JIMMA UNIVERSITY

JIMMA INSTITUTE OF TECHNOLOGY

SCHOOL OF CIVIL AND ENVRONMENTAL ENGINEERING

CIVIL ENGINEERING DEPARTMENT

HIGHWAY ENGINEERING STREAM

SUITABILITY OF BLENDED RECYCLED ASPHALT AGGREGATE WITH FRESH AGGREGATE AS A BASE COURSE MATERIAL

By: EZERA JIHAD

A Final Thesis Submitted to School of Post Graduate Studies of Jimma University, Jimma Institute of Technology in Partial Fulfilment of the Requirement for Master of science Degree in highway Engineering

June, 2017

Jimma ,Ethiopia

JIMMA UNIVERSITY

JIMMA INSTITUTE OF TECHNOLOGY

SCHOOL OF CIVIL AND ENVRONMENTAL

ENGINEERING

CIVIL ENGINEERING DEPARTMENT

HIGHWAY ENGINEERING STREAM

SUITABILITY OF BLENDED RECYCLED ASPHALT AGGREGATE WITH FRESH AGGREGATE AS A BASE COURSE MATERIAL

By: EZERA JIHAD

A Final Thesis Submitted to School of Post Graduate Studies of Jimma University, Jimma Institute of Technology in Partial Fulfilment of the Requirement for Master of science Degree in highway Engineering

Advisor Ass.pro Elmer C.Agon

Coadvisor Eng.Jemal Jibril

June, 2017

Jimma ,Ethiopia

JIMMA UNIVERSITY

POST GRADUATE STUDIES

JIMMA INSTITUTE OF TECHNOLOGY

SCHOOL OF CIVIL AND ENVIRONMENTAL ENGINEERING

HIGHWAY ENGINEERING STREAM

MSc. Thesis on:

SUITABILITY OF BLENDED RECYCLED ASPHALT AGGREGATE WITH FRESH AGGREGATE AS A BASE COURSE AGGREGATE

BY

EZERA JIHAD

Approved by board of examiners:

External Examiner	Signature	Date
…………………………	…………………	………………
Internal Examiner	Signature	Date
…………………………	………………….	………………
Chairman	Signature	Date
…………………………	………………….	………………
Advisor	Signature	Date
…………………………	………………….	………………
Co. Advisor	Signature	Date
…………………………	…………………..	………………

Acknowledgement

I am indebted to a number of personalities without whom my thesis would not have been a succeeded. First and foremost Almighty God for the abundant grace and care.

Secondly, my advisor (ASS.Professor) Elmer C.Agon, and Eng. Jemal for their expert guidance and assistance during the progress of this research work.

Thirdly, to the academic and technical staffs at the department of Civil and Environmental Engineering for their skilful and kind help.

Of course, this work would not be possible without the unconditional support provided by my family and friends.

ABSTRACT

Globally, large amount of construction waste is produced each year. Recycled materials offer viable solutions to the concern, which is beneficial to both environment and economy. Recycled Asphalt Aggregate is one of the most commonly used recycled materials. Road construction has become very expensive due to the increase costs on raw materials. Aggregate is one of the main ingredient in asphalt (>95%) and Recent statistics showed the increasing demand of construction aggregate to reach 2.6 million metric tons by the year 2013 in Ethiopia. The production of aggregate materials increased by 31% compare to the year 2012. A study is needed to evaluate the suitability of high percentage (greater than 20%) recycled asphalt aggregate as base course material, without compromising the pavement performance.

The main objective of this thesis was to evaluate the suitability of Blended recycled asphalt aggregate with fresh aggregate as a base course material concerning physical and mechanical properties of both fresh and recycled aggregates compare to ERA specification. The thesis was designed to be an experimental study and Recycled and fresh aggregate samples were collected from ERCC, Jimma town. Laboratory tests were performed on fresh aggregate and recycled aggregate samples at a mixture of 0%, 20%, 50% and 100% recycled aggregate by weight of fresh aggregate sample. The tests were Gradation, Flakiness index, Aggregate crushing value, Aggregate impact value, Los Angeles Abrasion value, Compaction and California Bearing Ratio. Then, the results of laboratory experiments were compared to standards to identify the suitability of recycled asphalt aggregate as a base course material by systematic analysis method.

The results from this study have shown that the addition of 0 %(Fresh aggregate), 20% and 50% RAA gave the technically qualified results on crushing, impact and abrasion value. The blend gave a ACV of 14.5%, 19.72% and 23.5% (maximum required 25%), AIV of 13%, 17.24% and 22% (Maximum required 24%), LAA value of 21%, 28% and 35%(maximum required 40%) respectively. But in the CBR test only 0% and 20% RAA mix by weight of fresh aggregate gave technically qualified results that is 107% and 92%(minimum required 80%) respectively. The test results by regression analysis to determine the maximum percent or optimum ratio of recycled asphalt aggregate used in base course regards to ERA specifications on crushing ,impact , abrasion and CBR values are 61%,62% ,58% and 40% respectively. This test results confirm that recycled aggregate is a good substitute for fresh aggregate with a limited ratio of 40% by weight of fresh aggregate for a base course material without compromising the pavement performance.

Based on the findings in this thesis concluded that Recycled Asphalt Aggregate not waste products and contribute in conserving non-renewable natural resources. As fresh aggregate become more limited and prices rise, the use of recycled aggregate in pavement construction is definitely an eco-friendly alternative.

Key words:-recycled asphalt aggregate, fresh aggregate, base course material, resource, pavement performance

Table of Content

List of Table

List of Figure

ACRONYM

AASHTO	American Association of State Highway and Transportation Officials
ACV	Aggregate Crushing Value
AIV	Aggregate Impact Value
ASTM	American Standard for Test Method
BS	British Standard
CBR	California Bearing Ratio
CDOT	Colorado Department of Transportation
DCP	Dynamic Cone Penetrometer
DGABC	Dense Graded Aggregate Base Course
DOT	Department of Transportation
ERA	Ethiopian Road Authority
ETB	Ethiopian Birr
EWCC	Ethiopian Works Construction Corporation
FDOT	Florida Department of Transportation
FHWA	Federal Highway Administration
FI	Flakiness Index
HMA	Hot Mix Asphalt
kPa	Kilo Pascal
LAA	Los Angeles Abrasion
MDT	Montana Department of Transportation
MDUW	Maximum Dry Unit Weight
MNDOT	Minnesota Department of Transportation
M/S	Meter per Second
NJDOT	New Jersey Department of Transportation

OMC	Optimum Moisture Content
RAA	Recycled Asphalt Aggregate
RCA	Recycled Concrete Aggregate
UDOT	Utah Department of Transportation
U.S	United States
WSDOT	Washington State Department of Transportation

CHAPTER ONE
INTRODUCTION

1.1. Background

Road infrastructure is the backbone to the development and it has of a bearing on sustainability of growth of any country. It supports the growth in agriculture and industry, open corridors, port links and tourism areas, and connect each region to the rest of the country. Ethiopia has been struggling to alleviate poverty and ensure sustainable economic development in the past twenty-five years of the multifaceted development sectors; road infrastructure is one among many other projects.

The Government of Ethiopia has been making a major expansion of the road network recognizant of its importance to meet its development goals of the country. The Ethiopian government has considered and has been carrying out enhancing road infrastructure as its vital instrument for sustainable development and poverty reduction, there are many materials recycled and virgin, b/c it was been use in the construction and rehabilitation of roadway.

Construction and maintenance of roads is not an easy investment. It requires a huge amount of expenditure with foreign currency. This huge amount of money is not affordable only by the government of Ethiopia from internal sources. Hence, the government of Ethiopia finance road construction and maintenance through loans and aid from other countries and organizations in addition to domestic finance .in recent periods, the lion share of finance is the government of Ethiopia followed by international development association. The other following finances are European Union and African development bank.

Recycled asphalt aggregate that has been properly processed and in most cases blended with conventional aggregates has demonstrated satisfactory performance as granular road base for more than 20 years and was now considered standard practice in many countries, but in Ethiopia recycled asphalt aggregate used as a capping layer and sub base material in many projects.

1.2 Statement of the Problem

Globally, large amount of construction waste produced each year. Recycled materials offer viable solutions to the concern, which is beneficial to both environment and economy. Recycled Asphalt Aggregate is one of the most commonly used recycled materials. Recycled Asphalt Aggregate is the term given to removed and/or reprocessed pavement materials containing asphalt and aggregates. RAA was generated when asphalt pavements are removed for reconstruction or resurfacing. An increased percentage of RAA in base course could offer economic and environmental benefits (Uhlmeyer, 2008).

The rapid economic growth in Ethiopia from 2004 to 2015(10.9%) and gauges the prospects for the future. In recent years, Ethiopia has dedicated three percent of GDP to road investments and investment program focuses mainly on rehabilitation, upgrading, and widening of the road (African Economic Outlook, 2015).

Road construction has become very expensive due to the increase costs on raw materials. Aggregate is one of the main ingredient in asphalt (>95 %). Demand for aggregate is high and will only increase in the future as cities grow and demand for infrastructure increases. Recent statistics showed the increasing demand of construction aggregate to reach 2.6 million metric tons by the year 2013 in Ethiopia. The production of aggregate materials increased by 31% compare to the year 2012(Araya, 2013). Fresh Aggregate is expensive; hence, the use of recycled asphalt aggregate, which is locally available and cheap, can either be used as partial replacement of fresh aggregate.

A study is needed to evaluate the suitability of high percentage (greater than 20%) recycled asphalt aggregate as base course material, without compromising the pavement performance. A successful application of high percentage (greater than 20%) RAA could contribute to the sustainability, in terms of costs, resource, and environmental hazard (McGarrah, 2007)

1.3 Research Questions

1. What are the engineering properties of recycled asphalt aggregate?
2. What are the engineering properties of blended recycled asphalt aggregate and fresh aggregate?
3. How much is the optimum ratio of recycled asphalt aggregate when blended with fresh aggregate as suitable for base course material?

1.4 Objective of the Study

1.4.1 General Objective

The general objective of the study is to evaluate the suitability of blended recycled asphalt aggregate with fresh aggregate as a base course material.

1.4.2 Specific Objectives

1. To identify the Engineering properties of recycled asphalt aggregate.
2. To determine Engineering properties of mixed recycled and fresh aggregates.
3. To quantify the optimum ratio of recycled asphalt aggregate with fresh aggregate as suitable to base course aggregate about engineering properties.

1.5 Significance of the Research

In recent years the road construction has become very expensive one of the reason are shortage of aggregate production. Recycled asphalt aggregate is readily and sufficiently available at asphalt pavements are removed for reconstruction or resurfacing. Fresh Aggregate is expensive; hence, the use of recycled asphalt aggregate, which is locally available and cheap, can be used as partial replacement of fresh aggregate. Therefore using it for base course to reduce the cost of road construction, save non-renewable natural resources and reducing the environmental hazards. Therefore, the results of this study will provide information to all stalk holders and consumers about the usage of recycled asphalt aggregate in various projects. This study will be surveying as a springboard for further studies in relation to this study.

1.6 Scope and Limitation of Study

This study was conducted to investigate the suitability of blended recycled asphalt aggregate and fresh aggregate in base course material with laboratory investigation. The tests used to determine suitability of blended recycled asphalt aggregate with fresh aggregate were gradation, flakiness index, aggregate crushing value, aggregate impact value, Los Angeles abrasion test, Compaction and California Bearing Ratio. The field application is out of the scope of this work and the study is limited to Jimma Town.

CHAPTER TWO
LITERATURE REVIEW

2.1 Overview

The principal factors behind recycling efforts include reduction of construction waste, preservation of non-renewable natural resources, and lower energy costs. Typically, economic savings and environmental benefits of using recycled materials are balanced by the performance requirements of pavement design. It is commonly acknowledged that the use of recycled construction materials to the maximum extent possible should be carried out in the overall context of maintaining a cost-effective, high quality, well performing, and environmentally sound pavement infrastructure (Copeland, 2011).

.

More than 90% of U.S. roads are constructed with HMA, and as the pavement infrastructure ages, there is a growing need to maintain and rehabilitate these roads. In principle, the same materials used to build the original highway system can be re-used to repair, reconstruct, and maintain it. Integral to pavement maintenance is the milling of the aged asphalt pavement during resurfacing, rehabilitation, and reconstruction operations.

In an NCHRP Synthesis of Highway Practice, noted that highway agencies have been proactive in the recycling of reclaimed and by product materials into construction materials, with RAA being the material most frequently used. In addition to its use in asphalt mixtures, they identified unbound base and sub base as “proven” applications for RAA, with grading identified as the limiting factor for use. Although 49 states indicated they used RAA, the primary use was in asphalt concrete. Thirteen states, including Virginia, indicated RAA use in base materials; four states used RAA in sub base material, and RAA was used in stabilized base and shoulder aggregate, each in two states.

Overall, the performance of granular base and sub base layers containing RAA material has been characterized as satisfactory to excellent (Collins and Ciesielski, 1994).

The majority of the eight studies discussed in the literature review were conducted for state DOTs from 1993 to 2005. The Taha (1999) and Trzebiatowski (2005) studies were conducted for the Sultanate of Oman and for the State of Wisconsin’s Solid Waste Research Program, respectively. All eight studies tested both similar and different types of engineering properties. In addition, the studies analysed different types of RAA, at different blends, with different types of virgin aggregate. As a result, no one study can be compared directly with another, but by comparing similar tests from all eight studies, some general trends appear. Table one list the general trends for five engineering properties from the eight studies. If the study did not blend virgin aggregate with RAA, the table compares 100% RAA to 100% virgin aggregate. Cooley (2005) investigated RAA for use in full-depth-reclamation rehabilitation methods. Although this method is not utilized by WSDOT, results are still relevant. Cooley (2005) conducted material classification and compaction tests, and evaluated the strength, stiffness and moisture susceptibility on two sources of RAA blended with two types of virgin aggregate at RAA contents of 0, 25, 50, 75 and 100%. Details whether the RAA material was blended with virgin aggregate the effect on the dry density of the material as the percent RAA was increased, effect on the optimum moisture content as the percent it was increased, effect on the permeability as the percent RAA was increased, effect on the California Bearing Ratio (CBR) as the percent RAA was increased, effect on the resilient modulus as the percent RAA was increased.

As seen in Table 1, three general conclusions can be made in regards to the use of RAA as a base course material: (1) dry density decreases as the percentage of RAA increases in the blend; (2) CBR Value decreases as the percentage of RAA increases; and (3) resilient modulus increases as the percentage of RAA increases.

Table 2.1 Literature Review Findings

REPORT	Blended	Dry Density	Moisture Content	Permeability	CBR	Resilient Modulus
Cooley (2005)	Yes	Decreased	Decreased	-	Decreased	
Garg& Thompson (1996)	No	Decreased	Decreased	-	Decreased	
MacGregor (1999)	Yes	Decreased	-	No Change	-	Increased
Bennert& Maher (2005)	Yes	Decreased	Decreased	Decreased	-	Increased
Papp (1998)	Yes	Decreased	Decreased	-	-	Increased
Sayed (1993)	No	Decreased	Decreased	-	Decreased	-
Taha (1999)	Yes	Decreased	No Change	Increased	Decreased	-
Trzebiatowsk (2005)	No	Decreased	-	Increased	-	-

2.2 Current Use of RAA as Base Course

The use of RAA as a base course material offers economic and environmental benefits. The WSDOT currently allows up to 20 percent RAA to be blended with virgin crushed Aggregates to form the base course materials.McGarrah(2007) Conducted a survey of current practices of State DOTs regarding the use of RAA as base course material and contacted 7 states including Colorado, Florida, Illinois, Minnesota, Montana, New Jersey and Utah .The Result for the survey is listed in Table 2.2

Table 2.2 State DOTs Survey Result [McGarrah 2007]

STATE	RAA ALOWED	MAX %	PROCESSED
Florida	No	-	-
Illinois	No	-	-
Montana	Yes	50-60%	No
New Jersey	Yes	50%	Yes – Gradation
Minnesota	Yes	3%	Yes – Gradation
Colorado	Yes	50%	Yes – Max Aggregate Size
Utah	Yes	2%	Yes – Gradation
Texas	Yes	20%	Unknown
California	Yes	50%	Unknown
New Mexico	Yes	Unknown	Unknown
Rhode Island	Yes	Unknown	Yes – Gradation
South Dakota	No	-	-

As shown in the table, the maximum percentage of RAA as base course material allowed by state DOTs vary from 0 percent to 60 percent based on the data collected from the survey. For the state of Montana, whether RAA may be used as base course material is decided on a project-by-project basis instead of being stated in the standard specifications, and the maximum percentage of RAA used as base course material may reach 60%. The maximum percentage of it used as base course was selected on the basis of the research conducted by Mokwa(2005), which proved that the blending of RAA with virgin aggregate only caused minor changes to the engineering properties of the mixed base course material.

2.2.1 Use of RAA in Montana State

Montana Department of Transportation (MDT) materials engineer, Matt Strizich, provided the following information. MDT does not state in their standard specifications whether RAA may be used as a base course material. Rather, they decide on a project-by-project basis whether to use RAA in the base course material and write project-specific specifications in the contract documents. Moreover, MDT blends the RAA with the virgin aggregate in a different manner than is typically seen in other DOTs. Due to vertical and horizontal alignment problems on state highways, MDT reconstructs more roadways than other states. Instead of blending previously milled asphalt with virgin aggregates, MDT mills the top few inches of the asphalt surface, hauls this material away, pulverizes the remaining asphalt and underlying base course and reuses this material as base course. For example, if the existing asphalt depth was 7-inches and MDT specified that 50% RAP would be allowed as base course and 4-inches of base course would need to be removed, the first 3-inches of the asphalt pavement would be milled and hauled off. The remaining 4-inches of asphalt and 4-inches of base would be pulverized, blended and used as base course material for the project. From previous experience, MDT believes the underlying base material is suitable to be reused.

Field compaction testing is followed in the same manner as is for virgin aggregate base course, although some subtle differences exist. Due to the hydrocarbons in the existing asphalt, MDT does not believe the nuclear gauges are accurate in determining density; therefore, a correction factor is added. The correction factor is determined by first testing the density and moisture of the blended material with the nuclear gauge. The material is then sampled and the actual moisture is determined. Actual moistures are used to determine the correction factor which is applied to the recorded density from the gauge.

