixing process, causing mixtures composed of virgin material to have weaker stripping resistance. They found that 40% RAP mixtures have higher TSR than 0% RAP mixtures, but lower TSR than the 20% RAP mixtures.

Celauro et al. (2010) evaluated the ITS test for the mixtures with close-graded, and (0%, 40%, and 50%) RAP content added to the mixture was investigated in the study. It was noticed from the results that the ITS decline with the rise in binder content. Furthermore, regardless of bitumen content, the indirect tensile strength increased with the increased in RAP content. This result matched what was observed for Marshall stability and has to be related, at a constant level of the bitumen content, with the steady increase in bitumen hardening due to the use of larger quantities of recycled asphalt.

2.6.4 Moisture damage for Recycled Mixture

Al-Rousan et al. (2008) studied the moisture damage of recycled mixtures. Two mixtures were prepared, one of the mixtures was composed of virgin asphalt and 100% fresh aggregate and the other mix was composed of 30% RAP and 70% fresh aggregate and virgin asphalt. Water susceptibility of the asphalt concrete mixes was evaluated by measuring the loss or reduction of the Indirect Tensile Strength (ITS) after immersion in water for 24 hours at 60°C due to RAP usage in asphalt mixes, according to AASHTO T-283 test procedure. It was prepared six samples of each mix. Initial ITS values for three samples were tested and the other three samples are tested after conditioned. Loss in ITS between "Control Mix" and "RAP Mix" is presented in Figure (2-9). It could be observed clearly from the figure that the loss in ITS for mixtures containing RAP is much lower than mixtures containing no RAP. This is attributed to the fact that RAP contains hardened asphalt that became more viscous as time passes. Thus, mixtures with more viscous materials will perform better under tension, which will lead to small reduction in tensile strength when exposed to severe conditions of high temperature and moisture.

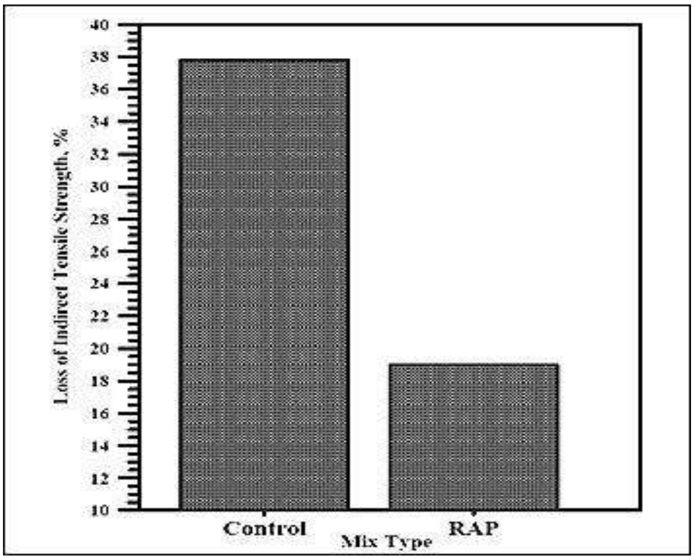

Figure (2-9) Loss of indirect tensile strength (ITS) between control mixes and RAP mixes, (Al-Rousan et al., 2008)

Tabaković et al., (2010) studied the moisture damage for recycled mixtures. RAP was introduced to the bituminous mixtures at levels of 10%, 20%, and 30%. Control samples were also investigated throughout the test. For each mix, six specimens 100 mm diameter and 70 mm in height with a target void content of 6% were prepared using the gyratory compactor. The indirect tensile test for both dry and wet specimens was determined. From the results presented in Table (2-6), it is evident that moisture damage is not an issue for the mixes containing 0%, 10%, and 20% RAP. With the inclusion of 30% of RAP in the mix, the ITS ratio decreases to below 90%, suggesting that further increases in RAP content could leave the mix vulnerable to moisture damage.

Table (2-6): Indirect tensile strength test and indirect tensile strength ratio for both wet and dry specimens, (Tabaković et al., 2010).

	ITST for specimen (KPa)			ITST for dry specimen (KPa)			
RAP content %	μ_1	α_1	c_1	μ_2	α_2	c_2	Indirect tensile strength ratio (%)
0	901.7	64.5	0.07	965.3	17.3	0.02	93.4
10	920.3	79.8	0.09	899.2	75.1	0.08	102.4
20	873.1	25.3	0.03	931.6	70.3	0.08	93.8
30	813.3	35.5	0.04	934.8	60.7	0.07	87.1

Miro et al. (2011) studied the behavior of high modulus bituminous mixtures with high percentages reclaimed asphalt pavement (RAP) and low penetration grade bitumen. Four mixtures with RAP percentages of 0%, 15%, 30% and 50%, respectively, were analyzed. In order to evaluate moisture sensitivity, specimens were conditioned by immersion in water for 72 hour at 40ºC. The Indirect Tensile Strength Ratio (ITSR) was determined for conditioned and unconditioned cylindrical specimens at 15ºC. Values were higher than 80% as presented in Figure (2-10), suggesting that mixtures had good resistance to the action of water. However, these values dropped with increasing RAP content.

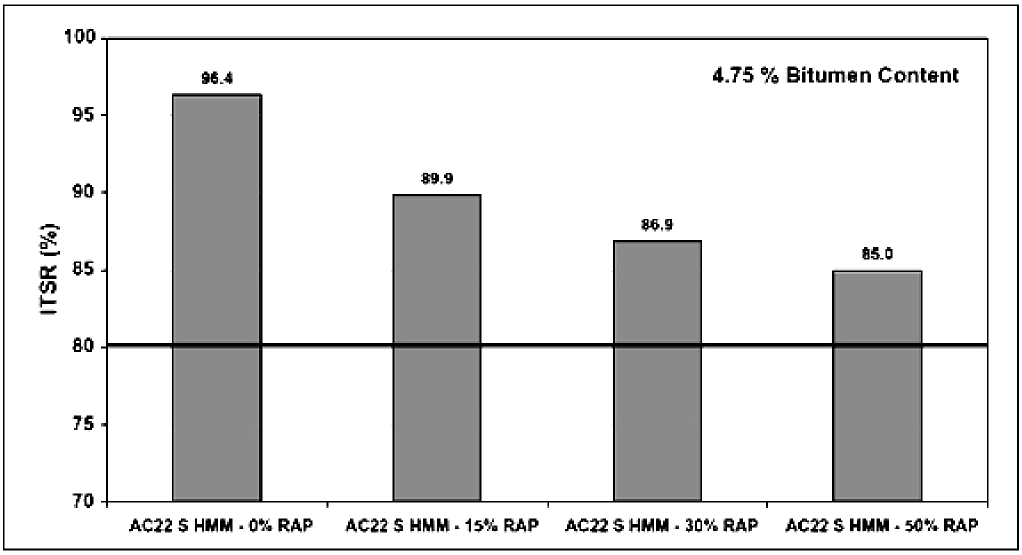

Figure (2-10): Indirect Tensile Strength Ratio Results, (Miro et al., 2011).

Qasim (2016) studied the effect of RAP content on TSR with three different percentages of RAP (5%, 10%, 15%) and two different types of filler. It has been concluded that the mixture containing RAP provide higher TSR than the control mixture as shown in Figure (2-11).

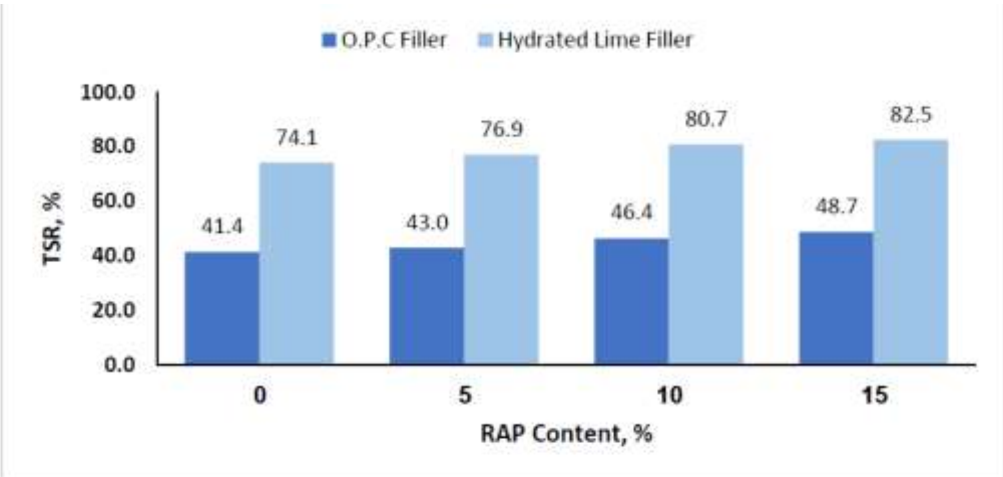

Figure (2-11): Effect of RAP on TSR (Qasim 2016).

2.6.5 Ultrasonic test

Cascante et al. (2006) studied the properties of asphalt mixture by ultrasonic test to measure the modulus of elasticity. The constraint elastic modulus of the mix from the wave velocity was determined for different gyrations and the results are shown in Figure (2-12). It can be noticed that whenever the number of gyration increased this will lead to increase in the values of constrains modulus.

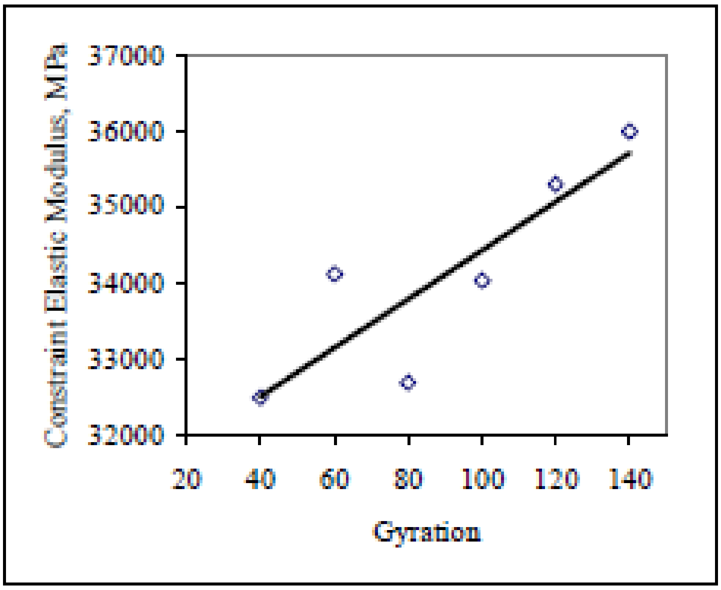

Figure (2-12): Constraint modulus vs. gyration (after Cascante et al., 2006).

Rose et al. in (2011) studied the properties of asphalt mixture by ultrasonic test to measure the modulus of elasticity .This test was conducted on two AC specimens of (100 mm*38 mm) and were compacted to 2.2% and 3.5% air void content. It was found that the value of modulus of elasticity was ranging from 25000 MPa to 35000 MPa.

2.7 Mechanistic – Empirical Approach

Mechanistic-empirical (M-E) method represents one step forward from empirical methods. According to the traffic loading and environmental conditions the induced state of stress and strain in a pavement structure is predicted using theory of mechanics. These structural responses model were linked to distress predictions through Empirical Mechanistic. Dormon and Metcalf (1965) first used the principle concepts for pavement design.

Asphalt Institute method (Shook et al., 1982) and Shell method (Claessen et al., 1977) incorporated strain-based criteria in their mechanistic-empirical procedures. Over the past fifteen years, several studies have advanced mechanistic-empirical techniques. Most of work, however, was based on variants of the same two strain-based criteria developed by Shell and the Asphalt Institute. The Departments of Transportation of the Washington State (WSDOT), North Carolina (NCDOT) and Minnesota (MNDOT), to name a few, developed their own M-E procedures. The National Cooperative Highway Research Program (NCHRP) 1-26 project report, Calibrated Mechanistic Structural Analysis Procedures for Pavements (1990), provided the basic framework for most of the efforts attempted by state DOTs. WSDOT (Pierce et al., 1993) and NCDOT (Corley-Lay and Judith, 1996) developed similar M-E frameworks incorporating environmental variables (e.g., asphalt concrete temperature to determine stiffness).

The NCHRP 1-37A project (NCHRP, 2004) delivered the most recent M-E based method that incorporates nationally calibrated models induced by traffic load and environmental conditions to predict distinct distresses. The methodology of NCHRP 1-37 is also incorporates load distributions and vehicle class in the design, a step forward from the Equivalent Single Axle Load (ESAL) approach used in the AASTHO design equation and other methods. The performance computation is done on a seasonal basis to incorporate the effects of climate conditions on the behavior of materials.

CHAPTER THREE
MATERIALS AND EXPERIMENTAL WORK

3.1 General

This chapter deals with the properties of all materials which have been used in this work. In addition, it focuses on the laboratory testing methods and experimental work.

All materials used in the study are locally available and widely used in roads paving in Iraq. Furthermore, all experimental works have been performed in the Asphalt and Materials Laboratories in the Building and Construction Engineering Department, University of Technology, and the Highway Laboratory in the National Center for Construction Laboratories (NCCL) in Baghdad. The scheme of the research is shown in Figure (3-1).

3.2 Superpave Mixture Design

Four steps are needed to design the Superpave mixture (Asphalt Institute, 2007):

1. Materials selection.
2. Design of aggregate structure.
3. Design of asphalt binder content.
4. Moisture sensitivity.

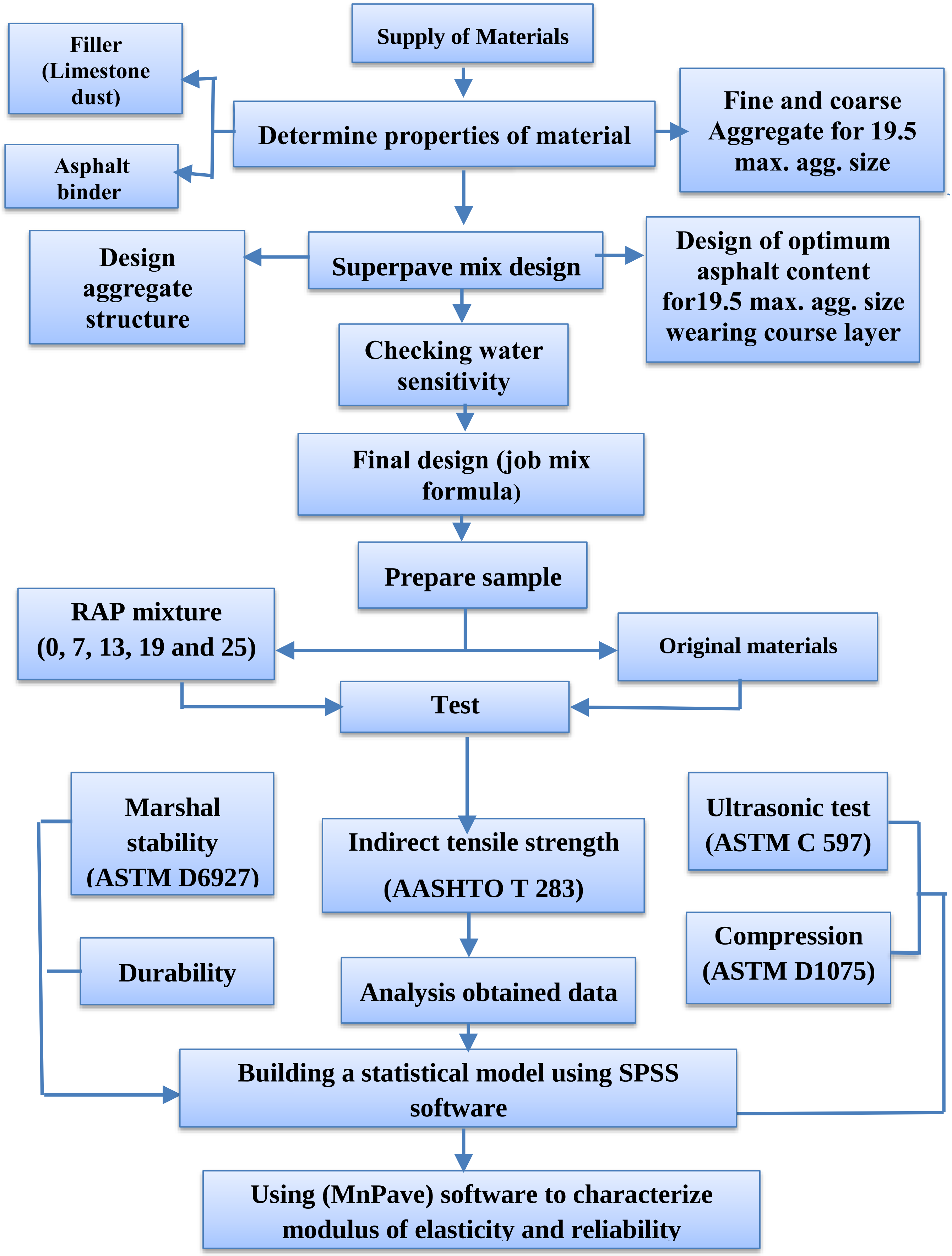

Figure (3-1) Scheme of the study.

3.2.1 Materials selection

The materials are divided as follows:

1. New materials: a- Asphalt cement, b-Aggregate, c- Mineral filler
2. Reclaimed materials: "Reclaimed asphalt pavement (RAP)".

3.2.1.1 New materials

1- Asphalt Cement

Two types of asphalt cement were utilized in this study; AC (40-50) and AC (60-70). They were obtained from Al- Daurah refinery, south-west of Baghdad. The physical properties of these types are shown in Table (3-1) and (3-2), respectively. The testing was carried out according to Iraqi specification (SCRB, 2003) and ASTM requirements.

Table (3-1): Physical properties of asphalt cement grade (40-50).

Test	Test conditions	Standard	Test value (measured)	Standard Limit using (NCCLR, 2013) according to SCRB / R9, 2003
Penetration	100 gm, 25°C, 5 sec., (0.1mm)	ASTM D5	44	40-50
Ductility	25°C, 5cm/min	ASTM D113	+113	+100
Softening point (ring & ball test)	(4±1)°C/min.	ASTM D36	54	>52 °C
Specific gravity asphalt	25°C	ASTM D70	1.032	-----
Flash and fire points	…..	ASTM D92	Flash 335°C	> 232 °C
			Fire 339°C	-------
Loss on heating	163 °C, 50gm, 5 hr	ASTM D1754	0.242%	
Kinematic viscosity	Pa.sec	ASTM D88	0.537 @ 135°C 0.15 @ 165°C	
Retained penteration of residue	25°C, 5cm/m	ASTM D113	72% cm	>55
Ductility of residue	25°C, 5cm/m	ASTM D113	66 cm	>25

Table (3-2): Physical properties of asphalt cement grade (60-70).

Test	Test Conditions	Standard	Test value (measured)		Standard limit using (NCCLR, 2013) according to SCRB / R9, 2003
Penetration	100 gm, 25°C, 5 sec., (0.1mm)	ASTM D5	66		60-70
Ductility	25°C, 5cm/min	ASTM D113	+125		+100
Specific gravity Asphalt	25°C	ASTM D70	1.025		-----
Flash and fire Points	…..	ASTM D92	Flash	296°C	> 232 °C
			Fire	320°C	-------
Loss on heating	163 °C, 50gm, 5 hr	ASTM D1754	0.365		< 0.75
Kinematic Viscosity	Pa.sec	ASTM D88	0.475 @ 135ºC* 0.113@ 165ºC**		

2- Aggregate Selection

The crushed aggregate used in this work was brought from Al-Sadour quarry. This aggregate is widely used in local asphalt paving in Baghdad city. The physical and chemical properties of aggregate used are shown in Tables (3-3) and (3-4), respectively.

Table (3-3): Physical properties of aggregate, according to ASTM requirements and Iraqi Specifications (SCRB/ R9, 2003).

Laboratory Test		**ASTM Designation and Specification**	**Results**			
Specific gravity	**Coarse aggregate**	ASTM C127	sieve size	Apparent Gs	Bulk Gs	Abs.%
			1/2" (12.5 mm)	2.64	2.623	0.41%
			3/8" (9.5 mm)	2.614	2.583	0.54%
			#4 (4.75 mm)	2.591	2.573	0.47%
			Crashed sand (< #4)	2.679	2.64	0.63%
Angularity for coarse aggregate		ASTM D 5821 Min 95%	97%			
Soundness for coarse aggregate		ASTM C88 10-20% Max	4.3%			
Equivalent sand (clay content)	**Crushed(<#4)**	ASTM D2419 Min 45%	56%			
Flat & elongation aggregate	**Flat**	ASTM D4791 Max 10%	1%			
	Elongation		2%			
Toughness, by (Los Angeles Abrasion)	**Aggregate Size < 25 mm**	ASTM C131 35 % Max	20.88%			

.

Table (3-4): Chemical Properties of Selected Aggregate

Chemical Compound	Content,%
Silica, SiO_2	84.73
Lime, CaO	3.37
Magnwasia, MgO	0.53
Sulphuric Anhydride, SO	2.9
Alumina,AlO	0.62
Ferric Oxide, FeO	0.58
Loss on Ignition	6.25
Total	98.98
Mineral composition	
Quartz	81.2
Calcite	10.02

***Test was conducted by the National Center for Construction Laboratories and Researches (NCCLR).**

3- Mineral filler selection

Filler is a non-plastic material passing sieve No. 200 (0.075 mm), which is usually used to fill the voids and improve mixture properties. The filler used in this work is limestone dust brought from the lime factory of Karbala' governorate. The physical properties of the lime are presented in Table (3-5).

Table (3-5): Physical properties of limestone filler

Property	Test Result
Specific gravity	2.72
%Passing sieve No. 200 (0.075 mm)	96%

3.2.1.2 Reclaimed asphalt pavement

The reclaimed asphalt pavement materials (RAP) are brought from Reclaimed Asphalt stoke of Mayoralty of Baghdad-project office at Altalbia-region in Baghdad City as shown in Figure (3-2). It was heavily crumbled by various cracks and potholes present on the surface. Recycled materials were gained through milling machines at a depth of 5 cm or more from the surface of the road.

***Figure (3-2): Reclaimed asphalt pavement* sampling location (google maps).**

1-Methodology of adding RAP

Virgin HMA mixtures were mixed with four different percentages of RAP; (7, 13, 19 and 25) percent by weight of total mix. First, the fractionated RAP is dried to make it workable and to mix it with the virgin materials. The RAP is heated to a temperature of 110°C (230°F) for a period of not more than 2 hours (McDaniel et al., 2001). In this study, the RAP was fractionated into coarse RAP (>4.75 mm) and fine RAP (<4.75 mm). Half of the weight of RAP selected to be added to the virgin HMA was coarse RAP and the other half was fine RAP. When batching out the RAP aggregates, it is important to remember that part of the weight of the RAP is binder. It is necessary to increase the weight of RAP and decrease the amount of new binder added to take the presence of this RAP binder into account. Batching a RAP mixture in this study is, perhaps, best illustrated by an example. Marshall Specimen is assumed to be prepared with RAP content of 7%.

- Wm = Weight of Marshall Specimen is 1200g.
- Rc = RAP content is 7%
- WR = Weight of RAP = 7% * 1200 = 84 gm.
- Fine RAP (Passing sieve No.4) to be added = 50% * WR

= 50% * 84 = 42 gm.

- Coarse RAP (Retained on sieve No.4) to be added = 50% * WR

= 50% * 84 = 42 gm.

- Weight of HMA mixture without RAP = 1200 – 84 = 1116 gm.
- The percent of virgin asphalt cement with presence of 7% RAP in the mix O.A.C = O. A. C of virgin mix $*\frac{\text{Weight of HMA mixture without RAP}}{\text{Weight of Marshall Specimen}}$

$= 4.8\% \ *\frac{1116}{1200}=4.46\%$

- Weight of new asphalt binder Wb ' = 4.46% * 1200 = 53.52
- Weight of aggregate with presence of 7% RAP in the mix = Wm – WR-Wb

  = 1200 – 84 – 53.52 = 1062.48 gm

2-Physical properties of RAP

After extraction test, Table (3-6) provides a summary of the typical ranges of physical properties of RAP and the gradation of aggregate is shown in Figure (3-3), after extraction and sieving on sieve #4.

Table (3-6) Physical Properties of Reclaimed Asphalt Mixture (RAP).

Laboratory test		**ASTM designation**	**Test results**
Coarse aggregate	Apparent specific gravity	C-127	2.63
	Bulk specific gravity	C-127	2.6
	Water absorption, %	C-127	0.69
Fine aggregate	Apparent specific gravity	C-128	2.53
	Bulk specific gravity	C-128	2.6
	Water absorption, %	C-128	0.615
Asphalt cement	Asphalt content	D2172	4%

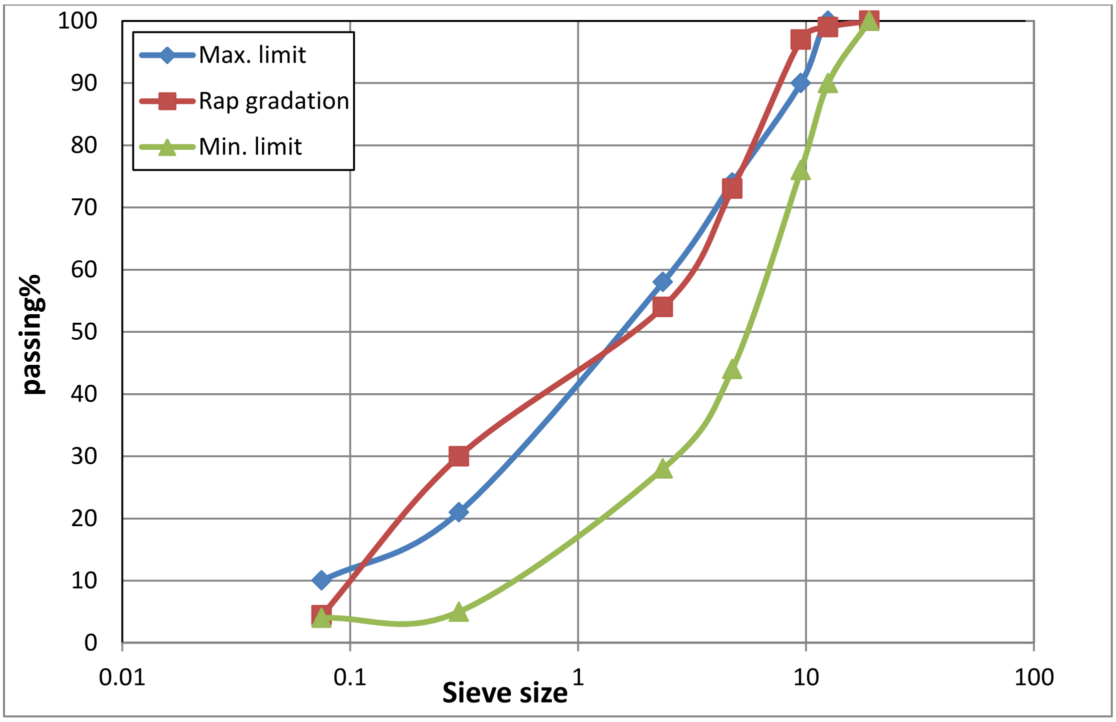

Figure (3-3) Specification limits and RAP gradation of (SCRB, 2003) for the surface course layer.

3.2.2 Selecting design aggregate structure

A new approach of specifying aggregate gradation was improved in superpave system. It uses a modification of a path already used by some agencies. The 0.45 power gradation chart was used to define a permissible gradation. A significant feature of the 0.45 power chart is the maximum density gradation. This gradation plots as a straight line from the maximum aggregate size through the origin. Superpave uses a standard set of ASTM sieves and the following definitions with respect to aggregate size (AASHTO M 323):

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

The maximum density gradation represents a gradation in which the aggregate particles fit together in their densest possible arrangement. Clearly this is a gradation to avoid because there would be very little aggregate space within which to develop sufficiently thick asphalt films for a durable mixture. To specify aggregate gradation, one additional feature are added to the 0.45 power chart:

- Control points.

Control points function as master ranges through which gradations must pass. They are placed on the nominal maximum size, an intermediate size (2.36 mm), and the dust size (0.075 mm). Figure (3-4) shows a 0.45 power gradation chart with a maximum size, nominal size and control points. This gradation practically always results in tender mix behavior, which is manifested by a mixture that is difficult to compact during construction and offers reduced resistance to permanent deformation during its performance life.

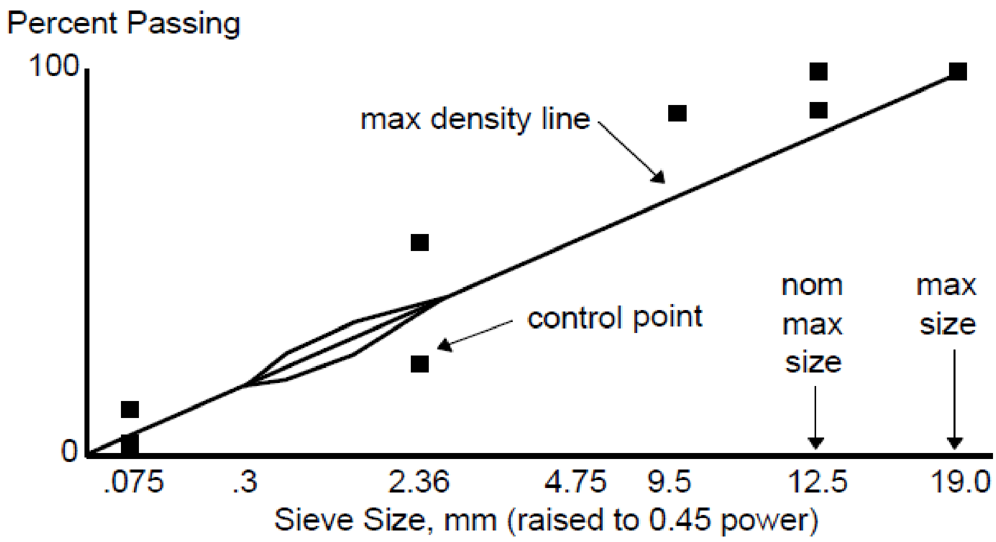

Figure (3-4): Superpave gradation limits (National Highway Institute, USA, 1998).

The nominal maximum size in this study was 12.5 mm selected for the three trial blends as illustrated in Table (3-7) and Figure (3-5).

Table (3-7): Percent passing by weight of selected aggregate gradation (12.5 mm nominal maximum size, surface course).

Sieve size		**Superpave Specification , 2007**		**Iraqi Specification (SCRP R9, 2003) surface layer type IIIA**		**%passing Blend (1)**	**%passing Blend (2)**	**%passing Blend (3)**
Standard Sieves	**English Sieves**	**max**	**min**	**Max**	**min**			
19 mm	3/4"	100	--	100	--	100	100	100
12.5 mm	1/2"	100	90	100	90	93	95	97
9.5 mm	3/8"	---	90	90	76	79	83	87
4.75 mm	#4	---	---	74	44	46	50	54
2.36 mm	#8	58	28	58	28	32	34	36
1.18 mm	#16			---	---	22	24	26
0.6 mm	#30			---	---	15	17	19
0.3 mm	#50			21	5	11	12	14
0.15 mm	#100	---	---	---	---	7	8	9
0.075 mm	#200	10	2	10	4	4	4.5	5
Filler						0	0	0

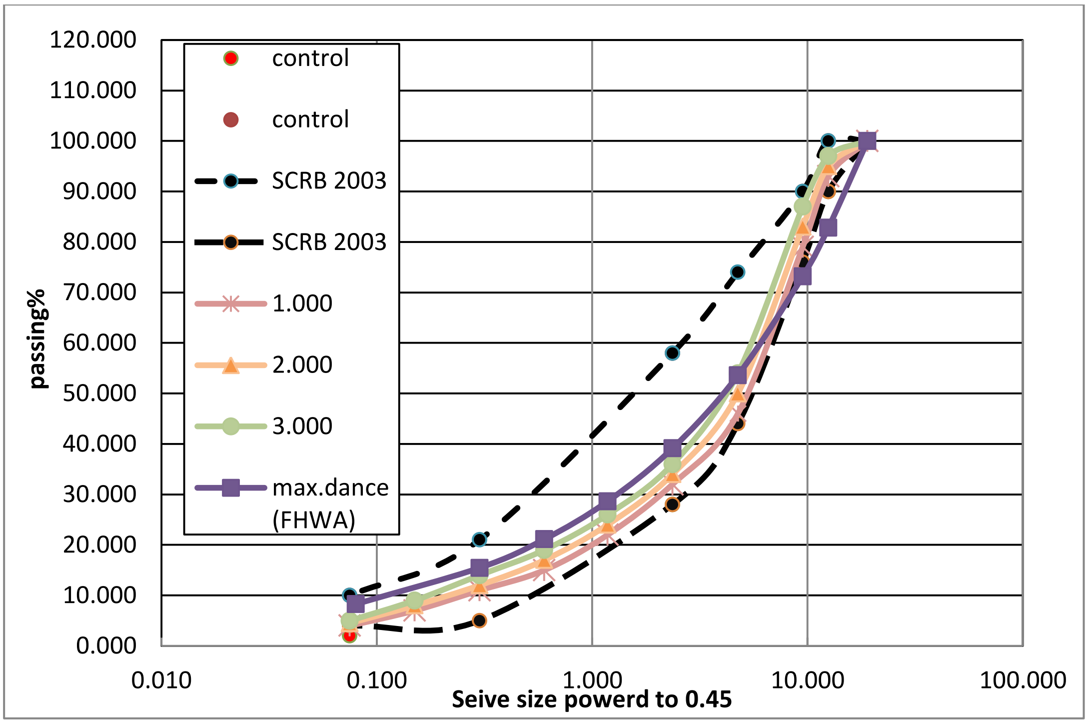

Figure (3-5): Aggregate gradation chart.