These maximum percentages were established from the results of a study conducted by Mokwa (2005). Mokwa conducted laboratory tests on four different types of virgin aggregates blended with varying percentages of RAA (20, 50 and 75%). The laboratory tests consisted of grain size analyses, specific gravity, modified Proctor compaction, relative density, Los Angeles abrasion, large-sample direct shear, R-value, permeability and X-ray computed tomography scans were conducted on the blends. Mokwa found that blending RAA with virgin aggregate results in only minor changes to the engineering properties of the virgin material.

2.2.2 Use of RAA in Florida State

The Florida Department of Transportation (FDOT) allows the use of RAA as a base course material but only in non-traffic applications such as paved shoulders and bike paths. The state's Geotechnical Material Engineer, David Horhota, also stated that no special provision exists within the state allowing the use of RAA as a base course material (Horhota Interview).

FDOT, however, has previously pursued the idea to use RAA as a base course material for roadway bases and subbases. In 2001, Florida Institute of Technology published a study that was conducted in order to develop specifications for using RAA as a base or subbase material (Consentino, 2001). A laboratory and field investigation was conducted to determine strength and deformation properties of RAA, effects of crushing unprocessed RAA, storage time at elevated temperatures and weather on the properties of the RAA. FDOT issued a *Summary of Final Report* in conjunction with the Florida Tech report (FDOT, 2001). A few of the results from the laboratory investigation are listed below:

1) Typical moisture density curves were not realized due to the fact that the dry density was relatively constant at moisture contents greater than 4%. Most of the curves were flat with no distinct peak.
2) Various compaction methods—Proctor, Marshall, vibratory and static—were utilized but none of the methods, except the static at 1,000 psi, met FDOT Lime rock Bearing Ratio (LBR) test specifications for base course material (LBR>100).
3) Increasing the temperature of the RAA has a significant effect on most of its triaxial properties.

The Lime rock Bearing Ratio (LBR) test—similar to the CBR test but correlated to lime rock that is predominantly used in Florida—is the most widely accepted FDOT specification for acceptance of a base course material (Consentino, 2001). As a result of the low LBR test values recorded in the laboratory investigation and during the warmer, summer months of the field investigation, FDOT did not adopt RAA as a possible base course material.

2.2.3 Use of RAA in New Jersey State

Currently, the state of New Jersey specifies that any percentage of RAA may be blended with virgin aggregate for base course material. According to the state materials engineer, Eileen Sheehy, however, this specification is being modified to allow a maximum of 50% RAA to be blended with virgin aggregate (Sheehy Interview) based on results from a Rutgers University study by Bennert and Maher (2005). This study will be discussed later.

RAA is utilized as a base course material by the New Jersey Department of Transportation (NJDOT) in the following manner. The standard specification employed by NJDOT allows up to 100% RAA to be used as base course material. The contractor for each project determines whether to incorporate RAA into the base course blend and at what percentage. Base course that consists of RAA shall meet the following requirements: (1) percent loss shall not exceed 50% when tested using the Los Angeles Machine; (2) RAA percentage shall be determined by the contractor and shall not vary by more than plus or minus 15% from the established value when measured at the source; and (3) the blend shall conform to the gradation listed in Table 4 (New Jersey DOT, 2001).

Although NJDOT does not require the RAA to be processed prior to blending, Sheehy said no contractor has been able to meet gradation requirements without processing. In some instances, the RAA only needed to be run over a scalping screen (Sheehy Interview). Sheehy further stated: (1) recycled concrete (RCA) is the preferred material to blend with virgin aggregate for base course material because RAA is more valuable when recycled for use in HMA and (2) only a few projects in New Jersey had used a RAA blend as a base course, but no failures have been noted for these projects.

Testing the in-place density of the blended material is performed according to AASHTO T191, T205 or T238, Method B and T239. If the nuclear gauge method is used, the procedure is slightly modified. The nuclear gauge is used to determine the in-place, wet density of the base course. Once the wet density is determined, a 1,000-gram sample is collected and taken to a laboratory where the percent moisture value is determined. Using the field-measured wet density and laboratory-determined moisture content, the dry density is calculated.

The study conducted by Bennert and Maher analyzed the permeability, triaxial shear strength, cyclic triaxial loading, California Bearing Ratio (CBR) and resilient modulus of base and sub base materials as well as RAA and RCA. The study found that permeability decreases as the percentage of RAA increases. This finding further illustrates the inconsistencies of a RAA-blended material between different reports. For the permeability tests—constant and falling head—RAA was blended with New Jersey's Central Region, naturally graded base (Dense Graded Aggregate Base Course - DGABC) and sub base material (I-3) at four different percentages. The results are listed in Table 5.

Table 2.3: Bennert and Maher (2005) Permeability Results

	Constant Head (ft/day)		Falling Head (ft/day)	
Blend Percentage	DGABC	I-3	DGABC	I-3
Natural (0% RAA)	172.7	55.8	121.05	43.2
25% RAA	121.4	2.2	27.8	2.4
50% RAA	113.7	8.3	39	7.7
75% RAA	1.7	3	2.1	3.3
100%RAA	16.9	16.9	13.9	13.9

As shown in Table 2.3, the difference between the permeability of the 100% natural aggregate and the 25% RAA – 75% natural aggregate blend is dramatic. The study conducted by Mokwa (2005), Taha (1999), Trzebiatowski (2005) and others, however, concluded just the opposite. A number of possible reasons exist for this discrepancy between the various studies. Bennert proposes the following possible reasons for those discrepancies (Bennert Interview): (1) level of compaction; (2) quality of virgin aggregate (hard, angular rock for the New Jersey study); (3) higher percent fines of virgin aggregate (10 to 11%); and (4) combining of a highly angular New Jersey DGABC to a more rounded, softer RAA source. The New Jersey study also noted that the RAA used for the various tests throughout the report was too fine to meet specifications for base course material; the percent passing the ¾" sieve was 100% and the specification requires a range between 55-90%. This further emphasizes the need for each state to conduct its own testing to determine how that state's aggregates and RAA specifications affect the properties of the blend.

Results from the CBR tests revealed that there was about a 50 and 55% decrease in

CBR values when the RAA blend was increased from 0 to 25% RAA and 50 to 75% RAA, respectively. Little change occurred in the CBR value when the RAA percentage was increased from 25 to 50%. In general, as the percentage of RAA increases, the CBR values decrease Bennert and Maher also conducted a triaxial test; the test was conducted only on a 100% RAA specimen. The specimen displayed similar results to the sub base material (I-3), but displayed lower shear strength than the base course material (DGABC). For the resilient modulus test, as the percentage of RAA increased, the resilient modulus increased.

Permanent deformation, however, increased as the percentage of RAA increased.

The 100% RAA mixture experienced the most deformation.

One of the trends that existed throughout the various tests was that a significant difference existed between a 0% RAA and 25% RAA mixture as well as the 50% RAA and 75% RAA mixture. But little difference existed in test results between 25% RAA and 50% RAA. As a result, if the 25% RAA blend is able to meet requirements for base course material, then it is likely that the 50% blend will also meet those requirements. This allows a public agency to set a higher maximum allowable RAA percentage.

2.2.4 Use of RAA in Minnesota

The Minnesota Department of Transportation allows both the asphalt and underlying base course to be reclaimed at the same time—as is done in Montana—as well as stockpile the reclaimed asphalt and mix it with virgin aggregate at a later time. The maximum bitumen content of the composite mixture—reclaimed asphalt and aggregate—shall not exceed 3% by weight. According to the State Grading and Base Engineer, Tim Andersen, this maximum percentage by weight corresponds to a volume percentage of 50-75% RAA due to low asphalt contents of previously placed HMA and degradation of the asphalt binder (Andersen Interview).

If a contractor chooses to use RAA as a base course material, the gradation of the RAA/aggregate blend must meet the gradation of the class of aggregate that was specified for the contract. However, the amount of RAP in the blend must exceed 10% (by volume) to be considered a recycled blend (Class 7). This means that a contractor may add a small percentage of RAA without changing the class of the aggregate. All the quality control specifications that apply to that class of virgin aggregate will apply to the blended mix if the RAA percentage does not exceed 10%.

Andersen said this
specification is currently being modified and will most likely be increased to allow up to 20-25% RAA to be added to a virgin class of aggregate without being considered a recycled blend. Minnesota requires the contractor to mechanically blend the RAA and virgin aggregate at the crushing site. They do not allow a stockpile of RAA and a virgin aggregate to be blended at the job site, with a grader. Andersen said that this type of blending—mechanically blending the two materials together at the crushing site—usually occurs in urban areas whereas the reclaiming of both the asphalt and underlying base course often happens in rural areas.

Minnesota does not use a nuclear gauge to determine the in-place density of the RAA blend. Instead, the state uses either a Quality Compaction method or the Dynamic Cone Penetrometer (DCP) method. The Quality Compaction method consists of visual inspection by the state inspector and is usually reserved for only small quantities. The DCP method is preferred. Andersen stated the permeability of the blended material decreases as RAA is increased. A typical base course aggregate has a permeability of 0.5 ft/day, but this value is reduced in half when RAA is added to the virgin aggregate.

2.2.5 Use of RAA in Colorado State

The Colorado Department of Transportation (CDOT) has lenient requirements with regards to the use of RAA as a base course material. Currently, CDOT allows up to 100% RAA. The State Materials/Geotechnical Engineer, Tim Aschenbrener, said this specification will likely be reduced to 50% due to recent findings (Aschenbrener Interview). Although RAA is not often used as a base course material in Colorado, recent studies have shown that the RAA blend has a high permeability. CDOT has no specific gradation for blended material, but a maximum aggregate size is specified to ensure larger pieces of reclaimed asphalt are not added to the blend. Aschenbrener was unsure of the exact maximum allowable size—either one or 2 inches—but he said that this requirement is usually met by running the RAA over a scalper and then re-crushing any of the larger pieces. In addition, RAA origin is not classified and a nuclear gauge is not used to test the in-place density. Rather, a roller compaction strip is completed to determine the roller pattern and visual observation is then used to ensure compaction is met during the construction process.

CDOT is also in the process of performing a material study on various properties of a RAA/aggregate blend. According to CDOT Research Engineer, Roberto de Dios, the study is similar to the one conducted by Bennert and Maher for New Jersey. Roberto de Dios said the study would be published at the end of 2007 or early 2008 (de Dios Interview).

2.2.6 Use of RAA in Utah State

The gradation that is used for a RAA base course material is the same as used for virgin aggregate, which is listed below in Table 6.

Table 2.4 : Utah Base Course Gradation

Gradation Limits – Single Value Job-Mix Formula

Sieve Size	**Percent Passing of Total Aggregate (Dry Weight)**		
	1-1/2 inch	1 inch	3/4 inch
1-1/2 inch	100	--	--
1 inch	--	100	--
3/4 inch	81 -91	--	100
1/2 inch	67 - 77	79 - 91	--
3/8 inch	--	--	78 – 92
No. 4	43 - 53	49 - 61	55 – 67
No. 16	23 - 29	27 - 35	28 – 38
No. 200	6 – 10	7 - 11	7 – 11

Utah requires the RAA/aggregate blend to be mechanically blended to ensure a homogenous material. Biel said that on a previous job the contractor placed the milled asphalt and dumped it onto base course material already placed. This process caused the base course to separate into lenses. A compaction curve is developed from a sample of the blended material and a nuclear gauge is used to measure field density.

However, if more RAA is used in the mix or nuclear gauge is producing results greater than the laboratory maximum density another method is used. A test section is rolled and the densities are taken. Once the base course densities reach a maximum and start decreasing, the maximum density can be determined. This process is conducted several times and an average Breakdown Curve Maximum Density is determined. The required construction density is usually 98% of the Breakdown Curve Maximum Density. Samples will also be taken to determine the moisture content to ensure proper watering if this testing procedure is used. Biel did not know of any previous areas where a RAA base course performed poorly and caused poor performance on the surface course and also said no changes regarding the use of RAA as a base course material would be made in the 2008 standard specifications.

2.2.7 Use of RAA in Illinois State

The state of Illinois does not allow RAA to be used as a base course material. Shelia Beshears, the state geotechnical engineer, stated that they sometimes use RAA as a sub base material but virgin aggregate is always placed overtop, prior to placing HMA or concrete (Beshears Interview). She also said that the main hurdle for using RAA as a base course material is the lack of quality control testing procedures for the material.

2.3 Past Studies on Other Engineering Properties of RAA

2.3.1 Moisture-density relationship

Cooley (2005) determined OMC and MDUW for samples containing different percentages of RAA using modified proctor compaction method. The results indicated that the increasing Percentage of RAA caused a decrease of OMC and MDUW. Attia (2009) found that RAA had a lower MDUW comparing to aggregate samples, based on results from both proctor Compaction tests and tests using gyratory compactor at 50 gyrations. For the 10gyratory compaction, increasing RAA decreased OMC whereas for standard proctor compaction, OMC increased with the increase of RAA percentage. Gupta (2009) conducted tests to determine the OMC and MDUW for samples containing different percentages of RAA using gyratory compactor at a compaction angle of 1.25 degrees, the compaction pressure of 600 KPa (87.02 psi), and 50 gyrations. It was concluded that increasing RAA increased MDUW but decreased OMC. Macgregor (1999) evaluated the relationship between OMC, MDUW and RAA content. The results indicated that no correlation was found between the RAA content and OMC or MDUW.

Malleable or brittle particles can lead to post-compaction deformations of the base or sub base layer if sufficient densification is not achieved during construction, which may be the cause of permanent strains sometimes reported when RAA was used as base material. Although construction methods for RAA are generally similar to those for conventional granular materials, field testing of moisture content and density by nuclear gauges is greatly affected by the presence of hydrogen ions in RAA material and requires using correction factors or resorting to other methods of quality control. The presence of asphalt reduces the amount of water needed to achieve the required compaction level of the RAA mixture, because of the surface coating of stone particles (Stroup-Gardiner and Wattenberg-Komas, 2013). This factor has to be considered when the suitable moisture content for compaction is determined. Locander (2009) observed that as the RAA fraction of the base layer increases, the optimum moisture content (OMC) required to achieve compaction decreases. This trend was confirmed by Guthrie et al. (2007), who found that the increase in RAA content leads to a decrease in the maximum dry density and OMC values. Observations by the Minnesota DOT (MnDOT) indicated that the standard Proctor test does not provide sufficient energy to achieve adequate compaction of RAA blends (J. Siekmeier, personal communication). Kim et al. (2007) reported that the gyratory compaction test method provided a closer correlation with field density measurements than the standard Proctor test. When compared to the Proctor test, gyratory compaction test results showed a large difference in the maximum dry density and a small difference in the OMC. As the RAA content of the blend increased, the OMC decreased for both test methods.

2.3.2 Permanent Deformation

Permanent deformation in base course greatly affects the pavement performance, such as Rutting. A series of repeated triaxial compression tests were conducted by Mohammad (2006) to determine the permanent deformation of base course materials. Two Vertical linear variable differential transducers (LVDT) were used to detect the displacements. A have sine load pulse of 0.1-second loading and 0.9-second rest period was applied to samples for 10,000 cycles. The samples were conditioned before the tests were conducted by applying a Number cycles of vertical stress and confining stress. The permanent deformation of RAA exhibited an initial acceleration and then reached a steady state. It was reported that the Mr was not sufficient in characterizing base course material of pavement structure and permanent deformation should be incorporated in the pavement design procedure [Mohammad et al. 2006]. Kim(2007) conducted 20 Mr Tests for samples with different percentages of RAA to investigate the effects of RAA percentage on resilient modulus. Specimens were prepared using the gyratory compactor and NCHRP 1-28A test protocol was followed. The test results showed that the RAA specimens were stiffer at high confining pressure when compared with virgin aggregate samples. However, the permanent deformation of specimens containing RAA was greater than that of virgin aggregates. In a study of recycled aggregates for use in unbound sub base; Ayan (2011) reported a decrease in CBR values with increasing RAA content. The results were attributed to sliding of the bitumen-coated aggregates over each other under the load application. Performance was satisfactory with the 50/50 mixture of RAA. The study recommended that shear strength measurements be carried out using large direct-shear box apparatus. The best results were achieved at moisture contents of 59% to 78% of the optimum value. However, the CBR results for this blend were marginally lower than the required design value of 80.

A study of geotechnical and geoenvironmental properties of construction and demolition waste conducted by Arulrajah et al. (2013) indicated that pure RAA does not meet the CBR and repeated load triaxial test requirements to qualify as an unbound sub base material in Australia.

The test provides resilient modulus /permanent deformation parameters that describe the material response to traffic loading. These parameters are used as input to the design and analysis of pavements (AustRoads, 2004). The study recommended blending RAA with high-quality aggregates to achieve the required strength and deformation requirements. Bennert and Maher (2005) confirmed the general trend of larger permanent deformations and lower CBR values as the RAA content was increased in the granular mixture. The authors recommended that RAA blended with virgin aggregate be limited to 50% by weight.

2.3.3 Permeability

Hydraulic conductivity is recognized as an important parameter for base course material. If the sub grade material is saturated, the pavement may deteriorate rapidly [Attia 2009, ARA 2004]. The moisture trapped between the particles in base layer may lead to the destruction of the pavement structure due to the loss of support.

For asphalt pavement, moisture can infiltrate into the base layer through surface cracking or shoulder over time. Compaction efforts during sample preparation reduce the volume of large pores and increase the volume of small pores [Gupta 2009].

Trzebiatowski (2005) conducted a study to determine the hydraulic conductivity of RAA as base course material. It was concluded that the saturated hydraulic conductivity of RAA ranged from 4.5× to 1.7× m/s when compacted with modified proctor efforts and from 2.4× to 9.0× m/s when compacted with standard proctor efforts. For the hydraulic conductivity testing conducted in the study by Trzebiatowski (2005), a rigid-wall, compaction-moldpermeameter was selected to conduct for sample preparation. By comparing the testing result on RAA and crushed stone, it was reported that the permeability of RAA is comparable to that of traditional base course material (Trzebiatowski, 2005). Another study by Gupta (2009) found that samples containing RAA had higher hydraulic conductivity when compared to aggregates. However, no correlation was detected between RAA percentage and the hydraulic conductivity.