3.2.2.1 Selection of trial asphalt binder content

For each trial blend, two specimens were compacted by using Superpave Gyratory compactor (SGC) and for the each trail blend the volumetric property is determined. For each specimen approximately 4500 grams of mixture was used. While for loose mixture 2000 grams was used to obtain a maximum theoretical specific gravity (G_{mm}) using AASHTO T 209. The (SGC) was conformed to AASHTO T 312. The ram was applied and maintained a pressure of 600 (± 18) kPa to the specimen during compaction. The compactor was tilted the specimen molds at a 1.25° (± 0.02°) angle and gyrate the specimen at a rate of 30.0 (± 0.5) gyrations per minute. The diameter of the mold was 150 mm from inside.

The mixture was heated to the mix temperature between (159-165) $^{\circ}$C. The range of compacting temperature was between 148 and 153ºC for asphalt grade (40-50) while the mixing and compacting temperatures for asphalt grade (60-70) were about (154-159) ºC and (144-149) ºC, respectively. The temperature of mixing and compaction are dictated using a plot of viscosity versus temperature after measuring the rotational or Brookfield viscosity at two temperatures (135 and 165°C) for asphalt using Brookfield viscometer test which is standardized by ASTM – D 4402-02 as shown in Figures (3-6) to (3-8). Asphalt mixtures are mixed and compacted at asphalt temperatures conforming to viscosities of 0.17± 0.02 and 0.28 ± 0.03 Pascal-seconds (Pa·s), respectively (Asphalt Institute, 2003), This test was carried out at Al-Nahrain University.

Figure (3-6): Rotational viscometer device at Al-Nahrain University.

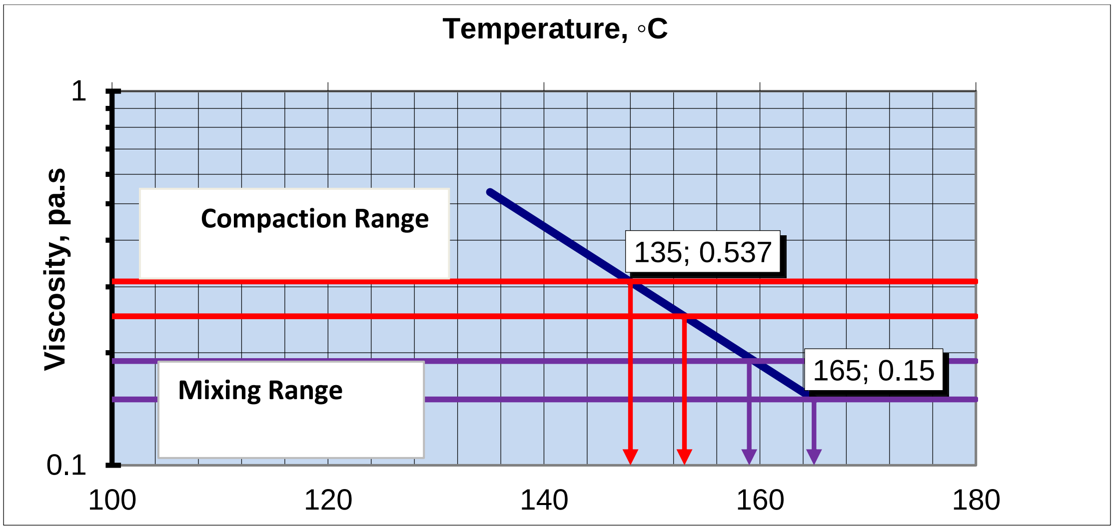

Figure (3-7): The mixing and compacting temperatures determination for asphalt penetration grade (40–50).

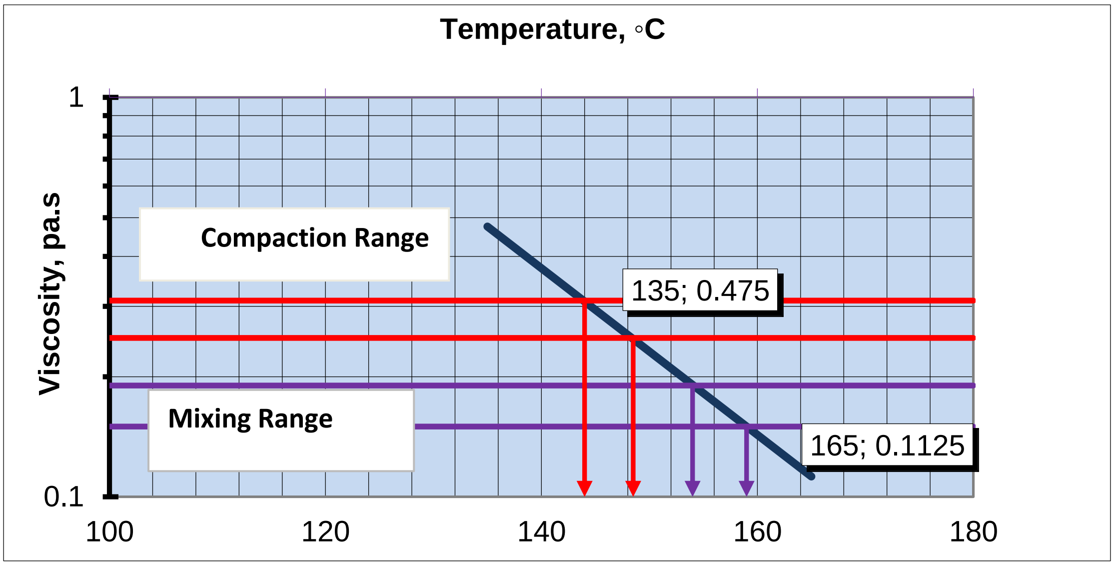

Figure (3-8): The mixing and compacting temperatures determination for asphalt penetration grade (60–70).

The specimens were then placed in oven at (135°C) for (2-4 hours) for short term aging, (Harman *et al*., 2009). Finally, the specimens are then removed and either compacted with standard specification ASTM D 6925 or allowed to cool loose (for Gmm determination) according to standard specification AASHTO T209. The number of gyration used for compaction is determined based on the traffic level according to AASHTO R35 as shown in Table (3-8).

Three gyration levels are of interest:

- Design numbers of gyrations (N_{design}).
- Initial number of gyrations ($N_{initial}$), and
- Maximum numbers of gyrations ($N_{maximum}$).

Table (3-8): Superpave gyratory compaction effort according to AASHTO R 35.

Superpave Design Gyratory Compactive Effort			
Design $ESAL_S$ (millions)	Compaction Parameters		
	$N_{initial}$	N_{design}	$N_{maximum}$
< 0.3	6	50	75
0.3 to < 3	7	75	115
3 to < 10	8	100	160
≥ 30	9	125	205

Test specimens are compacted using N_{des} gyrations. The relationship between N_{des}, N_{max}, and N_{ini} are:

$\boldsymbol{Log_{10}\ N_{max} = 1.10 \times Log10\ N_{des}}$ ………………………… (3-1)

$\boldsymbol{Log_{10}\ N_{ini} = 0.45 \times Log10\ N_{des}}$ ………………………….. (3-2)

The number of gyrations required for compaction is determined based on the traffic levels which are 3 to < 10 millions ESALs for Baghdad, Iraq.

3.2.3 Design of asphalt binder content

Once the design aggregate structure is determined, trial blend 2 is selected according to the Superpave specifications. The mixture properties are estimated to determine design asphalt binder content. Two specimens are compacted at each of the following asphalt contents:

- Estimated asphalt content
- Estimated asphalt content ± 0.5%, and
- Estimated asphalt content + 1.0%.

For Trial Blend 2, the binder contents for the mix design were 4.1% AC, 4.61% AC, 5.1% AC, and 5.61% AC.

This process is applied to both types of asphalt (40-50) and (60-70). In addition, two specimens are adapted for determination of G_{mm} at the estimated asphalt content. After short term aging, the samples are then removed and either compacted or allowed to cool loose mix for (G_{mm}) determination. Each specimen is compacted by using the maximum number of gyrations (N_{Max}=160), the final calculated bulk specific gravity of each compacted specimen is then compared to the final measured bulk specific gravity of that specimen, and then the correction factor is calculated. The bulk specific gravity at other gyration is subsequently adjusted (G_{mm} (corr)) using the correction factor. Finally, the % G_{mm} for the various gyration levels is calculated by dividing the corrected bulk specific gravity (G_{mb}(corr)) by the measured value for G_{mm}. The bulk specific gravity (G_{mb}) of the specimen is determined using AASHTO T166. G_{mm} of each blend is determined by using AASHTO T209. In this work, the design asphalt binder content is 4.83% for asphalt grade (40-50) and 4.7% for asphalt grade (60-70), these values are corresponds to 4.0 percent air voids at the design number of gyrations N_{des}= 100 gyrations. The design aggregate structure containing the design asphalt binder content becomes the design asphalt mixture. Figure (3-9) illustrates the procedure of testing and sample preparation.

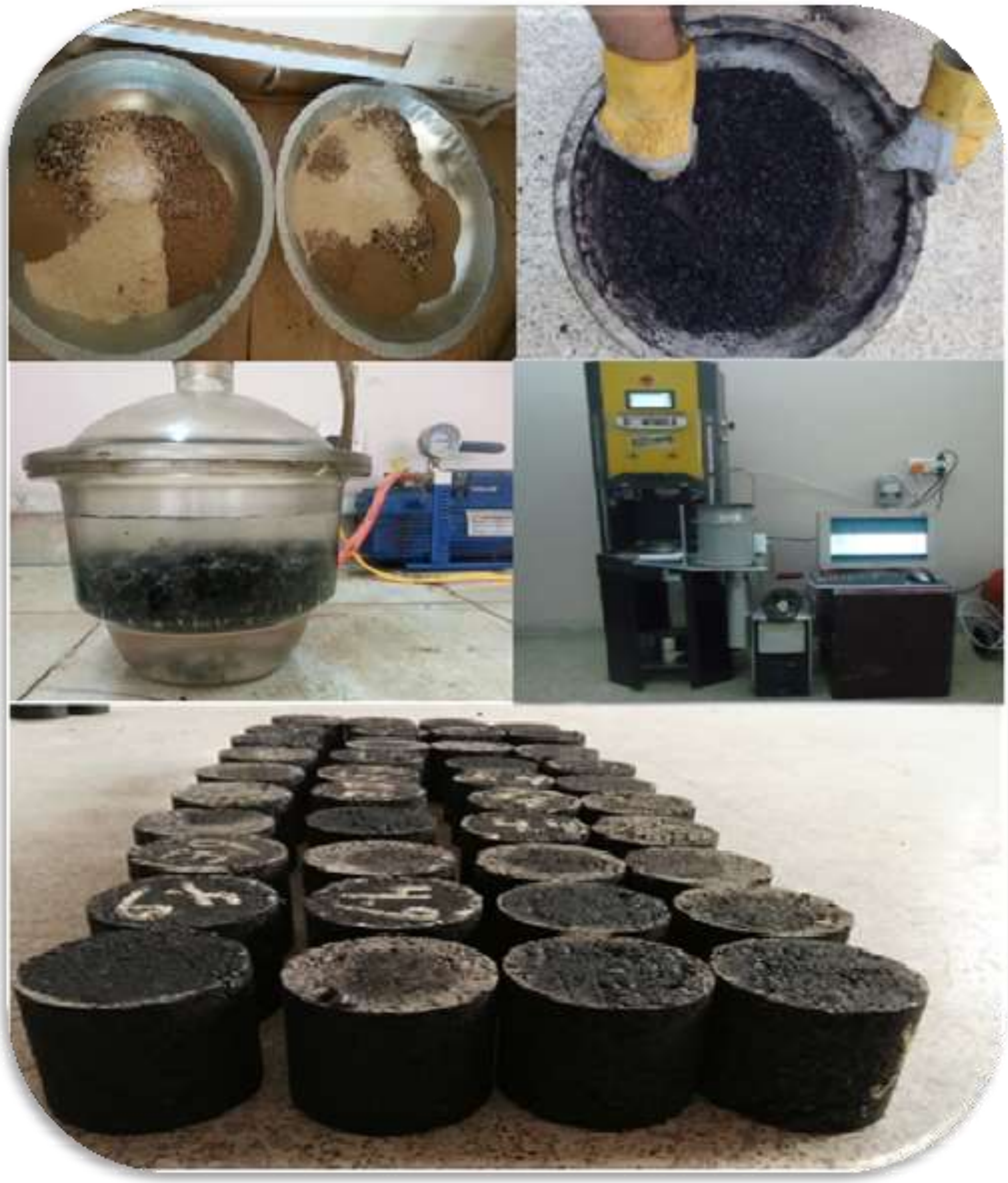

Figure (3-9) Samples preparation for Superpave gyratory compactor.

3.2.4 Moisture sensitivity

The last step in the "Superpave mix design" procedure is to appreciate the moisture susceptibility of the designing mixture, where the adhesion between the asphalt and aggregate is an important, yet complex and not well understood, property that helps ensure good pavement performance, (Asphalt Institute, 1996). This step is accomplished by performing AASHTO T 283 “Resistance of Compacted Bituminous Mixtures to Moisture Induced Damage” testing on the design aggregate blend at the design asphalt binder content. Specimens are compacted to approximately 7% air voids (height of 67 mm).

One subset of three specimens is considered control specimens. The other subset is the conditioned subset. The conditioned subset is subjected to vacuum saturation followed by an optional freeze cycle, followed by a 24-hour thaw cycle at 60 °C. After conditioning, both subsets are tested for indirect tensile strength, which is accomplished by Indirect Tensile Machine in the University of Technology in a condition of equal speed (50.8 mm/min), and the maximum load is recorded. The test was adopted in the Building and Construction Engineering Department at the University of Technology.

The moisture susceptibility is determined as a ratio of the tensile strengths of the conditioned subset divided by the tensile strengths of the control subset. Figure (3-10) shows some of apparatuses used in the test. Indirect Tensile strength is determined as follow:

$$S_t = 2000P/\pi t\, D \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3\text{-}3)$$

where: St = indirect tensile strength, kPa

P = maximum load, N

t = specimen height immediately before tensile test, mm, and

D = specimen diameter, mm.

Then the Tensile Strength Ratio is determined as follows:

$$TSR = (Stm/Std) * 100 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3\text{-}4)$$

where: TSR= tensile strength ratio, percent

Stm = average tensile strength of the moisture conditioned samples, kPa, and

Std = average tensile strength of the dry samples, kPa.

The minimum Superpave criterion for tensile strength ratio is 80%. In this study, the tensile strength ratio for both asphalt (40-50) mixture and asphalt (60-70) mixture were (91%) and (88%), respectively.

Figure (3-10): Apparatuses used in indirect tensile strength test.

3.3 Laboratory Testing

The testing has been conducted as follows:

1- Extraction test.
2- The Marshall stability and flow of bituminous mixture.
3- Durability of Hot Asphalt Mixtures containing reclaimed asphalt Pavements for (1, 3 and 7 days).
4- Indirect tensile strength.
5- Immersion–Compression test.
6- Ultrasonic test.

Each specimen is compacted by using Superpave Gyratory Compacter (SGC) for achieving a design number of gyrations (100 cycles). The diameter of samples was 100 mm for all tests as shown in Figure (3-11).

Figure (3-11) Equipment of Superpave gyratory compacter.

3.3.1 Extraction test:

The test was conducted on the reclaimed asphalt pavement for extraction of the asphalt from aggregate and filler. The testing procedure is according to (*ASTM-D2172*). Figure (3-12) shows the test procedure and Table (3-9) shows the gradation of aggregate according to the Iraqi specifications' R9, 2003).

Figure (3-12) Extraction test procedure.

Tables (3-9): Sieve analysis of Extracted aggregate.

Sieve size		% Passing by weight Requirements by (SCRB, 2003)	%Passing by weight (RAP)
in	mm		
3/4	19	100	100
1/2	12.5	90-100	99
3/8	9.5	76-90	97
No.4	4.75	44-74	73
No.8	2.36	28-58	54
No.50	0.3	5-21	30
No.200	0.075	4-10	4.4

3.3.2 The Marshall stability and flow of bituminous mixture

For determining the stability and flow values, the specimen was immersed in a water bath at a temperature of 60° ± 1°C for a period of (30-40) minutes then the sample was placed in the Marshall stability testing machine. The load is at a constant rate of deformation of 50.8 mm (2 in) per minute until failure. The maximum loading (that causes failure of the sample) was reordered as Marshall stability and the total amount of deformation had been taken as Marshall flow. The obtained stability value was corrected for volume. Figure (3-13) shows the test setup. The test was adopted in the Building and Construction Engineering Department of the University of Technology.

Figure (3-13) Marshal Test procedure at University of Technology.

3.3.3 Durability of hot asphalt mixtures

To evaluate durability, a mixture is subjected to environmental conditioning, and a mixture property associated with load-related or environmental distress is measured before and after the conditioning process. Abrasion characteristics of the aggregate in the mixture must also be considered in the evaluation of durability. The greater the protection by asphalt concrete, more durable the mix will be. The fewer air voids in the total mix, the slower will be the deterioration of the asphalt concrete itself (Putri and Suparma, 2010).

1- Theoretical Approach:

It includes three steps as following

1- Marshall stability

2- Retained Marshall stability (RMS)

3- Durability index (DI)

1- Marshall Stability

Marshall Stability is calculated from the following equation (ASTM D6927-15)

$$\boldsymbol{So} = \boldsymbol{o} * \boldsymbol{R} * \boldsymbol{T} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (3-5)$$

where:

So = stability numeral (kN)

o = stability (kN)

R = Proving ring calibration (kN)

T = the matter test correction factor

2- Retained Marshall stability (RMS)

The retained marshall stability is expressed as a percentage and is defined in terms of the Marshall stability of the composition after an immersion process under set conditions (as defined later) as a percentage of the initial (absolute) Marshall stability of the composition. The RMS values were determined as follows (ASTM D 1075 – 96):

$\boldsymbol{RMS}=(\boldsymbol{Si}/\boldsymbol{So})*\mathbf{100}\%$………………………………… (3−6)

where:

RMS = Retained Marshall Stability (%)

Si = maximum stability in conditioned set based on times series, and

So = maximum stability in unconditioned set (0 days).

3- Durability Index (DI)

In this study, the formula used to calculate durability index is adopted from durability index formula when Marshall test. Durability index is calculated from the following equation (Putri and Suparma, 2010):

$$\boldsymbol{DI} = \left(\frac{\mathbf{1}}{\mathbf{2}\boldsymbol{tn}}\right) \sum_{\boldsymbol{t}=\mathbf{1}}^{\boldsymbol{n}-\mathbf{1}} (\boldsymbol{S}_{\boldsymbol{i}} - \boldsymbol{S}_{\boldsymbol{i}+\mathbf{1}}) * \left(\mathbf{2}\boldsymbol{tn} - (\boldsymbol{t}_{\boldsymbol{i}+\mathbf{1}} - \boldsymbol{t})\right) \ldots\ldots\ \ldots.(\mathbf{3}-\mathbf{7})$$

where:

Si+1 = percent of retained strength at time ti+1,

Si = percent retained strength at time ti, and

ti, ti+1 = immersion time (calculated from beginning of test) in days.

Durability Index is defined as the average strength loss area enclosed between the durability curves. Figure (3-14) shows the schematic description of durability curve. The test was conducted in the Building and Construction Engineering Department of the University of Technology.

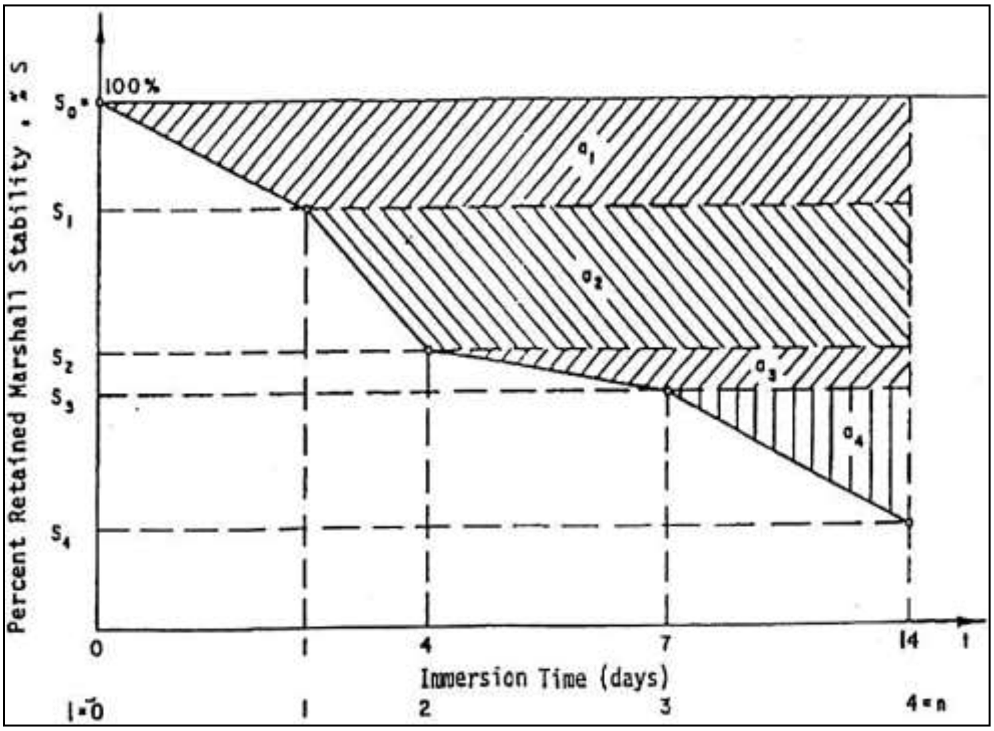

Figure (3-14) Schematic description of durability (Putri and Suparma, 2010).

3.3.4 Immersion–compression test

In this test, two sets of three specimens were prepared for both recycled mixtures by using gyratory compactor, because field compaction can be simulated in a progressive way using this method of compaction. This test was conducted according to ASTM D1075. An air void content of 6 percent was attained. One set of specimens was tested for the compressive strength at 25.0± 1ºC without conditioning and the other set of specimens were conditioned by immersing them in water bath at 60.0 ± 1ºC for 24 hours. After conditioning, the set was transferred to another water bath where temperature was maintained at 25.0 ± 1ºC. After storing the specimens for 2 hours in this bath, the compressive strength of each conditioned specimen was determined in accordance with ASTM D1074 as shown in Figure (3-15). A numerical index of resistance of bituminous mixtures to the damaging impact of water as the percentage of the main Strength that was retained after the immersion period, which should be a minimum of 0.7 (or 70%) as adopted by (SCRB/R9, 2003) for binder course as follows:

$$\boldsymbol{Index\ of\ Retained\ strength} = \frac{S_2}{S_1} * \mathbf{100} \ldots\ldots\ldots\ldots\ldots\ldots.(3-8)$$

Where:

S_1 = compressive strength of dry specimens (Set 1), and

S_2 = compressive strength of immersed specimens (Set 2).

Figure (3-15): Compressive strength test University of Technology.

3.3.5 Ultrasonic testing

The ultrasonic device is a portable seismic device that measures travel time of seismic wave pulses through a material. The seismic waves are generated by a built-in pulse generator, which transforms an electrical pulse to a mechanical vibration though a transducer. The seismic wave arrival time is recorded by a receiver, which is connected to an internal clock. The internal clock has the capability of automatically measuring and displaying the travel time of the waves. The travel time and the density of the specimen are used to determine the resilience modulus of the HMA specimens. The main advantage of this test is that it is nondestructive. In addition, the test can be performed on both laboratory-prepared specimens and field cores (Mashkoor, 2015).

3.3.5.1 Test procedure and calculations for ultrasonic test:

The specimens prepared for the test described above can be used to perform ultrasonic tests. The ultrasonic apparatus used in this study is shown in Figure (3-16). The elastic modulus of a specimen is measured by using an ultrasonic device containing a pulse generator and a timing circuit, coupled with piezoelectric transmitting and receiving transducers. The dominant frequency of the energy imparted to the specimen is 54 kHz. The timing circuit digitally displays the time needed for a wave to travel through a specimen. To ensure full contact between the transducers and a specimen, special removable epoxy couplant caps are used on both transducers. The receiving transducer, which senses the propagating waves, is connected to an internal clock. The clock automatically displays the travel time **"tv"** that can be used to calculate the constrained modulus **"Mv"** according to ASTM (C 597 – 02).

$$\mathbf{Mv = \rho Vp^2 = \rho\ (L/tv)^2} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \text{(3-9)}$$

where:

Mv= constrained modulus

ρ = Density, gm/mm^3

Vρ = Compression wave velocity mm/ms, and

L = Average length of the specimen mm.

tv = Travel time ms.

This equation may be simplified to:

$$\mathbf{Mv} = \frac{4ml}{\pi d^2 tv^2} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ (3\text{-}10)$$

where:

m: Mass of the specimen gm,

d: Average diameter of the specimen mm, and

Then Young´s modulus "**Ev**" may be determined from

$$\mathbf{Ev = Mv}\ \left\{\frac{(1-2v)(1+v)}{(1-v)}\right\} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ (3\text{-}11)$$

The Poisson´s ratio, υ, can be assumed based on experience, for asphaltic material generally assumed from (0.3 - 0.4) (Huang, 2010).

Figure (3-16) Ultrasonic testing device and samples.

CHAPTER FOUR

RESULTS AND DISCUSSIONS

4.1 General

This chapter introduces a summary of test results and discusses the effect of RAP addition on volumetric properties and the performance of HMA which achieved by using flexible pavement tests such as: Marshall stability, indirect tensile strength, compressive strength and ultrasonic test.

4.2 Selection of Design Aggregate Structure

The initial asphalt binder content (4.5%) was estimated based on previous local research, to evaluate the trial blends by compacting specimens and determining the volumetric properties of each trial blend as will be presented in following sections. Table (4-1) shows the specific gravity and initial asphalt content of the trial blends.

Table (4-1): Result of different specific gravity and initial asphalt contents.

Blend No.	Initial asphalt content (P_i) %	Bulk specific gravity, ($\mathbf{G_{mb}}$)	Effective specific gravity, ($\mathbf{G_{se}}$)	Apparent specific gravity, ($\mathbf{G_{sa}}$)
1	4.5	2.611	2.636	2.643
2	4.5	2.613	2.639	2.646
3	4.5	2.615	2.641	2.648

The volumetric properties for three blends are calculated at the initial asphalt content, as well as, the maximum number of gyrations for each trial blend. The air voids, voids in mineral aggregate (VMA), voids filled with asphalt (VFA), G_{mm} and dust proportion (Dp) are determined as shown in Tables (4-2), (4-3), and (4-4).

Table (4-2): Dust proportion of the trial blends.

Trail Blend	Filler, %	Dust Proportion
1	4.00	0.89
2	4.50	1.06
3	5.00	1.19

Table (4-3): Estimated mixture volumetric properties @ N_{des} .

Blend	Initial %AC ($Pbe_{ini.}$)	Trial %AC (Pb_{est}.)	%Air Voids	%VMA	%VFA
1	4.5	4.84	4.85	14.97	73.27
2	4.5	4.61	4.27	14.96	73.26
3	4.5	4.58	4.19	15.93	74.88

Table (4-4): Estimated mixture compaction properties @ N_{des} and @N_{max}.

Blend	% $\mathbf{Pbe_{ini.}}$	% $\mathbf{Pb_{est.}}$	%$\mathbf{G_{mm}}$ @$\mathbf{N_{ini}}$	%$\mathbf{G_{mm}}$ @ $\mathbf{N_{des}}$	%$\mathbf{G_{mm}}$@$\mathbf{N_{max}}$
1	4.5	4.84	87.33	95.15	97.27
2	4.5	4.61	87.3	95.73	97.22
3	4.5	4.58	87.2	95.81	97.5

For the surface (wearing) course, the nominal maximum aggregate size was 12.5 mm and the Superpave compaction criteria were (VMA=14%, VFA =65-78%, Va =4%, and DP between 0.6 and 1.2) according to standard specification of AASHTO M323-12. At the last, Blend 2 was selected as the best blend according to the Superpave specifications as detailed in Appendix (A).

4.3 Selection of Optimum Asphalt Binder Content

Once the aggregate structure is selected, the selection of optimum asphalt content started. The specimens are compacted at varying asphalt binder contents (estimated asphalt binder content %, estimated asphalt binder content ±0.5%, and estimated asphalt binder content +1.0%). The volumetric properties (%AV, %VMA, %VFA, G_{mm} and dust proportion) for (Blend 2) are calculated as shown in Tables (4-5) and (4-6).

Table (4-5): Asphalt contents versus $\%G_{mm}$ $@N_{Ini}$, N_{des} and N_{max} for Blend2.

% Asphalt content	$\%G_{mm}@ N_{Ini}$	$\%G_{mm}@ N_{des}$	$\%G_{mm}@ N_{max}$
4.1	88.76	94.43	95.71
4.61	88.56	95.64	96.84
5.1	88.3	96.3	97.54
5.61	87.63	96.93	97.87

Table (4-6): Mix volumetric properties at N_{des} for Blend 2.

% Asphalt content	%AV	%VMA	VFA	Dust Proportion
4.1	5.57	15.15	73.59	1.09
4.61	4.36	15.11	73.53	0.97
5.1	3.7	15.1	73.51	0.88
5.61	3.37	16.23	75.36	0.80

The design asphalt binder content is established at 4.0% air void, the design optimum asphalt binder content is determined as 4.8% for asphalt grade (40-50) and 4.7% for asphalt grade (60-70), all other mixture properties are checked at the design asphalt binder content to verify that they meet the Superpave criteria. Table (4-7) shows the design mixture properties that meet the Superpave specifications at 4.8% and 4.7% binder content for asphalt grade (40-50) and (60-70), respectively.

The graphs of air voids, %VMA, %VFA, G_{mm} % and dust proportion versus asphalt binder content can be generated as shown in Figures (4-1) to (4-5) for asphalt grade (40-50) and Figures (4-6) to (4-10) for asphalt grade (60-70).

Table (4-7): Design mixture properties of asphalt grades (40-50) and (60-70).

Mix Property	Results		Criteria According to AASHTO M323-12
	(40-50)	(60-70)	
Opt. AC, %	4.8	4.7	-
VA, %	4.0	4.0	4.00%
VMA, %	15.05	14.97	14 % min
VFA,%	73.44	73.3	65% - 75%
Dust proportion	0.93	0.955	0.6 - 1.2
$G_{mm}@N_{ini}$,%	88.47	87.8	less than 89%

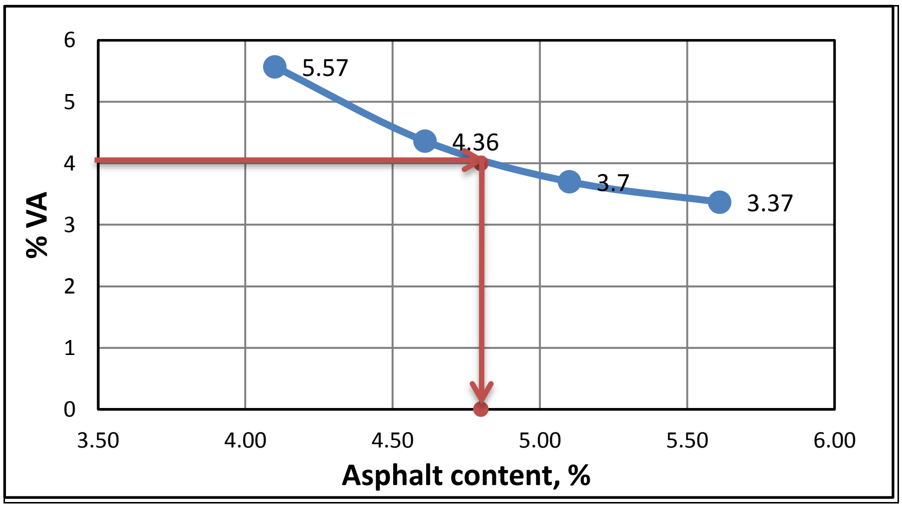

Figure (4-1): Asphalt content versus air void content (Av %) for asphalt (40-50).

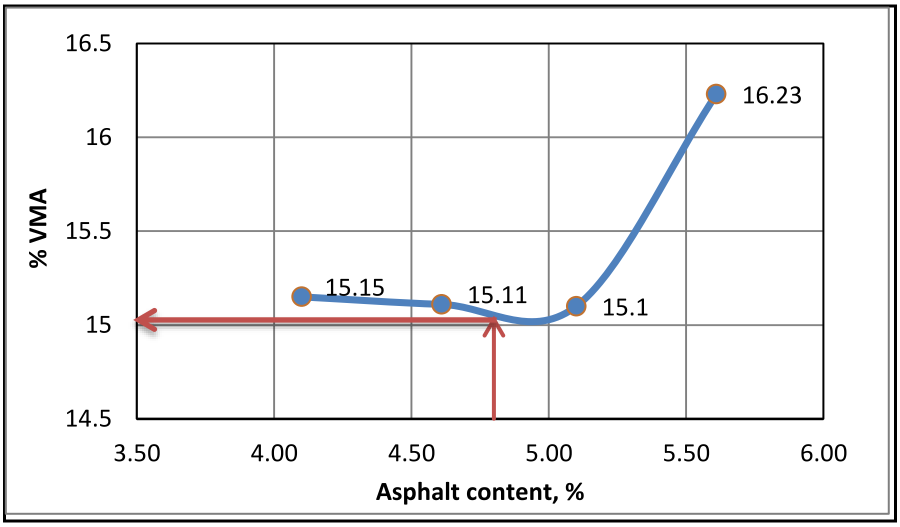

Figure (4-2): VMA versus asphalt content for asphalt (40-50).

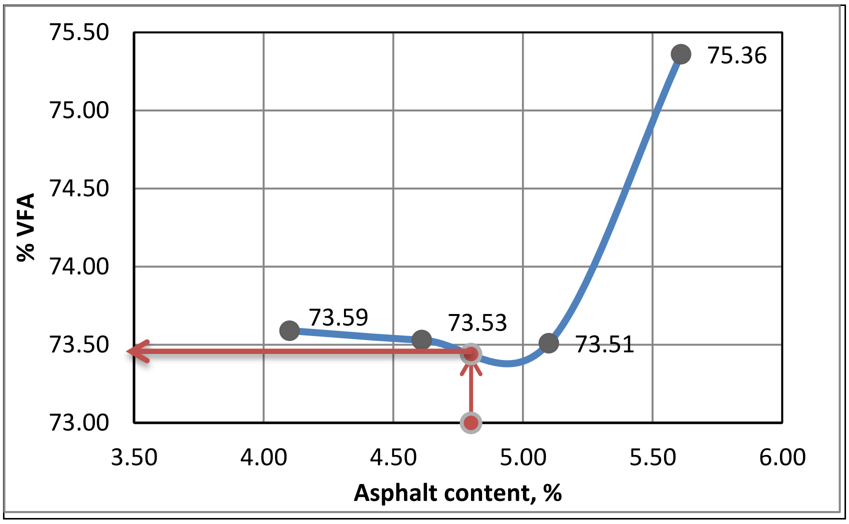

Figure (4-3): VFA versus asphalt content for asphalt (40-50).

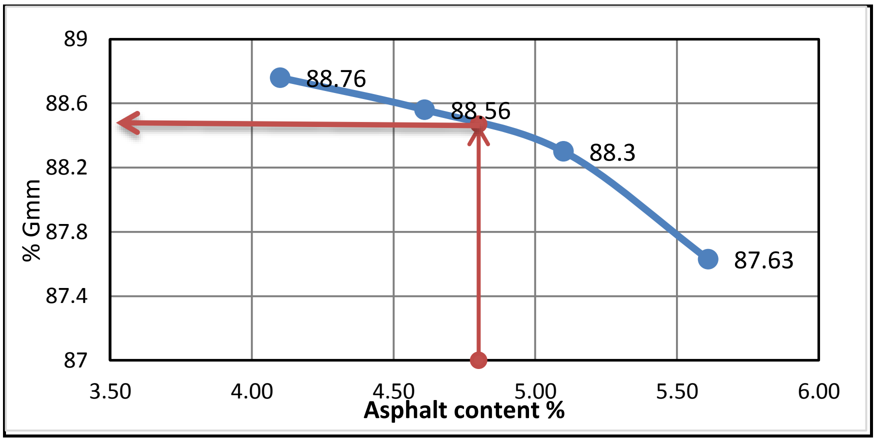

Figure (4-4) % G_{mm} versus asphalt content for asphalt (40-50).

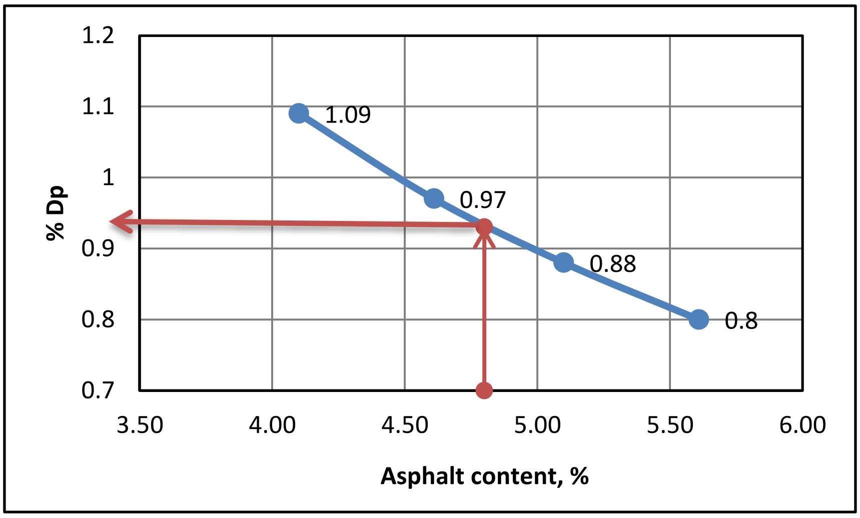

Figure (4-5) Dust proportion versus asphalt content for asphalt (40-50).

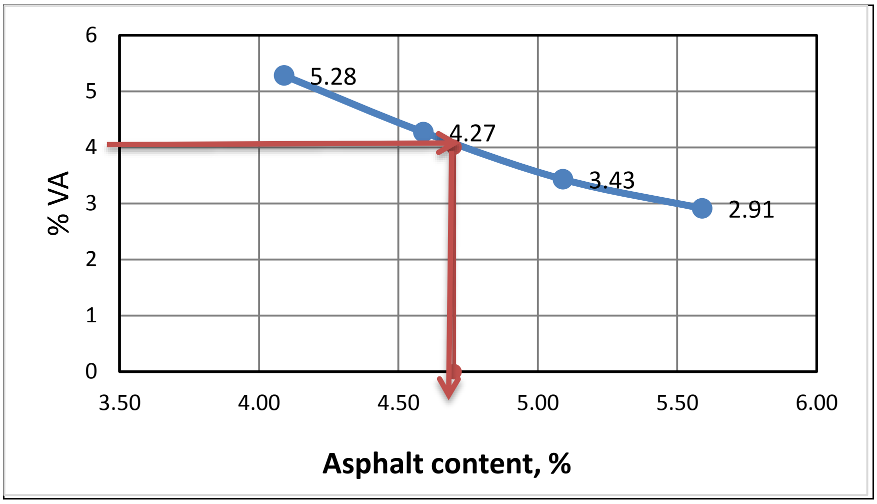

Figure (4-6) Asphalt content versus air void content (Av %) for asphalt (60-70).

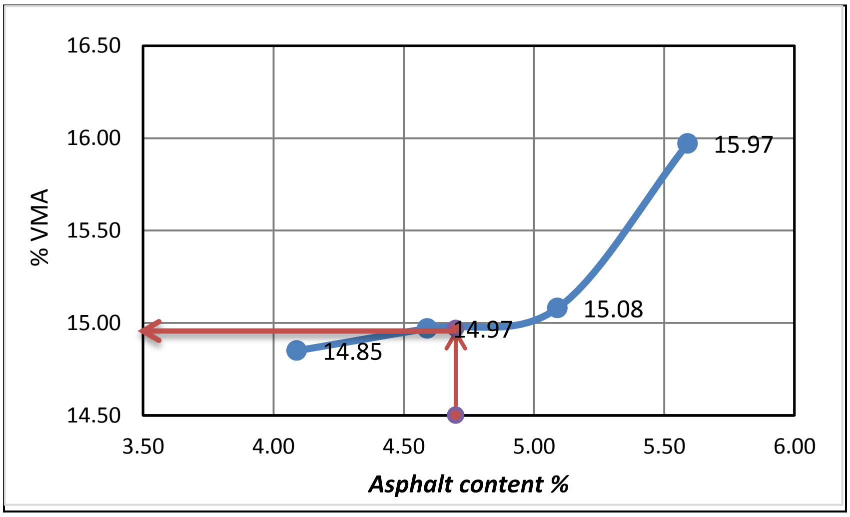

Figure (4-7): %VMA versus asphalt content for asphalt (60-70).

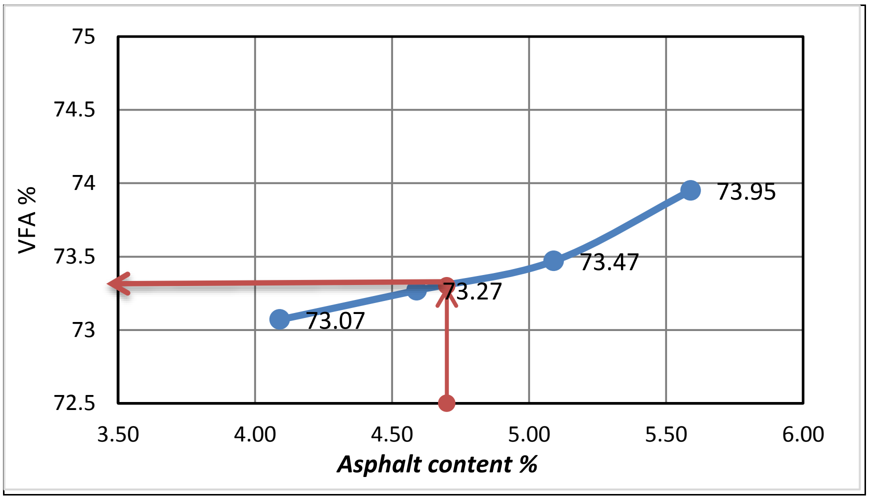

Figure (4-8): %VFA versus asphalt content for asphalt (60-70).

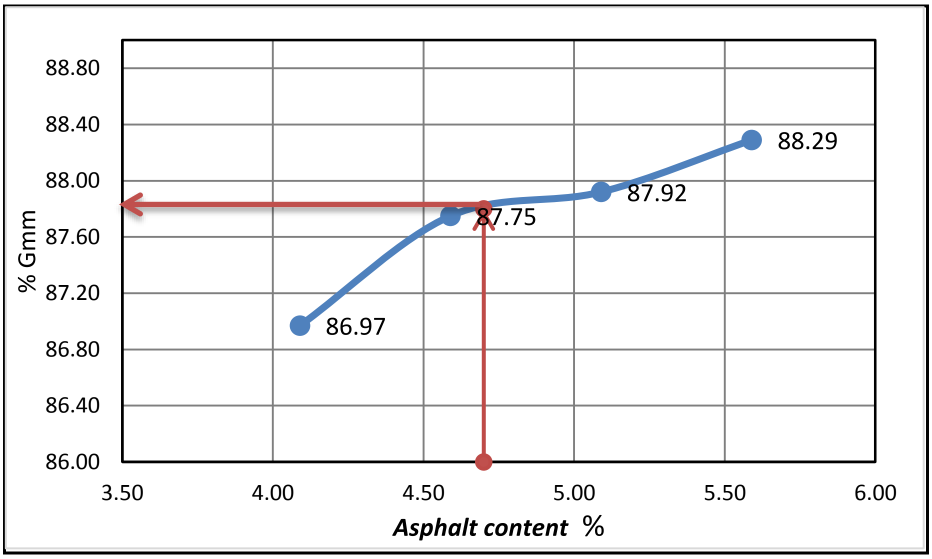

Figure (4-9): % G_{mm} versus asphalt content for asphalt (60-70).

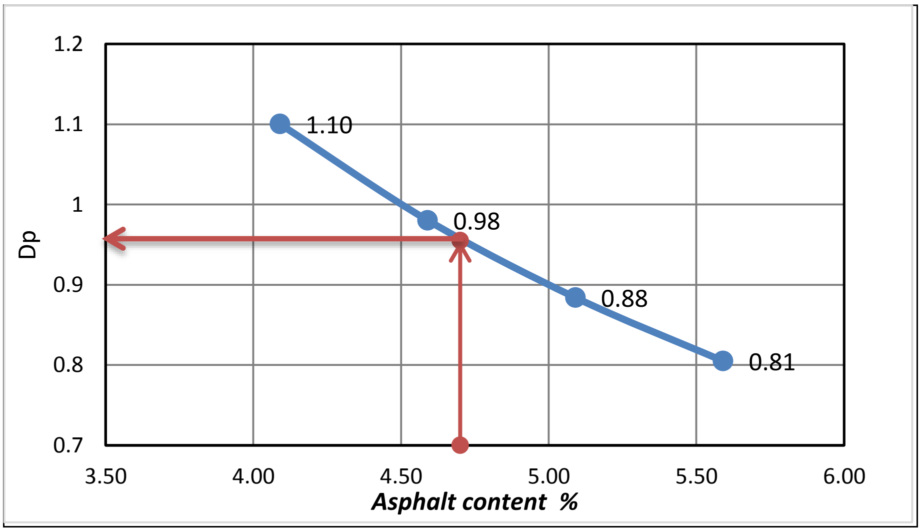

Figure (4-10): Dust proportion versus asphalt content for asphalt (60-70).

4.4 Mechanical and durability properties of HMA with RAP

The following paragraphs present the results of Marshall stability test and flow, durability test, indirect tensile strength, immersion-compression test and ultrasonic test.

4.4.1 Marshall stability and flow test

4.4.1.1 Marshall stability and flow test at optimum asphalt content

Marshall stability and flow test results for tested specimens are presented in Figures (4-11) and (4-12), respectively. Two specimens were compacted for each percentage and the average is taken according to the standard specifications (ASTM D6926-10). The specimen was immersed in a bath of water at a temperature of 60 ± 1°C for a period of 30 minutes. Then the sample was placed in the Marshall stability testing machine at a constant load rate of deformation of 50.8 mm (2 in) per minute until failure.

Figure (4-11) clearly shows that adding of the RAP to asphalt mixture has improved the Marshall stability, it is noticed that stability increased by about 13.82%, 21.4%, 26.73% and 34.5% when adding 7%, 13%m 19% and 25% RAP content, respectively.

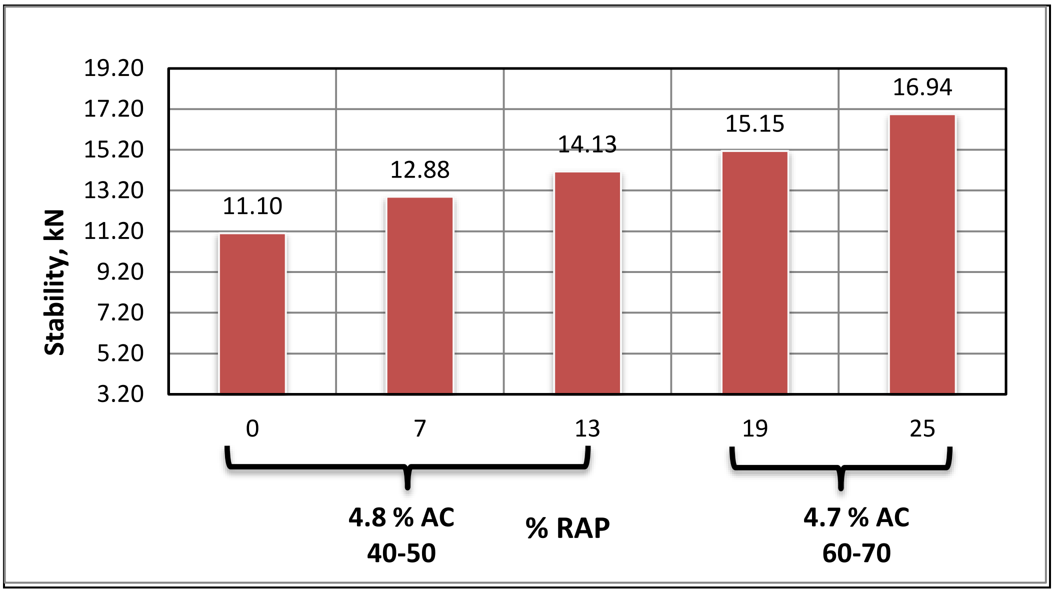

Figure (4-11) Relation between stability and RAP content.

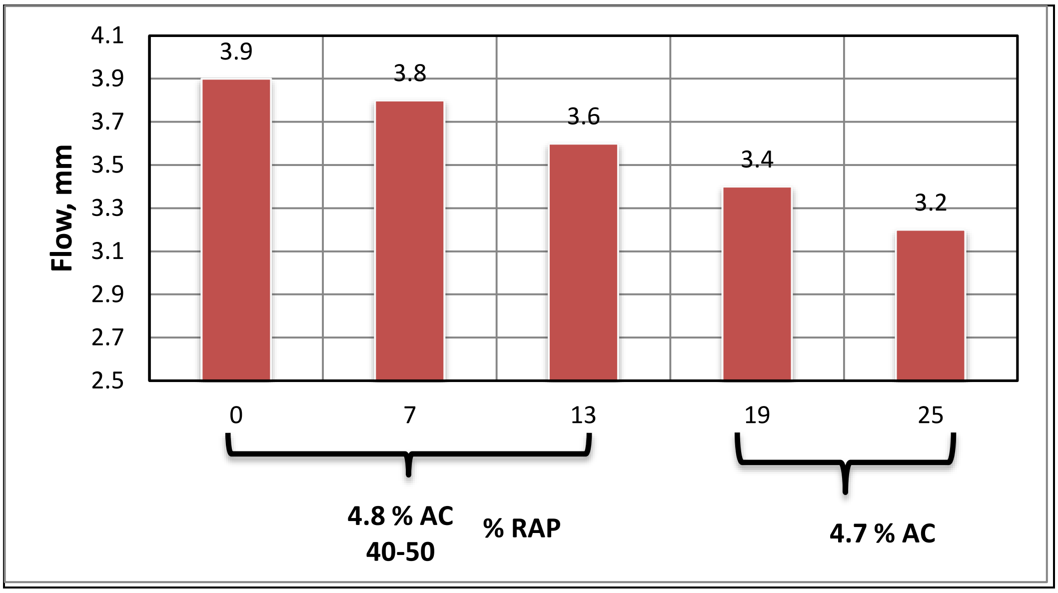

Figure (4-12) Relationship between flow and RAP percent content.

Flow is defined as the amount of vertical deformation of the specimen at failure and it whenever flow values are high, this denotes mix flexibility. The results showed that the flow decreases with increasing in RAP content. For example, the flow decreases by 17.95% when RAP increases in mixture to 25%. It is noted that all results are within the specification range which is (2-4) mm according to (SCRB/R9, 2003).

4.4.1.2 Marshall stability and flow test at optimum +0.5 asphalt content

Figures (4-13) and (4-14) show the laboratory results of stability and flow with the RAP ratios at + 0.5 optimum asphalt content. It is observed that the values increase from the original mixture as a quantity of RAP is added, for example when RAP increased to 7% and 19%, the stability values increased by 14.18% and 28.1%, respectively. For flow results, it is noticed that flow decreases gradually when the RAP ratios increase. The flow value is 3.8 at original mix and it decreased to 3.7 and 3.5 and 3 percent at % RAP of 7, 13 and 25, respectively. When comparing the results of Marshall stability, it was found that the stability values decrease when the amount of asphalt in the mixture increases compared to the mixture that contains the optimum asphalt ratio and due to the fact that increasing the amount of asphalt in the mixture increases the elasticity of the mixture and thus decreases its resistance.

Result of the stability in Marshall test clearly shows that the inclusion of RAP aggregate and asphalt in the "RAP mix" has improved the stability of the Marshall stability and reduced the loss of Marshall's stability to the "control mix". This is believed to be attributed to the fact that the RAP contains hardness asphalt, which will lead to increase stability due to the rise of asphalt viscosity. Hardened asphalt is more viscous than virgin asphalt of the same type and grade. The stability of the "RAP blend" will be less affected by the hot water than that of the "control blend" that is made entirely of virgin mixture. These results of stability are supported by the findings of Kandhal et al. (1995).

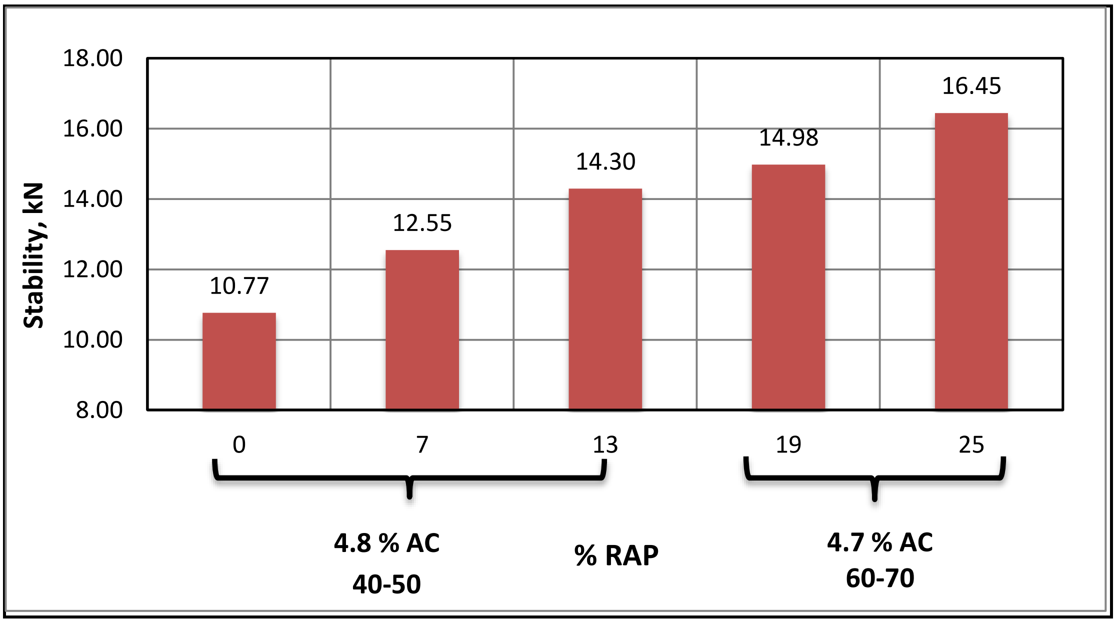

Figure (4-13): Effect of RAP content on stability at optimum +0.5 asphalt content.

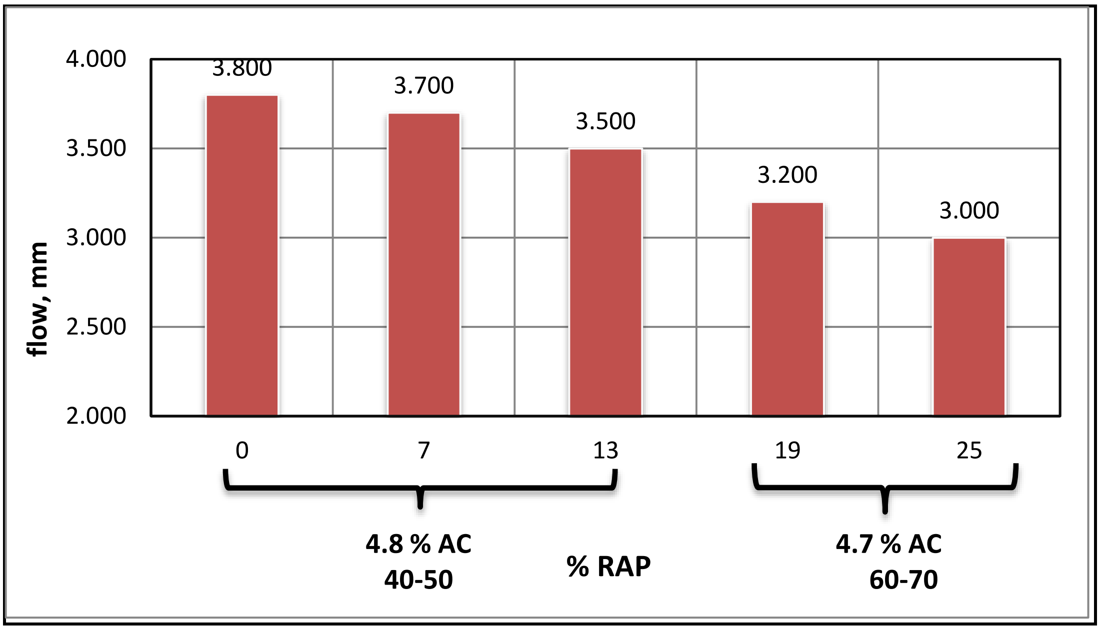

Figure (4-14): Effect of RAP content on Flow at optimum +0.5 asphalt content.

4.4.2 Retained Marshall stability (RS)

Retained of stability test was used to measure mix durability by evaluating the resistance of the investigated mixes to moisture damage. This test is intended to measure the loss of stability resulting from the action of water on compacted asphalt mixtures by comparing the stability of dry specimens which have been immersed in water path at 60° C for (1, 3 and 7) days. The result are listed in Tables (4-9) and (4-10).

From Tables (4-8) and (4-9), it is observed that the retained stability (RS) decreased with increasing of immersion day for all percents of addition at optimum asphalt content (AC) and optimum +0.5 asphalt content (Opt. +0.5 AC). For example, for 19% RAP content, the RS is 95.4% for one day emersion and is 87.1% for seven day immersion. It can be observed that the RS increases with increasing of RAP content; for example, the RS for 7 days is 86%, for 7% addition and is 88.3% for 25% RAP content. These results are at Opt. AC. As for the results of the asphalt increase by 0.5%, it is almost observed that the same increase in the optimum asphalt ratio is obtained. Where the value of RS increases as a quantity of RAP is added, it is also noted that this percentage decreases gradually by increasing the immersion periods.

Table (4-8): Retained stability (%) with days at optimum AC.

% RAP	RS for 1 day	RS for 3 day	RS for 7 day
0	0.921	0.874	0.847
7	0.929	0.901	0.860
13	0.951	0.926	0.880
19	0.956	0.923	0.871
25	0.954	0.931	0.883

Table (4-9): Retained stability (%) with days at optimum +0.5 AC.

% RAP	RS for 1 day	RS for 3 day	RS for 7 day
0	0.919	0.854	0.822
7	0.927	0.871	0.842
13	0.940	0.892	0.858
19	0.944	0.901	0.870
25	0.953	0.929	0.879

Figures (4-15) to (4-17) show comparison between the results of the stability and immersion day for all percents of RAP additions at optimum asphalt content and optimum +0.5 asphalt content. It can be seen that the stability for all percents is higher than the control mix. Stability increases with increasing RAP content for all immersing periods for both AC and +0.5 AC. The results show that the stability value of the original mixture at the optimum asphalt ratio is higher than that for the mixture containing an increase in the amount of asphalt by 0. 5%. When adding RAP by 7%, the stability value will increase by about 14. 53% and 15% from the original mixture at opt. AC and + 0.5 opt. AC, respectively. Stability value is roughly equal for opt. and +0.5 opt.AC. at 13% RAP, but it increases again at AC when compared to the mixture that contains an increase in asphalt when the RAP ratio rises to 19% and 25%. The reason that makes the stability increases and decreases the loss of stability was RAP containing hardened asphalt, which will lead to increase stability due to higher asphalt viscosity. Hardened asphalt is more viscous than virgin asphalt of the same type and grade. Therefore, when the two mixes are subjected to immersion, the stability of the "RAP Mix" is less affected by hot water than that of the "Control Mix".

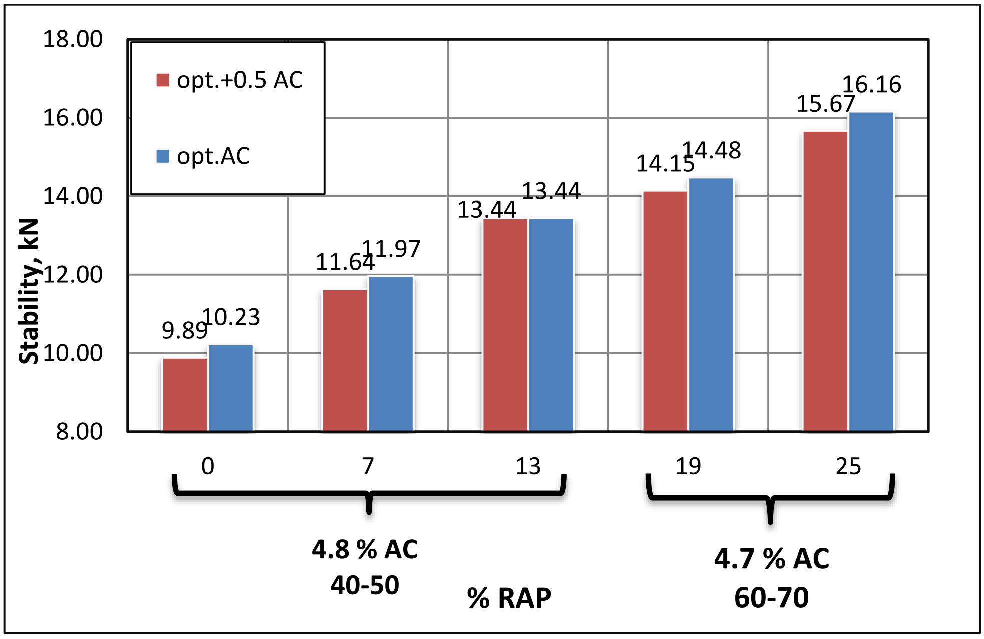

Figure (4-15): Stability with % RAP for 1 day immersion.

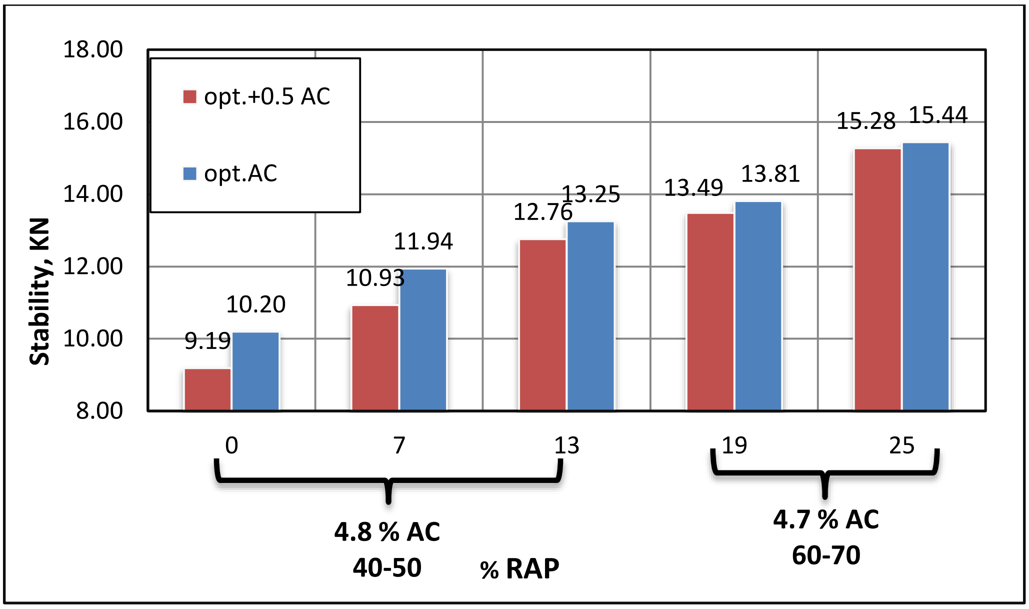

Figure (4-16) Stability with % RAP for 3 day immersion.