Bouchedid (2001) tested base course materials for coefficient of permeability in the triaxialpermeameter as well as in the rigid wall permeameter, respectively. It was founded that the difference between the two methods was caused by different boundary conditions and sample preparation methods. Based on the results of field permeability measurements, triaxialpermeameter was recommended to be used for lab testing since the average field permeability was close to that from the triaxial permeability. Macgregor (1999) conducted 12 hydraulic conductivity tests with samples containing RAA, crushed-stone base materials and gravel-borrow sub base materials. It was found that hydraulic conductivity was not significantly affected by the change of RAA percentage in the RAA/crushed stone mixtures while the hydraulic conductivity of RAA/gravel-borrow mixtures increased by nearly an order of magnitude with the increase of RAA percentage from 0% to 50%. The uniform gradation of RAA was believed to be the reason for the increased hydraulic conductivity. Since factors such as compaction efforts, type of soil and gradation affect hydraulic conductivity, it is difficult, based on the literature, to determine whether the RAA percentage affects the hydraulic conductivity of mixtures.

RAA tends to behave as a strongly hydrophobic material. With regard to soil-water characteristic curves, RAA exhibited the best ability to drain water when compared to other recycled and natural aggregates (Edil et al., 2012). As a result, RAA is expected to provide more efficient subsurface drainage as compared to hydrophilic materials having the same pore size.

In a study designed to evaluate the suitability of using RAA as an additive to crushed angular aggregate, Mokwa and Peebles (2005) concluded that the use of RAA mixtures in base and sub base courses is viable. The permeability of RAA blends reportedly increased as the percentage of asphalt millings was increased, indicating improved drainage properties.

2.3.4 Moisture Damage

The base materials are subjected to moisture damage and freeze-thaw cycles. When RAA is used in base course, asphalt may strip off the aggregates and affect the permeability. In the laboratory, pavement materials are subjected to freeze-thaw conditioning for determining stripping. For aggregates, AASHTO T102 introduces procedures for freezing and thawing in Which samples should be cooled until the centre of the samples reaches -23C° ±3C° (-9F°±5F°) and the temperature shall be held for a minimum of 2 hours prior to the thaw cycle which lasts a minimum of 30 minutes at 21C° ±3C° (70F°±5F°). According to AASHTO T102, the procedure of alternate freezing and thawing should be repeated for 25 cycles.

2.3.4.1 Effect of Freeze-thaw on resilient modulus

The modulus of base course exhibits seasonal variations due to variation of moisture content and/or temperature. The stresses and strains induced in the pavement by traffic loads also vary with the modulus of the pavement layers [Mohammad et al. 2006]. Attia(2009) subjected a set of samples to two freeze-thaw cycles to evaluate the effect of freeze-thaw on the resilient modulus of RAA as compared to virgin aggregate . One cycle of freeze-thaw conditioning consisted of 24 hours of freeze conditioning at -12F° followed by 24 hours thawing conditioning at room temperature. Based on test results, samples containing RAA compacted at OMC did not show loss of strength due to freeze-thaw cycles. It was reported that the moisture Content was decreased, which indicated loss of moisture during conditioning and/or testing. The decreased moisture content could be a reason for higher modulus after freeze-thaw conditioning for samples.

2.3.5Strength and Stiffness

Particles of original coarse aggregate can be presumed to have good strength and be resistant to deformation, whereas agglomerations of fine aggregate and asphalt mastic may tend to be brittle or malleable depending on the asphalt condition (age and oxidation) and temperature.

Bennert (2000) reported that 100% RAA specimens have higher stiffness, higher resilient modulus values, and lower shear strengths than dense-graded aggregate base course specimens. Even though RAA is stiffer than the dense-graded aggregate base course, 100% RAA material accumulates the greatest amount of permanent strain. Several studies have shown relatively high resilient modulus values for RAA, accompanied by large permanent deformations. Bennert(2000) reported that the resulting contrast between the 100% RAA resilient modulus and its permanent deformation might be attributable to the progressive breakdown of asphalt binder under loading. Dong and Huang (2014) also indicated that RAA materials tended to have a higher resilient modulus and larger permanent deformations when tested as unbound aggregates. Triaxial creep test results demonstrated viscous properties and temperature dependency of unbound RAA base mixture. Locander (2009) reported that the shear strength decreases as the quantity of RAA increases. Taha (1999) indicated that the presence of RAA results in the lower bearing capacity of the material as compared to virgin aggregates.

Cosentino (2012) reported that all granular blends containing RAA exhibited some amount of creep. The study recommended that unsterilized RAA material be blended with a minimum of 75% approved aggregate for use in traffic base applications. Alternatively, blends should be proportioned so that the asphalt binder content does not exceed 1.5% by weight. In a study of material properties, Bleakley and Cosentino(2013) concluded that granular RAA and lime rock mixture without a stabilizing agent can meet the strength and creep requirements for base course if blended up to a maximum of 25% RAA and 75% lime rock. Blends with a maximum of 50% RAA may be used with a chemical stabilizing agent, such as cement.

The amount and type of the stabilizing agent should be determined by a mix design method that results in a blend that meets the required performance specifications.

McGarrah (2007) examined published studies on the properties of RAA blends used in unbound base applications and concluded that 100% RAA does not produce a product of adequate base course quality and should not be allowed. As the RAA content increased, the shear strength of the blend decreased below the required level. McGarrah (2007) recommended limiting the RAA content to 25% and blending RAA with the virgin aggregate. Onsite blending was found to be unsatisfactory, resulting in base course separation into lenses. Dong and Huang (2014) recommended that no 100% unbound RAA base be used under asphalt pavements. Schaefer et al. (2008) concluded that 20% to 50% RAA content is typically used in actual construction.

Ooi (2010) concluded that limiting RAA to 50% might be prudent as long as the material meets all other requirements in the specifications that a virgin aggregate would satisfy.

CHAPTER THREE
RESEARCH METHODOLOGY

3.1. Study Area

The research carried out at Jimma Town, situated in south-western Ethiopia, Formerly found under Region Oromia administrative zone. It has a latitude and longitude of 7°40′N36°50′E. It features a long annual season from March to October. Temperatures with the daily mean staying between 20°C and 25°C year-round and the average rainfall is 5mm.

3.2 Study Population

Study population for this research were recycled asphalt aggregate and fresh aggregate.

3.3 Sample Size and Sampling Procedures

3.3.1AggregateSamples

Sampling method used in this study was purposive sampling. Material used in this study includes crushed aggregates and recycled asphalt aggregate. Crushed aggregates were sampled from Ethiopian works construction corporation (EWCC), Jimma site. Recycled asphalt aggregate was collected from Ethiopian works construction corporation construction site in jimma town, around police station. The sample from around police station Road was collected after the removing of the existing pavement section. The samples were filled in 100-kilogram sacks and packed, then transported to the laboratory.

3.4 Study Variables

3.4.1 Independent Variables

Independent variables of this study are results of laboratory tests which are:

- Gradation
- Flakiness Index
- Aggregate crushing value
- Aggregate impact value
- Los Angeles abrasion value
- Compaction
- California Bearing Ratio

The values of these tests describe engineering property of fresh and recycled aggregate mixture. They can represent the suitability of material for use as a base course.

3.4.2 Dependent Variable

The dependent variable in this study is suitability of blended recycled asphalt aggregate with fresh aggregate for base course material, which is determined using the independent variables stated above.

3.5 Study Design

The research was designed to be an experimental study. Different laboratory tests were conducted in order to assess the properties of fresh and recycled asphalt aggregate samples. Then, the results of laboratory experiments were compared to standards to identify the suitability of recycled asphalt aggregate as a base course.

After all necessary materials for the research were acquired and transported to the laboratory, preparation of samples for testing commenced. The laboratory test were carried out according to ASHTO T -27, BS 812 Part 105-1990, BS 812 Part 110-1990, BS 812 Part 112-1990, ASTM C 131 – 89, ASTM D 1557 and ASTM D 1883.

3.5.1 Gradation

Particle gradation was conducted according to AASHTO T -27. Since the objective of this study was to evaluate the suitability of RAA, in order to eliminate the effect of gradation on the material properties.

This was done to determine the percentage particle size distribution of a given sample of recycled asphalt aggregate and fresh aggregate. Dry sieving analysis was performed. This procedure is suitable for coarse aggregate. This test aimed at determining the particle size distribution or gradation of the aggregate used. This was presented in form of graph plotted on grading chart.

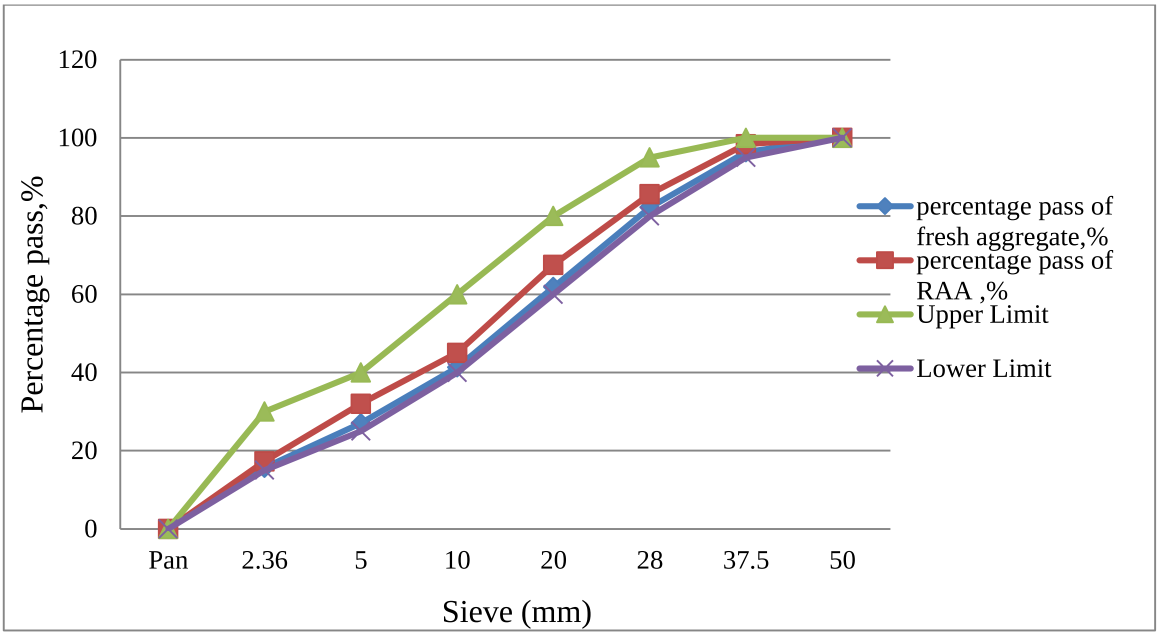

Figure 3.1 Particle size distribution of fresh and recycled aggregate

As shown in Figure 3.1 the gradation of fresh aggregate and recycled aggregate were determined by AASHTO T-27 and the results are com6paring to the specification of ERA manual. The figure clearly shows that the percentage passing of both recycled and fresh aggregates are beyond on upper and lower limit of percentage passing. The maximum particle size of an unbound layer is constrained by the layer thickness because roller compaction is only effective up to a depth of approximately 250 mm and layer workability gets difficult if stones of about 30% of layer thickness are present in the layer (Thom, 2008). Therefore, particle size and gradation are significant factors to be considering when constructing granular pavement layers.

3.5.2Flakiness Index

The flakiness index shall not exceed 30% when determined in accordance with BS 812 Part105-1990. Flakiness Index is one of the tests used to classify aggregates and stones. In Pavement Design there are specific requirements regarding the Flakiness Index of materials.

For base course and wearing course aggregates, the presence of flaky particles is considered undesirable as they may cause inherent weakness with possibilities of breaking down under heavy loads. The Flakiness Index of an aggregate sample is found by separating the flaky particles and expressing their mass as a percentage of the mass of the sample. The value of the Flakiness Index is calculated from the expression:

Flakiness Index, FI = (M3/M2)*100

Where

M3 is weight passed after discarding 5% or less (g)

M2 is sum of mass of after discarding 5% or less (g)

Figure 1.2 Determination of Flakiness index value

3.5.3 Aggregate Crushing Value

Aggregate used in road construction should be strong enough to resist crushing under traffic wheel loads. If the aggregate are weak, the integrity of the pavement structure is likely to be adversely affected. The aggregate crushing value gives a relative measure of the resistance of an aggregate to crushing under a gradually applied load and it determined by measuring the material passing a specified sieve after crushing under a load of 400KN. The test is applicable to a standard fraction aggregates passing a 14mm sieve and retained on a 10mm sieve.

If the standard size fraction 14 - 10 mm on BS 812: Part 110:1990.

ACV=(M2/M1)*100

Where

M_1 Is the mass of the test specimen (in g)

M_2 Is the mass of the material passing the 2.36 mm sieve (in g)

If the individual results differ by more than 7 % of the mean value, the test shall be repeated for two further specimens. The median value shall be reported as the ACV.

Figure 3.3 Determination of Aggregate crushing value

3.5.4 Aggregate Impact Value

The Aggregate Impact Value (AIV) gives a relative measure of the resistance of an aggregate to sudden shock or impact. The test can be performed in either a dry condition or in a soaked condition. The test is applicable to a standard fraction aggregates passing a 14 mm sieve and retained on a 10 mm sieve in BS 812 : Part 112 : 1990.

Calculate the Aggregate Impact Value (AIV) expressed as a percentage to the first decimal Place for each test specimen from the following equation:

AIV= (M2/M1)*100

where

M_1 Is the mass of the test specimen (in g)

M_2 Is the mass of the material passing the 2.36 mm sieve (in g)

The mean is reported as the Aggregate Impact Value, unless the individual results differ by more than 15 % of the mean value. In this case the test shall be repeated for two more specimens. The median value shall be reported as the AIV.

Figure 3.2 Determination of aggregate impact value

3.5.5 Los Angeles Abrasion Value

The objective of the test is to assess the durability of coarse aggregates used in pavement construction. Due to the movement of traffic, the road stones used in the surface course are subjected to wearing action at the top. Resistance to wear or hardness is hence an essential property for road aggregates, especially when used in wearing course.

The Los Angeles test is a measure of degradation of mineral aggregates of standard grading resulting from a combination of actions including abrasion and grinding in a rotating steel drum containing a specified number of steel spheres. As the drum rotates, a shelf plate picks up the sample and the steel spheres, carrying them around until they are dropped to the opposite side of the drum, creating an impact/crushing effect.

The contents then roll within the drum with an abrading and grinding action until the shelf plate impacts and the cycle is repeated specification on ASTM C 131 – 89.

Express the loss from the equation.

LAA value (%) = (M1-M2)/M1*100

Where

M1 is the mass of the test specimen (in g)

M2 is the mass of the material retained on 1.7 mm sieve

Table 3.1 Standard Sieve and Mass of samples for Abrasion Test

Sieve Size		Mass of indicated Sizes (g)			
Passing	Retained on	Grading			
		A	B	C	D
37.5 mm	25.0 mm	1250 ± 25			
25.0 mm	19.0 mm	1250 ± 25			
19.0 mm	12.5 mm	1250 ± 10	2500 ± 10		
12.5 mm	9.5 mm	1250 ± 10	2500 ± 10		
9.5 mm	6.3 mm			2500 ± 10	
6.3 mm	4.75 mm			2500 ± 10	
4.75 mm	2.36 mm				5000 ± 10
Total		5000 ± 10	5000 ± 10	5000 ± 10	5000 ± 10

Grading A: Suitable for Graded Crushed Stone and Natural Gravel for Base Course.
Grading B: Suitable for chippings for Surface Dressing, nominal sizes 20 mm and 14 mm.
Grading B: Suitable for chippings for Surface Dressing, nominal size 10 mm.
Grading B: Suitable for chippings for Surface Dressing, nominal size 7 mm.

Figure 3.3 Determination of Los Angeles Abrasion Value

3.5.6 Dry Density - Moisture Content

The objective of this test is to obtain relationships between compacted dry density and aggregate moisture content. The test is used to provide a guide for specifications on field compaction.

The first is a light compaction test using a 2.5Kg rammer (standard proctor).the second is a heavy compaction test using a 4.5Kg rammer with a greater drop on thinner layers of aggregate(modified proctor). The dry density which can be achieved for an aggregate depends on the degree of compaction applied and the moisture content. The moisture content which gives the highest dry density is called the optimum moisture content for that type of compaction.

The modified proctor compaction test was conducted to determine the optimum moisture Content (OMC) and Maximum dry unit weight (MDUW) in accordance with ASTM D 1557 Method, because less than 30 percent by mass of the material is retained on the 19 mm (3/4 inch) sieve. This procedure uses a 48 N (10 lb) hammer and a 45.72 cm (18 inches) drop height.

Particles retained on the 19- mm (0.75 inch) sieve were removed prior to Compaction and samples were compacted in five lifts in a 152- mm (6 inches) mould using 56 blows per layer.

This test was done to determine the maximum dry density (MDD) and optimum moisture content (OMC) of the material. It was done on the aggregate sample and then various percentages of recycled asphalt aggregate were added and MDD and OMC determined.

The wet density, moisture content and dry density are calculated from the following equation:

$$W1 = (A-B)/V$$

where,

W1 is wet density;

A is the mass of compacted specimen and mold;

B is the mass of mold;

V is the volume of mold.

3.5.7 California Bearing Ratio

CBR test was done to determine the strength of a given material and how it will behave when subjected to loading. This was determined by measuring the relationship between force and penetration when a cylindrical plunger of cross sectional area 1935mm^2 is made to penetrate the aggregate at given rate. At any penetration value, the ratio of the force to a standard force is defined as the California Bearing Ratio.

The strength of the base course is the main factor in determining the required thickness of flexible pavements for roads and airfields. The strength of a sub grade, sub base and base course materials were expressed in terms of their California Bearing Ratio (CBR) value.

The CBR-value is a requirement in design for pavement materials of natural gravel.

The CBR test described in ASTM Standards D1883-05 (for laboratory-prepared samples).This method covers the laboratory determination of the California Bearing Ration (CBR) of a compacted sample of soil dynamically compacted by metal rammers – one point method.

3.6 Data Collection Process

The data collection process begun with desk study of related researches and literature on recycled asphalt aggregate as a base course material. Then, collection of test samples of recycled aggregate from scarified asphalt of Jimma town and fresh aggregate from ERCC. Samples were collected by using simple instruments and labor. After that, the samples were taken to the laboratory and data with respect to material property were recorded for each of the tests specified.

3.7 Data Analysis

Quantitative data obtained from test results and results were analysed according to the specifications. Among the systematic analysis method, this particular research was employed percentage, tabulation, graphs and regression analysis. The manuals used were AASHTO, ERA, BS and ASTM.

3.8 Ethical Consideration

All pertinent data were collected after ethical permission was given from respective authorities and institution .The purpose of the study was clearly described to the authorities and concerned local communities.