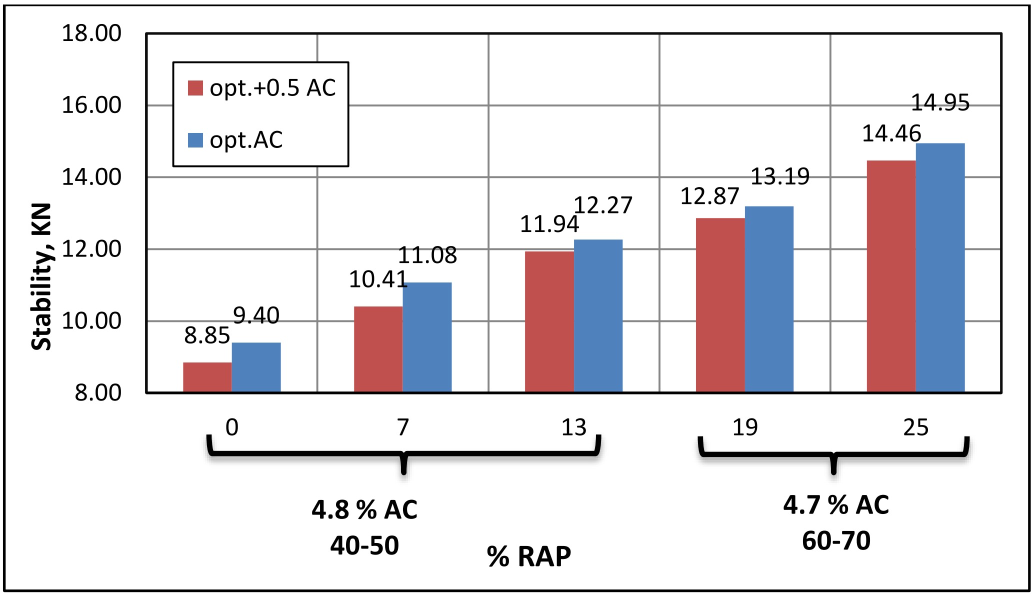

Figure (4-17): Stability with % RAP for 7 day immersion.

4.4.2.1 Durability index (DI)

Table (4-10) shows the results of the durability index with the RAP ratio at optimum and optimum +0.5 AC. Figure (4-18) shows that the values of DI are gradually reduced when the amount of RAP added to the mixture increases. For example, the value of DI is reduced when the RAP quantity increases from 0 to 7% by 14.75% and by 33.18% when the amount of RAP is increased to 13% at the optimum asphalt ratio. While at opt. + 0.5 the value of DI is decreases. It has a 11.17% and 23.71 when increasing the amount of RAP added from 0 to 7% and 13%, respectively. The results also show that the DI values when increasing the optimum asphalt ratio by 0.5% are higher when compared with the values at the optimum asphalt ratio. It increases by 16.71% and 13.55 when adding a RAP rate of 13% and 19% respectively.

Table (4-10): DI with RAP at optimum AC and optimum +0.5 AC

RAP %	DI for Optimum AC	DI for +0.5 Optimum AC
0	11.45	13.07
20	9.76	11.61
30	7.65	9.97
40	7.91	9.15
50	7.28	7.49

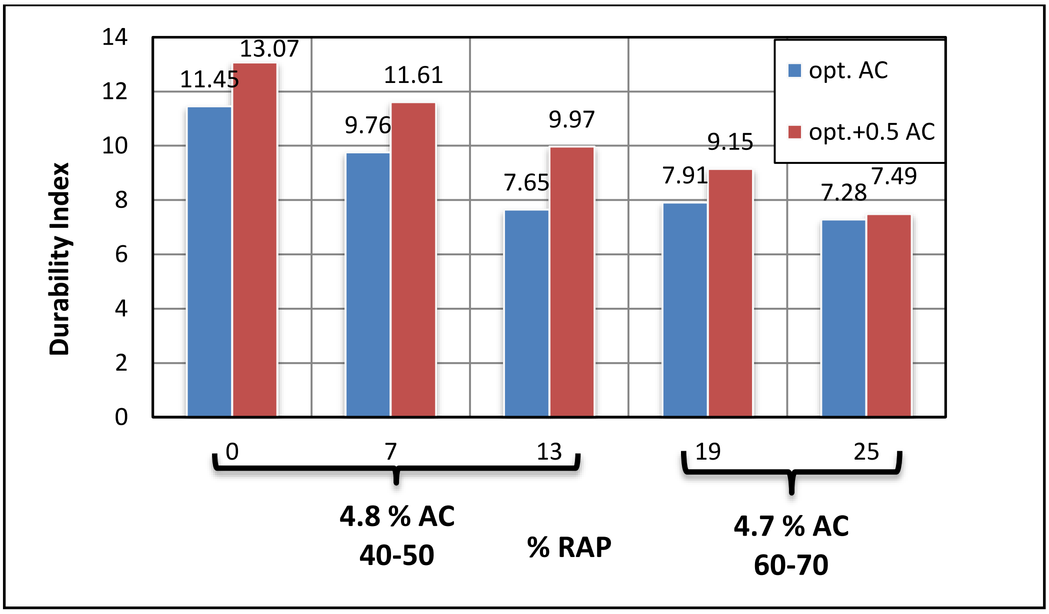

Figure (4-18): Durability index with RAP at optimum AC and optimum +0.5 AC.

4-4-3 Indirect tensile strength test

4.4.3.1 Indirect tensile strength test at optimum asphalt content

Indirect Tensile Test (ITS) is a method of determining the tensile strength of a sample by applying a compressive load on a cylindrical specimen. The load is applied vertically, creating intense stress pressure, and the failure load is measured. Tensile strength can be used to predict the water susceptibility of the sample. In this case, the tensile strength was measured before and after water treatment to determine the retained strength percentage. A high percentage retained predicts a good resistance of the sample to moisture damage. The tensile strain at the failure point is often used to predict the susceptibility of the pavement to cracking. This step is accomplished by performing AASHTO T 283 “Resistance of Compacted Bituminous Mixtures to Moisture Induced Damage” testing on the design aggregate blend at the design asphalt binder content and +0.5 from optimum asphalt content. Specimens are compacted to approximately 7% air voids (height of 67 mm).

Figure (4-19) shows comparison between the results of indirect tensile strength and RAP percentage for unconditioned sample at 25˚C and 60˚C. Indirect tensile strength value for unconditioned sample increases with increasing RAP percentage. For example, it increases from 9.36% to 33.1% when increasing RAP percentage from 7% to 25% at 25˚C. The value for all percentages of RAP decreases with increase of temperature from 25˚C to 60˚C by 36.95 % and 35.71% for 7% and 25% RAP.

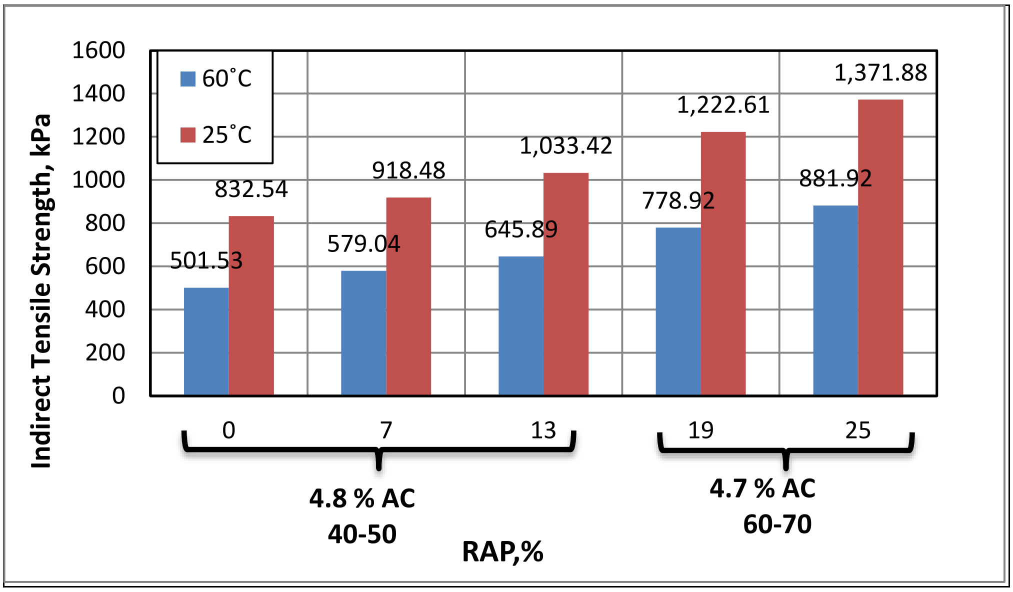

Figure (4-19): Indirect tensile strength for unconditioned sample at 25˚C and 60˚C.

Figure (4-20) illustrates a comparison between the results of indirect tensile strength and RAP percentage for conditioned sample at 25˚C and 60˚C. The results for all percentages of RAP are higher than the control mix. For example the value of 19% RAP percent is rising by 27.67% when increasing RAP from 7% to 19% at 25˚C.

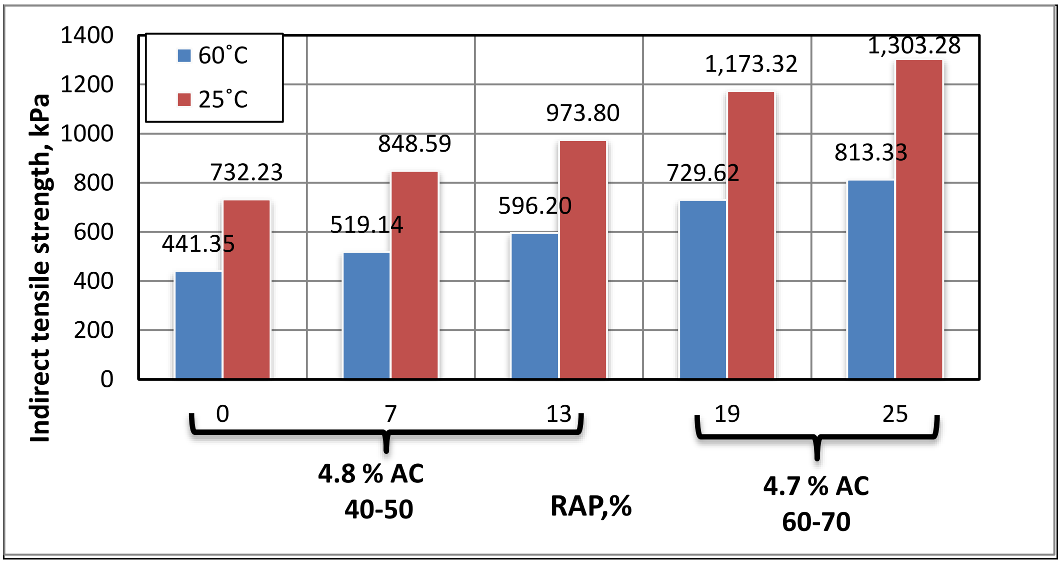

Figure (4-20) Indirect tensile strength for conditioned sample at 25˚C and 60˚C.

Comparison of loss in ITS between "Control Mix" and "RAP Mix" is presented in Figure (4-21). It can be noticed clearly that the loss in ITS for mixtures containing RAP is much lower than mixtures containing no

RAP. The percent excepted of the value of RAP percentage 7% at 60ºC was lower than control mix at 25C. The figure also shows that the result increases with increasing RAP content until 19% and then dropped at 25% RAP content. Result of indirect tensile strength ratio with mixture containing RAP is higher than mixture not containing RAP. These results are above the minimum value of superpave criteria 0.8.

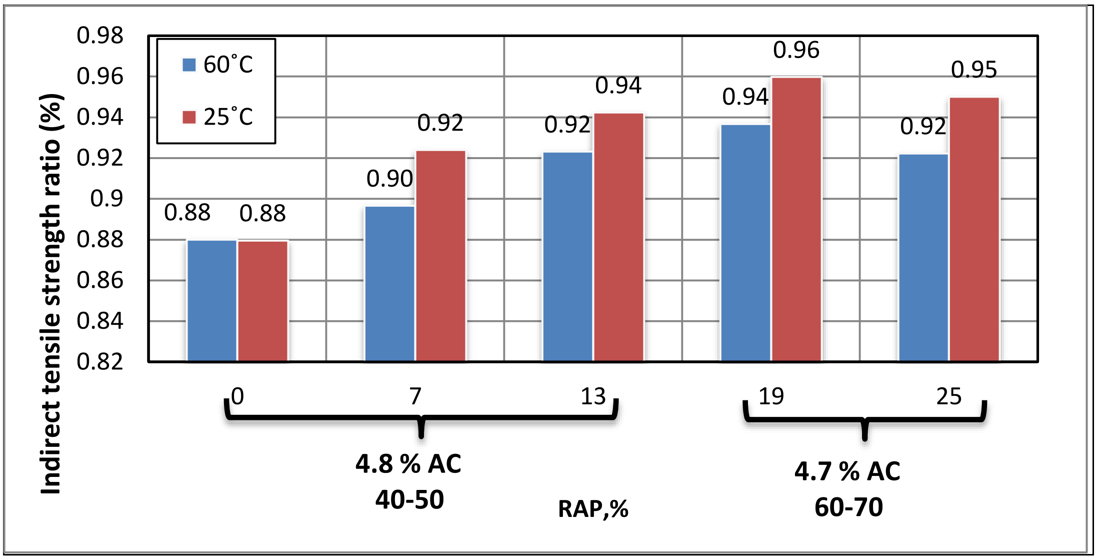

Figure (4-21): Indirect tensile strength ratio (%).

4.4.3.2 Indirect tensile strength test at optimum +0.5 asphalt content

Figures (4-22) and (4-23) show comparison between the results of indirect tensile strength and RAP percentage at optimum +0.5 asphalt content for unconditioned and conditioned samples at 25 ˚C, and 60 ˚C. It can be noticed from figures that the result of indirect tensile strength at optimum +0.5% asphalt content is rising with adding RAP at any percentage. For example, when adding a 13% RAP rate in the mixture, an increase of about 23% is observed from the original mixture value at 25˚C. At 60ºC, it can be noticed that this percentage is about 34% more than the original mixture at the same value as the added RAP for unconditioned sample, while increasing by 29.08% and 36.43% from the control mix for conditioned sample at 25˚C and 60˚C, respectively.

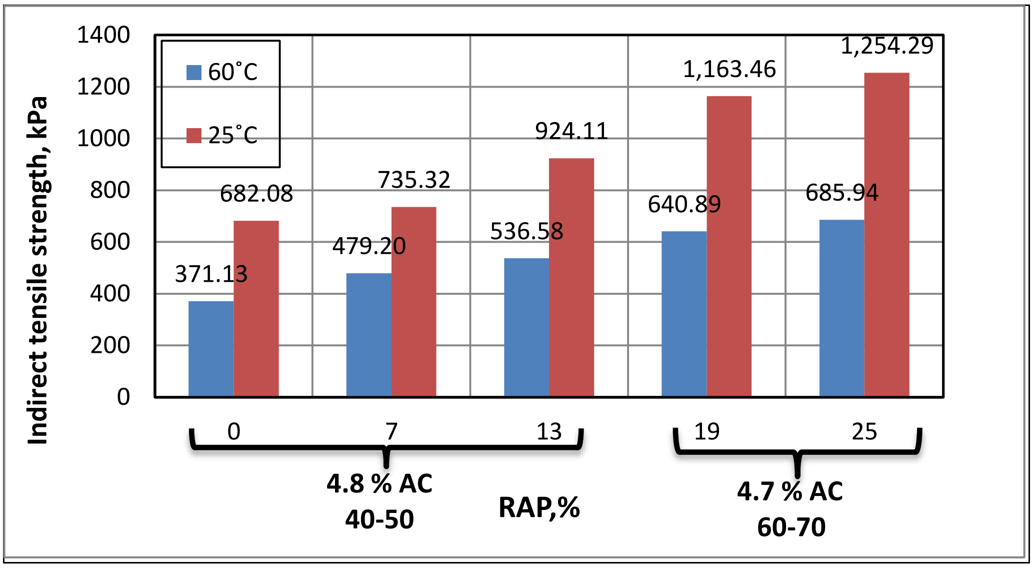

Figure (4-22): Results of indirect tensile strength and RAP percentage at Opt. +0.5 AC content for unconditioned sample.

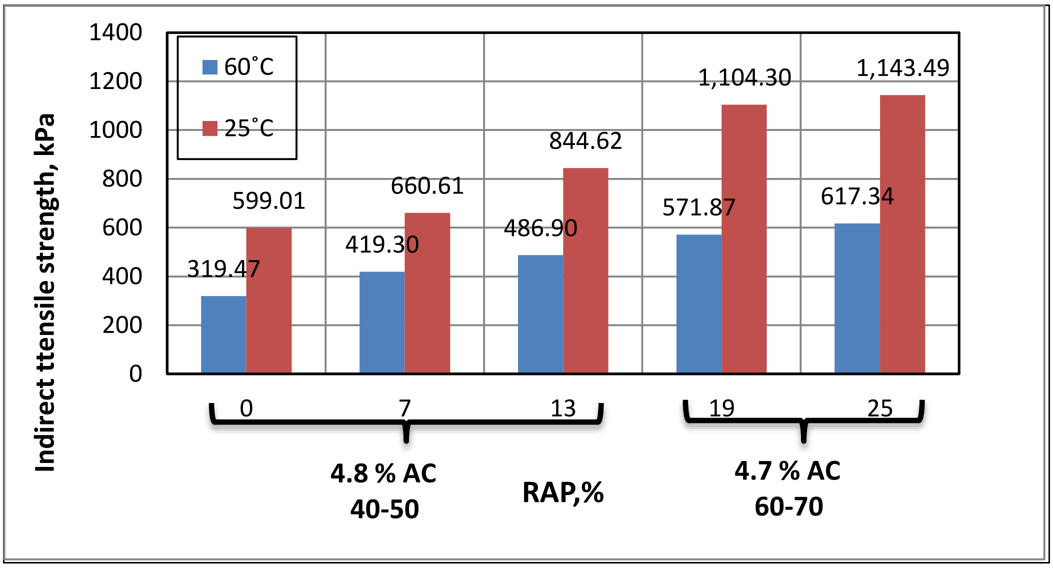

Figure (4-23): Results of indirect tensile strength and RAP percentage at Opt. +0.5 AC content for conditioned sample.

Figure (4-24) shows the results of indirect tensile strength ratio and RAP percentage at +0.5% optimum asphalt content at 25 ºC and 60 ºC. It can be noticed that the value of indirect strength ratio increases with the addition of RAP percent up to 13% when tested at 60 ºC and then begins to decreases at 19% and 25% RAP percent. While the indirect strength ratio increases at 19% and then decreases at 25% when tested at 25 ºC.

The figure shows that the maximum value is 0.949 % at 19% RAP percent when tested at 25 ºC.

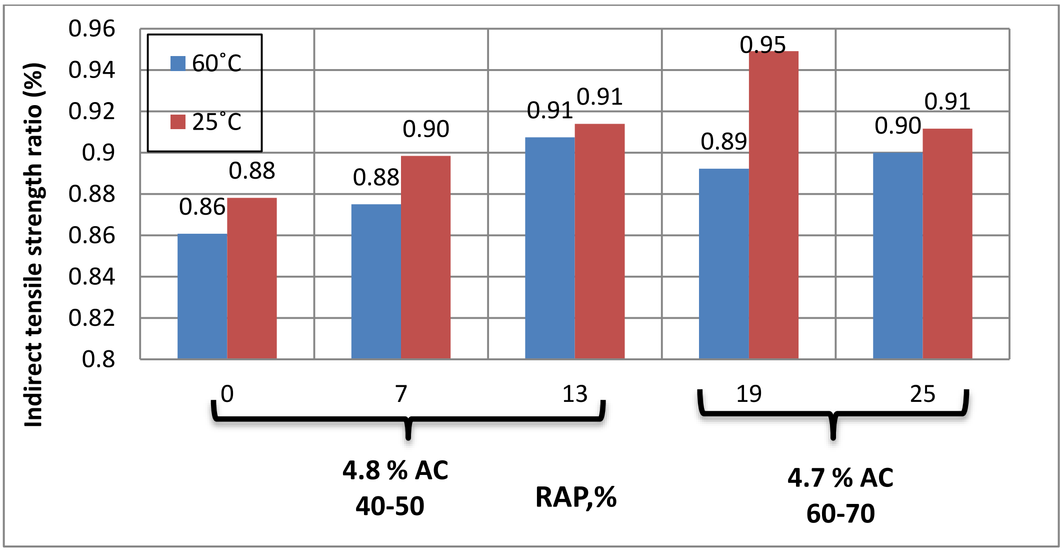

***Figure (4-24) results between indirect tensile strength ratio and RAP percentage at 0ptimum +0.5 asphalt content at* 25 ºC *and* 60 ºC.**

It can be seen clearly that the increase in optimum asphalt ratio by 0.5% in the mixture, leads to a decrease in TSR values. It can be also noticed that the loss of the mixtures containing the RAP is much lower than the mixtures that do not contain RAP. This is due to the fact that the RAP contains hardened asphalt, which that became more viscous over time. The hardened asphalt will reduce the tendency of the mixture to strip when exposed to moisture which enhances the indirect tensile strength ratio. Moreover, mixtures with the most viscous substances will play a better role under tension, resulting in lower reduction in tensile strength when exposed to extreme conditions of high temperature and moisture. This result is supported by the findings of Kiggundu and Newman (1987), who pointed out that recycled mixtures carry better resistance to the operation of water than virgin mixtures.

4.4.4 Immersion-Compression test results

4.4.4.1 Compressive strength of mixture contained optimum asphalt content

Compressive strength test was conducted in accordance with ASTM D1075. The results illustrated in Figure (4-25) shows the values of compressive strength. The results indicate that the compressive strength values increase by increasing the amount of RAP added. For example, the value of the compressive strength increases by 12.18%, 19.9%, 27.99% and 31.7% from the original mixture for dry specimen, respectively, when 7%, 13%, 19% and 25% RAP is added.

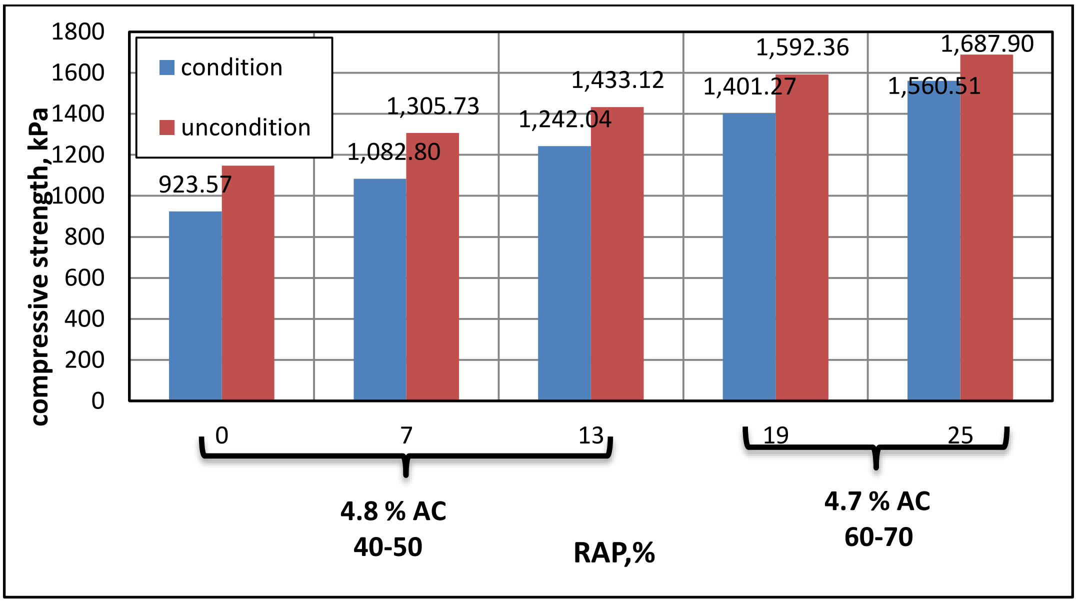

Figure (4-25): Compressive strength and RAP content for both dry and wet sample at optimum asphalt content.

4.4.4.2 Compressive strength of mixture contained optimum +0.5 asphalt content

Figure (4-26) illustrates the results of compressive strength for both dry and wet samples at optimum +0.5 asphalt content. The results show that the compressive strength values increase by increasing the amount of RAP added where it was observed that when adding 7% of the RAP, the value of the compressive strength increases by 13.9% and 15.62% from the original mixture for dry and wet forms, respectively. The results also show that the compressive strength of the conditioned mixtures is less than the unconditioned mixture by 19.64% at a rate 19% of RAP. From these results, it can be noticed that the value of compressive strength increases when the optimum asphalt ratio is increased by 0.5%.

The addition of RAP to the asphalt mixture would increase its resistance and thus increasing the value of the compressive strength. This is due to the fact that the voids of RAP aggregate are filled or almost filled with asphalt which may prevent the excessive absorption of asphalt by the aggregate added to the mixture also the RAP that added to the mixture contains rigid asphalt as a result of atmospheric circumstances as well as broken particles in aggregate due to the loading.

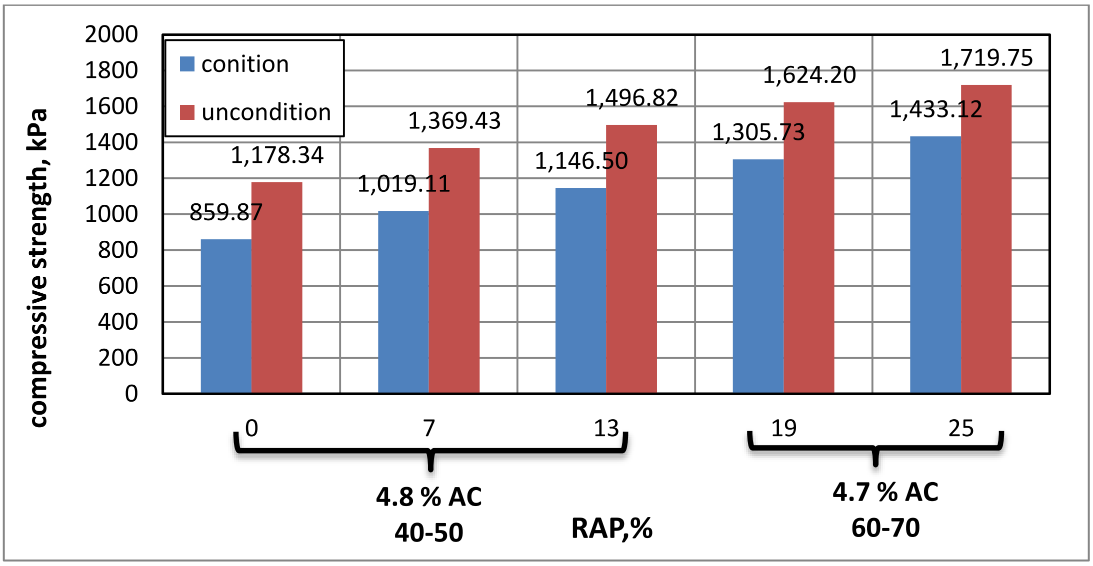

Figure (4-26): Compressive strength and RAP content for both dry and wet sample at optimum +0.5 asphalt content.

4.4.4.3 Index of retained strength results

Index of Retained Strength (I.R.S) is a reference for mixture impedance to water damage. It is obtained as the ratio of average compressive strength of conditioned specimens (wet) to that of unconditioned (dry) specimens in each category. Figure (4-27) shows the results of I.R.S at optimum asphalt content and optimum +0.5 asphalt content. The results show that the IRS values increase as the amount of RAP in the mixture is increase. Which is observed to be 1.18% and 2.77% at a rate of 7% for the original blend of AC and + 0.5 AC. It was also noted that, when the asphalt rate is increased by 0.5%, the IRS rate was 11.64% at 13% RAP.

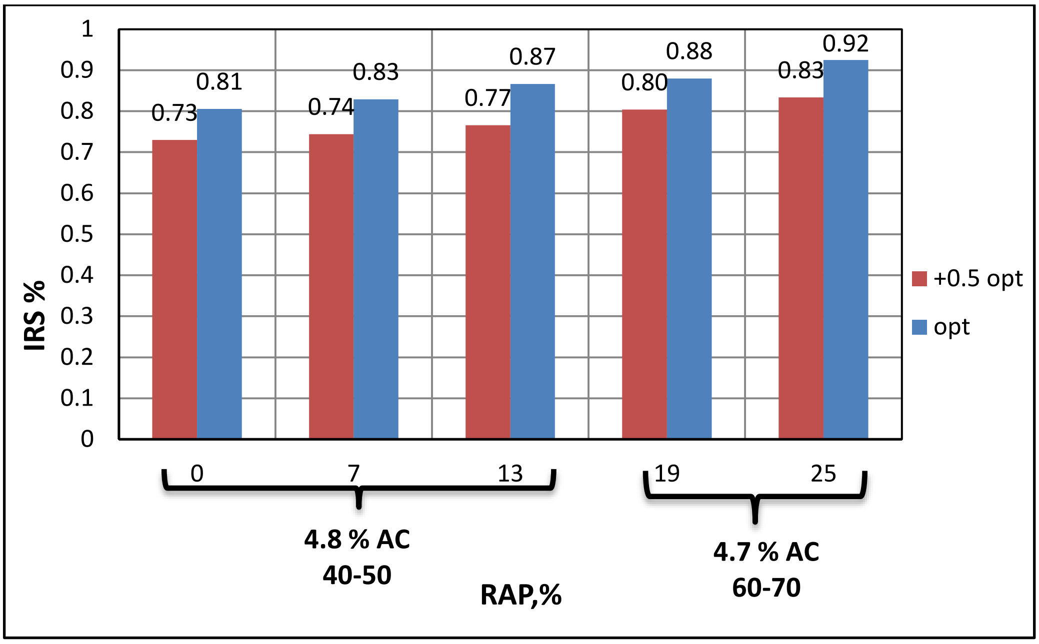

Figure (4-27): Index of retained strength results.

4.4.5 Modulus of elasticity results

The results of modulus of elasticity at 60, 80, 100 and 120 gyrations from ultrasonic test shows that for each percent of RAP content, the modulus of elasticity increase as the RAP content increases.

At 60 gyrations, the results indicate that modulus of elasticity with RAP percent is higher than control mix and increase with increasing RAP percentage to 25%. Figure (4-28) shows that the result of modulus of elasticity at (7%, 13%, 19% and 25%) RAP content is higher than control mix by 5.34%, 6.44%, 7.26% and 9.26%, respectively.

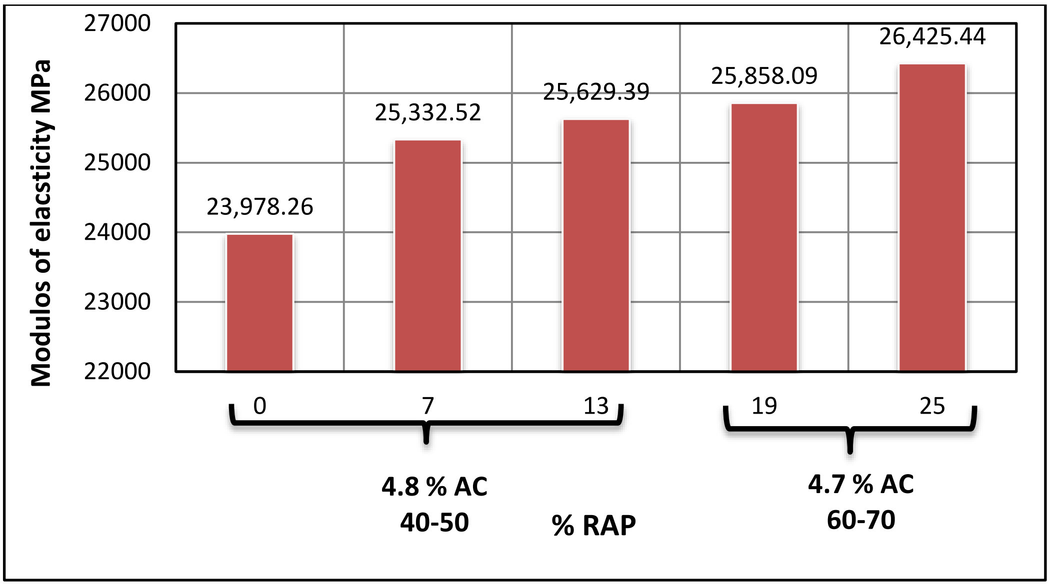

Figure (4-28): Results of modulus of elasticity with RAP content at 60 gyrations.

At 80 gyrations, Figure (4-29) shows that the results of the modulus of elasticity were gradually increases with the increasing of RAP. For example, the modulus of elasticity values increase by 2.42%, 3.6%, 5.32% and 6.67% when 7%, 13%, 19% and 25% RAP, respectively are added.

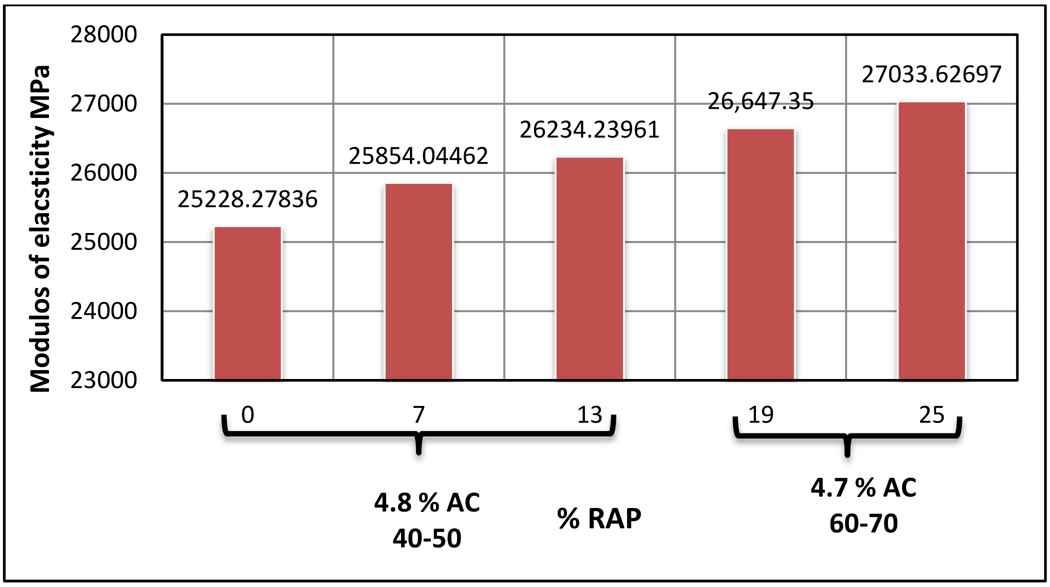

Figure (4-29): Results of modulus of elasticity with RAP content at 80 gyrations.

At 100 gyrations, it can be seen from Figure (4-30) that the results of the modulus of elasticity are increased in the mixture that contains RAP when compare with the mixture that does not contain RAP. It can be noted that an increase by 1.87%, 3.77%, 5.25% and 6.73% from the original mixture when 7%, 13% 19% and 25% RAP, respectively are added.

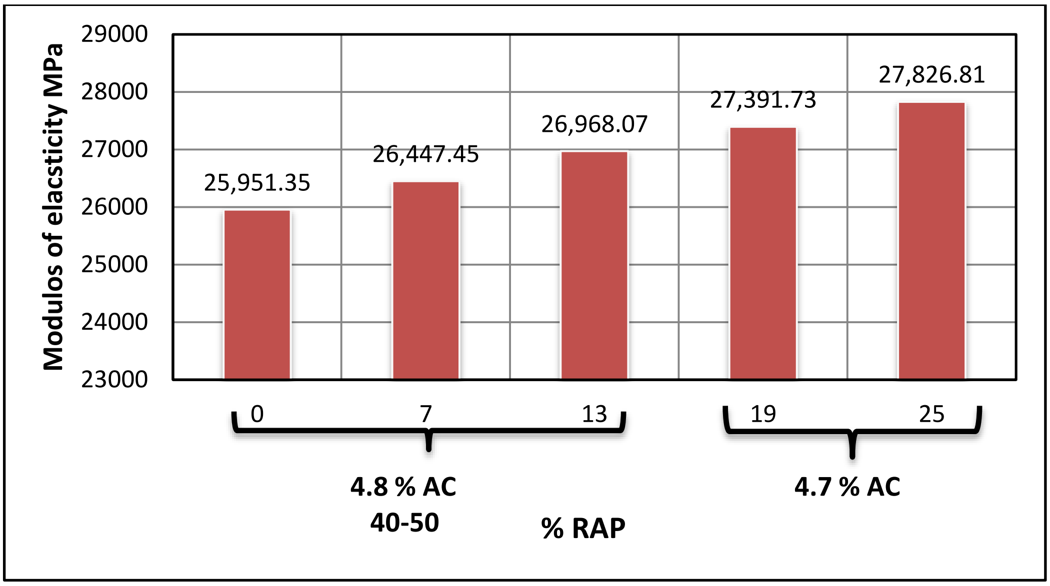

Figure (4-30): Results of modulus of elasticity with RAP content at 100 gyrations.

At 120 gyrations, the results in Figure (4-31) show an increase in the modulus of elasticity values when RAP in the mixture increase as compared to the original mixture. For example, an increase of 1.48%, 4.84%, 6.77% and 9.14% was observed when 7%, 13%, 19% and 25% RAP, respectively are added.

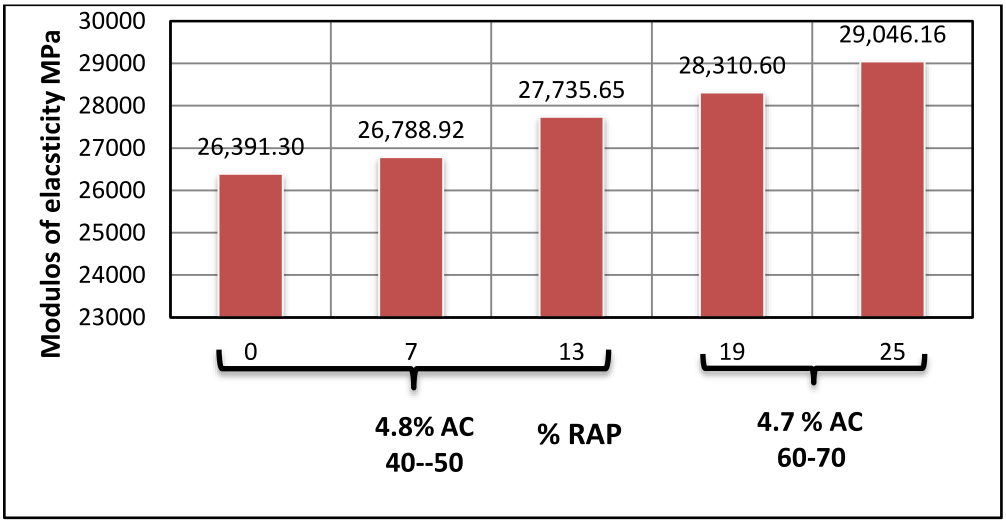

Figure (4-31): Results of modulus of elasticity with RAP content at 120 gyrations.

The addition of the RAP to the original mixture would improve its modulus of elasticity, as the amount of RAP in the mixture increases. It will also increase the mixture's elasticity factor. The reason for this is that the RAP contains rigid asphalt due to the conditions in which it was exposed during service such as traffic and atmospheric conditions. This led to hardened asphalt and increased viscosity and thus increases the elasticity factor of the asphalt mixture.

4.4.5.1 Effect of compaction on modulus of elasticity

In Figure (4-32), the results show that the increase in number of gyrations would increase the intensity of the mixture and thus increasing its resistance, which would mean increasing the hardness of the mixture. It can be noted that the higher the number of gyrations, the greater the hardness (increase modulus of elasticity) and reduction in air voids of the material.

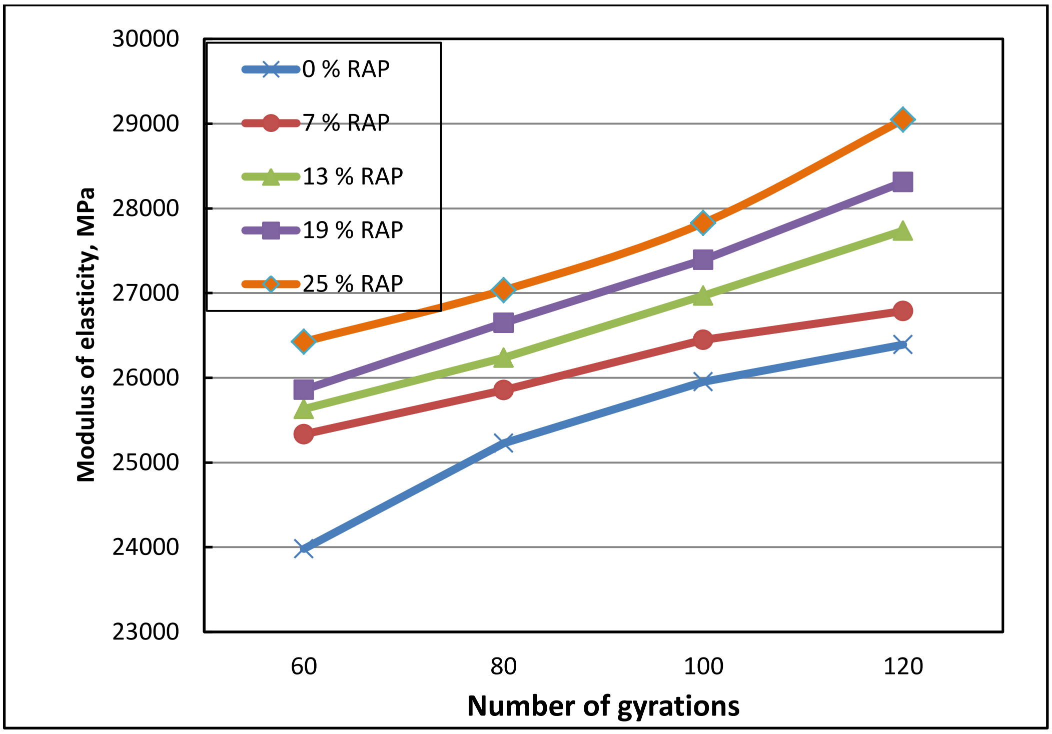

Figure (4-32): Effect of No. of gyration on the modulus of elasticity.

CHAPTER FIVE
BUILDING STATISTICAL MODELS

5.1 General

An asphalt concrete mixture must be designed, produced and placed in order to obtain the following desirable mix properties: Stability, Durability, Impermeability, Workability, Flexibility, Fatigue Resistance, and Skid Resistance (O'Flaherty, 2007). Stability requirements should be high enough to handle traffic adequately. Also, high stability value produces a pavement that is also stiff for that it is less durable than required. The durability of an asphalt pavement is its capability to resist factors such as changes in the binder (polymerization and oxidation), disintegration of the aggregate, and stripping of the binder films from the aggregate. These factors can be the results of weather and traffic or a combination of the two. The mixture should be designed and compacted to give the pavement maximum impermeability that minimizes the intrusion of air and water into the pavement. A lack of sufficient durability in a pavement can have several causes and effects. One of the important inputs for the efficient management of pavement systems are stability and durability models, due to their role in the allocation of cost responsibilities to various vehicle classes for their use of the highway system, and the design of pavement structures (Majeed, 2016).

This chapter presents the statistical approach used to develop regression models to predict the stability, retained stability, and indirect tensile strength produced in the wearing asphalt concrete mixtures at different additional percents of reclaimed asphalt. The overall objective of statistical modeling is to develop predictive equations that correlate these depended variables with the independent variables which include: asphalt viscosity, asphalt content, testing temperature, immersion period, and percent of RAP. There are three parts in process of modeling: the response variable, the mathematical function and the random errors. The mathematical function used to describe the deterministic variation in the response variable is called the regression function or regression equation which will be explained in section 5.2.

5.2 Regression Analysis Approach

Regression analysis is a statistical technique that attempts to explore and model the relationship between two or more variables. Equation (5-1) represents the dependent variable "y_i" as a linear function of one independent variable" x_i" subject to a random ‘disturbance’ or ‘error’, u_i:

$$y_i=\beta_0+\beta_1 x_i+ui \quad \text{………………..} \quad (5\text{-}1)$$

where:

y_i = the dependent variable, β_0 = constant, β_1= the slope, and, x_i= the independent variable.

The error term u_i is assumed to have a mean value of zero, a constant variance, and it will be uncorrelated with itself across observations. Multiple linear regressions make certain assumptions about the data, (as cited by Ahmed, 2002):

1. Linearity. The model assumes that the relationship between the dependent variable and the independent variables can be well-estimated by a straight line.
2. Normality of residuals. Residuals refer to the distances between the line and the points. Multiple linear regressions assume that these distances are normally distributed with a mean of 0.
3. Homogeneity and independence of residuals. The residuals should be normally distributed with equal variance (called homogeneity) and they must not be related to the independent variables.

5.2.1 Identification of the dependent and predictor variables

The following variables represent the dependent and independent variables to simulate environmental and traffic loading subjected to pavement. The first three are the dependent variable, whereas the rest are the independent variables:

- S= Marshall stability for surface asphalt mixture, (kN),
- RS = Retained stability for surface asphalt mixtures, (%),
- ITS= Indirect tensile strength (kN),
- Ac = Asphalt content in mixture, (%),
- V_b = Viscosity of the asphalt binder (Pa.sec),
- P = Condition period (days),
- C = Conditions susceptible to asphaltic mix in indirect tensile test.
- T = Testing temperature (°C),
- R = Reclaimed asphalt pavement as a percentage of an asphalt mixture, (%).

5.2.2 Selection of sample size

The following issues can be considered for the selection of sample size;

- What population parameters need to be estimated,
- Cost of sampling (importance of information),
- How much is already known,
- Spread (variability) of the population,
- Practicality: how hard is it to collect data, and
- How precise of the final estimates will be.

There is a law of diminishing returns of the sample size in statistical investigations. Standard errors and the corresponding lengths of confidence intervals decrease not in proportion to (n), but only in proportion to the square root of (n). For example, doubling the sample size does not reduce the standard error to 50% of its previous value, but only to 71%. Thus, the contribution of each additional observation decreases as the sample size increases, (Al-Moula, 2012). In this framework 75 percent from samples database were used for building the statistical models. In this sense, about 25 percent of the sample data used for validation process. At least 25% of the database should be reserved for this purpose. To determine the required sample size, Kennedy and Neville (1986) present the following equation to calculate percent of error according to sample size, as cited by (Ahmed, 2002):

$$E = CV \times t/n^{0.5} \quad \text{.................................} \quad (5\text{-}2)$$

where: E: error of the mean,

CV: coefficient of variation,

t : t-statistics, and

n : sample size.

Table (5-1) shows the percentage of error according to the sample size that is used in model building process. The margin of error can be accepted if less than 5 percent is achieved. It has been noticed from Table (5-1) that the sample size is accepted with the presented percent of error for the stability, retained stability and indirect tensile strength models.

Table (5-1): Percentage of error according to the sample size.

Model	Sample Size	CV	T-Statistical	D.F n-1	Error
S	30	$=\sigma/\mu = 0.17$	2.042	29	0.049
RS	30	0.0654	2.042	29	0.0243
ITS	30	0.371	2.042	29	0.064

5.2.3 Scatter plots

For the requirements of the modeling process, scatter plot is carried out between the dependent and independent variables. From the plots, the nature of relation between dependent and independent variables can be expected and the best relations are selected.

5.2.4 Checking for outliers

"Outlier" is the name given to one or more of the observations which are different significantly from all others, The cause of a faulty observation can be a mistake. The outliers and influential observations are checked by using Chauvenet's criterion (Kennedy and Neville, 1986) to examine outliers of all data used to ensure accuracy. Table (5-2) shows the results of these tests. It can be seen that all results are less than the tabulated values. Therefore, there is no outlier. Furthermore, dependent variables for all models were tested for outliers and again no outlier can be noticed.

Table (5-2): Results of chauvenet's test for outliers.

Variable	Minimum	Maximum	Mean	Standard Deviation	(Xmin-x')/std	(Xmax-x')/std	(Xm-x')/std-tab.
S	8.85	16.94	12.92	1.72	1.80	1.77	5.57
RS	82	100	92.7	6.07	1.76	1.20	2.82
ITS	319.47	1371.87	845	297.5	1.61	1.92	5.94

5.2.5 Testing of normality

Kolmogorov-Smirnov (or K-S test) is used to check if the variables are normally distributed. Scheaffer and McClave (1990) stated that the K-S statistics D is based upon the maximum distance between F(y) and F_n (y), that is:

$$D = Max\ |F(y) - F_n(y)| \text{ .... } (5\text{-}3)$$

where: F(y): Normal cumulative probabilities (From normal distribution table), and

$F_n(y)$: Sample cumulative distribution function.

$$D_{+} = Max[\frac{i}{n} - F(yi)] \ldots\ldots\ldots\ldots\ldots (5\text{-}4)$$

$$D^{-} = Max\ \{f(yi) - \frac{i-1}{n}\} \ldots\ldots\ldots\ldots\ldots (5\text{-}5)$$

Since:

$$D = Max\ (D_{+}, D_{-}) \ldots\ldots\ldots\ldots\ldots (5\text{-}6)$$

Kolmogorov-Smirnov test results for the dependent predicted models are tabulated in Table (5-3). The results indicate that K-S calculated values are less than the critical values presented by Scheaffer and McClave (1990). Accordingly the distributions of models are normal.

Table (5-3): D-value and K-S test results.

Residual value	D+	D-	Absolute D	K-S (Dn, 0.05)
S	0.101	0.103	0.103	0.24
RS	0.116	0.184	0.184	0.24
ITS	0.121	0.088	0.121	0.24

5.2.6 Multicollinearity

Multicollinearity (collinearity and intercorrelation) is a statistical procedure to find the correlation between independent variables with one another. The adverse effect of multicollinearity is that the estimated regression coefficients tend to have large sampling variability, SPSS software version (23) is employed for the development of the models. A confidence level of 95 percent, (a significant level of 0.05) is employed. Based on the intercorrelation analysis, the independent variables are eliminated one-by-one depending on significance. The process is repeated until significant predictor variable remained at that point interactions among the variables are considered. A correlation matrix is produced to determine the correlation coefficients for the variables. The decision to add or delete a variable is made on the basis of that variable improves the model or not. By using SPSS software, the correlation coefficients between all of the variables are calculated and the correlation matrix is set up. Tables (5-4) to (5-6) show the bivariate correlation coefficients, which are determined to identify the underlying form of the relationship between the dependent variable and each of the predictor variables for stability, retained stability, and indirect tensile strength values.

Table (5-4): Correlation coefficient matrix (R) for the stability (S).

Variable	Ac	Vb	P	RAP	S
Ac	1	.192	.117	-.139	-.245
Vb	.192	1	-.058	-.855**	-.732**
P	.117	-.058	1	-.008	-.351
RAP	-.139	-.855**	-.008	1	.925**
S	-.245	-.732**	-.351	.925**	1

**. Correlation is significant at the 0.01 level (2-tailed).
*. Correlation is significant at the 0.05 level (2-tailed).

Table (5-5): Correlation coefficient matrix (R) for retained stability (RS).

Variable	Ac	Vb	P	RAP	S	RS
Ac	1	.192	.117	-.139	-.245	-.217
Vb	.192	1	-.058	-.855**	-.732**	-.162
P	.117	-.058	1	-.008	-.351	-.894**
RAP	-.139	-.855**	-.008	1	.925**	.283
S	-.245	-.732**	-.351	.925**	1	.607**
RS	-.217	-.162	-.894**	.283	.607**	1

**. Correlation is significant at the 0.01 level (2-tailed).
*. Correlation is significant at the 0.05 level (2-tailed).

Table (5-6): Correlation coefficient matrix (R) for indirect tensile strength (ITS).

Variable	Ac	Vb	C	RAP	T	ITS
Ac	1	.192	.115	-.139	.071	-.389*
Vb	.192	1	-.082	-.855**	.027	-.558**
P	.115	-.082	1	.022	.144	-.234
RAP	-.139	-.855**	.022	1	-.077	.626**
T	.071	.027	.144	-.077	1	-.743**
ITS	-.558**	-.234	.626**	-.743**	1	-.558**

**. Correlation is significant at the 0.01 level (2-tailed).

5.2.7 Regression modeling

The regression modeling is the statistical process being used to determine the relationships between two or more quantitiy variables to generate a model that predicts one variable from the other (s) in order to present the data in the best fit. The term multiple linear regression is employed when a model is a function of more than one predictor variable. The goal of multiple linear regressions is to develop best model at selected confidence level and satisfying the basic assumptions of regression analysis.

- High intercorrelation does not exist among predictor variables,
- Influential observation or outliers do not exist in the data,
- The distribution of error is normal, and
- The mean of error distribution is zero.

The objective is accomplished by selecting the model, which provides the highest adjusted coefficient of determination (R^2) and the lowest mean square error (MSE), for given data (Montgomery and Peck, 1992). The best and communally method usually used to determine parameter of prediction model is stepwise procedure. The procedure begins by entering variable that has the smallest significant value, largest F-value SPSS software at significant level 5%. The software, then, will examine if the variable verifies the condition of remaining in the model or not. after the selection of all variables. To improve the model, the second variable (which has F-statistic greater than F to enter) will be examined by comparing with first model (which have the first variable). If it exceeds the standard value, the procedure is repeated to the end. If two independent variables are highly correlated, only one of them will enter the equation. Once the first variable is included, the added explanatory power of the second variable will be minimal and its F-statistic will not be large enough to enter the model. In this way, multicollinearity is reduced. The procedures continue by adding another independent variable at each step. The significant value of all variables is computed (at each step) and compared with F to remove. If F-statistics of variable falls below this standard, it should be removed from the equation. These steps are repeated until no variables are added or removed.

5.2.8 Model limitations

The model of stability is developed for a single uniform structure section. The limitations of the data used to establish models for stability and retained stability and durability are presented in Tables (5-7) to (5-9). The intention of the limitation is not to suggest the modeling effort has not been successful. It merely serves to alert of the limitations of the data.

Table (5-7): Summary of the limitation of data used for S model.

	Ac %	Vb, (Pa.sec)	P, (days)	RAP, (%)	S, (kN)
Max.	5.30	0.475	7	25	16.94
Min.	4.70	0.537	0	0	8.85
Mean	5.0100	0.51220	2.63	12.57	12.9281

Table (5-8): Summary of the limitation of data used for RS model.

	Ac %	Vb, (Pa.sec)	P, (days)	RAP, (%)	S, (kN)	RS, (%)
Max.	5.30	0.475	7	25	16.94	100.00
Min.	4.70	0.537	0	0	8.85	82.00
Mean	5.01	0.51220	2.63	12.57	12.9281	92.7333

Table (5-9): Summary of the limitation of data used for ITS model.

	C, (days)	RAP, (%)	T, (°C)	ITS, (kN)
Max.	1	25	60	1371.88
Min.	0	0	25	319.47
Mean	0.53	12.57	40.17	845

5.2.9 Goodness of fit

The measures of goodness of fit are aimed to quantify how well the proposed regression model obtained fit the data. The two measures that are usually presented are coefficient of determination (R^2) and standard error of regression (SER) (Devore, 2000). For more accuracy, several statisticians use the adjusted coefficient of multiple determinations, adjusted R^2 which refer to magnitude increasing of R^2 when new parameter inters the model. The second parameter SER is estimated by the following equation (Devore, 2000):

$$SER = \left[\frac{SSE}{n-(k+1)}\right]^{0.5} \quad \text{.....................} \ (5\text{-}7)$$

where: SER: Standard error of regression, SSE: (Sum Squares of Error) = Σ (yi-yi'), y_i : actual value of response variable for the i_{th} case, y_i': value of the regression prediction for the i_{th} case,

n-(k+1): degree of freedom (Df),

n :number of sample, and k : number of independent variables.

The results of ANOVA and summary of stepwise regression, for several possible models can be seen in Tables (5-10) to (5-18) for stability, retained stability and durability model, respectively.

Table (5-10): Results of ANOVA for stability model.

ANOVA[a]						
Model		Sum of Squares	df	Mean Square	F	Sig.
1	Regression	126.847	1	126.847	164.875	.000[b]
	Residual	21.542	28	.769		
	Total	148.389	29			
2	Regression	144.402	2	72.201	488.956	.000c
	Residual	3.987	27	.148		
	Total	148.389	29			
3	Regression	145.294	3	48.431	406.847	.000d
	Residual	3.095	26	.119		
	Total	148.389	29			
4	Regression	146.339	4	36.585	446.172	.000e
	Residual	2.050	25	.082		
	Total	148.389	29			

a. Dependent Variable: Marshall stability
b. Predictors: (Constant), % RAP
c. Predictors: (Constant), % RAP , Condition period
d. Predictors: (Constant), % RAP , Condition period, Asphalt content
e. Predictors: (Constant), % RAP , Condition period, Asphalt content, Asphalt viscosity

Table (5-11): Stepwise regression summary for stability model.

Coefficients[a]						
Model		Unstandardized Coefficients		Standardized Coefficients	t	Sig.
		B	Std. Error	Beta		
1	(Constant)	10.061	.275		36.618	.000
	% RAP	.228	.018	.925	12.840	.000
2	(Constant)	10.786	.138		78.439	.000
	% RAP	.227	.008	.922	29.226	.000
	Condition period	-.272	.025	-.344	-10.903	.000
3	(Constant)	14.248	1.271		11.211	.000
	% RAP	.225	.007	.911	31.854	.000
	Condition period	-.265	.023	-.335	-11.740	.000
	Asphalt content	-.688	.251	-.079	-2.737	.011
4	(Constant)	8.210	1.993		4.118	.000
	% RAP	.259	.011	1.050	23.015	.000
	Condition period	-.255	.019	-.323	-13.487	.000
	Asphalt content	-.809	.211	-.093	-3.827	.001
	Asphalt viscosity	12.079	3.383	.165	3.570	.001

a. Dependent Variable: Marshall stability

Table (5-12): Model summary of stability.

Model Summary				
Model	R	R Square	Adjusted R Square	Std. Error of the Estimate
1	.925a	.855	.850	.87713
2	.986b	.973	.971	.38427
3	.990c	.979	.977	.34502
4	.993d	.986	.984	.28635

a. Predictors: (Constant), % RAP
b. Predictors: (Constant), % RAP , Condition period
c. Predictors: (Constant), % RAP , Condition period, Asphalt content
d. Predictors: (Constant), % RAP , Condition period, Asphalt content, Asphalt viscosity

Table (5-13): Results of ANOVA of retained stability model.

ANOVA[a]						
Model		Sum of Squares	Df	Mean Square	F	Sig.
1	Regression	854.297	1	854.297	110.964	.000b
	Residual	215.569	28	7.699		
	Total	1069.867	29			
2	Regression	959.562	2	479.781	117.439	.000c
	Residual	110.305	27	4.085		
	Total	1069.867	29			
3	Regression	997.326	3	332.442	119.154	.000d
	Residual	72.540	26	2.790		
	Total	1069.867	29			

a. Dependent Variable: Retained Stability
b. Predictors: (Constant), Condition period
c. Predictors: (Constant), Condition period, Marshall stability
d. Predictors: (Constant), Condition period, Marshall stability, % RAP

Table (5-14): Stepwise regression summary of retained stability model.

Coefficients[a]						
Model		Unstandardized Coefficients		Standardized Coefficients	t	Sig.
		B	Std. Error	Beta		
1	(Constant)	97.733	.694		140.784	.000
	Condition period	-1.899	.180	-.894	-10.534	.000
2	(Constant)	85.447	2.473		34.557	.000
	Condition period	-1.649	.140	-.776	-11.759	.000
	Marshall stability	.899	.177	.335	5.076	.000
3	(Constant)	53.039	9.043		5.865	.000
	Condition period	-.825	.252	-.388	-3.270	.003
	Marshall stability	3.930	.837	1.463	4.697	.000
	% RAP	-.711	.193	-1.073	-3.679	.001

a. Dependent Variable: Retained Stability

Table (5-15): Model summary of retained stability.

Model Summary				
Model	R	R Square	Adjusted R Square	Std. Error of the Estimate
1	.894a	.799	.791	2.77469
2	.947b	.897	.889	2.02123
3	.966c	.932	.924	1.67033

a. Predictors: (Constant), Condition period
b. Predictors: (Constant), Condition period, Marshall stability
c. Predictors: (Constant), Condition period, Marshall stability, % RAP

Table (5-16): Results of ANOVA for indirect tensile strength model.

ANOVAa						
Model		Sum of Squares	Df	Mean Square	F	Sig.
1	Regression	1416472.246	1	1416472.246	34.479	.000b
	Residual	1150298.839	28	41082.101		
	Total	2566771.085	29			
2	Regression	2251181.679	2	1125590.840	96.299	.000c
	Residual	315589.406	27	11688.497		
	Total	2566771.085	29			
3	Regression	2427788.729	3	809262.910	151.392	.000d
	Residual	138982.356	26	5345.475		
	Total	2566771.085	29			
4	Regression	2464267.977	4	616066.994	150.256	.000e
	Residual	102503.108	25	4100.124		
	Total	2566771.085	29			

a. Dependent Variable: ITS
b. Predictors: (Constant), Temprature
c. Predictors: (Constant), Temprature, % RAP
d. Predictors: (Constant), Temprature, % RAP , Asphalt content
e. Predictors: (Constant), Temprature, % RAP , Asphalt content, Conditions

Table (5-17): Model summary of indirect tensile test.

Model Summary				
Model	R	R Square	Adjusted R Square	Std. Error of the Estimate
1	.743a	.552	.536	202.68720
2	.937b	.877	.868	108.11335
3	.973c	.946	.940	73.11276
4	.980d	.960	.954	64.03221

a. Predictors: (Constant), Temprature
b. Predictors: (Constant), Temprature, % RAP
c. Predictors: (Constant), Temprature, % RAP , Asphalt content
d. Predictors: (Constant), Temprature, % RAP , Asphalt content, Conditions

Table (5-18): Stepwise regression summary of indirect tensile strength model.

Coefficients[a]						
Model		Unstandardized Coefficients		Standardized Coefficients	t	Sig.
		B	Std. Error	Beta		
1	(Constant)	1304.228	93.350		13.971	.000
	Temperature	-12.529	2.134	-.743	-5.872	.000
2	(Constant)	1040.997	58.733		17.724	.000
	Temperature	-11.782	1.142	-.699	-10.322	.000
	% RAP	18.561	2.196	.572	8.451	.000
3	(Constant)	2571.113	269.150		9.553	.000
	Temperature	-11.510	.773	-.682	-14.882	.000
	% RAP	17.406	1.499	.536	11.612	.000
	Asphalt content	-304.699	53.010	-.265	-5.748	.000
4	(Constant)	2516.953	236.420		10.646	.000
	Temperature	-11.224	.684	-.666	-16.408	.000
	% RAP	17.595	1.314	.542	13.387	.000
	Asphalt content	-289.081	46.721	-.252	-6.187	.000
	Conditions	-71.119	23.843	-.121	-2.983	.006

a. Dependent Variable: ITS

5.2.10 Validation of the developed model

The objective of validation is to assess the accuracy of the proposed prediction model, and to measure the error or accuracy of the prediction for the validation period.

5.2.10.1 Selection of validation methods

Neter et al., (1990) suggested the following methods for validation of a regression model:

1. Checking on models predictions and coefficients attempts to make sure that the selected model agrees with the physical theory. This essentially has been already checked during the development process.
2. Collection of the new data suggests that new data set should be collected.
3. Comparison with previously developed models. The results of a newly developed model are compared with the previously developed model or with a theoretical model.
4. Data splitting recommends that one should not consider data splitting unless $N>2P+25$, where N is a sample size and P is number of estimated parameters.
5. Prediction Sum of Squares is a form of data splitting and it is not feasible because of the available large sample size.

Following the previous discussion and due to the nature of the available data, data splitting procedure is selected to assess the predictive ability of the stability, retained stability, and indirect tensile strength models. The existing data were splitted in two sets; the first set named the model building set, while the second named the validation or prediction set, which is used to evaluate the reasonableness and predictive ability of the selected model.

5.2.10.2 Diagnostic (Q-Q) plots

To develop models in this study, 25 percent of the data have been split and used of validation for stability, retained stability, and indirect tensile strength models. The observed values are plotted against those estimated by using developed models as shown in Figures (5-1) to (5-3).

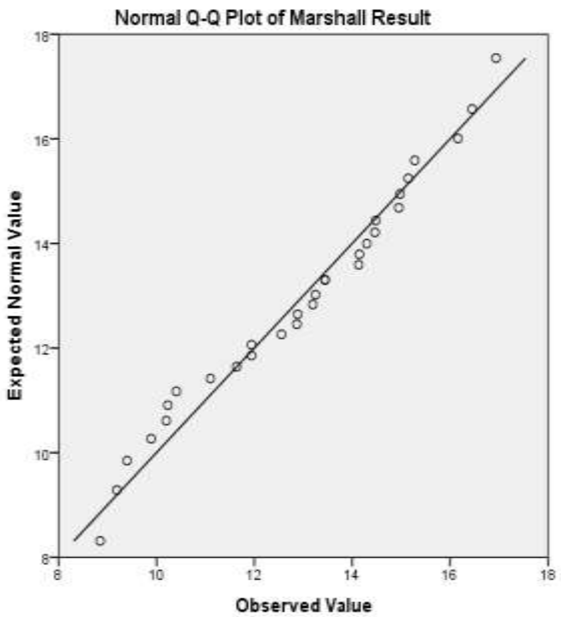

Figure (5-1): Developed stability S model versus observed S.