CHAPTER FOUR
RESULTS AND DISCUSSIONS

After the completion of laboratory tests, the test results were analysed to determine physical and mechanical properties of base course aggregate. The aggregate was tried for its suitability for use as improved material for base course. Crushed aggregate is required to meet maximum Flakiness Index 30%, Maximum crushing value of 25%, Maximum impact value 24%, Maximum Abrasion Value 40% and Minimum CBR Value of 80% is the requirement for base course aggregate in ERA manual.

4.1. Results

Summary of the results for the tests conducted are presented below as in Table 4.1 and their graphical presentation shown in the Appendix.

Table 4.1: Result of Tests

RAA (%)	FI	ACV	AIV	LAA	CBR	MDD (g/cc)	OMC
0	19.86	14.5	13	21	107	2.4	5.4
20		19.72	17.24	28	92	2.32	6
50		23.5	22	35	75	2.23	7.2
100	17.04	32.5	33	52	63	2.14	8
ERA STANDARD	MAX 30%	MAX 25	MAX 24	MAX 40%	MIN 80%		

Table 4.1 provides a summary of the results from some physical and mechanical properties of aggregate with compare to ERA specification. The results included in Table 4.1 are recycled asphalt aggregate with zero percentage (fresh aggregate), 20% recycled aggregate, 50% recycled aggregate and fully recycled aggregate by a weight of fresh aggregate. Values obtained from the evaluation of the physical and mechanical properties of recycled aggregate are compared against the values obtained from the same evaluation process conducted on fresh aggregate.

The results showed that recycled aggregate has higher crushing value, higher impact value, higher abrasion loss, lower dry density and lower CBR value than fresh aggregate, because of formed a weak bond between aggregates in the case of some impurities of material such as asphalt coating, plastics, dust etc…

4.2. Discussions

4.2.1. Flakiness Index

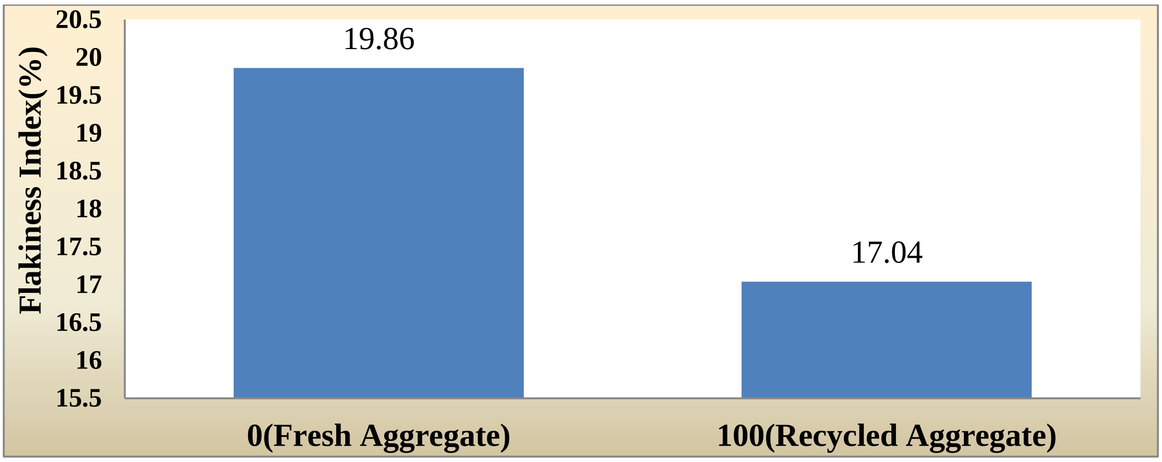

Figure 4.1 Result of Flakiness Index with different amount of RAA

As we can see from Fig 4.1, flakiness test was conducted to evaluate the shape of the aggregate recycled and fresh by using BS 812 part 105-1990 and it specify the maximum flakiness index was 30%.As depicted in figure 4.1, the horizontal direction indicated percentage of different amount of recycled aggregate and the vertical direction shows the flakiness index by percentage .The flakiness index values are 19.86% and 17.04% with 0%(fresh) and 100%(fully recycled) respectively, it shows that almost there is no difference amount between fresh and recycled aggregate.. In many respects, the physical properties of RAA are similar to those of crushed stone.

4.2.2 Aggregate Crushing Value

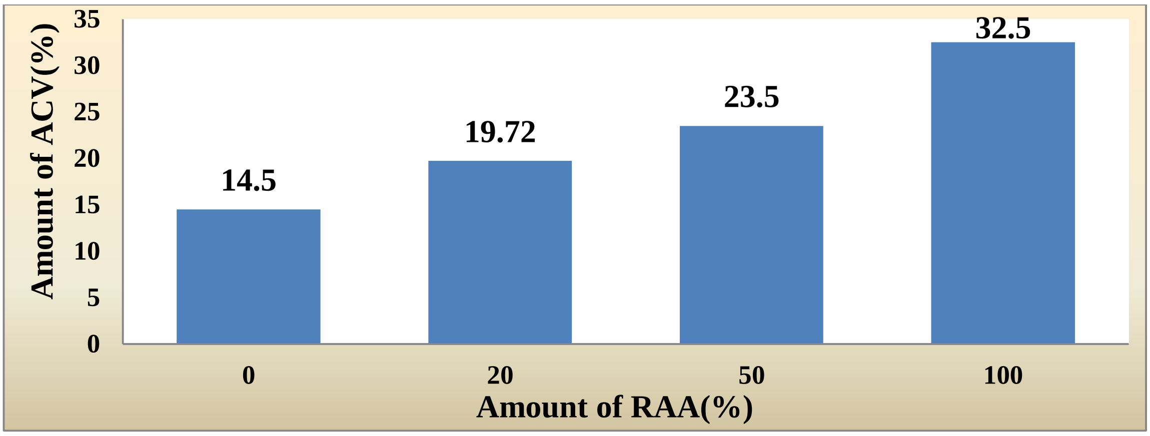

Figure 4.2: The relationship between aggregate crushing value and amount of RAA

Figure 4.2 shows the result of aggregate crushing value of the sampled data by using BS 812: Part 110:1990.

The result clearly indicated that, aggregate crushing value from a fresh value was (14.5%), (19.72%), (23.5%) and 32.5% after mixed with zero %(fresh aggregate), 20%, 50% and 100% of recycled aggregate respectively.

Based on the result, aggregate crushing value increased with the increase of recycled material for samples containing different percentages of RAA varying from 0% to 100%. The samples with higher recycled aggregate percentage, it has poor crushing resistance properties. As shown in Figure 4.2, samples containing higher recycled aggregate percentages increased crushing value compared to samples with lower recycled aggregate percentages, which implies that the base course in recycled aggregate was more sensitive to crushing compared to fresh aggregate.

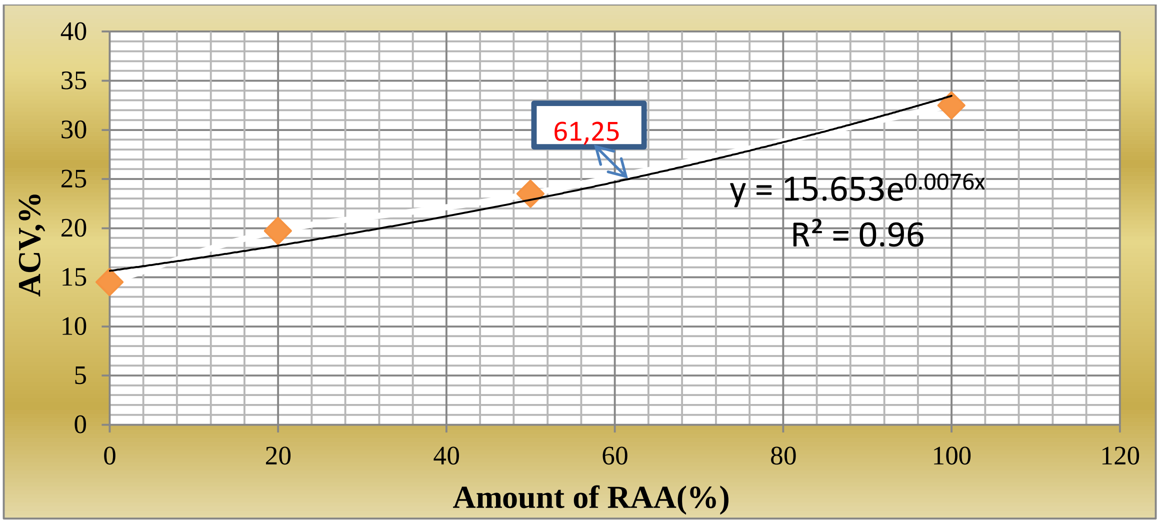

Figure 4.3: Correlations between amount of RAA and crushing value

Figure 4.3 shows that, the addition of recycled aggregate on fresh was found to have an effect increasing the aggregate crushing value remained within the allowed limit up to 58% of mixed with the fresh aggregate in the regression value.

However the aggregate crushing value above 58% recycled aggregate mixed with fresh was not found to be acceptable.

As we noted from the above results, we conclude that a fixed quantity mix i.e. 58% and below is not changing the performance of pavement condition with reference to ERA specification. Therefore all stakeholders involving in road construction sector should consider the recommendation value i.e. >58% of mix as to maintain the specification of ERA.

4.2.3 Aggregate Impact Value

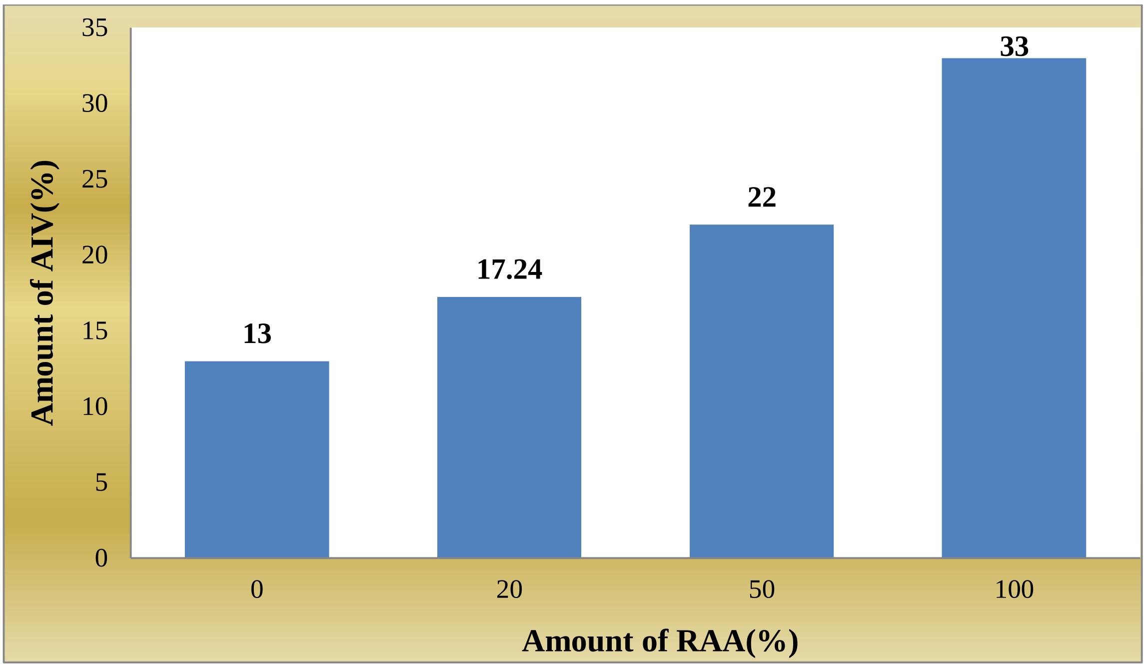

Figure 4.4:The relationship between Aggregate impact value and different amount of RAA

As shown in figure 4.4, aggregate impact value was computed as BS 812: Part 112:1990 to find out the impact capacity of fresh aggregate and recycled aggregate. The results of this study were revealed that, the data of the maximum impact value were significantly increase with the addition of recycled aggregate.

In relation to this ,aggregate impact value ranges from 13%,17.24%,22%, and 33% aggregate after mix with 0%(fresh),20%,50%,59% and 100% of recycled aggregate respectively. From the above data, one can safely arrive at a conclusion that, the resistance against impact decreases with increasing the percentage of recycled aggregate in the mixture.

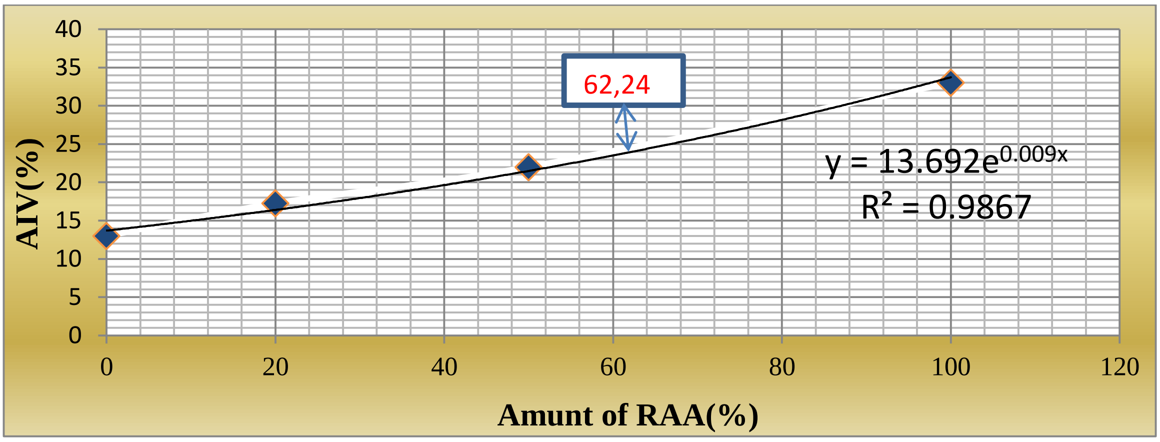

Figure 4.5 Correlation between RAA and Impact value

The most important things to be consider in figure 4.5, the aggregate impact value still remained in regression analysis within the acceptance of limit as far as the mixture is below 60%.On the contrary the aggregate impact value above 60% of recycled aggregate mix with fresh aggregate was not acceptable.

The data clearly indicated that a certain amount of quantity of mix, i.e. up to 60% is not affecting the performance of pavement condition according to ERA specification.

4.2.4 Abrasion Value

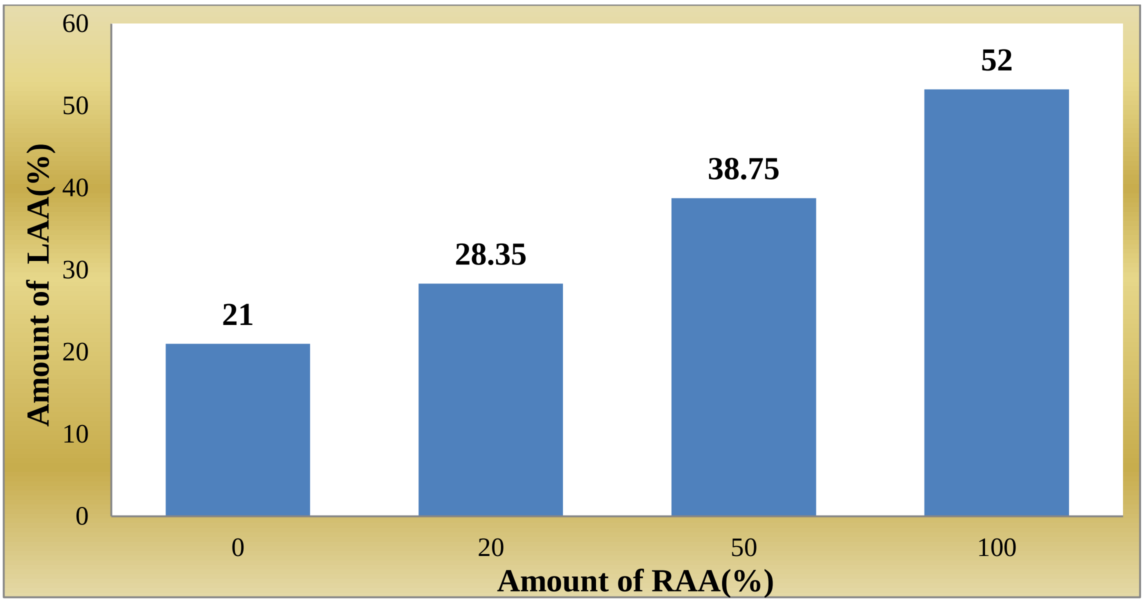

Figure 4.6 Relationship between Abrasion value and Amount of RAA

Figure 4.6 elaborated the results of the abrasion value in the mixture of recycled aggregate and fresh aggregate.

Accordingly In order to evaluate the resistance of the coarse fraction of the mixtures against abrasion, Los Angeles abrasion test was conducted on the mixtures according to ASTM C131 standard method.

Figure 4.6 shows the results of the Los Angeles tests, where the L.A. abrasion loss value in percentage has shown for the mixtures. As can be seen from the figure, the mixtures containing recycled aggregates have a higher abrasion value (52%) than the fresh aggregate (21%).When amount of recycled aggregate increased on the contrary wearing resistance decreased. The durability of construction aggregates will therefore depend upon the quality of aggregate mechanical properties.

It indicated that the presence of Recycled asphalt aggregate in base course results the base course has lower wearing resistance compared to fully fresh aggregates.