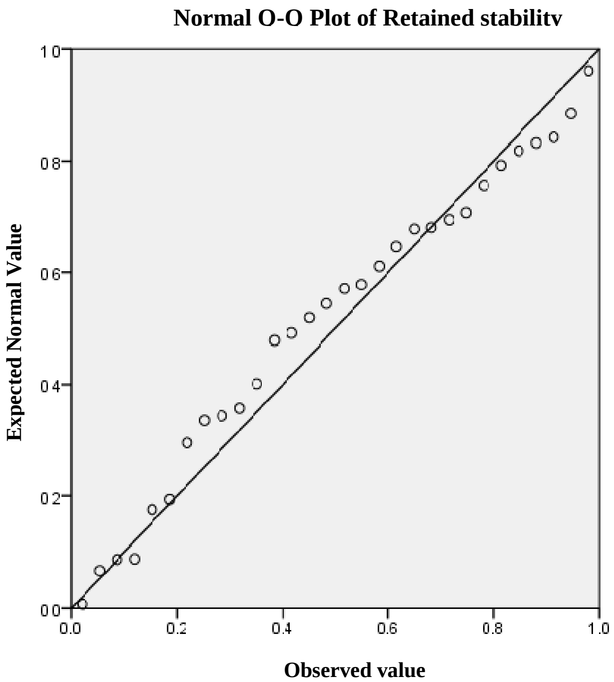

Figure (5-2): Developed retained stability RS Model versus observed RS.

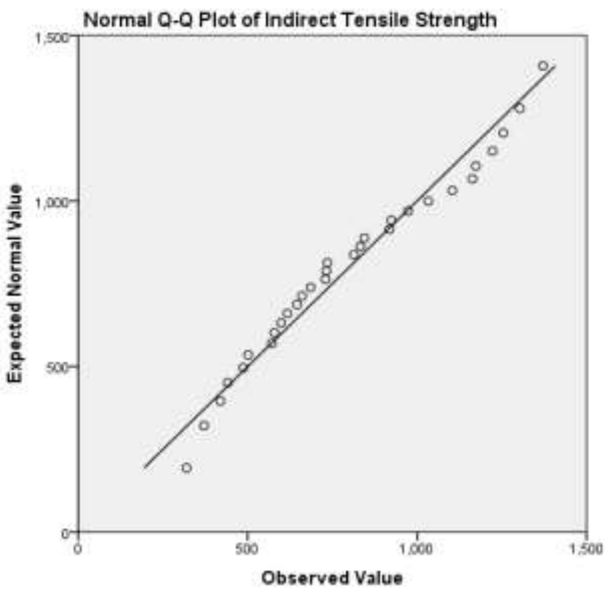

Figure (5-3): Developed indirect strength ITS model versus observed ITS.

5.2.11 Checking of R-Critical

A high correlation coefficient R value does not guarantee that the model fits the data well. The correlation between x and y is considered significant at the given probability level when the calculated R exceeds the tabulated R value. The correlation coefficient of the final form of the S model=0.99> $R_{tabulated}$=0.361 and the correlation coefficient of the final form of RS model = 0.96> $R_{tabulated}$ =0.361 and the correlation coefficient of the final form of ITS model = 0.98 > $R_{tabulated}$ = 0.361. Therefore, there is a real correlation between dependent and independent variables in the stability, retained stability and indirect tensile strength models and this correlation is not due to chance.

5.3 Regression Results Analysis

Multiple regression analysis was used to know the effect of the independent variables (asphalt content, asphalt viscosity, RAP%, period, temperature) on the dependent variable (S, RS, ITS). SPSS program was used to predict the model of stability, retained stability, indirect tensile strength.

It is obvious from Table (5-19) that the values of standardized coefficient (Beta) for the stability model; the first independent variable asphalt content has lowest effect in the prediction of the dependent variable (S) because the Beta value is (-0.809). Asphalt viscosity is the second independent variable and the value of Beta is (12.079). The value of Beta for the third independent variable condition period is (-0.255). The last independent variable is RAP%. Its beta value is (0.259) and has slightly effect on the prediction variable (S). All of the independent variables in Table (5-19) have a significant level less than 5%.

Table (5-19) Coefficients of stability model.

Model		Unstandardized Coefficients		Standardized Coefficients	t	Sig.
		B	Std. Error	Beta		
1	(Constant)	8.210	1.993		4.118	.000
	Asphalt content	-.809	.211	-.093	-3.827	.001
	Asphalt viscosity	12.079	3.383	.165	3.570	.001
	Condition period	-.255	.019	-.323	-13.487	.000
	% RAP	.259	.011	1.050	23.015	.000

a. Dependent Variable: Marshall stability

Table (5-20) presents the values of standardized coefficient (Beta) for the retained stability model. The values of Beta for independent variable (asphalt content, RAP % and condition period) are (2.486, 1.209 and -0.433), respectively. These values have slightly affected in the prediction of the (RS), while for (asphalt viscosity and Marshall stability) the values of Beta are (-45.795 and 5.583), which has affected in the prediction of the (RS) model.

Table (5-20) Coefficients of retained stability model.

Coefficients[a]

Model		Unstandardized Coefficients		Standardized Coefficients	t	Sig.
		B	Std. Error	Beta		
1	(Constant)	47.899	14.339		3.340	.003
	Asphalt content	2.486	1.478	.106	1.682	.106
	Asphalt viscosity	-45.795	23.081	-.233	-1.984	.059
	% RAP	+1.209	.294	-1.825	-4.107	.000
	Condition period	-.433	.302	-.204	-1.432	.165
	Marshall stability	5.583	1.110	2.079	5.028	.000

a. Dependent Variable: Retained Stability

Table (5-21) also presents the values of standardized coefficient (Beta) for indirect tensile strength model. The values of Beta for independent variable (asphalt content, asphalt viscosity, RAP %, conditions and temperature) are (-274.62, -1419.9, 13.558, -77.035 and -11.309), respectively. This refers to that Beta has significant effect on the predication of the (ITS). Also all of the independent variables have a significant level less than 5%.

Table (5-21): Coefficients of indirect tensile strength model.

Model		Unstandardized Coefficients		Standardized Coefficients	t	Sig.
		B	Std. Error	Beta		
1	(Constant)	3229.091	423.607		7.623	.000
	Asphalt content	-274.623	44.810	-.239	-6.129	.000
	Asphalt viscosity	-1419.920	717.223	-.147	-1.980	.059
	% RAP	13.558	2.389	.418	5.676	.000
	Conditions	-77.035	22.759	-.131	-3.385	.002
	Temprature	-11.309	.649	-.671	-17.432	.000

a. Dependent Variable: ITS

5.3.1 Predictive models

Analysis of results, calculation of standard error, coefficient of variation for the stability (S), retained stability (RS) and indirect tensile strength (ITS) in term of models are presented in Table (5-22).

Table (5-22): Summary results of final model.

Model	R^2 (%)	SER
$S = 8.21 - 0.809Ac + 12.079\ V_b - 0.255\ P + 0.259\ R$	0.986	0.286
$RS = 47.899 + 2.486\ Ac - 45.795\ V_b + 1.209\ R - 0.433\ P + 5.583\ S$	0.943	1.589
$ITS = 3229.09 - 274.623\ Ac - 1419.92\ V_b + 13.558\ R - 77.035\ C - 11.309\ T$	0.966	60.59

5.3.2 Analysis of results

Analysis of results, calculation of standard error, coefficient of variation for the stability, retained stability and indirect tensile models are presented previously in Table (5-22).

The coefficient of determination values are found to be (0.986), (0.943) and (0.966) for the stability (S), retained stability (RS) and indirect tensile strength (ITS) model, respectively. The values seem to give good correlation between the observed and estimated results, with the standard error of regression (0.286), (1.589) and (60.59). Furthermore, it can be seen that the dependent variable (S) increases as the asphalt viscosity and reclaimed asphalt pavement increase and decreases with the increase of asphalt content and condition periods (days). For retained stability variable (RS) it can be seen that it increases with the increase of asphalt content, RAP and stability and decreases with increasing of asphalt viscosity and condition periods (days). For last independent variable indirect tensile strength (ITS), it increases as the reclaimed asphalt pavement increases and decreased with increasing the asphalt content, asphalt viscosity, conditions and temperature. The coefficient of determination (R^2) is 0.986, 0.943 and 0.966 for the three models, respectively, which means that there is a good correlation between the actual and estimated values of stability, retained stability and indirect tensile strength, and only 1.4, 5.7 and 3.4 percent of observed variation is unexplained by the developed model. This indicates that these three models can be explained with high degree of accuracy in terms of test conditions and mix parameters.

The plots of the observed values versus the estimated values obtained from three models as shown in Figures (5-1) , (5-2) and (5-3) illustrate a good correlation between the predictors and measured of the responses variables.

CHAPTER SIX

APPLICATION OF MECHANISTIC EMPIRICAL APPROACH USING MnPAVE SOFTWARE

6.1 General

This chapter presents the application of Mechanistic-Empirical approach using MnPAVE (Version 6.3, 2014) program and presents its output results which are used to make detailed prediction of pavement performance. In other words, the output will allow estimating the performance life of HMA through achieving a certain level of permanent deformation. The Mechanistic-Empirical Pavement Design Guide is developed under the (NCHRP 1-37, 2004), a project represents a major advancement to pavement design and analysis. It uses site specific traffic, climatic conditions and materials properties to predict cracking and rutting performance of flexible pavement structures. Inclusion of basic material properties into distress prediction models through fundamentally based test procedures has facilitated the design of pavements based on site and material specific characteristics.

Material properties, traffic and climatic which required as an input data for the design analysis process are discussed. Input data which are required to be provided for the purpose of the program application are presented as well as the output data are discussed at the end of this chapter.

6.2 Mechanistic-Empirical Analysis Approach

Mechanistic empirical design methods are based on the mechanics of materials that relate an input, such as wheel load and material properties to an output of pavement response, such as stress or strain. In the M-E Design Guide procedure, the pavement is regarded as a multi-layered elastic system. The materials in each of these layers are characterized by modulus of Elasticity (E*) and Poisson's ratio (ν). This method requires the determination of critical stress, strain, or deflection in the pavement by some mechanistic method and the prediction of resulting damages by some empirical failure criteria prior to the thickness design and remaining life of the existing pavement which will be evaluated. In the M-E design process, the multi-layer structure is analyzed mechanistically to estimate the critical strains developed within the structure. These strain values are used to estimate the structural capacity in terms of repeated traffic loading by using the empirically derived transfer functions. The results of the elastic modulus (E) are compared with the results obtained from ultrasonic laboratory results to validate the mechanistic component of asphalt concrete mixtures. Figure (6-1) shows a flowchart of the steps used in this study.

Powell and Leech (1983) showed that the dynamic stiffness of the mixture increases by 30 % if the void content of the material is reduced by 3%. Linear elastic analysis of the construction as a whole shows that, by reducing the void content, the thickness of the construction can be reduced by 8%. The other advantage of adequate compaction is the increase in the resistance to the moisture damage. In the case of the perpetual pavement, enough stiffness in the upper pavement layers is needed to preclude rutting and enough total pavement thickness and flexibility in the lowest layer to avoid fatigue cracking from the bottom of the pavement structure. Since the HMA pavement is tailored to resist specific distresses in each layer, the materials selection, mix design, and performance testing need to be specialized for each material layer. Figure (6-2) shows the Mechanistic – Empirical Pavement Design Process.

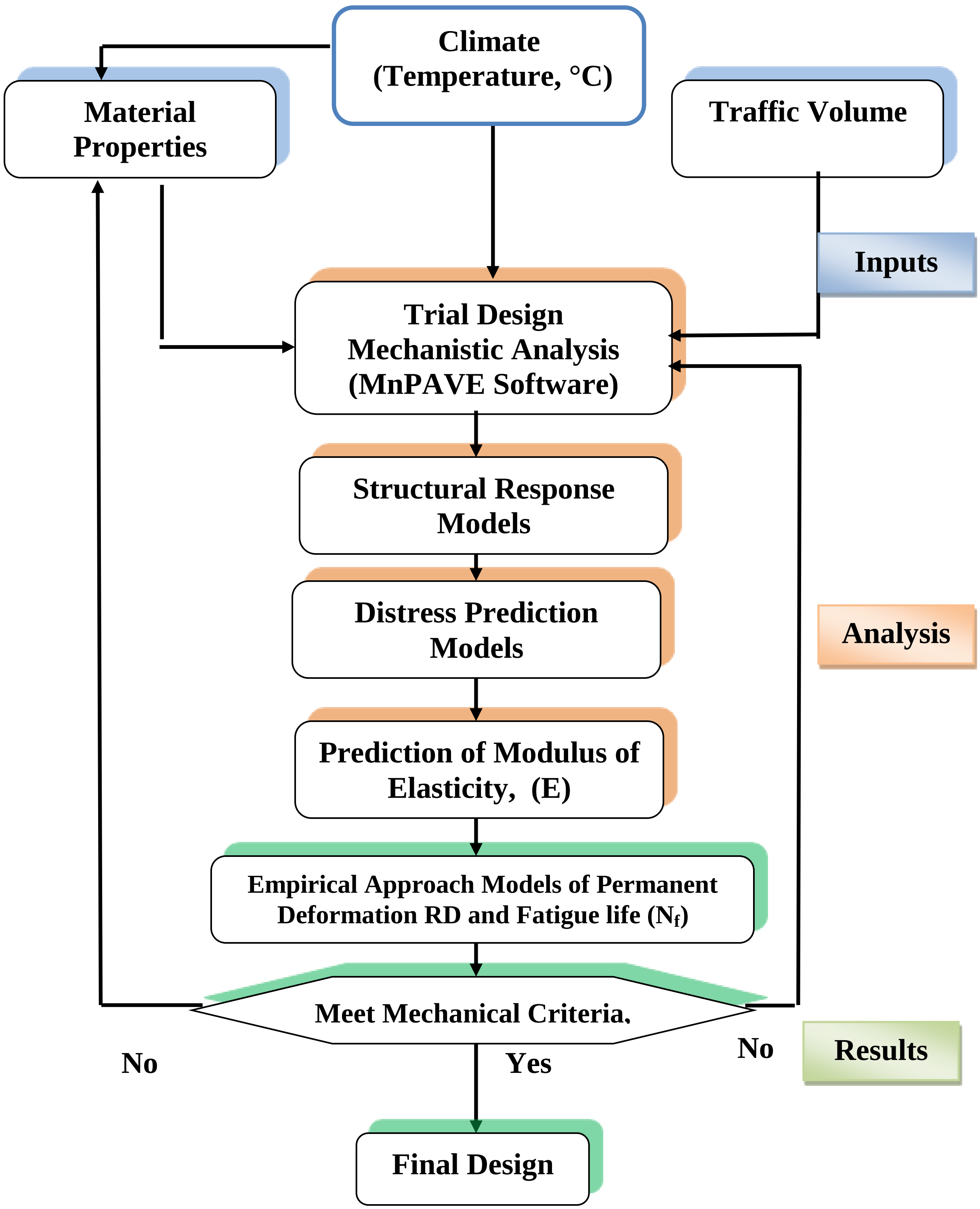

Figure (6-1): M-E flexible pavement design flow chart.

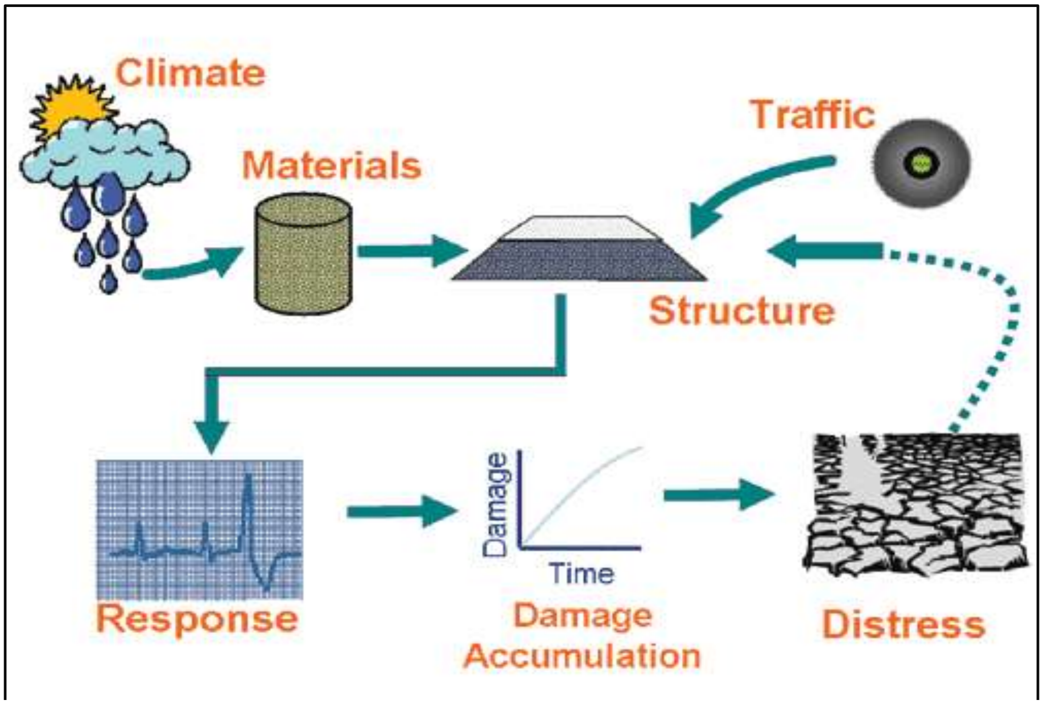

Figure (6-2): Mechanistic – empirical pavement design process, (M-E Design Guide Web site).

6.3 MnPAVE Flexible Pavement Design Approach

6.3.1 Introduction

MnPAVE is a computer program developed through a joint effort by the Minnesota Department of Transportation and the University of Minnesota version 6.3 on March, 2014. MnPAVE combines known empirical relationships with a representation of the physics and mechanics behind flexible pavement behavior. The mechanistic portions of the program rely on finding the tensile strain at the bottom of the asphalt layer, the compressive strain at the top of the subgrade, and the maximum principal stress in the middle of the aggregate base layer.

MnPAVE consists of three input modules: Climate, Structure, and Traffic, and three design levels: Basic, Intermediate, and Advanced. The level is selected based on the amount and quality of information known about the material properties and traffic data. In the basic mode, only a general knowledge of the materials and traffic data are required. The intermediate level corresponds to the amount of data currently required for projects. The advanced level requires the determination of modulus values for all materials over the expected operating range of moisture and temperature.

Material inputs include layer thickness, dynamic modulus, Poisson's ratio, and an index indicating the degree of slip between layers. MnPAVE assumes zero slip at all layer interfaces. Other inputs include load and evaluation locations. Loads are characterized by pressure and radius.

MnPAVE output includes the expected life of the pavement and the damage factors. Reliability also has been incorporated into the latest version. There is also a batch section for testing a range of layer thicknesses which includes various pavement responses for each season.

6.3.2 Mix design properties

Hot Mix Asphalt pavement mixtures are expected to perform over extended periods under a variety of traffic and environmental conditions. HMA properties are very important in resisting permanent deformation under traffic loads (Huang, 2004).

- ***Pavement Layers***

The thickness of the asphalt concrete surface course plays a crucial role in bearing load repetitions because a given percentage of increase in the expected loads can be accommodated by a much smaller percent increase in pavement thickness.

- ***Material Properties of Pavement Components***

According to the multi-layered elastic theory, the material properties of each layer such as resilient modulus and Poisson's ratio will contribute to the magnitudes of stress and strain in and between each layer and thus can directly reflect the permanent deformation behavior of pavements.

One of the material properties required in the M-E PDG which is considered innovative for pavement design methods are the dynamic modulus for asphalt concrete. Loading time and temperature dependency of asphalt mixtures are characterized by the dynamic modulus, |E*|. The dynamic modulus master curve models the variation of asphalt concrete stiffness due to rate of loading and temperature variation (hardening with low temperature/high frequency and softening with high temperature/low frequency). The nonlinear elastic behavior of unbound granular materials is modeled by a stress-dependent resilient modulus included as variables input.

The complex dynamic modulus |E*| is the principal material property input for asphalt concrete. It is a function of mixture characteristics: (binder, aggregate gradation, and volumetric properties), rate of loading, temperature, and age. For inputs, the dynamic modulus master curve is constructed based on time-temperature superposition principles by shifting laboratory frequency sweep test data, (Huang, 2004 and Pellinen et al., 2004). Binder viscosity measured in University of Al-Nahrain using the Superpave Rotational Viscometer (RV) is also a required input. For inputs, the dynamic modulus master curve is obtained via an empirical predictive equation. The |E*| predictive equation is an empirical relationship between |E*| and mixture properties as shown below Schwartz and Carvalho (2007):

$$\log E^{*} = 3.750063 + 0.02932\,.\rho_{200} - 0.001767.(\rho_{200})^2 - 0.002841\,.\rho_4 - 0.058097.V_a - 0.802208.\left(\frac{V_{beff.}}{V_{beff.}+V_a}\right) + \frac{3.871977+0.0021\,.\rho_4+0.003958\,.\,\rho_{38} - 0.000071\,.(\rho_{38})^2+0.005470.\rho_{34}}{1+e^{(-0.603313-0.313351.\log(f)-0.393532\,.\log(\vartheta))}} \quad \ldots(6\text{-}1)$$

In which:

E^* = dynamic modulus, 10^5 psi

η = binder viscosity, 10^6 Poise

f = loading frequency, Hz

V_a = air void content, %

V_{beff} = effective binder content, % by volume,

ρ_{34} = cumulative % retained on the 19-mm sieve,

ρ_{38} = cumulative % retained on the 9.5-mm sieve,

ρ_4 = cumulative % retained on the 4.75-mm sieve,

ρ_{200} = % passing the 0.075-mm sieve.

The values of viscosity for the binders and test temperatures relevant to this study are listed in Tables (6-1) and (6-2).

Table (6-1): Binder viscosity determined by RV for asphalt (40-50).

Binder Grade	Test Temperature	Viscosity, Pa.sec
AC (40-50)	25°C	3.97
AC (40-50)	60°C	2.28

Table (6-2): Binder viscosity determined by RV for asphalt (60-70).

Binder Grade	Test Temperature	Viscosity, Pa.sec
AC (60-70)	25°C	2.25
AC (60-70)	60°C	1.79

6.3.3 Factors affecting pavement performance

To use MnPAVE software according to Mechanistic-Empirical approach, the following parameters should be estimated:

1. Climate

The environmental conditions are simulated by using data from weather stations contained temperatures within the M-E PDG software database. The latitude and the longitudes of Baghdad are 33º21´S and 44º25´ E, respectively, with a continental climate hot and dry in summer and cold and rainy in winter with also short spring and autumn seasons. Baghdad's climate is not different form the general atmosphere of Iraq (Fadhil, 2007).

Temperature change affects the existing insitu resilient modulus of HMA. When the pavement surface cools, the asphalt binder will slowly transform from a ductile into a brittle material. Inherent in the pavement structure is a large number of flaws that are unable to transmit loads and will therefore act as stress concentrators (Ahmed, 2002).

Seasonal air and pavement temperatures were entered into MnPave software of Baghdad city as sourced from Funding Seismographic and Meteorological Commission. Each seasonal air temperature value represents the average daily temperature for that season. The equation used to convert air temperature to pavement temperature can be seen by clicking the temperature equation, as shown in Figure (6-3).

2. Traffic Loading and Volume

Traffic loads are simulated in MnPAVE to estimate the life of a given pavement design. In MnPAVE, the Equivalent Single Axle Load (ESAL) is a means of simplifying traffic data for pavement design. An ESAL is defined as an 18 kip (80 kN) dual tire axle with a tire pressure of 80 psi (552 kPa). Other axle loads and configurations can be converted to $ESAL_s$ by using Load Equivalency Factors (LEF) as defined in AASHTO Guide for Design of Pavement Structures. A LEF is the number of 18 kip $ESAL_s$ required to cause the same amount of damage as the axle in question. Lifetime $ESAL_s$ are the number of ESAL$_s$ expected during the number of years specified in Design Period Length. The First Year value is calculated based on the Design Period Length and Growth Rate.

If only First Year ESAL$_s$ are known, it can be entered here and Lifetime ESAL$_s$ will be calculated based on the Design Period Length and Annual Growth Rate (%). The Design Period Length is typically 20 years. This value can be adjusted by the user, as shown in Figure (6-4).

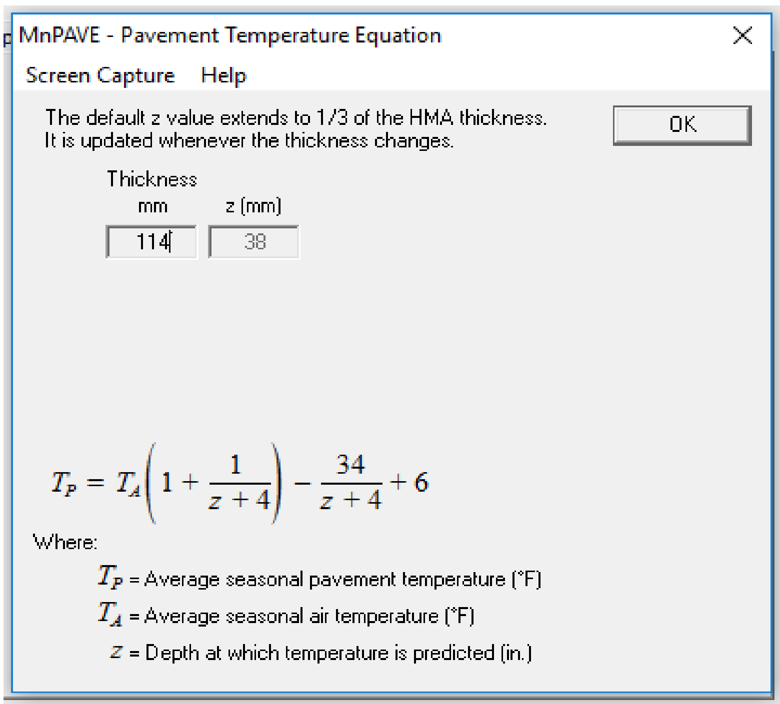

Figure (6-3): Temperature equation of MnPAVE software.

The Annual Growth Rate (3.2%) determines the amount that traffic increases during each year of the Design Period. Traffic analysis conducted by MnPAVE program has indicated that a simple growth model is appropriate for most routes, i.e. traffic increases by a fixed amount each year.

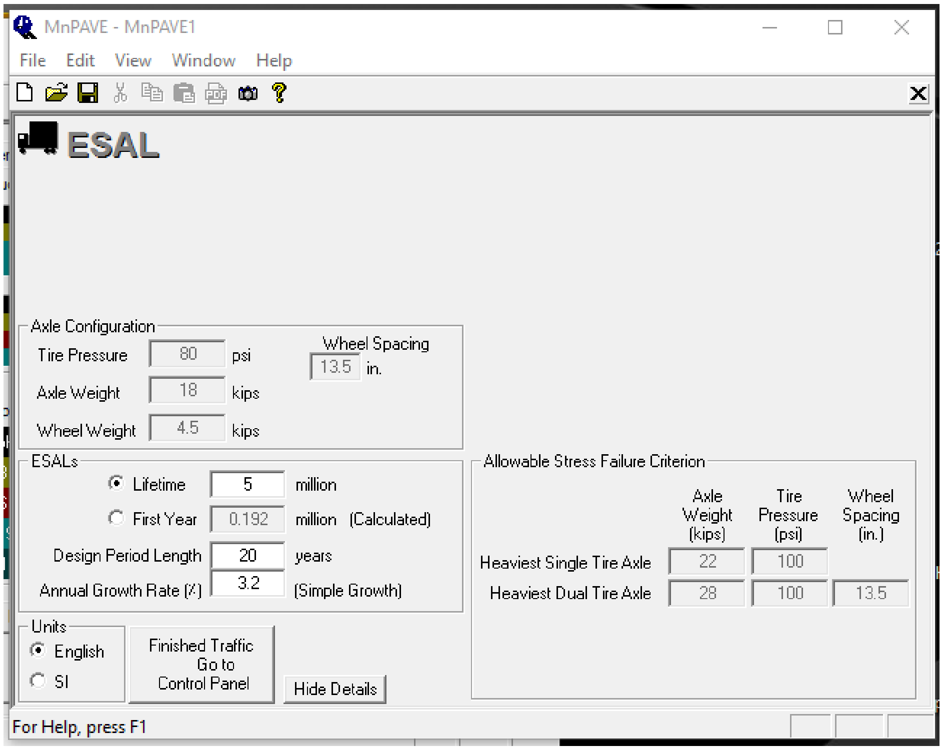

Figure (6-4): Traffic loading and volume of MnPAVE software.

The procedure for determining the 18-kip ESAL is summarized as follows (Huang, 2004):

1. Estimating the number of vehicles of different types, such as passenger cars, single-unit trucks, including buses, and multiple unit trucks of various types expected on the proposed or existed facility.
2. Determination of the number of each type of truck on the design lane during the first year of analysis. It is calculated according to the following equation:

$$n_i = 365 \,.\, AADT.\, P_i \,.\, D_d \,.\, L_d \;\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ (6\text{-}2)$$

where:

n_i = Number of vehicles type (*i*) on the design lane during the first year,

AADT= Annual Average Daily Traffic,

P_i = Percentage of i^{th} truck in the AADT,

D_d = Directional distribution factor, which is usually assumed to be 0.5 unless the traffic in two directions is different, and,

L_d = Lane distribution factor. For two lane highway, the lane distribution factor is 1 whereas for multilane highways its value ranges from 0.85 to 0.70 for two and three lanes in each direction, respectively.

3. Determination of a truck factor for each vehicle type. Truck factor is defined as the number of 18-kip axle load applications contributed by one passage of a truck. The values of truck factors for different vehicle types are presented elsewhere, (AASHTO, 2010).
4. For the given analysis period, the traffic growth factor is calculate for all vehicles or separate factors for each vehicle type, as appropriate. Growth factor can be calculated according to the following relationship, (AASHTO, 2010):

$$\boldsymbol{G_f} = \frac{(\mathbf{1}+\boldsymbol{r})^{\boldsymbol{Y}}-\mathbf{1}}{\boldsymbol{r}} \quad \text{………………………..} \ (6\text{-}3)$$

Where:

G_f = Growth factor,

r = Annual growth rate (decimal), and

Y = Analysis period, (years).

5. Multiply the number of trucks of each type during the first year by the truck factor and the growth factor and sum the values determined to obtain the 18-kip ESAL applications during the analysis period.

3. Structure

All mix design information is entered in the HMA mix properties window correctly. Mix design information such as asphalt binder content and gradation are required to estimate the HMA dynamic modulus, as shown in Figure (6-5). Currently, the selection of asphalt binder of (40-50) and (60-70) penetration serve to document the binders used in the design.

Gradation can be defined by entering numbers in the “Percent Passing” edit boxes through clicking on the colored bar as shown in Figure (6-6).

Basic Structure is intended for intermediate-volume roads or designs that don’t require a high degree of reliability, MnPAVE uses default design modulus values for them. These modulus values are adjusted for seasonal variations in moisture and temperature. The number of layers in the pavement structure is selected by clicking on a layer button. The bottom layer is always semi-infinite. MnPAVE pavement structures have between two and five layers, as shown in Figure (6-7).

Material Types for each layer are selected on the left side of the Structure window under Edit Structure. Layers with a white pointer arrow can be clicked to select a different subtype, as shown Figure (6-7).

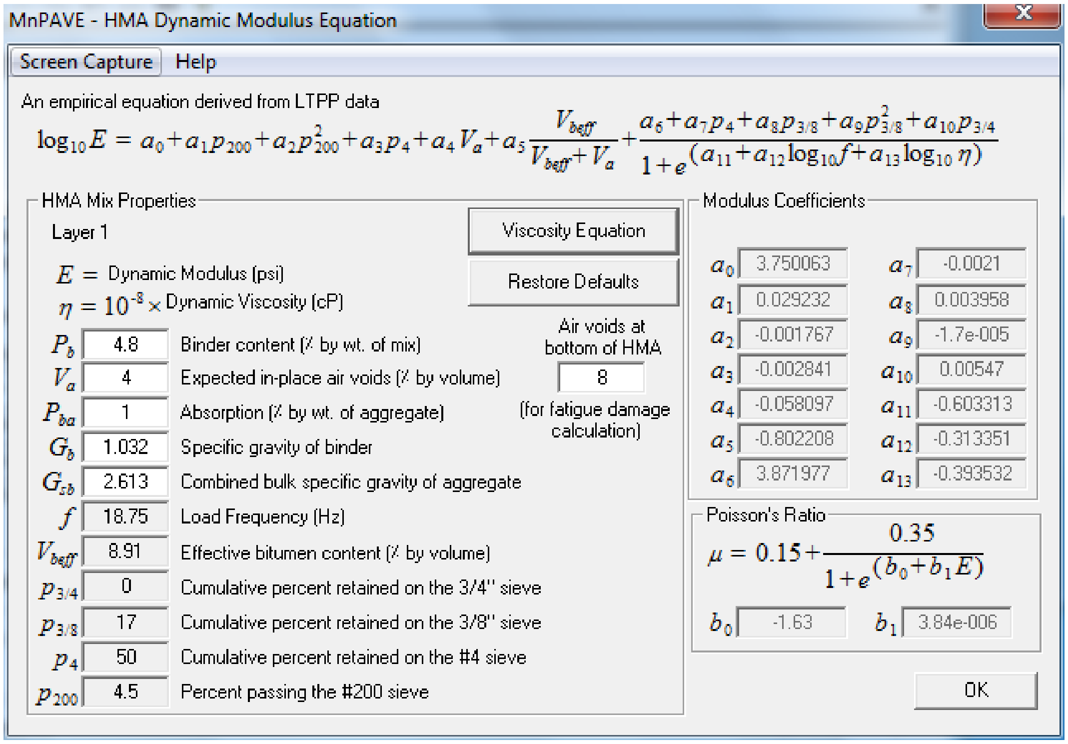

Figure (6-5): HMA dynamic modulus (E) equation.*

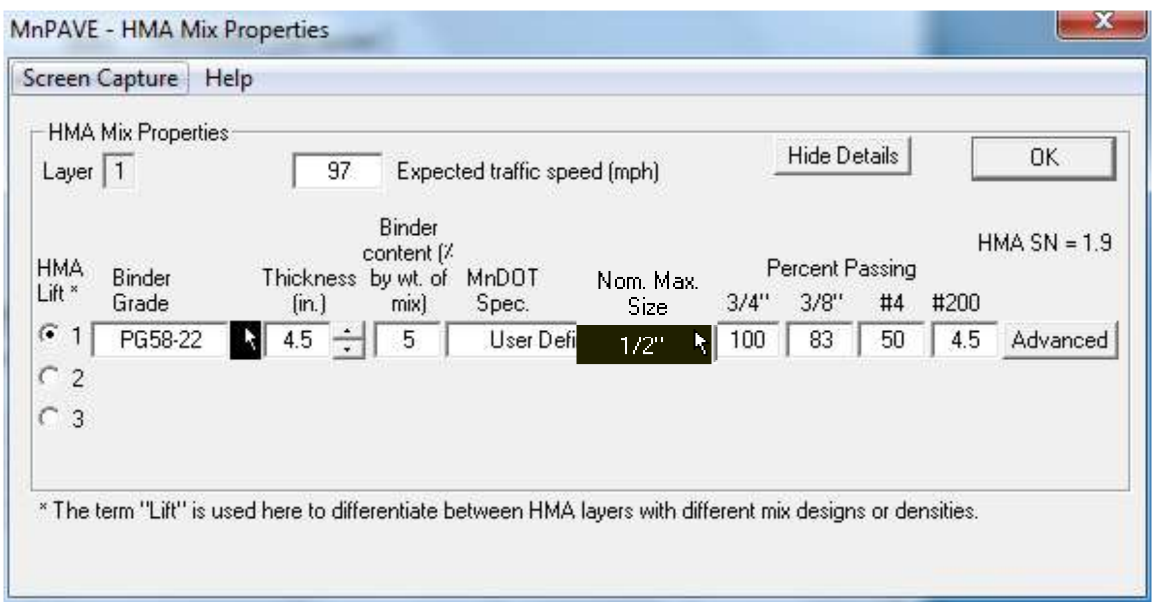

Figure (6-6): Mix properties of MnPAVE software.

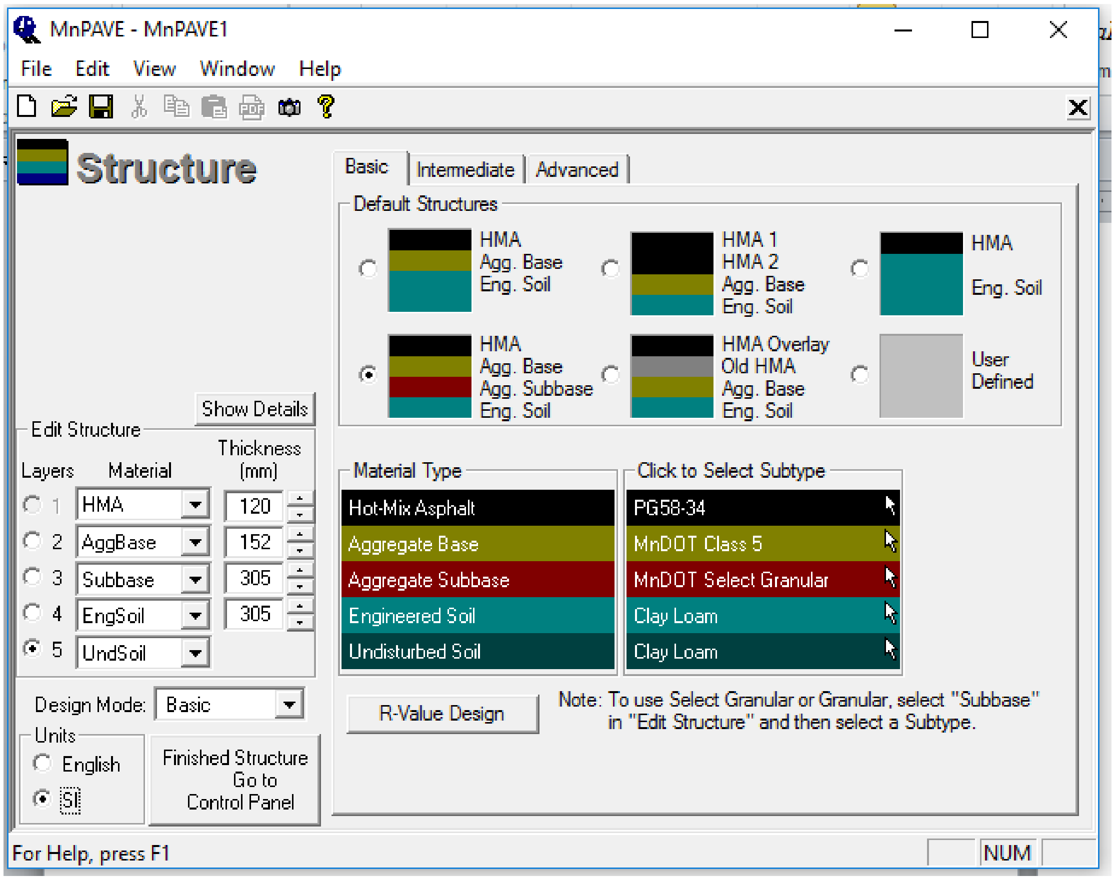

Figure (6-7): Pavements structures layers of MnPAVE software.

6.3.4 Application of MnPAVE Program

The M-E PDG requires a large set of material properties. Three components of the design process require material properties: the climate model, the pavement response models, and the distress models. Climate properties are used to determine temperature and moisture variations inside the pavement structure. The pavement response models use material properties; for temperature and moisture effects; to compute the state of stress/strain at critical locations in the structure due to traffic loading and temperature changes. These structural responses are used by the distress models along with complementary material properties to predict pavement performance.

- **Rutting and Fatigue Models**

The expected life of a pavement is calculated by simulating the strains due to traffic loads and using an empirical transfer function to determine the Allowed Repetitions for each load. If the Applied Load Repetitions exceed the allowed repetitions, the pavement is assumed to have failed.

Basic Output displays the expected years of pavement life based on calculated rutting and fatigue damage.

A summary report can be viewed at the final of Data analysis which contains expected life, damage factor, project information, and limited structural and traffic information as shown in Figure (6-8).

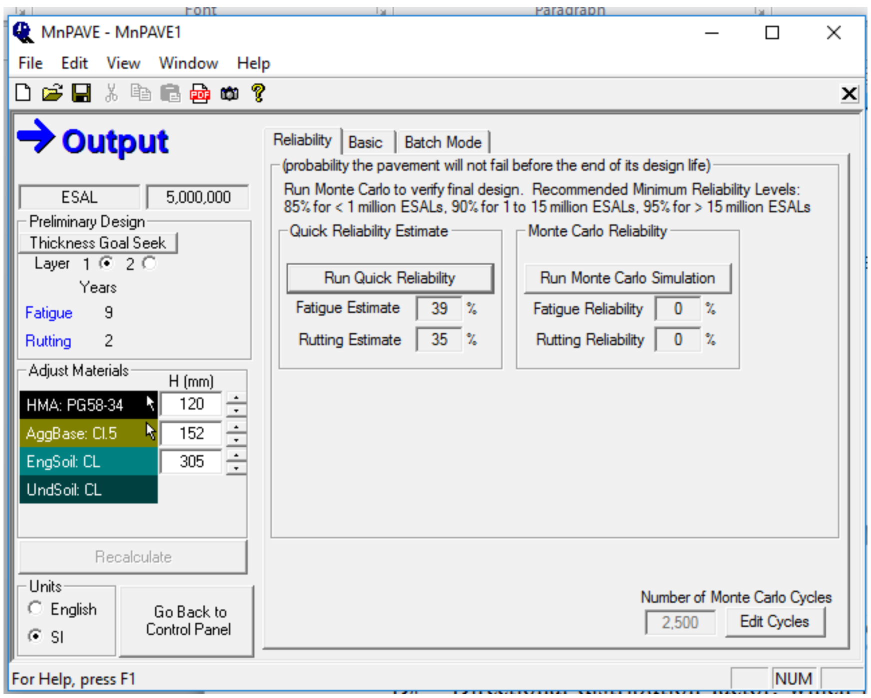

Figure (6-8): A Summary report of data analysis.

- **Reliability**

Reliability is defined as the probability that the design pavement will achieve its design life with serviceability higher than or equal to the specified terminal serviceability. Although the reliability factor is applied directly to traffic in the design equation. It does not imply that traffic is the only source of uncertainty.

Table (6-3) suggests appropriate levels of reliability for various highway classes. High volume and high speed highways have higher reliability factors than minor roads and local routes.

Output MnPAVE reliability considers the variability of the thickness, selected axle type and modulus values for each layer to determine a reliability value for the pavement design. In this program, the reliability will not necessarily agree with the confidence level selected in Structure because the confidence level selects the worst case thickness and modulus value for each layer while the reliability analysis considers a random combination of thickness and modulus values. The allowed repetitions are then calculated. Once a sufficient number of cycles have been completed, a distribution of allowed repetitions can be generated.

Table (6-3): Suggested levels of reliability for various highway classes (AASHTO, 1993).

Functional Classification	Recommended Level of Reliability	
	Urban	Rural
Interstate and Freeways	85-99.9	80-99.9
Principal arterials	80-99	75-95
Collectors	80-95	75-95
Locals	50-80	50-80

6.3.5 MnPAVE results and data analysis

Table (6-4) presents output data from this program which are used to make detailed prediction of pavement performance.

In order to evaluate the effect of input variability on the calculated reliability; the influence of air voids, asphalt type, asphalt content and RAP percent on dynamic modulus are evaluated, as described in the following section:

Table (6-4): Output data of MnPAVE Program for wearing course.

Sample No.	Ac, %	Av, %	No. of Gyrations	Asphalt viscosity, Pa.sec	RAP,%	Elastic Modulus, (E*), Psi	Reliability, %
1	4.8	7	60	0.537	0	651,188	64
2	4.8	7	60	0.537	7	701,453	70
3	4.8	7	60	0.537	13	821,598	75
4	4.7	7	60	0.475	19	913,868	77
5	4.7	7	60	0.475	25	1,155,831	80
6	4.8	5.4	80	0.537	0	913,888	66
7	4.8	5.4	80	0.537	7	1,014,567	72
8	4.8	5.4	80	0.537	13	1,121,598	80
9	4.7	5.4	80	0.475	19	1,303,868	82
10	4.7	5.4	80	0.475	25	1,355,831	88
11	4.8	4	100	0.537	0	1,078,667	82
12	4.8	4	100	0.537	7	1,193,843	85
13	4.8	4	100	0.537	13	1,342,884	88
14	4.7	4	100	0.475	19	1,413,548	96
15	4.7	4	100	0.475	25	1,513,888	98
16	4.8	3.6	120	0.537	0	1,117,667	66
17	4.8	3.6	120	0.537	7	1,253,843	72
18	4.8	3.6	120	0.537	13	1,342,884	80
19	4.7	3.6	120	0.475	19	1,513,548	82
20	4.7	3.6	120	0.475	25	1,613,888	88

6.3.5.1 Effect of air voids

Air voids is an important parameter which has a pivot role of the performance of asphalt pavement. Dynamic modulus of asphalt mixture at four levels of air voids are evaluated: (a) 7% air voids level and (b) 5.4% air voids level (c) 4% air voids level, and (d) 3.6 % air voids with two types of asphalt (40-50) and (60-70) penetration, and two optimum asphalt contents in addition to four percentages of reclaimed asphalt. The results are shown in Figure (6-9). It is found that increasing of air voids from 4% to 7% will decrease E* and reliability by 11.2% and 12%, respectively for asphalt of (40-50), as shown in Figure (6-10).

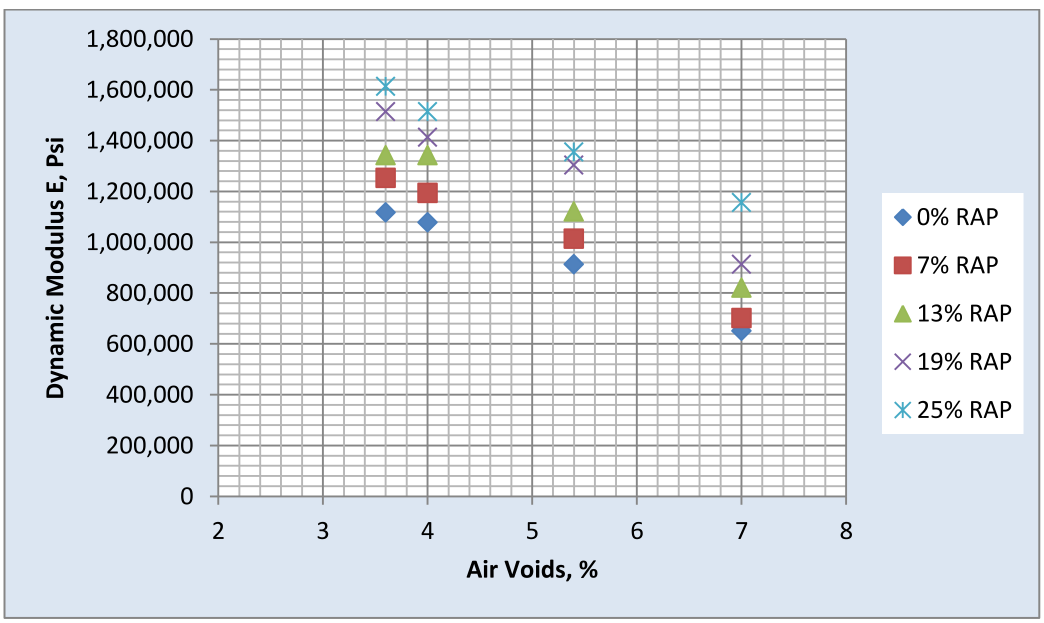

Figure (6-9): Effect of air voids on dynamic modulus (E*).

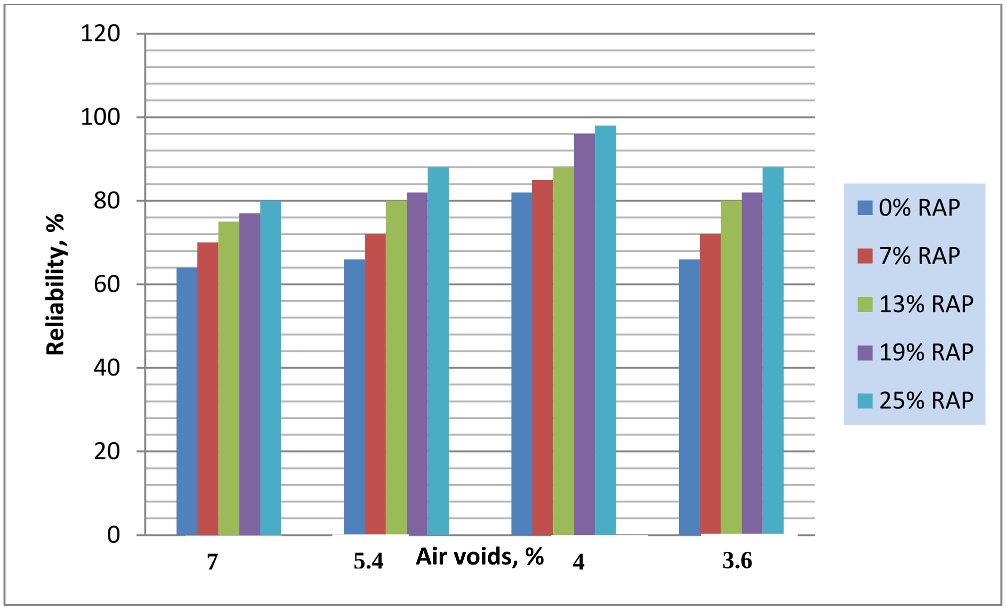

Figure (6-10): Effect of air voids on reliability.

6.3.5.2 Effect of reclaimed asphalt content

The effect of reclaimed asphalt content on the dynamic modulus was evaluated; Figure (6-11) shows the results of dynamic modulus for two asphalt contents at 4 percent air voids. It is observed that a change of reclaimed asphalt content from (7% to 13 %) and from (19 to 25 %) causes a 12.4 % and 7 % increase in E* at 25º C test temperature, while a change of reclaimed asphalt content from (7% to 13 %) and from (19 to 25 %) causes 3.5 % and 2.08 % increase in reliability at 25º C test temperature, as shown in Figure (6-12).

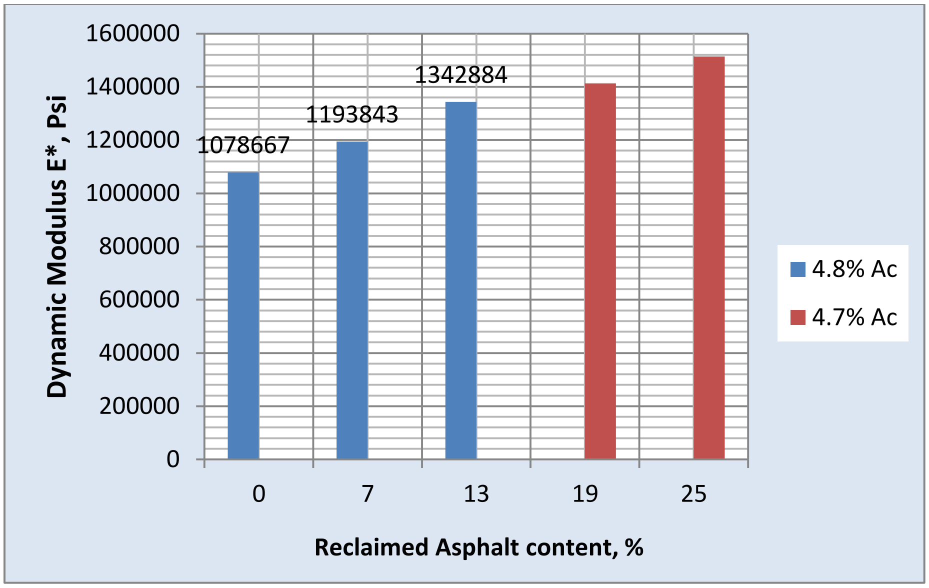

Figure (6-11): Effect of asphalt content on dynamic modulus.

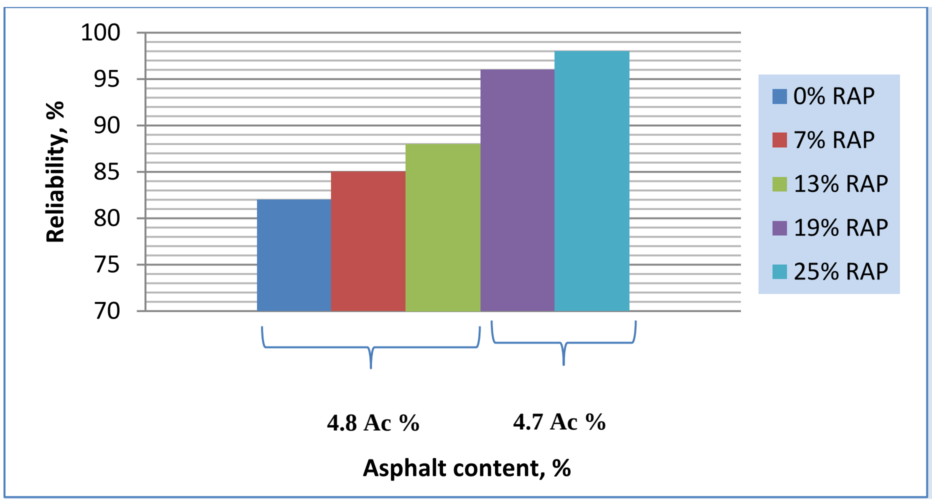

Figure (6-12): Effect of asphalt content on reliability.

6.3.5.3 Effect of asphalt binder type

Asphalt binder is an essential component of asphaltic mixtures. The performance of an asphaltic mixture is directly related to mechanical characteristics of the binder. Therefore, there is a need to evaluate the relationship between the properties of binders and asphaltic mixtures such that a proper understanding and selection of an asphalt binder can be made to improve the performance of an asphaltic mixture. In this study, the effects of asphalt binder properties on asphaltic mixtures were evaluated.

The results show that the asphalt types; grades (40-50) and (60-70) have a significant influence on the dynamic modulus at different reclaimed asphalts; higher PG grade leads to lower E*. It is found that a change of binder from grade (40-50) to (60-70) causes 5.2 % decrease in E* and 9 % decrease in reliability.

CHAPTER SEVEN
CONCLUSIONS AND RECOMMENDATIONS

7.1 Conclusions

The following conclusions are limited to the materials used and test conditions under which the experimented works were conducted:

1. Marshal stability results show that the increase in RAP content will increase the stability at both optimum asphalt content and +0.5 optimum asphalt content. The stability increased by (13.8 %, 21.4%, 26.7% and 34.5%) % for (7%), (13%), (19%) and (25%) of RAP content, respectively, at optimum asphalt content.
2. The durability results show that increasing of RAP will lead to increase the resistance to water along immersion day. The durability increases by (14%, 22.2%, 29.4% and 36.7%) for (7%), (13%), (19%) and (25%) of RAP content, respectively, for one-day emersion.
3. Results of indirect tensile strength show that increasing of RAP will increase the value of indirect tensile strength for condition and unconditioned samples. For condition samples, the resistance increases by 9.35%, 19%, 31% and 39.3% for (7%), (13%), (19%) and (25%) of RAP content, the indirect tensile strength ratio (TSR) are increases to 4.3% and 6.4% when increasing RAP percentages to 7% and 13%, respectively.
4. Inclusion of RAP material results in an increase in the mixtures resistance to compressive strength, the value increases by 12.19%, 19.9%, 27.9% and (32.1%) for (7%), (13%), (19%) and (25%) of RAP content, respectively, for unconditioned sample at optimum asphalt content.

5. The use of RAP material has an effect on modulus of elasticity in Ultrasonic Test. When the RAP content is increase to 13% and 19%, this will lead to increase in the elasticity modulus by 3.77% and 5.25%. The value of modulus elasticity is increased by 3%, 6% and 8.66% when number of gyrations increased from 60 to 80, 100 and 120 at 19% RAP content.

6. An increase in the asphalt content by 0.5% by weight results in decrease in stability value and indirect tensile strength but leads to an increase in compressive strength test.

7. Based on the Indirect Tensile Test, the HMA with RAP additive of (19%) provides a high resistance value (96%) to moisture damage.

8. Based on test results, models were developed to predict stability, retained stability, and indirect tensile strength of local asphalt concrete surface course mixtures for different test conditions and mix properties using statistical technique. The following forms were found:

$$S = 8.21 - 0.809Ac + 12.079\ V_b - 0.255\ P + 0.259\ R \quad \text{(7-1)}$$

$$RS = 47.899 + 2.486\ Ac - 45.795\ V_b + 1.209\ R - 0.433\ P + 5.583S \quad \text{(7-2)}$$

$$ITS = 3229.09 - 274.623\ Ac - 1419.92\ V_b + 13.558\ R - 77.035\ C - 11.309\ T \quad \text{(7-3)}$$

9. Mechanistic Empirical design approach through MnPAVE 2014 software was used to characterize the dynamic modulus in HMA and reliability as a function of expected traffic loads, material properties, and environmental conditions. The influence of variables on the dynamic modulus E* and reliability is evaluated.

10. From the M-E Design Guide procedure results, it is found that increasing of air voids from 4% to 7% will decrease E* and reliability by 11.2% and 12% respectively for asphalt grade of (40-50),

11. Increasing of reclaimed asphalt content influences on Dynamic modulus, a change of reclaimed asphalt content from (7% to 13 %) and from (19 to 25 %) causes a 12.4 % and 7 % increase in E* at 25º C test temperature. while a change of reclaimed asphalt content from (7% to 13 %) and from (19 to 25 %) causes 3.5 % and 2.08 % increase in Reliability at 25º C test temperature.

7.2 Recommendations

Based on the current findings, the following recommendations can be given:

1. Only one source of RAP has been studied in this study. Multiple RAP sources should be investigated.
2. Evaluating the mixture performance of the designed asphalt mixture containing high RAP content is recommended. There are a variety of performance tests available for evaluating the probable permanent deformation, fatigue, and thermal cracking performance of compacted asphalt mixtures.
3. Further field testing is recommended to validate the performance characteristics of field compacted mixtures to laboratory compacted mixtures.
4. Further documentation of the production, construction, and long-term performance of high RAP mixtures is needed.
5. Consideration should be given to include documentation of RAP use in a pavement management system with details concerning RAP quantities used, sources, and placement details.

References

- AASHTO Guide for Design of Pavement Structures, (2010), The American Association of State Highway and Transportation Officials, Civil Engineering Department, Washington, D. C., USA.
- Ahmed, N. G., (2002), “The Development of Models for the Prediction of Thermal Cracking in Flexible Pavements”, Ph.D. Thesis, Civil Engineering, Baghdad University.
- Al-Moula, Z. I., (2012), “Prediction Model For Pavement Deformation Of Superpave Asphalt Concrete Mixtures”, Ph. D Thesis, University of Baghdad.
- Al-Rousan, T., Asi, I., Al-Hattamleh, O., and Al-Qablan, H., (2008), ‘‘Performance of Asphalt Mixes Containing RAP’’, Jordan Journal of Civil Engineering, Vol. 2, No. 3, pp. 218-227.
- Al-Qadi, I., Elseifi, M., and Carpenter, S., (2007), “Reclaimed Asphalt Pavement—A Literature Review”, Report No. FHWA-ICT-07-001, Illinois Center for Transportation, Rantoul, IL.
- Al-Qadi, I.L., Carpenter S. H., Roberts G., Ozer H. and Aurangzeb Q., (2009), Determination of Usable Residual Asphalt Binder in RAP (Illinois Department of Transportation Research Report ICT-09-031), Springfield, IL: IIIinois Department of Transportation. 2009.
- Al-Qadi, I. L., Aurangzeb Q. and Carpenter S. H., (2012). “Impact of High RAP Contents on Structural and Performance Properties of Asphalt Mixtures”, Research Report FHWA-ICT-12-002, Illinois Department of Transportation.
- Apeagyei A., Diefnderfer B. & Diefnderfer S. (2011). “Rutting Resistance of Asphalt Concrete Mixtures that Contain Recycled Asphalt Pavement”, Transportation Research Record: Journal of the Transportation Research Board, Vol 2208, pp. 9-16.

- ARRA. (2001). Basic Asphalt Recycling Manual, Asphalt Recycling and Reclaiming Association. U.S. Department of Transportation, Federal Highway Administration.
- Asphalt Institute, (1996), "Superpave Mix Design", Superpave Series No. 2 (SP-2), the Asphalt Institute, Lexington, Kentucky.
- Asphalt Institute, (2003). "Superpave Mix Design SP-2", Third Edition, Lexington, KY, USA.
- Asphalt Institute, (2007). "Asphalt Hand Book‖, Manual series No. 04, Sixth Edition, Kentucky, USA.
- ASTM, (2009). "Road and Paving Materials", Annual Book of ASTM Standards volume 04.03.American Society for Testing and Materials", West Conshohocken, USA.
- Beam, J. and D. Maurer (1991). "Evaluation of Recycled Hot-mix Asphaltic Concrete in Pennsylvania". Final report, Mar 83-Feb 91, Pennsylvania Dept. of Transportation, Harrisburg, PA (United States). Bureau of Bridge and Roadway Technology.
- Brown E., Kandhal P. S., Roberts F.L., Kim Y.R., Lee D. and Kennedy, T.W. (2009). "Hot-Mix Asphalt Materials, Mixture Design, and Construction". Third Edition, NAPA Research and Education Foundation, Lanham, Maryland.
- Brownie R. and Hironaka M. (1979). "Recycling of Asphalt Concrete". Airfield Pavement. A Laboratory Study, DTIC Document.
- Cascante G., Ponniah J. and Jiang Z. (2006). "Improved Ultrasonic Pulse Velocity Technique for Bituminous Material Characterization", Civil Engineering Department,University of Waterloo, 200 University Avenue. West Waterloo, ON., Canada N2L 3G1.
- Celauro, C., Bernardo, C. and Gabriele B. (2010). "Production of Innovative, Recycled and High-performance Asphalt for Road

Pavements", Resources, Conservation and Recycling, Vol. 54, pp. 337-347.

- Chiu, C., Hsu, T. and Yang W., (2008). "Life Cycle Assessment on using Recycled Materials for Rehabilitating Asphalt Pavements", Resources Conservation and Recycling. Vol. 52(3), pp. 545-556.
- Claessen, A., Edwards P., Sommer P. and Uge P. (1977). "Asphalt Pavement Design: the SHELL Method", Proceedings, Fourth International Conference on the Structural Design of Asphalt Pavements, the University of Michigan, Ann Arbor, Michigan, USA.
- Copeland, A. and Opeland A. 2011. Reclaimed Asphalt Pavement in Asphalt Mixtures: State of the practice. Report No. FHWA-HRT-11-021
- Corley-Lay and Judith B. (1996). "Transportation to Develop a Mechanistic Pavement Design System", Efforts by North Carolina Dept. of. Preprint Paper - Transportation Research Board Meeting, Washington, D.C.
- Cosentino P. J., Kalajian, Shieh E. H., Mathurin C., Gomez W., Cleary F. E. and Treeratrakoon A. (2003). Developing specifications for using recycled asphalt pavement fill materials, as base, subbase or General. Phase II. Florida Institute of Technology. Vol. 352. p.p 955-2924.
- Devore J. (2000), "Probability and Statistics for Engineering and the Sciences", 5th Edition, Duxbury/Thomson Learning, USA.
- Doh Y.S., Amirkhanian S.N. and Kim K.W., (2008), "Analysis of Unbalanced Binder Oxidation Level in Recycled Asphalt Mixture Using GPC", Construction and Building Materials, Vol. 22, pp. 1253–1260.
- Dormon G. M. and Metcalf C. T., (1965). "Design Curves for Flexible Pavements Based on Layered System Theory." Highway Research Record, Number 71, Flexible Pavements Design, Highway Research Board, Washington, D.C. 1965, pp. 69-84.

- Fadhil, A.M., (2007), “Environmental Change Monitoring by Geoinformation Technology for Baghdad and its Neighboring Areas”, GIS Research Project, Technical College, Baghdad.
- Harman, T., Angelo, J.D., Bukowski, J.(2009), “Superpave Asphalt Mixture Design Workshop”, FHWA National Asphalt Training Manuals Asphalt Institute MS & SP Series.
- Huang, Y. (2004). “Pavement Analysis and Design”, 2nd Edition, Prentice Hall, Englewood Cliffs, New Jersey, USA.
- Hussain, A. and Yanjun, Q., (2012), “Evaluation of Asphalt Mixes Containing Reclaimed Asphalt Pavement for Wearing Course”, 2012 International Conference on Traffic and Transportation Engineering, IPCSIT vol. 26 , Singapore, pp. 43-48.
- Kallas, B. F. (1984). “Flexible Pavement Mixture Design Using Reclaimed Asphalt Concrete”, FHWA/RD-84/088.
- Kandhal, P.S., Rao, S.S., Watson D.E. and Young, B., (1995), “Performace of Recycled Hot Mix Asphalt Mixtures”, National Center for Asphalt Technology (NCAT), Report No. 95- 1.
- Kandhal, P.S. and Mallick R.B., (1997), “Pavement Recycling Guidelines for State and Local Governments Participant's Reference Book”, Report No. FHWA-SA-98-042, Federal Highway Administration Office of Engineering.
- Kelly, T. (1998), “Crushed Cement Concrete Substitution for Construction Aggregates-a Material Flow Analysis”, U.S. Department of the Interior, U.S.