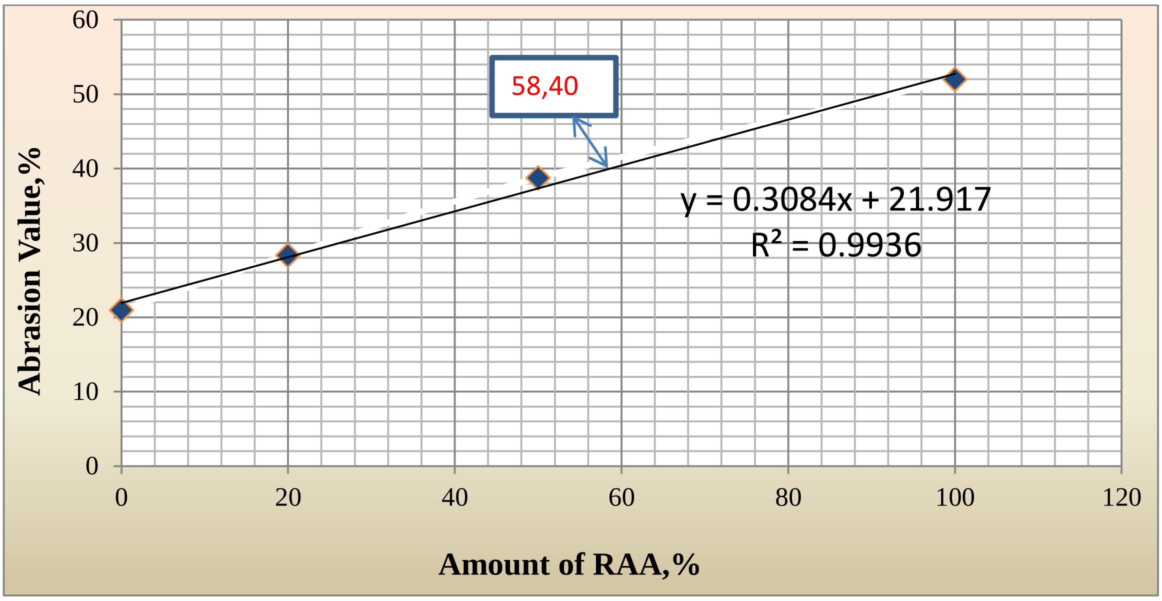

Figure 4.7 Correlation between RAA and abrasion value

According to the requirements of ERA specifications, the maximum abrasion value of base course is limited to 40. As shown in figure 4.8, this was satisfied by up to 53% of the mixtures. Therefore, the result of this data indicated that the use of limited percent of recycled asphalt aggregates in base would not cause any problem related to the abrasion.

The result of this research implies that, special attention should be given in order to fit with the specification of ERA.

4.2.5 Effect on compaction characteristics

4.2.5.1 Optimum moisture content

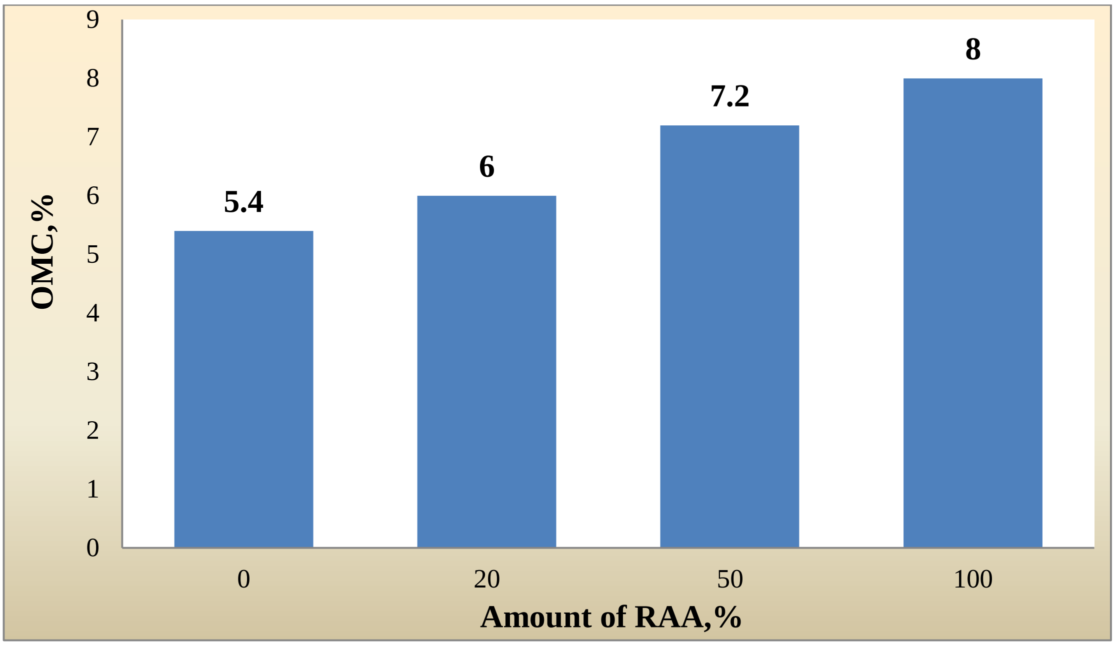

Figure 4.8: The relationship between OMC and Amount of RAA

The above figure 4.8 indicate that the optimum moisture content of samples increased with increase in recycled aggregate content of samples. OMC increased from 5.4% to 8% when recycled aggregate content increased from zero %(fresh aggregate) to 100% RAA.

4.2.5.2 Maximum dry density

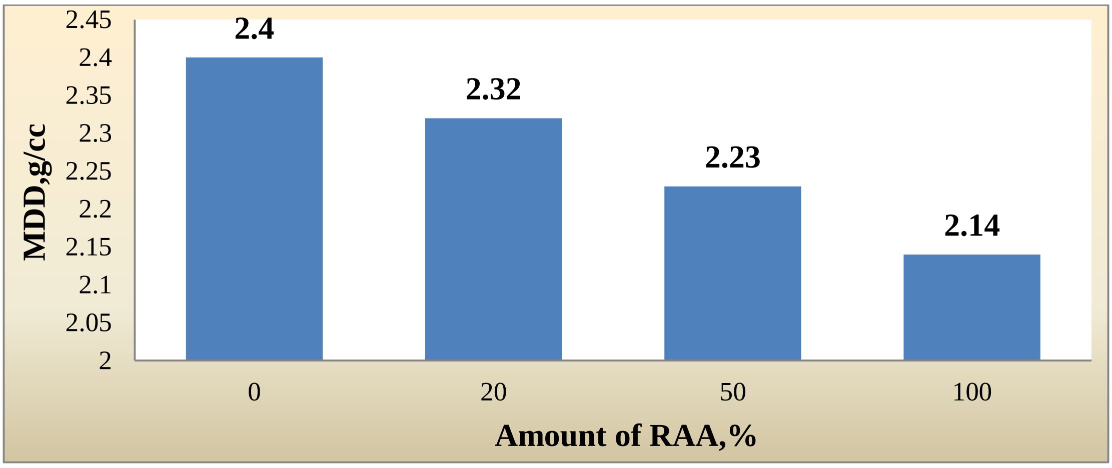

Figure 4.9: The relationship between MDD and Amount of RAA

The above figure 4.9 implies that decrease in MDD is observed in recycled aggregate treated samples. MDD of decreased from 2.4 to 2.14 by increase in recycled aggregate 0% (fresh) to 100% RAA content . The decrease may be a result of addition of a relatively lightweight aggregate material in place of base course material.

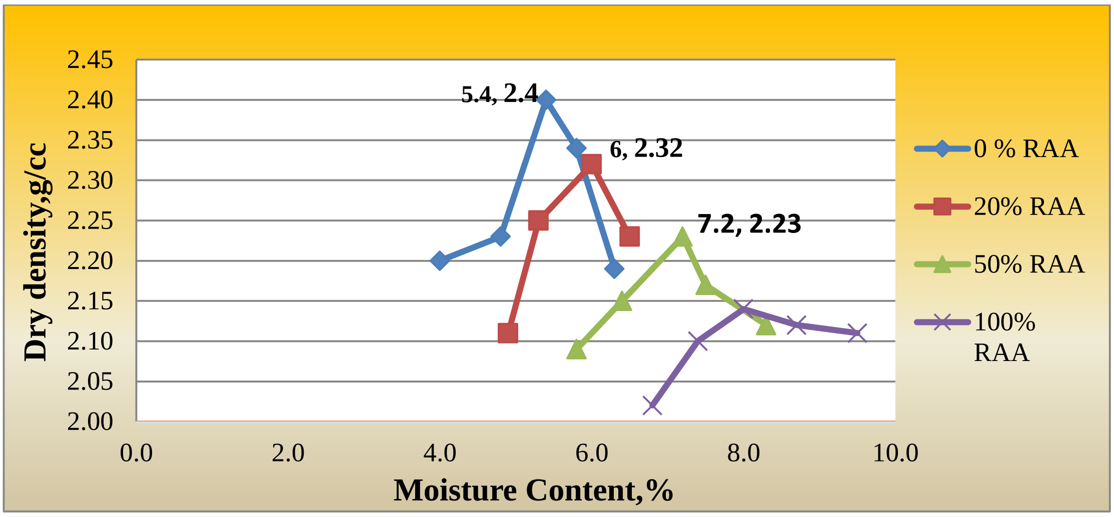

Figure 4.10: The relationship between Dry Density and Moisture Content of RAA

The above figure 4.10 shows the values for the maximum dry densities were noted to significantly decrease with the addition of recycled aggregate from a neat maximum value of 2.4g/cc to a value of 2.14g/cc attained in the blend of 0% RAA to 100% RAA respectively. Thus, the materials used to improve the base course were found to not facilitate the closer packing of the aggregate particles and thus a decrease in the maximum dry density. The OMC was found to increase from 5.4%-8% this may be attributed to the addition of recycled aggregate which increased the quantity of water taken to achieve the desired compaction.

Malleable or brittle particles can lead to post-compaction deformations of the base or layer if sufficient densification is not achieved during construction, which may be the cause of permanent strains sometimes reported when RAA was used as base material. Although construction methods for RAA are generally similar to those for conventional granular materials.

The presence of asphalt reduces the amount of water needed to achieve the required compaction level of the RAA mixture, because of the surface coating of stone particles .This factor has to be considered when the suitable moisture content for compaction is determined. Locander (2009) observed that as the RAA fraction of the base layer increases, the optimum moisture content (OMC) required to achieve compaction decreases. This trend was confirmed by Guthrie et al. (2007), who found that the increase in RAA content leads to a decrease in the maximum dry density.

4.2.6 Effect on California bearing ratio

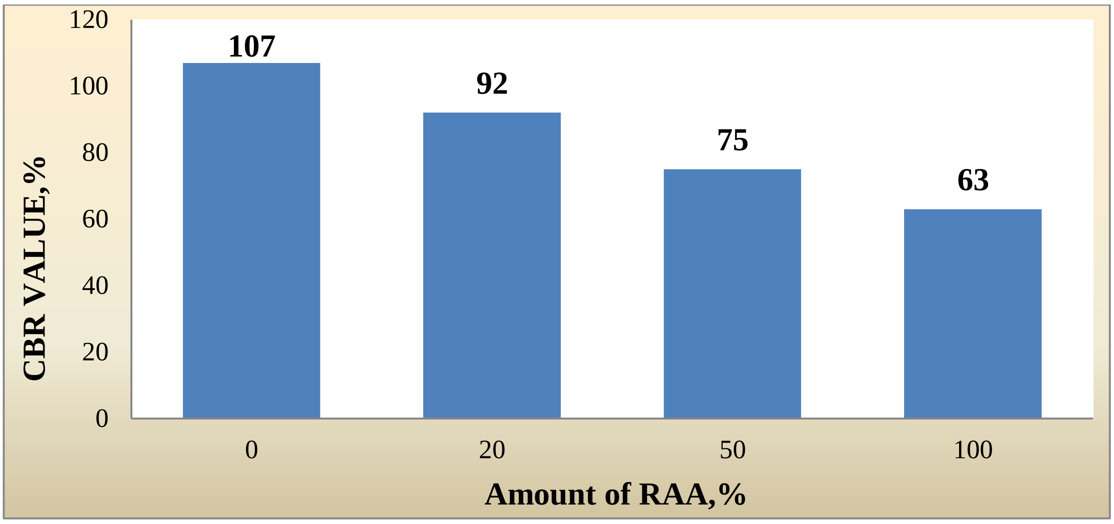

Figure 4.11: The relationship between CBR Value and Different Amount of RAA

Greater attention was given to CBR tests because the main aim of this research is to obtain suitability of blended high RAA values with fresh aggregate to attain the strength of base course aggregate compare to standard specification. From the results of the above Figure 4.11 for all mixes, it is evident that in every case the strength of base course aggregate continues to increase with decrease the recycled amount of aggregate, for the given mix proportion, base course made from the fresh aggregate have a higher strength or higher CBR value than those from recycled aggregate. But the result clearly shows that compare to ERA specification.

Bennert et al. (2000) reported that 100% RAA specimens have higher stiffness, higher resilient modulus values, and lower shear strengths than dense-graded aggregate base course specimens. Even though RAA is stiffer than the dense-graded aggregate base course, 100% RAA material accumulates the greatest amount of permanent strain. Several studies have shown relatively high resilient modulus values for RAA, accompanied by large permanent deformations.

In a study of recycled aggregates for use in different review reported a decrease in CBR values with increasing RAA content. The results were attributed to sliding of the bitumen-coated aggregates over each other under the load application.

Particles of original coarse aggregate can be presumed to have good strength and be resistant to deformation, whereas agglomerations of fine aggregate and asphalt mastic may tend to be brittle or malleable depending on the asphalt condition.

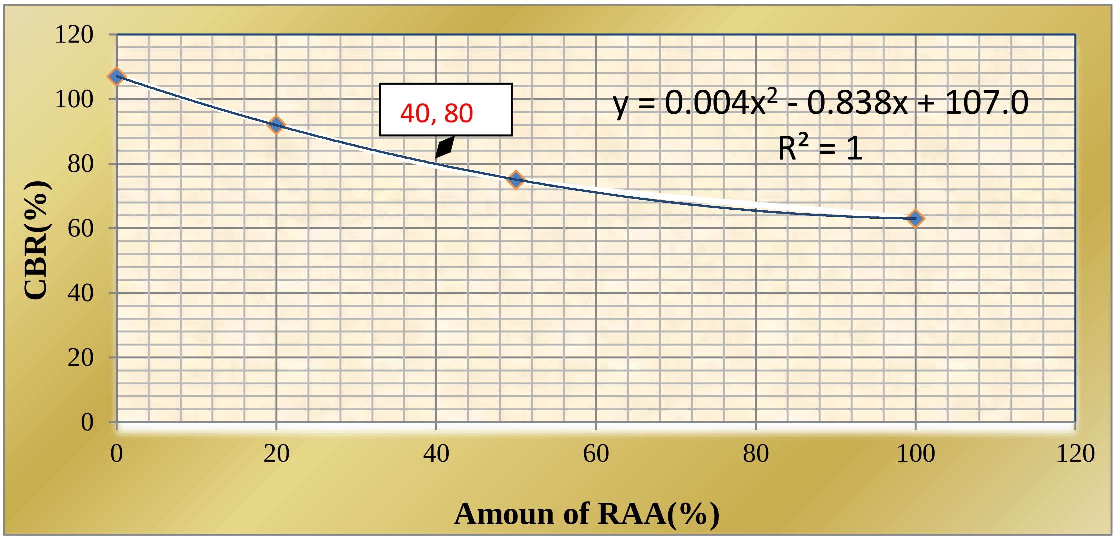

Figure 4.12: Correlation between CBR and amount of RAA

From the above figure 4.12 by regression analysis, conclude that limited amount of recycled aggregates such that 40% of RAA mixed are suitable for base course material according to ERA manual. Also fully recycled asphalt aggregates are a good substitute for sub base material compare to ERA manual. A study of geotechnical and geo-environmental properties of construction and demolition waste conducted by Arulrajah et al. (2013) indicated that pure RAA does not meet the CBR and repeated load triaxial test requirements to qualify as an unbound base material in Australia. Therefore, the above data implies that the concerned bodies in road sector should consider the maximum limit of mix (40%) in road construction process.

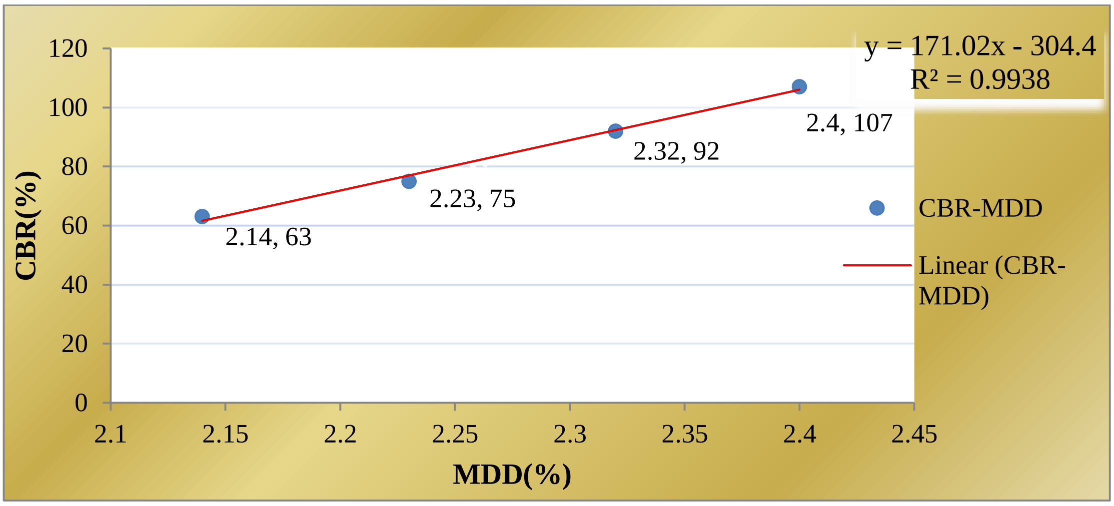

Figure 4.13: Correlation between CBR and MDD

As we can see in Figure 4.13, the relationship between MDD and CBR value with different mix of fresh and recycled aggregate by regression analysis. The result clearly indicated that there is a positive correlation (R^2=0.993) between CBR and MDD it means high CBR value attain at maximum dry density. On the other hand When the mix have high-recycled materials by weight of fresh aggregate contrary the value of maximum dry density and California bearing ratio were decreased. Different review materials reported that, there is a negative correlation between recycled aggregate with California bearing ratio and dry density.

4.2.7 Effect on CBR-swell

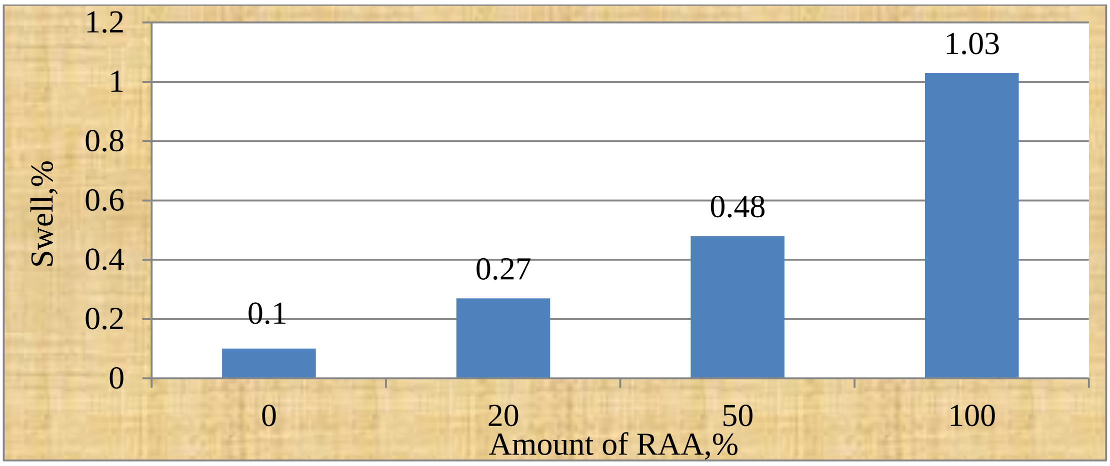

Figure 4.14: The relationship between CBR-Swell and Different Amount of RAA

The above figure 4.14 shows that the CBR-Swell value increased from 0.1% to 1.03%.It increases with each increment of recycled aggregate in a significant amount. This shows increases in swelling potential of aggregate with treatment of recycled aggregate. It means recycled aggregate absorb more moisture than the fresh one. Volume expansion of RAA material may be a factor if steel slag aggregates are present. These aggregates are used to improve frictional characteristics of the HMA surface course. The potential for expansion depends on the origin of the slag, grain size distribution, and age of the stockpile.

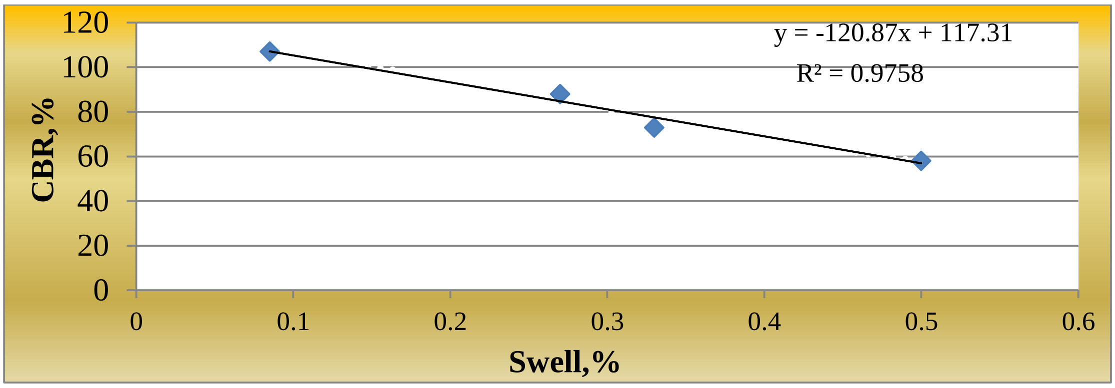

Figure 4.15: The relationship between CBR and CBR-Swell

The figure 4.15 clearly shows that an indirectly proportional relationship between CBR value and CBR-Swell when the value of CBR-Swell increases contrary the CBR value decreases. It means a good base course material aggregate have low voids and water absorption.

4.2.8 Correlation between ACV, AIV, Abrasion Value and CBR

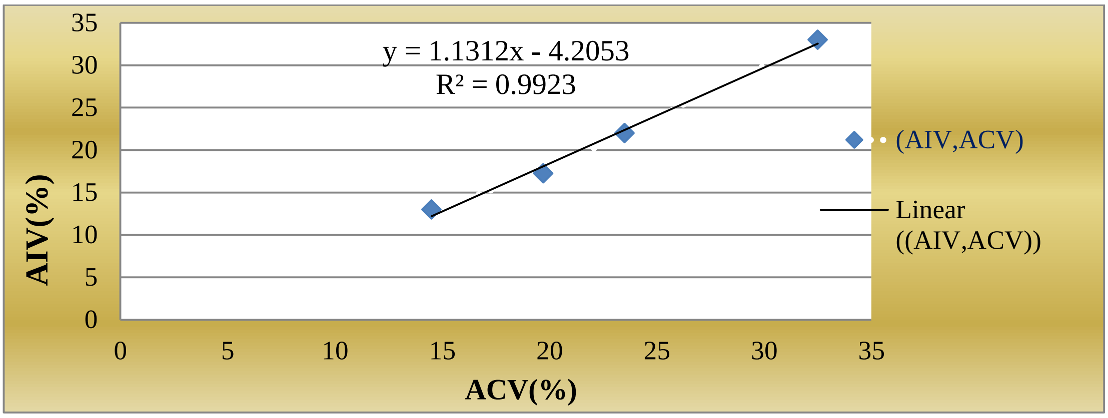

Figure 4.16: Correlation between Aggregate Impact and crushing value.

In the following sections, the correlation between some engineering properties of aggregate presented in Figures 4.16, 4.17, 4.18. It can be seen in Figure 4.16 the positive (R^2=0.992) correlation between the Aggregate Crushing Value and Aggregate Impact Value. Increasing in ACV Amount has direct effect on AIV amount.

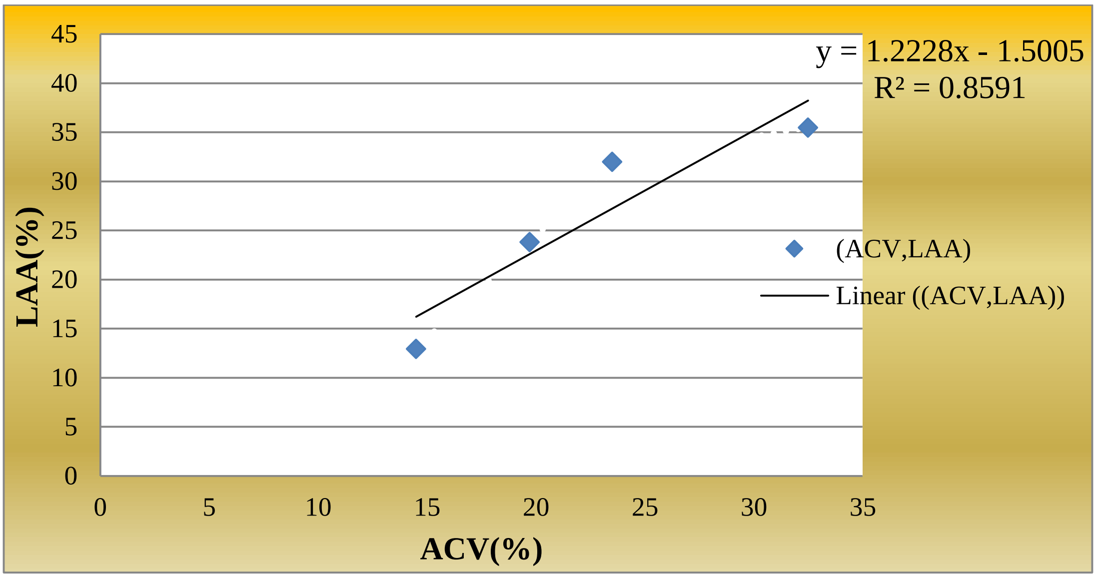

Figure 4.17: Correlation between Abrasion and Crushing Value of Aggregate

It can be seen in Figure 4.17 the positive (R^2=0.859) correlation among the Aggregate crushing value and Los Angeles abrasion value amounts. When the ACV amount increased also the amount of LAA increased.

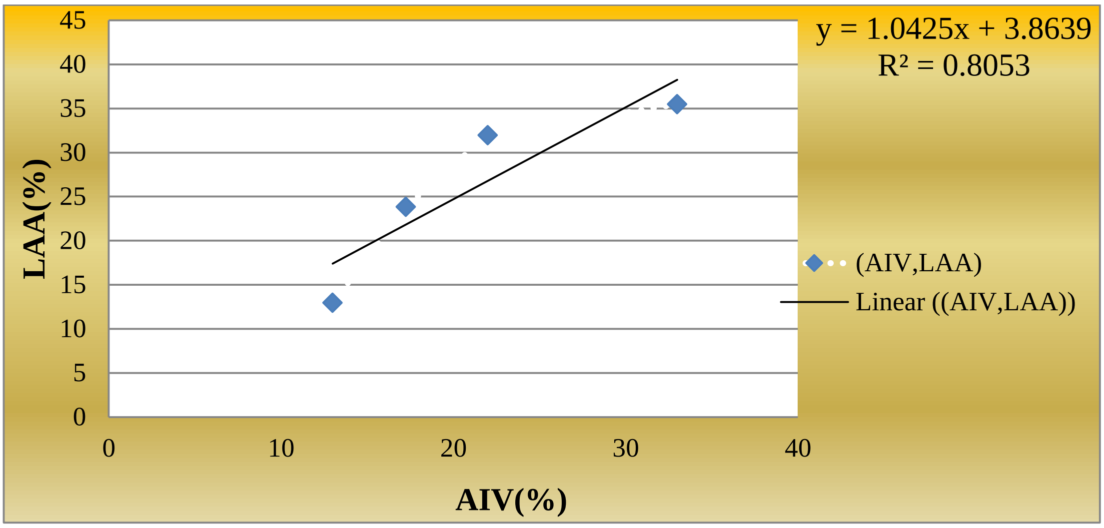

Figure 4.18: Correlation between Abrasion and Impact Value of Aggregate

Also in Figure 4.18, There is a positive correlation (R^2=0.805) between Aggregate impact value and Abrasion value. It means that lower AIV resulted lower values of LAA.

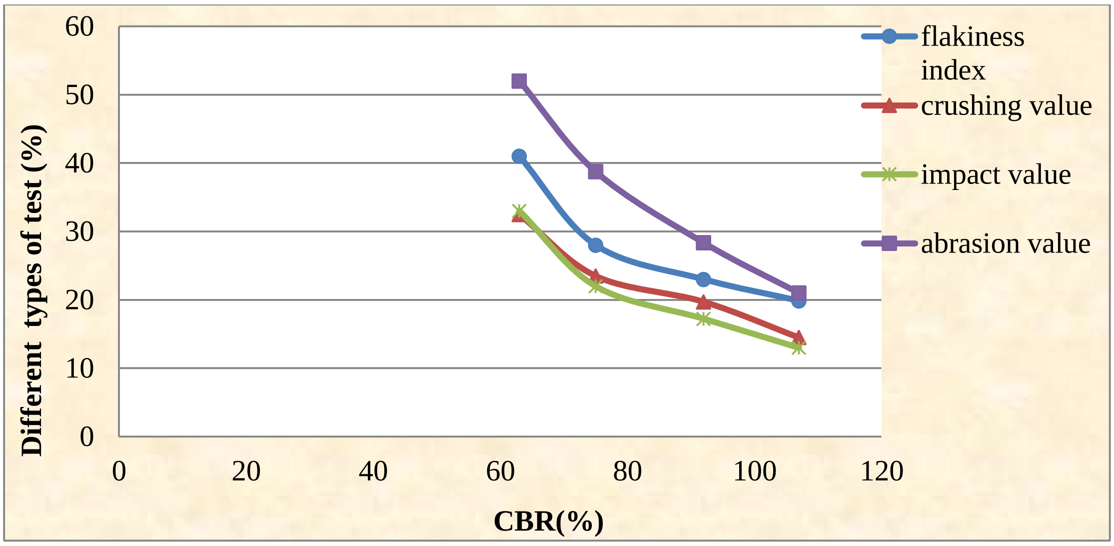

Figure 4.19: Relationship between CBR, ACV, AIV and LAA

As shown in Fig 4.22, the positive correlation between CBR, ACV, AIV and LAA values are directly proportional relationship. From this result, one can conclude that recycled material losses their strength. Finally, the test results imply that aggregate crushing; impact and abrasion values were directly proportional to the aggregate quality. When the material has a weak bond between particles contrary, the resistance of impact, crushing, abrasion and bearing loads were decreased. One of the reasons behind the weak bond is impurities of material such as asphalt, wood, plastics, dust etc...

CHAPTER FIVE

CONCLUSIONS AND RECOMMENDATTIONS

5.1. Conclusions

The work presented in this thesis evaluates the effect of recycled asphalt aggregate quality on the properties of base course material. Some physical and mechanical properties of the mixtures were evaluated. For the materials used in this research and over the range of the variables value investigated, the following results can be drawn from this study.

- Recycled asphalt aggregates have almost the same physical properties to those fresh aggregate. Moreover, Recycled asphalt aggregates have high water absorption, abrasion, crushing, impact value and moisture content than the fresh aggregates. In addition, recycled aggregates have low dry density and bearing ratio than fresh aggregate.
- Potential to resist crushing, impact, abrasion and compaction depends on the mixing ratio of recycled and fresh aggregate .When the ratio of recycled material increase, the resistant load decreased. It indicated that recycled aggregate loss some strength compare to the fresh one.
- The results from this study have shown that the addition of 0 %(Fresh aggregate), 20% and 50% RAA gave the technically qualified results on crushing, impact and abrasion value. The blend gave a ACV of 14.5%, 19.72% and 23.5% (maximum required 25%), AIV of 13%, 17.24% and 22% (Maximum required 24%), LAA value of 21%, 28% and 35%(maximum required 40%) respectively. But in the CBR test only 0% and 20% RAA mix by weight of fresh aggregate gave technically qualified results that is 107% and 92%(minimum required 80%) respectively.
- Additionally, the test results by regression analysis to determine the maximum percent of recycled asphalt aggregate used in base course compares to ERA specifications on flakiness , crushing , impact, abrasion and CBR values were 61%,62% , 58% and 40% respectively.

This test results confirm that recycled aggregate is a good substitute for fresh aggregate with a limited ratio of 40% for base course aggregate and fully recycled asphalt aggregates are a good substitute as a sub base material according to the CBR value. Finally, I conclude that Recycled Asphalt Aggregate was not waste products and contributes in conserving natural resources. As virgin resources become more limited and prices rise, the use of recycled aggregate in pavement construction is definitely an eco-friendly alternative.

5.2. Recommendations

The following recommendations could be drawn from the study:

- It was recommended that before using recycled aggregate, the material should be clear prior to using in base course as to increase its quality.
- Recycled asphalt aggregate alone cannot be used effectively to improve the base course for use in road construction. Therefore, recycled asphalt aggregate should be mixed with fresh aggregate by considering the ERA specification.
- Different mix ratios should be tried before the ratio that provides the best alternative is selected to economize on cost while attaining the highest possible standards
- Further studies should be carried out in order to identify the long term effects that RAA has on the durability and strength of the road pavement structures

REFERENCE

- American Association of State Highway and Transportation Officials (AASHTO). (1993) Guide for Design of Pavement Structures, Washington, D.C.
- Araya and Beyene (2013) African Development Bank Group, 2014; Ministry of Finance and Economic Development, 2014, p. 22, 24, 43–44).
- Attia, M., Abdelrahman, M. (2010) Modeling the effect of moisture on resilient modulus of untreated reclaimed asphalt pavement. Transportation Research Record: Journal of the Transportation Research Board, No 2167, pp. 30-40.
- African Economic Outlook, 2015,p.13,21
- AustRoads. *Guide to the Structural Design of Road Pavements*. Sydney, NSW, Australia, 2004.
- Ayan, V. Assessment of Recycled Aggregate for Use in Unbound Sub base of Highway Pavement .Ph.D. Thesis, KingstonUniversity, London, UK, 2011.
- Bennert, T., and Maher, A. The Development of a Performance Specification for Granular Base and Sub base Material .Rutgers University, Piscataway, NJ, 2005.
- Bouchedid, M.B. and Humphrey, D.N. (2005) Permeability of Base Material for Maine Roads. *Transportation Research Record: Journal of the Transportation Research Board*, No1936,pp. 142-149.
- Carruthers,Robin, Ranga Rajan Krishnamani, and Siobhan Murray.2009. ―Improving Connectivity: Investing in Transport Infrastructure in Sub-Saharan Africa.‖ AICD Background Paper 7, Africa Region, World Bank, and Washington, DC.
- Collins, R. J. and S. K.Ciesielski. *Recycling and Use of Waste Materials and By-Products in Highway Construction*. National Cooperative Highway Research Program Synthesis of Highway Practice 199, Transportation Research Board, Washington, DC, 1994
- Cooley, D.A. (2005) Effects of Reclaimed Asphalt Pavement on Mechanical Properties of Base Materials, submitted for degree, Brigham Young University.

- Copeland, A. *Reclaimed Asphalt Pavement in Asphalt Mixtures: State of* thePractice.FHWA-HRT-11-021.Federal Highway Administration, McLean, VA, 2011.
- Cosentino, P.J., Kalajian, E.H., Bleakley, A.M., Diouf, B.S., Misilo, T.J., Petersen, A J., Krajcik, R.E., and Sajjadi, A.M. *Improving the Properties of Reclaimed Asphalt Pavement for Roadway Base Applications*. Florida Institute of Technology, Melbourne, 2012.
- Edil, T.B., Tinjum, J.M., and Benson, C.H. *Recycled Unbound Materials*. Final Report 2012-35. Minnesota Department of Transportation Research Services, Office of Policy Analysis, Research &Innovation. University of Wisconsin-Madison, 2012.
- Garg, Navneet, and Thompson, Marshall R. (1996). Lincoln Avenue Reclaimed Asphalt Pavement Base Project. Transportation Research Record 1547. pp. 89-95.
- Gupta, S.C., Kang, D.H., and Ranaivosoon, A. *Hydraulic and Mechanical Properties ofRecycled Materials.* Minnesota Department of Transportation, Research Services Section, St. Paul, 2009.
- Guthrie, W. S., Cooley, D.A. and Eggett, D. L. (2007) Effects of Reclaimed Asphalt Pavement on Mechanical Properties of Base Materials. Transportation Research Record: Journal of the Transportation Research Board, No 2005, pp. 44-52.
- Kim, W., Labuz, J. F., and Dai, S. Resilient Modulus of Base Course Containing Recycled Asphalt Pavement. In *Transportation Research Record: Journal of the Transportation Research Board,* No. 2006. Transportation Research Board of the National Academies, Washington, DC, 2007, pp. 27-35.
- Locander, R. *Analysis of Using Reclaimed Asphalt Pavement (RAP) As a Base Course Material.* Colorado Department of Transportation - Research, Denver, 2009.
- Macgregor, Jennifer A.C., et al. (1999). Structural Numbers for Reclaimed Asphalt Pavement Base and Sub base Course Mixes. Transportation Research Record 1687. pp. 22-28.