  Geological Survey Circular 1177, Online access: http://purl.access.gpo.gov/GPO/LPS22459.

- Kennedy, J.B. and Neville, A.M. (1986), “Basic Statistical Methods for Engineers and Scientists”, 3rd Edition Thomas Y., Crowell Company, Inc. USA.
- Kennedy, T. W., W. O. Tam, and M. Solaimanian, (1998) “Optimizing Use of Reclaimed Asphalt Pavement with the SuperPave System,” Journal of the Association of Asphalt Paving Technologists, Vol. 67, , pp. 311-333.
- Kiggundu, B. and Newman, J. (1987). Asphalt-aggregate Interactions in Hot Recycling. Final Report ESL-TR- 87-07, New Mexico Engineering Research Institute.
- Majeed A. D., (2016), “Implementation of Using the Reclaimed Asphalt Pavement (RAP) for Road Construction”, Msc Thesis, the Highway and Transportation Engineering Department, College of Engineering, University of Technology.
- Malpass, G. A. (2003). “The Use of Reclaimed Asphalt Pavement in New Superpave Asphalt Concrete Mixtures”. North Carolina State University.
- Mashkoor O. G., (2015). “Durability Evaluation of Polymer Modified Open Graded HMA Mixtures Based on Experimental Work and Fe Modeling”, Msc Thesis, the Highway and Transportation Engineering Department, College of Engineering, Al-Mustansiriayah University.
- McDaniel, R. and Anderson, R.M. (2001). Recommended Use of Reclaimed Asphalt Pavement in Superpave Mix Design Method: Technician’s Manual. NCHRP Report 452. Transportation Research Board- National Research Council, Washington, D.C.
- McDaniel, R. S., Soleymani, H., Anderson, R. M., Turner, P. and Peterson R. (2000). “Recommended Use of Reclaimed Asphalt Pavement in the Superpave Mix Design Method”, NCHRP Web document, 30.

- Miro, R., Valdés, G., Martinez, A., Segura, P. and Rodriguez, C., (2011), “Evaluation of High Modulus Mixture Behavior with High Reclaimed Asphalt Pavement (RAP) Percentages for Sustainable Road Construction”, Construction and Building Materials, Vol. 25, pp. 3854–3862.
- Montgomery, D. C. and Peck E. A. (1992), “Introduction to Linear Regression Analysis”, 2nd Edition, John Wiley and Sons, Inc., New York.
- National Highway Institute, (1998), “Superpave Fundamentals Reference Manual” course #131053, U.S. Department of Transportation, Federal Highway Administration.
- NCHRP, (2004), “Mechanistic-Empirical Design of New and Rehabilitated Pavement Structures”, NCHRP Project 1-37A Draft Final Report, Transportation Research Board, National Research Council, Washington, D.C.
- Neter, J., Wassermann W. and Kutner, M.H. (1990), “Applied Linear Statistical Models: Regression, Analysis of Variance and Experimental Design”, 3rd edition, Richard D. Irwin, Inc., Homewood, IL, 1181.
- NRA 2005; Use of Recycled Glass Bottles in Road Pavement Subsurface Layers.
- O' Flaherty, A.M. (2007), “Highways: The location, design, construction and maintenance of road pavements”, Fourth edition published by Butterworth- Heinemann,
- Pellinen, T.K., Song, J. and Xiao, S. (2004). “Characterization of Hot Mix Asphalt with Varying Volumetric Composition Using Triaxial Shear Strength Test”, 8th Conference of Asphalt Pavements in South Africa, CAPSA-04, September, p.p 12-16.
- Perez F., Rodriguez M., De Visscher J., Vanelstraete A. and De Bock L., (2004), “Design and Performance of Hot Mix Asphalts with High Percentages of Reclaimed Asphalt- Approach Followed in the PARAMIX

Project", Proceedings of the 3rd Eurasphalt & Eurobitume Congress, Vienna, paper-098.

- Pierce, L.M., Jackson, N.C., and Mahoney, J.P. (1993). "Development and Implementation of a Mechanistic-Empirical Based Overlay Design Procedure for Flexible Pavements", Transportation Research Board No. 1388, Washington DC.
- Powell W. D. And Leech D., (1983), "Compaction of Bituminous Road Materials Using Vibratory Rollers". Transport Research Laboratory, Crowthorne, LR 1102.
- Putri, A. M. and Suparma, L. B. (2010), Laboratory Study on the Durability Characteristics (moisture damage evaluation) of Asphalt Concrete Wearing Course (Ac-Wc) Utilizing bantak and Clereng as Aggregate (using Marshall Methods). Journal of the Eastern Asia Society for Transportation Studies, 8, 1555-1567.
- Qasim, H. A., "Experimental Evaluation for Local Reclaimed Asphalt Pavement", Msc Thesis, the Highway and Transportation Engineering Department, College of Engineering, Al-Mustansiriayah University.
- Ramanujam, J.M., (2000), "Recycling of Asphalt Pavements", Department of Transport & Main Roads, Queensland, Internet link Http://Www.Docstoc.Com/Docs/42047013/Recycling-Of-Asphalt-Pavements.
- Reyes, O., Pérez, F., Miro, R., and Botella, R. (2009). Proyecto fénix. Mezclas semicalientes. Proceedings del XV Congreso Iberolatinoamericano del Asfalto. Lisboa, Portugal.
- Roberts, F.L.; Kandhal, P.S.; Brown, E.R.; Lee, D.Y. and Kennedy, T.W. (1991). "Hot Mix Asphalt Materials, Mixture Design, and Construction", National Asphalt Pavement Association Education Foundation. Lanham, MD.

- Santucci, L., (2007), “Recycling Asphalt Pavements—A Strategy Revisite”, Tech Topic No.8, Technology Transfer Program, Institute of Transportation Studies, University of California Berkeley.
- Sarsam S.I., (2007), “A Study on Aging and Recycling of Asphalt Concrete Pavement”, University of Sharjah, Journal of Pure & Applied Sciences, Vol. 4, No. 2, pp. 79-96.
- Scheaffer, L. R., and McClave J. T., (1990), “Probability and Statistics for Engineers”, 3rd Edition, PWS-KENT Publishing Company, Boston.
- Schwartz C. W. and Carvalho R. L. (2007), “Implementation of the NCHRP 1-37A Design Guide”, Department of Civil and Environmental Engineering, the University of Maryland, Vol.2.
- SCRB / R9, (2003), “General Specifications for Roads and Bridges”, Section R/9, Hot Mix Asphalt Concrete Pavement, Revised Edition. State Corporation of Roads and Bridges, Ministry of Housing and Construction, Republic of Iraq.
- Shook, J., Finn F., Witezak M., and Monismith C. (1982), “Thickness Design of Asphalt Pavements-The Asphalt Institute Method”, Proceedings of the Fifth International Conference on the Structural Design of Asphalt Pavements, Vol., Delft, Netherlands.
- Shu, X., Huang, B., and Vukosavljevic, D., (2008), “Laboratory Evaluation of Fatigue Characteristics of Recycled Asphalt Mixture”, Construction and Building Materials, Construction and Building Materials, vol. 22, pp. 1323-1330.
- Silva, H.M.R.D., Oliveira J.R.M., and Jesus C.M.G., (2012), “Are Totally Recycled Hot Mix Asphalts a Sustainable Alternative for Road Paving”, Resources, Conservation and Recycling, Vol. 60, pp. 38-48.
- Smith, L. (1996). “Wasting No Time: Florida's History of Successful Recycling for Transportation”, TR News No. 184:4-7.

- Stroup-Garinder, m. and Wagner C. (1999), “Use of Reclaimed Asphalt Pavement in Superpave Hot-mix Asphalt Applications”, Transportation research record: journal of the transportation research board, Vol. 1681, pp. 1-9.
- Sullivan, J. (1996), “Pavement Recycling Executive Summary and Report”, Report FHWA-SA-95-060 from the Federal Highway Administration. Washington, D.C.
- Tabaković, A., Gibney, A., McNally, C. and Gilchrist, M.D., (2010), “Influence of Recycled Asphalt Pavement on Fatigue Performance of Asphalt Concrete Base Courses”, Journal of Materials in Civil Engineering, ASCE, Vol. 22, No. 6, pp. 643-650.
- Tabakovic, A., (2013), “Recycled asphalt (RA) for pavements. Handbook of Recycled Concrete and Demolition Waste”, PP. 394-423.
- Taylor, N. (1997), “Life Expectancy of Recycled Asphalt Paving. Recycling of bituminous pavements”, (L. E. Wood, Ed.). ASTM STP 662, pp. 3–15.
- Terrel, R.L. and Lee S.Q.S., (1997), “Hot In-place Recycling of Asphalt Pavements – Experience in an Urban Setting Using New Technology”, Arab Urban Development Institute, Al-Ain.
- Valdés, G., Jiménez, F.P., MirO, R., MarTInez A. and Botella R., (2011), “Experimental Study of Recycled Asphalt Mixtures with High Percentages of Reclaimed Asphalt Pavement (RAP)”, Construction and Building Materials, Vol. 25, pp. 1289-1297.
- Rose J.L., Van Velsor J. K., Premkumar L. and Chehab G. (2011), “Measuring the Complex Modulus of Asphalt Concrete Using Ultrasonic Testing”, Journal of Engineering Science and Technology Review 4 (2) p.p 160-168.

- Vollor, T. W. (1986), “Asphalt Pavement Recycling Primer”, DTIC. Document.
- Watson, D.E., Nordcbeck A.V., Moore J., Jared D. and Wu P., (2008), “Evaluation of the Use of Reclaimed Asphalt Pavement in Stone Matrix Asphalt Mixtures”, In Transportation Research Record: Journal of the Transportation Research Board, No. 2051, TRB, National Research Council, Washington, D.C., p.p. 64-70.
- West, R. (2010), “Reclaimed Asphalt Pavement Management: Best. Practices”, National center for asphalt technology, Aubum University.
- West, R., Willis, J. and Marasteanu, M. (2013a), NCHRP Report 752: Improved Mix Design, Evaluation, and Materials Management Practices for Hot Mix Asphalt with High Reclaimed Asphalt Pavement Content. Transportation Research Board of the National Academies, Washington, DC.
- West, R. C., Rada, G. R., Willis, J. R. and Marasteanu, M. O., (2013b), Improved Mix Design, Evaluation, and Materials Management Practices for Hot Mix Asphalt with High Reclaimed Asphalt Pavement Content, Transportation Research Board.
- Widyatmoko I. (2008), “Mechanistic-empirical mixture design for hot mix asphalt pavement recycling”, Construction and Building Materials, Vol. 22, pp. 77-87.

APPENDIX A

CRITERIA OF HOT MIX ASPHALT DESIGHN

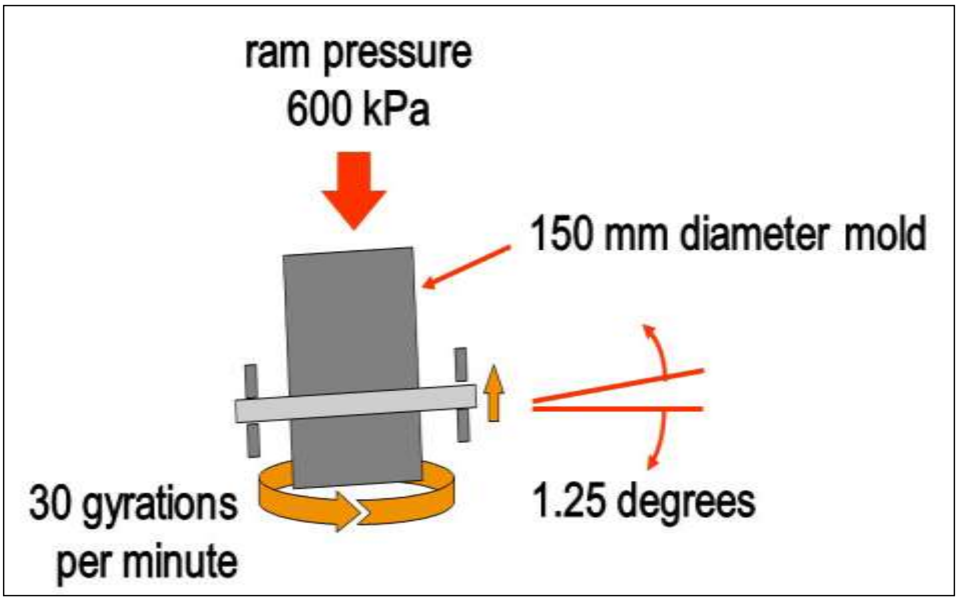

Figure (A-1) : Superpave gyratory compactor.

Table (A-1) : Superpave design gyratory compactive effort.

Design ESALs (millions)	Compaction Parameters				
	N initial	N design	N max.	Max. % $G_{mm}@N_{ini}$.	Max. % $G_{mm}@N_{max}$
DENCE GRADED					
< 0.3	6	50	75	91.0	98.0
0.3 to < 3	7	75	115	90.5	98.0
3 to < 10	8	100	160	89.0	98.0
≥ 30	9	125	205	89.0	98.0
OPEN GRADED					
All ESALs	NA	20	NA	NA	NA
SMA					
All ESALs	NA	100	NA	NA	NA

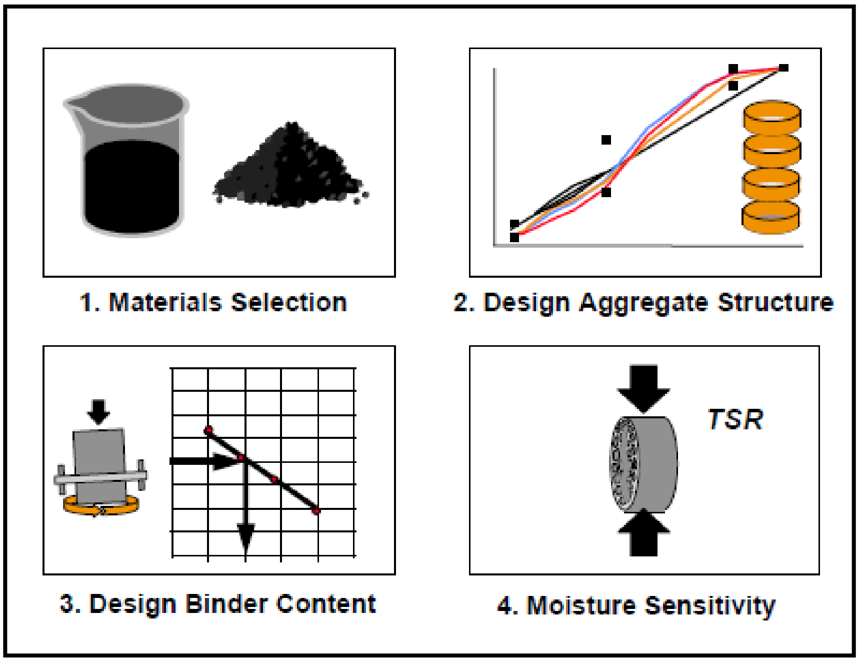

Figure (A-2) : Steps of Superpave mix design

STEP1: Selection Of Materials

Table (A-2) : Superpave Coarse Aggregate Angularity Requirements.

Traffic, million ESALs	Percent, Minimum Depth from Surface	
	< 100 mm	> 100 mm
< 0.3	55/-	-/-
0.3 to < 3	75/-	50/-
3 to < 10	85/80	60/-
10 to < 30	95/90	80/75
≥ 30	100/100	100/100
Note: "85/80" means that 85 % of the coarse aggregate has one fractured face and 80 % has two fractured faces.		

Table (A-3) : Superpave fine aggregate angularity requirements.

Traffic, million ESALs	Percent, Minimum Depth from Surface	
	< 100 mm	> 100 mm
< 0.3	-	-
0.3 to < 3	40	40
3 to < 10	45	40
10 to < 30	45	40
≥ 30	45	45
Note: Criteria are presented as percent air voids in loosely compacted fine aggregate.		

Table (A-4) : Superpave flat, elongated particle requirements.

Traffic, million ESALs	Percent, Minimum
< 0.3	-
0.3 to < 3	10
3 to < 10	10
10 to < 30	10
≥ 30	10
Note: Criteria are presented as maximum percent by weight of flat and elongated particles.	

Table (A-5) : Superpave clay content requirements.

Traffic, million ESALs	Sand Equivalent, Minimum
< 0.3	40
0.3 to < 3	40
3 to < 10	45
10 to < 30	45
≥ 30	50

STEP2: Select Design Aggregate Structure

Table (A-6) : Gradation criteria for 12.5mm nominal mixture.

Gradation Control Item	Sieve Size, mm	Minimum, %	Maximum, %
Control Points	19	100.0	100.0
	12.5	90.0	100.0
	9.5	-	90
	2.36	28	58
	0.075	2	10

Table (A-7) : Superpave mixtures designation criteria.

Superpave Mixture Designation	Nominal Maximum Size, mm	Maximum Size, mm
37.5 mm	37.5	50
25 mm	25	37.5
19 mm	19	25
12.5 mm	12.5	19
9.5 mm	9.5	12.5

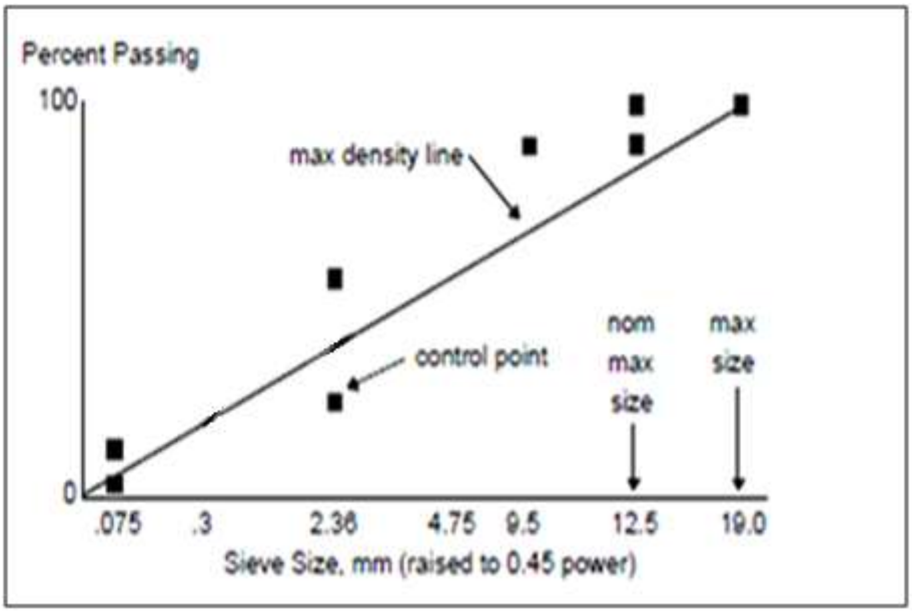

Figure (A-3) : 0.45 Power gradation chart.

Table (A-8) : Estimated aggregate blend properties.

Property	Criteria
Coarse Ang.	95%/90% min.
Fine Ang.	45% min.
Thin/Elongated	10% max.
Sand Equivalent	45 min.
Combined G_{sb}	n/a
Combined G_{sa}	n/a

• Dust Proportion: Requirement applied to all mixtures

$$0.6 \leq \frac{\%\ \text{weight of } 0.075 \text{ material}}{\%\ \text{weight of effective asphalt}} \leq 1.2$$

STEP3: Select Trial Asphalt Binder Content

Table (A-9) : Superpave VMA requirements.

Nominal Maximum Aggregate Size	Minimum VMA, %
9.5 mm	15.0
12.5 mm	14.0
19 mm	13.0
25 mm	12.0
37.5 mm	11.0

Table (A-10) : Superpave VFA requirements.

Design ESALs (million)	Design VFA, %
< 0.3	70 -80
0.3 to < 3	65 -78
3 to < 10	65 –75
10 to < 30	65 –75
≥ 30	65 –75
- For 9.5-mm nominal size mixtures, the VFA shall be 73% to 76% for design traffic levels > 3 million ESALs. - For 25-mm mixtures, the VFA lower limit shall be 67% for < 0.3 million ESALS. - For 37.5-mm mixtures, the VFA lower limit shall be 64% for all design traffic levels.	

STEP4: Moisture Sensitivity Evaluation of the Mixture

According to (AASHTO T283) the:

• Samples produced at design binder content and 7% air voids

• Three samples are conditioned by vacuum-saturation and freezing and thawing, another three that are not

• Tensile strength ratio (TSR) is: a ratio of average tensile strength of conditioned samples to unconditioned samples

• Minimum criterion is a ratio of 70- 80%.

Table (A-11) : Moisture sensitivity evaluation of The mixture.

Test Parameter	Test Requirement
Short-Term Aging	Loose mix*: 16 hrs at 60° C Compacted mix: 72-96 hrs at 25° C
Air Voids Compacted Specimens	6 to 8 %
Sample Grouping	Average air voids of two subsets should be equal
Saturation	55 to 80 %
Swell Determination	None
Freeze	Minimum 16 hrs at -18° C (optional)
Hot Water Soak	24 hrs at 60° C
Strength Property	Indirect tensile strength
Loading Rate	51 mm/min at 25° C
Precision Statement	None

* Short-term aging protocol of AASHTO T 283 does not match short-term aging protocol of Superpave. Suggest using T283 procedure of 16 hours at 60° C.

APPENDIX B

Superpave Mix Design

B1- Step one

1- Gradation of aggregate

Select weight of binder for each gradation by use roll

$$G_{se} = G_{sb} + 0.8(G_{sa} - G_{sb}) --- (\text{Asphalt Institute SP} - 2)$$

$$G_{se} = effective\ specific\ gravity\ of\ the\ aggregate\ blend.$$
$$G_{sb} = bulk\ specific\ gravity\ of\ the\ aggregate\ blend.$$
$$G_{sa} = apparent\ specific\ gravity\ of\ the\ aggregate\ blend.$$

$$G_{bulk} or G_{app} = \frac{1}{\frac{P_1}{100G_1} + \frac{P_2}{100G_2} + \frac{P_2}{100G_2}}$$

$$(See\ Appendix\ XI\ ASTM\ C127\ or\ C128)$$

where:

G = average specific gravity. All forms of expression of specific gravity can be averaged in this manner.

G_1, G_2 ... G_n = appropriate specific gravity values for each size fraction depending on the type of specific gravity being averaged.

P_1, P_2, ... P_n = weight percentages of each size fraction present in the original sample.

$$\mathbf{A}V_{be} = 0.176 - 0.0675Log(s_0)$$

$$W_s = \frac{P_s * (1 - V_a)}{(\frac{P_b}{G_b} + \frac{P_s}{G_{se}})}$$

$$V_{ba} = \frac{p_s*(1-V_a)}{(\frac{P_b}{G_b}+\frac{P_s}{G_{se}})} * \left(\frac{1}{G_{sb}} - \frac{1}{G_{se}}\right)$$

$$P_{bi} = \frac{100 * G_b * (V_{be} + V_{ba})}{(G_b * (V_{be} + V_{ba})) + W_s}$$

$s_o = the\ nominal\ maximum\ sieve\ size\ of\ agg.$

$V_{ba} = volume\ of\ absorbed\ asphalt\ binder\ \frac{cm^3}{cm^3} in\ mix$

$P_b = percent\ of\ binder$

$P_s = percent\ of\ aggregate$

$G_b\ = specific\ gravity\ of\ binder$

$P_{bi} = percent\ of\ binder\ by\ mass\ of\ mix$

$W_s = mass\ of\ aggregate\ , gram$

$$G_{mb} = \frac{sample\ mass\ in\ air\ (dry)}{sample\ mass\ wet\ surface\ dry\ (S.S.D) - sample\ weight\ in\ water\ (wet)}$$

$G_{mb} = bulk\ specific\ gravity\ of\ mix, all\ mass\ taked\ by\ gram.$

$$C = \frac{G_{mb(measure)}}{G_{mb(estimation)}}$$

$G_{mb(measure)} = measure\ bulk\ specific\ gravity\ after\ N_{max}$

$G_{mb(estimation)} =\ estimated\ bulk\ specific\ gravity\ at\ N_{max}$

$C = correction\ factor$

$$G_{mb(correction)} = C * G_{mb(estimated)}$$

$G_{mb(correction)}$
$= corrected\ bulk\ specific\ gravity\ of\ the\ specimen\ at\ any\ N_{max}$

$G_{mb(estimated)} =$
$= estmated\ bulk\ specific\ gravity\ of\ the\ specimen\ at\ any\ N_{max}$

$$\%G_{mm}@\mathrm{N} = \frac{G_{mb(correction)}}{G_{mm(measure)}} * 100\%$$

$\%G_{mm} = maximum\ theoretical\ specific\ gravity\ @N_{desgin} percent$

$G_{mm(measure)} = maximum\ theoretical\ specific\ gravity$

The height of specimens for each blend in (mm)

$$\%Va = 100 - \%G_{mm}@N_{desgin}$$

$$\%VMA = 100 - \frac{\%G_{mm}@N_{desgin} * G_{mm} * P_s}{G_{sb}}$$

$$\%VMA = void\ in\ mineral\ aggregate\ , percent\ of\ bulk\ volume$$

Check if air void = 4% ok else

$$P_{b,estmated} = P_{bi} - (0.4 * (4 - V_a))$$

$$C = constant = 0.1\ if\ V_a\ is\ less\ then\ 4\%$$

$$= 0.2\ if\ V_a\ is\ greater\ then\ 4\%$$

$$\%VMA_{estmited} = \%VMA_{initial} + C * (4 - V_a)$$

$$\%VMA_{initial} = \%VMA\ from\ trial\ asphalt\ binder\ content$$

$$\%VFA = 100 * \frac{\%VMA_{estimated} - 4.0}{\%VMA_{estimated}}$$

$$\%VFA = percent\ of\ voids\ filled\ with\ asphalt.$$

$$\%G_{mm\ estimated}@N_{ini} = \%G_{mm\ Trial}@N_{ini} - (4.0 - V_a)$$

$$\%G_{mm\ estimated}@N_{max} = \%G_{mm\ Trial}@N_{max} - (4.0 - V_a)$$

$$\%G_{mm\ estimated}@N_{ini}\ and\ \%G_{mm\ estimated}@N_{max}$$
$$= the\ maximum\ allowable\ mixture\ density\ atNumber\ cercal$$

$$Initial\ and\ Maximum$$

$$P_{be} = -(P_s * G_b) * \frac{G_{se} - G_{sb}}{G_{se} * G_{sb}} + P_{b,estimated}$$

$$P_{be} = effective\ asphalt\ content\ , percent\ by\ total\ mass\ of\ mixture\ .$$

$$0.6 < DP = \frac{P_{0.075}}{P_{be}} < 1.2$$

$$DP = dust\ proportion$$

$$P_{0.075} = aggregate\ content\ passing\ the\ 0.075mm\ sieve\ percent.$$

المستخلص

بينما تزداد عمليات التوسع في انشاء واعادة تأهيل الطرق، تزداد كلف مواد التبليط، وهناك نقصان في النوعية الجيدة من مصادر هذه المواد. ان عملية اعادة التدوير هي واحدة من الحلول المهمة لهذه لمشكلة، ان هذه العملية تقدم طبقة تبليط مستدامة عن طريق استخدام المواد القديمة التي يتم قشطها من هذه الطبقة، هذه المواد من الممكن مزجها مع المواد الجديدة ومعاملات اعادة التدوير لانتاج خلطات الخرسانة الاسفلتية المعاد تدويرها.

ان الاهداف الرئيسية لهذه الدراسة هي تقييم مستوى الاداء للخرسانة الاسفلتية المعاد تدويرها من خلال دراسة تاثير مختلف النسب من هذه المواد القديمة على اداء الخلطه الاسفلتية، دراسة تاثير مختلف المتغيرات للخلطة الاسفلتية الكونكريتية على الثباتية، الثباتية المتبقية، مقاومة الانضغاط، معامل المرونة وحساسية الرطوبة. ولتحقيق هذه الأهداف وإجراء الجزء العملي من هذه الدراسة؛ تم استخدام المواد المحلية المتاحة التي تتضمن نوعين من الأسفلت ذو درجة الاختراق (٤٠ -٥٠) و (٦٠-٧٠) ، والركام ذو المقاس الاسمي الاقصى ١٢,٥ ملم واستخدام مادة الغبرة كمادة مالئة، بينما المواد القديمة التي تم قشطها من التبليط فانها استخدمت باربعة نسب مختلفة من القشط (٧%، ١٣%، ١٩%، ٢٥%) ويتم تهيئتها وفقا للمواصفات القياسية، وتعديلها حسب المحددات للمواد المعاد تدويرها.

تم اعتماد طريقة التصميم فائق الاداء (السوبربيف) وقد تم استخدام جهاز الرص الدوراني (SGC) لضغط (٢٠) نموذج من الخلطة الاسفلتية. وشملت أعمال المختبر تصنيع قالب مختبري (SGC) بقطر (١٠٠) ملم لضغط النماذج، حيث تم استخدمه لتهيئة النماذج للاختبارات الميكانيكية الاخرى . تم فحص الخلطة الاصلية والمضاف اليها نسب من المواد المعاد تدويرها ومقارنتها مع الخلطة الاصلية من خلال عدة فحوص وهي : خصائص مارشال، فحص مقاومة الشد غير المباشر ، فحص مقاومة الانضغاط، فحص الديمومة ومعامل المرونة للخلطه الاسفلتية باستخدام جهاز الموجات الفوق الصوتية; ان منهجية العمل شملت نسب مختلفة من المحتوى الاسفلتي، مختلف درجات الحرارة بالاضافة الى فترات الاغمار (١، ٣، ٧) يوم.

اظهرت النتائج ان الخلطات مع المواد القديمه المستحصلة من التبليط اظهرت اداء افضل من الخلطات التي لا تحتوي على المواد القديمة المستحصلة; وقد وجد ان الخلطات المعاد تدويرها بنسبه ٢٥% من المواد القديمة لها زيادة في خصائص مارشال، قوة الشد عند ٢٥ درجه مئوية ، نسبه مقاومة الشد غير المباشر ، مقاومة الانضغاط وفحص الموجات فوق الصوتية وعند نسبة رص ١٠٠

دورة بنسبة :٣٤,٤٧%، ٩,٣٥%، ٨,٤٢%، ٣٢,٧٥% و٦,٧٤% عند نسبة الاسفلت المثلى على التوالي. ولكن هذه النتائج هي أقل تقريبا عندما يتم زيادة محتوى الأسفلت الأمثل بنسبة ٠,٥%.

استنادا الى الفحوصات المختبرية; تم تحليل النتائج وتطوير الموديلات الاحصائيه للثباتية، الثباتية المتبقية ومحتوى الرطوبة باستخدام البرنامج الاحصائي (SPSS) الاصدار (٢٢). تحليل النتائج، حساب نسبة الخطأ المعياري ومعامل الارتباط اظهرت علاقات جيدة بمعامل ارتباط جيد (R^2) وكان بنسبة ٩٨،٦، ٩٤,٣ و ٩٦,٦ بالمائة على التوالي.

وأخيرا، يستخدم برنامج دليل التصميم الميكانيكي-التجريبي (MEPDG) الإصدار ٦,٣ للتنبؤ بأداء هيكل التبليط المرن مع المواد القديمه المضافة للخلطة الاسفلتية الحارة للطبقة السطيحة. مختلف عمليات التصميم قد اجريت باستخدام بعض مستويات التسلسل الهرمي للتحليل في برنامج (MEPDG). التصاميم قد اجريت مع تغيير في خصائص HMA (محتوى المواد المعاد تدويرها، نوع الاسفلت، كمية الاسفلت والفراغات الهوائية) للطبقة السطحية. معامل المرونة للخلطات الاسفلتية تم احتسابه مختلف الخلطات; وقد وجد ان اضافه ٧% من المواد القديمة يزيد من معامل المرونة بحوالي ٧,١% لنوع الاسفلت ذو درجة اختراق (٤٠-٥٠) على مدى ٢٠ عاما عند التحليل باستخدام برنامج التصميم الميكانيكي-التجريبي MEPDG.

وزارة التعليم العالي والبحث العلمي

الجامعة التكنولوجيــة

قسم هندسة البناء والإنشاءات

فرع هندسة الطرق والجسور

تصميم الخلطات الاسفلتية الحارة المعاد تدويرها بأستخدام الاسلوب الميكانيكي-التجريبي

رسالة

مقدمة الى قسم هندسة البناء والانشاءات في الجامعة التكنولوجية وهي جزء من متطلبات الحصول على درجة الماجستير علوم في هندسة الطرق والنقل

من قبل

ياسين عطا زهير

بكلوريوس هندسة بناء وانشاءات ٢٠١٥

باشراف

أ. د. محمد يوسف فتاح | **أ. م. د. زينب ابراهيم قاسم**

محرم 1439 | تشرين الاول 2017