- Maher, M.H., et al. (1997). Recycled Asphalt Pavement as a Base and Sub-Base Material. ASTM Special Publication 1275. pp. 42-53.
- McGarrah, E.J. (2007) "Evaluation of Current Practices Reclaimed Asphalt Pavement/Virgin Aggregate as Base Course Material."Report No. WA-RD 713.1, Washington State of Department of Transportation, Olympia, WA.
- Mohammad, L. N., Herath, A., Rasoulian, M. and Zhang, Z. (2006) Laboratory Evaluation of Untreated and Treated Pavement Base Material from a repeated Load Permanent Deformation Test. Transportation Research Record: Journal of the Transportation Research Board, No 1967, pp. 78-88.
- Mokwa, Robert L, and Peebles, Cole S.(2006). Strength, Stiffness and
- Compressibility of RAP/Aggregate Blends. Pavement Mechanics and Performance – GeoShanghai International Conference. pp. 247-255.
- Ooi, P.S.K. *Application of Recycled Materials in Highway Projects*. University of Hawaii at Manoa, Honolulu, 2010.
- Papp, W.J., et al. (1998). Behaviour of Construction and Demolition Debris in Base and Sub base Applications. Geotechnical Special Publication, No. 79. pp. 122-135.
- Senior, S. A., S. I. Szoke, and C. A. Rogers, "Ontario's Experience With Reclaimed Materials for Use in Aggregates." Presented at the International Road Federation Conference, Calgary, Alberta, 1994.
- Taha, Ramzi, et al. (1999). Evaluation of Reclaimed Asphalt Pavement Aggregate in Road Bases and Subbases.Transportation Research Record 1652. pp. 264-269
- Uhlmeyer, J. "Construction and Performance of Reclaim Untreated Aggregate Base." Research Need Statement, WashiTransportation, Olympia, WA.
- Viyanant, C., et al. (2004). Compaction Control of Crushed Concrete and Recycled Asphalt Pavement Using Nuclear Gauge. Geotechnical Engineering for Transportation Projects. Pp. 958-966.

APPENDEX A: DETAILED TEST RESULTS

Table A1: Gradation of Fresh Aggregate

GRADING OF AGGREGATE						
	TEST METHOD : (AASHTO T-27)					
Sampling						
Station	Jimma City(Around Police Station)					
Source	ERCC(Jimma District)					
Lab	Fresh Aggregate					
Material for	Base Course					
Sieve(mm)	Retained Mass(g)	% Mass retained	Cum % retained	% Pass	Lower Limit	Upper limit
50	0	0	0	100	100	100
37.5	237.43	3.68	3.68	96.32	95	100
28	909.08	14.09	17.77	82.23	80	95
20	1314.9	20.38	38.15	61.85	60	80
10	1325.9	20.55	58.7	41.3	40	60
5	277.44	4.3	63	27	25	40
2.36	1377.5	21.35	84.35	15.65	15	30
pan	1009.7	15.65	100	0		
	6451.95					

Table A2: Gradation of Recycled Asphalt Aggregate

GRADING OF AGGREGATE						
	TEST METHOD : (AASHTO T-27)					
Sampling						
Station	Jimma City(Around Police Station)					
Source	ERCC(Jimma District)					
Lab	100 % RAA					
Material for	Base Course					
Sieve(mm)	Retained Mass(g)	% Mass retained	Cum % retained	% Pass	Lower Limit	Upper limit
50	0	0	0	100	100	100
37.5	93.86	1.64	1.64	98.36	95	100
28	730.34	12.76	14.4	85.6	80	95
20	1034.85	18.08	32.48	67.52	60	80
10	1288.98	22.52	55	45	40	60
5	744.1	13	68	32	25	40
2.36	844.52	14.75	82.75	17.25	15	30
pan	987.05	17.25	100	0		
	5723.7					

Table A3: Flakiness Index of Fresh Aggregate

sieve size (mm)	weight taken (g)	sum of the fraction on trays, m1(g)	% individual fraction ,m(g)	sum of mass after discarding 5% or less	weight passed (g)	sum of m3 (g)	FI=(M3/ M2)*100
50-37.5	0	5017	0	4647	0	923	19.86
37.5-28	145		2.89		52		
28-20	1622		32.33		280		
20-14	2025		40.36		423		
14-10	1000		19.93		220		
10-6.3	225		4.5		65		

Table A4: Flakiness Index of Recycled Asphalt Aggregate

sieve size (mm)	weight taken (g)	sum of the fraction on trays, m1(g)	% individual fraction ,m(g)	sum of mass after discarding 5% or less	weight passed (g)	sum of m3 (g)	FI=(M3/ M2)*100
50-37.5	0		0		0		
37.5-28	200		3.75		45		
28-20	1822	5322	34.23	4647	246	907	17.04
20-14	2125		39.93		450		
14-10	850		15.97		159		
10-6.3	325		6.12		52		

Table A7: Fresh Aggregate Crushing Value

ACV			
TEST METHOD : (BS 812 PART 110:1990) Sampling Station Jimma City(Around Police Station) Source ERCC(Jimma District) Lab Fresh Aggregate Material for Base Course			
Specification Limits : 25% max			
Standard Aggregate Size (14 - 10mm)			-
Test No		1	2
Mass of aggregate before test, passing 14mm and retained on 10mm sieve(m1)	M1(g)	3500	3500
Mass of aggregate after compression, passing 2.36mm sieves(m2)	M2(g)	560	455
AggregateCrushing Value(ACV)=(M2)/M1*100		16	13
Average ACV(%)=(Test1+Test2)/2		14.5	

Table A8: Blended of Fresh Aggregate With 20% Recycled Asphalt Aggregate Crushing Value

ACV			
TEST METHOD : (BS 812 PART 110:1990)			
Sampling			
Station	Jimma City(Around Police Station)		
Source	ERCC(Jimma District)		
	Fresh Aggregate with 20%		
Lab	RAA		
Material for	Base Course		
Specification Limits : 25% max			
Standard Aggregate Size (14 - 10mm)			-
Test No		1	2
Mass of aggregate before test, passing 14mm and retained on 10mm sieve(m1)	M1(g)	3500	3500
Mass of aggregate after compression, passing 2.36mm sieves(m2)	M2(g)	715	665
Aggregate CrushingValue(ACV)=(M2)/M1*100		20.4	19
Average ACV(%)=(Test1+Test2)/2		19.7	

Table A9: Blended of Fresh Aggregate With 50% Recycled Asphalt Aggregate Crushing Value

ACV			
TEST METHOD : (BS 812 PART 110:1990) Sampling Station: Jimma City(Around Police Station) Source: ERCC(JimmaDistrict) Lab: Fresh Aggregate with 50% RAA Material for: Base Course			
Specification Limits : 25% max			
Standard Aggregate Size (14 - 10mm)			-
Test No		1	2
Mass of aggregate before test, passing 14mm and retained on 10mm sieve(m1)	M1(g)	3500	3500
Mass of aggregate after compression, passing 2.36mm sieves(m2)	M2(g)	840	805
Aggregate CrushingValue(ACV)=(M2)/M1*100		24	23
Average ACV(%)=(Test1+Test2)/2		23.5	

Table A10: Recycled Asphalt Aggregate Crushing Value

ACV			
TEST METHOD : (BS 812 PART 110:1990)			
Sampling Station	Jimma City(Around Police Station)		
Source	ERCC(Jimma District)		
Lab	100% RAA		
Material for	Base Course		
Specification Limits : 25% max			
Standard Aggregate Size (14 - 10mm)			-
Test No		1	2
Mass of aggregate before test, passing 14mm and retained on 10mm sieve(m1)	M1(g)	3500	3500
Mass of aggregate after compression, passing 2.36mm sieves(m2)	M2(g)	1225	1050
AggregateCrushing Value(ACV)=(M2)/M1*100		35	30
Average ACV(%)=(Test1+Test2)/2		32.5	

Table A11: Fresh Aggregate Impact Value

AIV			
TEST METHOD : (BS 812 PART 112:1990) Sampling Station Jimma City(Around Police Station) Source ERCC(Jimma District) Lab Fresh Aggregate Material for Base Course			
Specification Limits : 24% max			
Standard Aggregate Size (14 - 10mm) -			
Test No		1	2
Mass of aggregate before test, passing 14mm and retained on 10mm sieve(m1)	M1(g)	2500	2500
Mass of aggregate after impact, passing 2.36mm sieves(m2)	M2(g)	300	350
Aggregate Impact Value(AIV)=(M2)/M1*100		12	14
Average ACV(%)=(Test1+Test2)/2		13	

Table A12: Blended of Fresh Aggregatewith 20% Recycled Asphalt Aggregate Impact Value

AIV			
TEST METHOD : (BS 812 PART 112:1990)			
Sampling Station	Jimma City(Around Police Station)		
Source	ERCC(Jimma District)		
Lab	Fresh Aggregate with 20% RAA		
Material for	Base Course		
Specification Limits : 24% max			
Standard Aggregate Size (14 - 10mm)			-
Test No		1	2
Mass of aggregate before test, passing 14mm and retained on 10mm sieve(m1)	M1(g)	2500	2500
Mass of aggregate after impact, passing 2.36mm sieves(m2)	M2(g)	412	450
Aggregate Impact Value(AIV)=(M2)/M1*100		16.2	18
Average ACV(%)=(Test1+Test2)/2		17.2	

Table A13: Blended Of Fresh Aggregate With 50% Recycled Asphalt Aggregate Impact Value

AIV			
TEST METHOD : (BS 812 PART 112:1990) Sampling Station: Jimma City(Around Police Station) Source: ERCC(Jimma District) Lab: Fresh Aggregate with 50% RAA Material for: Base Course			
Specification Limits : 24% max			
Standard Aggregate Size (14 - 10mm)			-
Test No		1	2
Mass of aggregate before test, passing 14mm and retained on 10mm sieve(m1)	M1(g)	2500	2500
Mass of aggregate after impact, passing 2.36mm sieves(m2)	M2(g)	575	525
Aggregate Impact Value(AIV)=(M2)/M1*100		23	21
Average ACV(%)=(Test1+Test2)/2		22	

Table A14: Recycled Asphalt Aggregate Impact Value

AIV			
TEST METHOD : (BS 812 PART 112:1990) Sampling Station Jimma City(Around Police Station) Source ERCC(Jimma District) Lab 100% RAA Material for Base Course			
Specification Limits : 24% max			
Standard Aggregate Size (14 - 10mm)			-
Test No		1	2
Mass of aggregate before test, passing 14mm and retained on 10mm sieve(m1)	M1(g)	2500	2500
Mass of aggregate after impact, passing 2.36mm sieves(m2)	M2(g)	875	775
Aggregate Impact Value(AIV)=(M2)/M1*100		35	31
Average ACV(%)=(Test1+Test2)/2		33	

LOS ANGELES ABRASION VALUE

Table A15: Fresh Aggregate Los Angeles Abrasion Value

LAA			
TEST METHOD : (ASTM C 131-89) Sampling Station Jimma City(Around Police Station) Source ERCC(JimmaDistrict) Lab Fresh Aggregate Material for Base Course			
Specification Limits : 40% max			
Sieve Size in mm(37.5,25,19,12.5 and 9.5) -			
Test No		1	2
Mass of aggregate before test, on standard sieve(m1)	M1(g)	5000	5000
Mass of aggregate after abrasion, retained on 1.7mm sieves and oven dried(m2)	M2(g)	4050	3850
LosAngelesAbrasion Value(LAA)=(M1M2)/M1*100		19	23
Average LAA(%)=(Test1+Test2)/2		21	

Table A16: Blended Of Fresh Aggregate With 20% Recycled Asphalt Aggregate Abrasion Value

LAA			
TEST METHOD : (ASTM C 131-89) Sampling Station Jimma City(Around Police Station) Source ERCC(JimmaDistrict) FreshAggregatewith Lab 20%RAA Material for Base Course			
Specification Limits : 40% max			
Sieve Size in mm(37.5,25,19,12.5 and 9.5) -			
Test No		1	2
Mass of aggregate before test, on standard sieve(m1)	M1(g)	5000	5000
Mass of aggregate after abrasion, retained on 1.7mm sieves and oven dried(m2)	M2(g)	3550	3615
LosAngelesAbrasion Value(LAA)=(M1M2)/M1*100		29	27.7
Average LAA(%)=(Test1+Test2)/2		28.35	

Table A17: Blended of Fresh Aggregate With 50% Recycled Asphalt Aggregate Abrasion Value

LAA			
TEST METHOD : (ASTM C 131-89) Sampling Station: Jimma City(Around Police Station) Source: ERCC(JimmaDistrict) Lab: FreshAggregatewith 50%RAA Material for: Base Course			
Specification Limits : 40% max			
Sieve Size in mm(37.5,25,19,12.5 and 9.5) -			
Test No		1	2
Mass of aggregate before test, on standard sieve(m1)	M1(g)	5000	5000
Mass of aggregate after abrasion, retained on 1.7mm sieves and oven dried(m2)	M2(g)	3105	3016
LosAngelesAbrasion Value(LAA)=(M1M2)/M1*100		37.83	39.67
Average LAA(%)=(Test1+Test2)/2		38.75	

Table A18: Recycled Asphalt Aggregate Abrasion Value

LAA			
TEST METHOD : (ASTM C 131-89) Sampling Station Jimma City(Around Police Station) Source ERCC(JimmaDistrict) Lab 100%RAA Material for Base Course			
Specification Limits : 40% max			
Sieve Size in mm(37.5,25,19,12.5 and 9.5) -			
Test No		1	2
Mass of aggregate before test, on standard sieve(m1)	M1(g)	5000	5000
Mass of aggregate after abrasion, retained on 1.7mm sieves and oven dried(m2)	M2(g)	2535	2265
LosAngelesAbrasion Value(LAA)=(M1M2)/M1*100		49.3	54.7
Average LAA(%)=(Test1+Test2)/2		52	

Table A19: Compaction value of fresh aggregate

Project (site): Around Police Station(jimma)	Date of sampled: 23/12/2017
Material source: ERCC	Tested date: 1/05/2017
Material description: Base course	Lab:-Fresh Aggregate
Test Method: ASTM D 1557	Sampled by:

A	Number of tests		1	2	3	4	5
B	water added %		1.5	3	4.5	6	7.5
C	weight of mold (g)		5269	5269	5269	5269	5269
D	weight of sample+mold(g)		10128.7	10232.9	10641.9	10527.4	10213.6
E	weight of wet sample (g)		4859.71	4963.87	5372.87	5258.43	4944.61
F	mold volume (cm^3)		2124	2124	2124	2124	2124
G	wet density(g/cc)	(E/F)	2.29	2.34	2.53	2.48	2.33
	Moisture determination						
H	Cone. no		F-10	A-6	A-2	C-25	A-5
I	wt.cone+wet Sam.(g)		887.0	1006.3	1026.1	1063.2	1123.3
J	wt.cone+dry Sam.(g)		867.0	978.3	993.1	1025.2	1078.3
K	weight of water (g)	(I-J)	20	28	33	38	45
L	weight of cone. (g)		367.0	395.0	382.0	374.0	364.0
M	Weight of dry Sam. (g)	(J-L)	500	583.33	611.11	651.17	714.3
N	Moisture content (%)	(K/M)*100	4.0	4.8	**5.4**	5.8	6.3
O	Dry density (g/cc)	(G/((N/100)+ 1))	2.20	2.23	2.40	2.34	2.19

Table A20: Compaction value of Blended fresh aggregate and 20% RAA

Project (site): Around Police Station(jimma)	Date of sampled: 23/12/2017
Material source: ERCC	Tested date: 2/05/2017
Material description: Base course	Lab:-20% RAA
Test Method: ASTM D 1557	Sampled by:

A	Number of tests		1	2	3	4	5
B	water added %		1.5	3	4.5	6	7.5
C	weight of mold (g)		5269	5269	5269	5269	5269
D	weight of sample+mold(g)		9970.24	10301.29	10492.34	10313.4	10259.6
E	weight of wet sample (g)		4701.2	5032.29	5223.34	5044.4	4990.6
F	mold volume (cm^3)		2124	2124	2124	2124	2124
G	wet density(g/cc)	(E/F)	2.21	2.37	2.46	2.37	2.35
	Moisture determination						
H	Cone. no		F-10	A-6	A-8	C-15	A-5
I	wt.cone+wet Sam.(g)		945	1070.5	1164.7	1119.3	1113.6
J	wt.cone+dry Sam.(g)		867.0	978.3	993.1	1025.2	1078.3
K	weight of water (g)	(I-J)	27	34	40	45	49
L	weight of cone. (g)		367.0	395.0	458.0	382.0	364.0
M	Weight of dry Sam. (g)	(J-L)	551.02	641.51	667.67	692.31	720.6
N	Moisture content (%)	(K/M)*100	4.9	5.3	**6**	6.5	6.8
O	Dry density (g/cc)	(G/((N/100)+1))	2.11	2.25	2.32	2.23	2.2

Table A21: Compaction value of blended fresh aggregate and 50% RAA

Project (site): Around Police Station(jimma)	Date of sampled: 23/12/2017
Material source: ERCC	Tested date: 3/05/2017
Material description: Base course	Lab:-50% RAA
Test Method: ASTM D 1557	Sampled by:

A	Number of tests		1	2	3	4	5
B	water added %		3	4.5	6	7.5	9
C	weight of mold (g)		5269	5269	5269	5269	5269
D	weight of sample+mold(g)		9963.04	10132.96	10345.36	10217.92	10154.2
E	weight of wet sample (g)		4694.04	4863.96	5076.36	4948.92	4885.2
F	mold volume (cm^3)		2124	2124	2124	2124	2124
G	wet density(g/cc)	(E/F)	2.21	2.29	2.39	2.33	2.30
	Moisture determination						
H	Cone. no		F-10	A-6	A-8	C-15	A-5
I	wt.cone+wet Sam.(g)		987.2	1010.1	1083.3	1070.0	1081.7
J	wt.cone+dry Sam.(g)		953.2	973.1	1041.3	1022.0	1026.7
K	weight of water (g)	(I-J)	34	37	42	48	55
L	weight of cone. (g)		367.0	395.0	458.0	382.0	364.0
M	Weight of dry Sam. (g)	(J-L)	586.21	578.13	583.33	640	662.65
N	Moisture content (%)	(K/M)*100	5.8	6.4	**7.2**	7.5	8.3
O	Dry density (g/cc)	(G/((N/100)+1))	2.09	2.15	2.23	2.17	2.12

Table A22: Compaction value of 100% RAA

Project (site): Around Police Station(jimma)	Date of sampled: 23/12/2017
Material source: ERCC	Tested date: 4/05/2017
Material description: Base course	Lab:-100% RAA
Test Method: ASTM D 1557	Sampled by:

A	Number of tests		1	2	3	4	5
B	water added %		4.5	6	7.5	9	10.5
C	weight of mold (g)		5269	5269	5269	5269	5269
D	weight of sample+mold(g)		9856.84	10069.24	10159.2	10175.44	10175.44
E	weight of wet sample (g)		4587.84	4800.24	4885.2	4906.44	4906.44
F	mold volume (cm^3)		2124	2124	2124	2124	2124
G	wet density(g/cc)	(E/F)	2.16	2.26	2.30	2.31	2.31
	Moisture determination						
H	Cone. no		F-10	A-6	A-9	C-15	A-5
I	wt.cone+wet Sam.(g)		963.8	1019.1	1015.5	1094.2	1067.1
J	wt.cone+dry Sam.(g)		925.8	976.1	967.5	1037.2	1006.1
K	weight of water (g)	(I-J)	38	43	48	57	61
L	weight of cone. (g)		367.0	395.0	367.5	382.0	364.0
M	Weight of dry Sam. (g)	(J-L)	558.82	581.1	600	655.17	642.11
N	Moisture content (%)	(K/M)*100	6.8	7.4	**8**	8.7	9.5
O	Dry density (g/cc)	(G/((N/100)+1))	2.02	2.10	2.14	2.12	2.11

Table A23: CBR Value of Fresh Aggregate

TEST METHOD: ASTM D 1883

Soaking condition	56 Blows/Layer	
	B.soak	Soaked
Mold number		P-10
Weight of mold + Wet. sample (g)	11526.46	11517.04
Weight of mold (g)	6292	6292
Weight of wet soil (g)	5234.46	5234.6
Mold Volume (cm^3)	2124	2124
Wet density (g/cm^3)	2.46	2.46
Dry density (g/cm3)	2.33	**2.30**

Moisture determination	56 Blows/Layer	
	before	After
Container number	A-7	D-2
Wt. Wet. sample + container (g)	668.86	683.87
Dry sample + container (g)	658.86	665.87
Weight of water (g)	10	18
Weight of container (g)	383	405
Weight of Dry sample (g)	275.86	260.87
Moisture content (%)	5.8	6.9

Swell Reading	Swell elapsed (date)	Soaking time @	56 Blows/Layer	
			mm	swell %
Initial Rdg.	From: 8/05/2017		0.2	0.12
Finally Rdg.	To: 12/05/2017		0.34	
Swell Difference after 94 hours (Mold Height=116.4)		MM	0.14	

Penetra. (mm)	std. load (KN)	56 Blows/Layer		
		Dial rdg.	Load(KN)	CBR %
0.00		0	0.00	
0.64		222	5.03	
1.27		369	8.36	
1.96		444	10.05	
2.54	13.34	565	12.8	96
3.18		672	15.22	
3.81		778	17.63	
4.45		856	19.39	
5.08	20.1	950	21.51	107
7.62		1096	24.83	

Table A24: CBR Value of Blended Fresh Aggregate with 20%RAA

TEST METHOD: ASTM D 1883

Soaking condition	56 Blows/Layer	
	B.soak	Soaked
Mold number	D-8	
Weight of mold + Wet. sample (g)	11475.6	11411.8
Weight of mold (g)	6378	6378
Weight of wet soil (g)	5097.6	5097.6
Mold Volume (cm^3)	2124	2124
Wet density (g/cm^3)	2.4	2.37
Dry density (g/cm3)	2.25	**2.2**
Moisture determination	56 Blows/Layer	
	before	After
Container number	A-7	D-2
Wt. Wet. sample + container (g)	645.15	695.8
Dry sample + container (g)	629.15	671.8
Weight of water (g)	16	24
Weight of container (g)	383	356
Weight of Dry sample (g)	246.15	315.8
Moisture content (%)	6.5	7.6

Swell Reading	Swell elapsed (date)	Soaking time @	56 Blows/Layer	
			mm	swell %
Initial Rdg.	From: 8/05/2017		0.22	0.27
Finally Rdg.	To: 12/05/2017		0.53	
Swell Difference after 94 hours (Mold Height=116.4)		MM	0.31	

Penetra (mm)	std. load (KN)	56 Blows/Layer		
		Dial rdg	Load(kN)	CBR %
0.00		0	0.00	
0.64		205	4.65	
1.27		279	6.32	79
1.96		364	8.25	
2.54	13.34	465	10.54	
3.18		516	11.68	
3.81		592	13.42	
4.45		679	15.39	
5.08	20.1	817	18.5	92
7.62		887	20.10	

Table A25: CBR Value of Blended Fresh Aggregate with 50% RAA

TEST METHOD: ASTM D 1883

Soaking condition	56 Blows/Layer	
	B.Soak	Soaked
Mold number	P-25	
Weight of mold + Wet. sample (g)	11164.2	11148.93
Weight of mold (g)	6279	6279
Weight of wet soil (g)	4885.2	4869.93
Mold Volume (cm^3)	2124	2124
Wet density (g/cm^3)	2.3	2.29
Dry density (g/cm3)	2.13	**2.10**
Moisture determination	56 Blows/Layer	
	Before	After
Container number	A-7	D-2
Wt. Wet. sample + container (g)	673.23	814.11
Dry sample + container (g)	652.23	786.11
Weight of water (g)	21	28
Weight of container (g)	383	475
Weight of Dry sample (g)	269.23	311.11
Moisture content (%)	7.8	9

Swell Reading	Swell elapsed (date)	Soaking time @	56 Blows/Layer mm	swell %
Initial Rdg.	From: 8/05/2017		0.22	0.48
Finally Rdg.	To: 12/05/2017		0.78	
Swell Difference after 94 hours (Mold Height=116.4)		MM	0.56	

Penetra. (mm)	std. load (KN)	56 Blows/Layer Dial rdg	Load(kN)	CBR %
0.00		0	0.00	
0.64		117	2.65	
1.27		179	4.05	
1.96		307	6.35	
2.54	13.34	365	8.27	62
3.18		407	9.21	
3.81		500	11.32	
4.45		571	12.34	
5.08	20.1	667	15.1	75
7.62		751	17.02	

Table A26: CBR Value of 100% RAA

TEST METHOD: ASTM D 1883

Soaking condition	56 Blows/Layer	
	B.soak	Soaked
Mold number	D-10	
Weight of mold + Wet. sample (g)	11145.8	11103.2
Weight of mold (g)	6473	6473
Weight of wet soil (g)	4672.8	4630.2
Mold Volume (cm^3)	2124	2124
Wet density (g/cm^3)	2.2	2.18
Dry density (g/cm3)	2.03	**1.98**
Moisture determination	56 Blows/Layer	
	before	After
Container number	A-7	D-2
Wt. Wet. sample + container (g)	736.58	780
Dry sample + container (g)	708.58	745
Weight of water (g)	28	35
Weight of container (g)	383	395
Weight of Dry sample (g)	325.58	350
Moisture content (%)	8.6	10.0

Swell Reading	Swell elapsed (date)	Soaking time @	56 Blows/Layer	
			mm	swell %
Initial Rdg.	From: 8/05/2017		0.26	1.03
Finally Rdg.	To: 12/05/2017		1.46	
Swell Difference after 94 hours (Mold Height=116.4)		MM	1.26	

Penetra. (mm)	std. load (KN)	56 Blows/Layer		
		Dial rdg	Load(kN)	CBR %
0.00		0	0.00	
0.64		82	1.85	
1.27		95	2.15	
1.96		217	4.32	
2.54	13.34	234	6.67	50
3.18		350	7.93	
3.81		402	9.10	
4.45		457	10.35	
5.08	20.1	559	12.66	63
7.62		629	14.25	

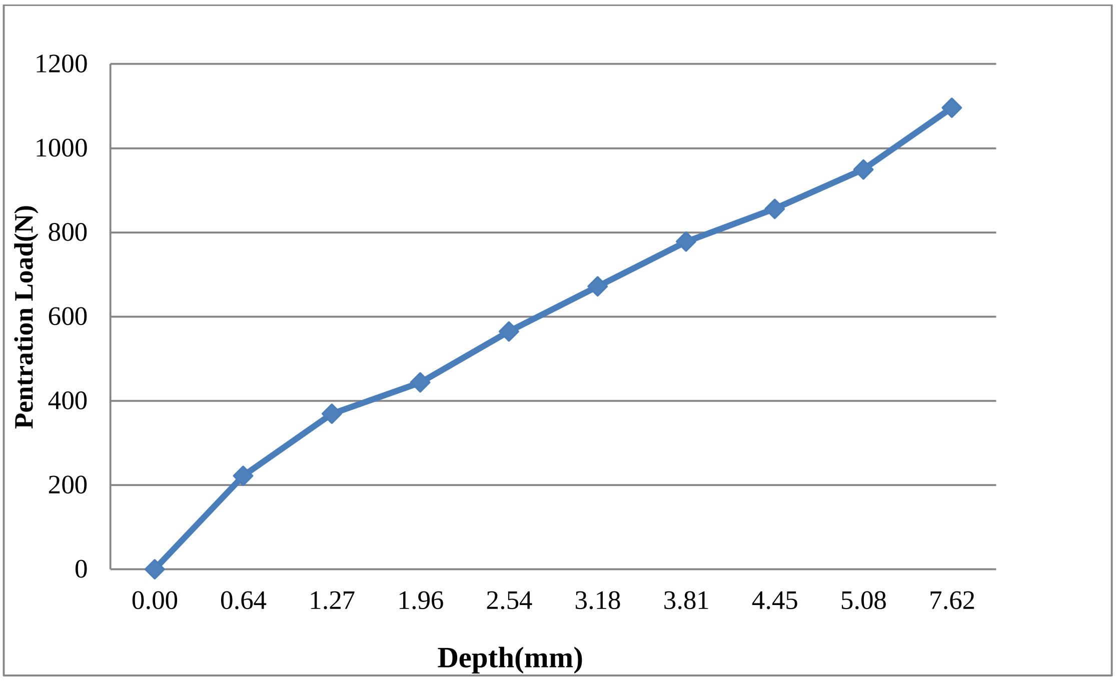

Figure A1: The relationship between penetration load and depth of fresh aggregate

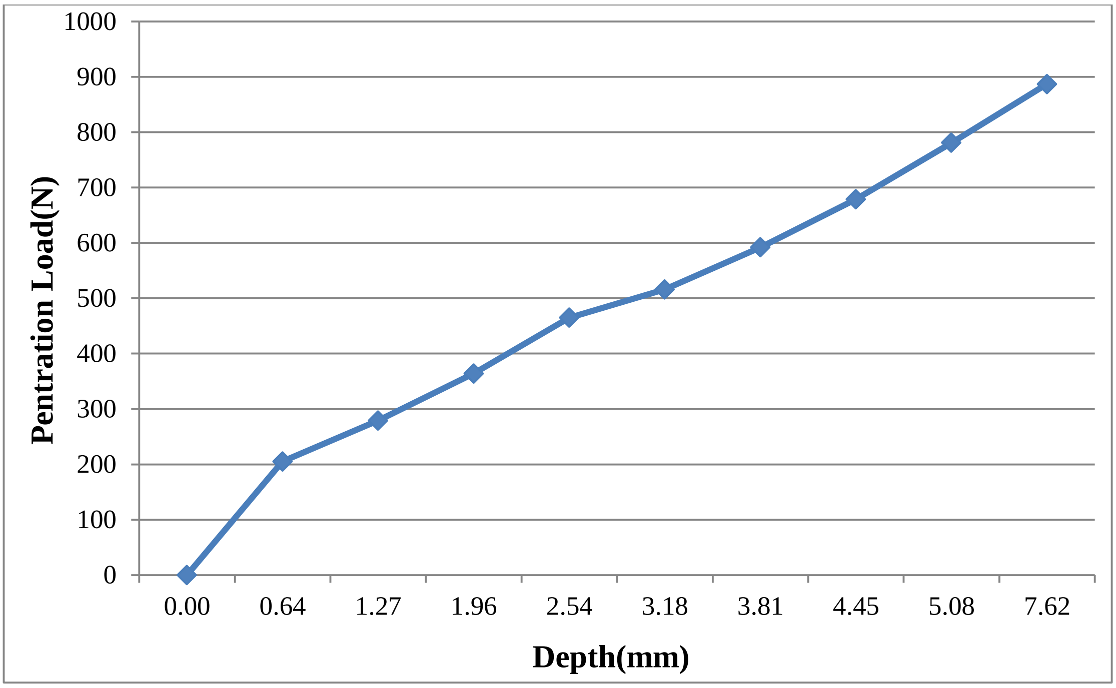

Figure A2: The relationship between penetration load and depth of Blended fresh aggregate with 20% RAA

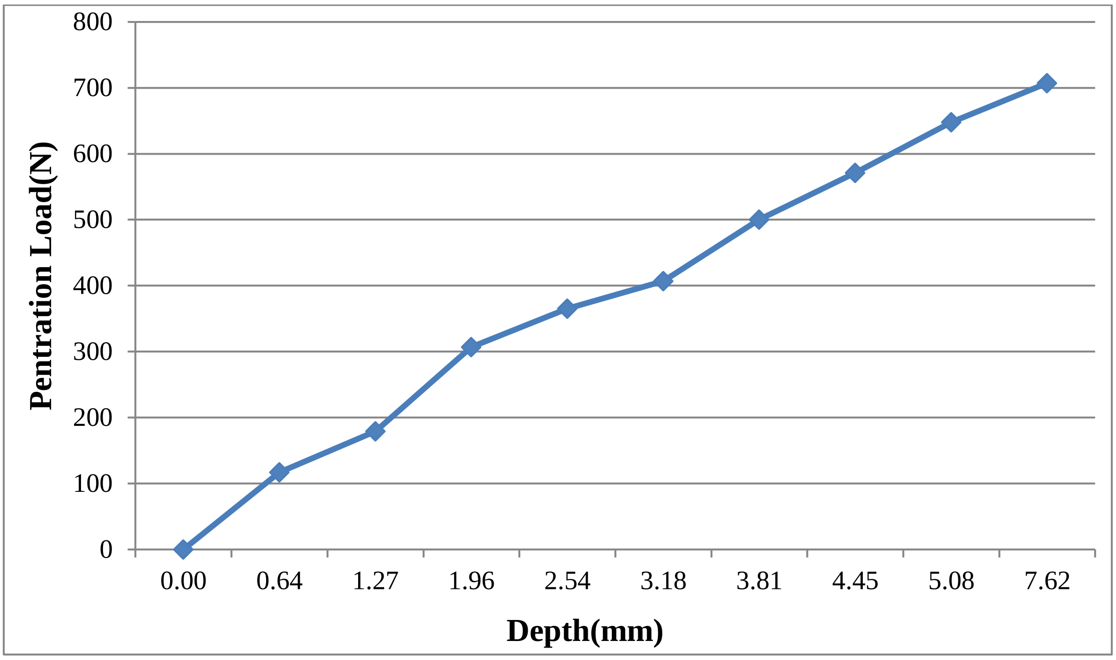

Figure A3: The relationship between penetration load and depth of Blended fresh aggregate with 50% RAA

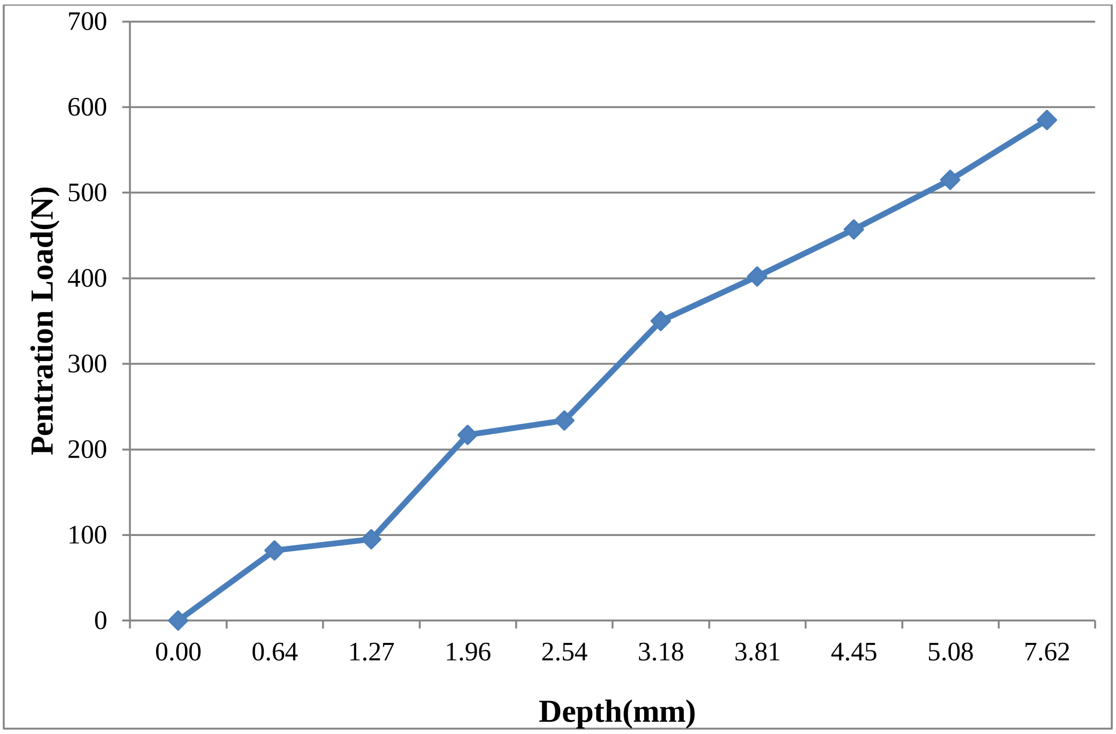

Figure A4: The relationship between penetration load and depth of 100% RAA

APPENDEX B: DIFFERENT PHOTOS ON THE LABRATORY


19/4/2017


19/4/2017


14/2/2017

Figure B2: Fresh aggregate sample


1/2/2017


30KGS
9/2/2017


13/2/2017


4/01/2017


1/2/2017


9/2/2017


9/2/2017


9/2/2017


9/2/2017




24/4/2017


24/4/2017


26/4/2017


24/4/2017


4/5/2017


25/4/2017


A9
4/5/2017


4/5/2017


4/5/2017


4/5/2017


CBR Tester
8/5/2017


old
8/5/2017
CBR Tester