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Research Article

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Effect of Air Voids Content on Ageing Characteristics of Bituminous Mix

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Abstract Highway pavement provides smooth surface over which vehicles can move safely from one place to another. Yet this vital need is sometimes not achieved due to failure of pavements before its design life which is common. Aging is one of the most important and dynamic characteristics of asphalt materials.Certain parameters are found to be essential in a comprehensive understanding of aging characteristics of asphalt binders such as air voids. Air voids determination as it affects the aging of the bituminous mix were conducted in intervals of five (5) years (5 years asphalt, 10 years asphalt, 15 years asphalt and 20 years asphalt). A fresh asphalt mix which served as control was prepared and subjected to Marshall Test, density determination, void content and binder content determination. The same test conducted on fresh asphalt concrete (including bitumen extraction test) was conducted on asphalt sample of 5 years, 10 years, 15 years and 20 years. A total of 15 samples (12 collected from different pavements within the age ranges and 3 samples from fresh asphalt mix) were tested to simulate the effect of air voids as the pavements ages towards its design life. The initial values of the parameters for the pavement considered were obtained from the Ministry of Works, AkwaIbom State, which were used as basis for comparisons. The study shows that the voids content of the different ages considered fell between 7.0 – 2.2 % with the fresh asphalt having the highest void content and the asphalt mix of 20 years having the lowest. The unit weight of the specimens ranged from 2.20g/cm^3 – 2.31g/cm^3 with the fresh asphalt having the lowest unit weight and the asphalt mix of 20 years having the highest. The binder content of the specimens ranged from 4.15 – 5.34% with the fresh asphalt mix having the lowest and the asphalt mix of 20 years having the highest. The stability of these specimens was seen, ranging from 3.78kN – 10.61kN. The fresh asphalt sample has the lowest stability while the asphalt mix of 20 years has the highest. The difference in void content could have been due to the viscoelastic behavior of the asphalt material that is; its properties are a function of time (or frequency) and temperature. The difference in unit weight of the samples must have been due to initial density of the material and traffic load. The difference in the binder content of the specimens could be as a result of oxidation and volatilization of the mix. As the asphalt binder increases in age, its viscosity increases and it becomes more stiff and brittle. The difference in stability of these specimens could be attributed to the internal friction and cohesion of the material. It is worthy of note that air voids contribute to the failure of flexible pavements before its design life.

Keywords Ageing, Void Content, Bituminous Mix, Binder Content, Stability, Asphalt

1. Introduction

1.1. Background of the study

Highway pavement is constructed to provide smooth surface over which vehicles can move safely from one place to another. Failure of pavements before its design life is common and the trend continues unabated. Road deterioration is still a major problem in Nigeria to this day. It is influence by many factors; (a) overloaded vehicles, (b) poor construction quality and (c) inadequate maintenance. At present, sustainability of pavement continues to pose a serious issue of concern to pavement researchers and engineers. In view of this, the changes of Asphalt Concrete's (AC's) properties with the age of pavement are an interesting area of research. Aging being one of the most important and dynamic characteristics of asphalt materials, helps predict the life of a flexible pavement owing to the adhesive characteristics of binders with aggregate particles. As the pavement ages, the asphalt mix components including binders oxidize, and volatilization of oils takes place, which render asphalt mixes higher stiffness, thus, increasing viscosity. Asphalt concrete, a high quality bituminous surfacing material consists of a mixture of aggregates continuously graded from a maximum size of 25mm to a fine filler of about 0.075mm size. Sufficient asphalt is added so that the mix when compacted will be impervious and will have viscous and elastic properties. The asphalt concrete mix is designed to have sufficient air voids in the total compacted mix to allow for a slight amount of additional compaction under traffic loading. A freshly laid asphalt concrete layer has air voids in the range of 5–8 percent [56]. Due to the traffic loading, consolidation occurs in the asphalt concrete layers due to two mechanisms. They are one-dimensional densification and plastic flow. One-dimensional densification occurs due to the reduction in air voids volume only, $V_a$in Figure 1.1. In the plastic flow mechanism, consolidation occurs due to reduction of the total voids in the mineral aggregates volume, V_{ma} in Figure 1.1.

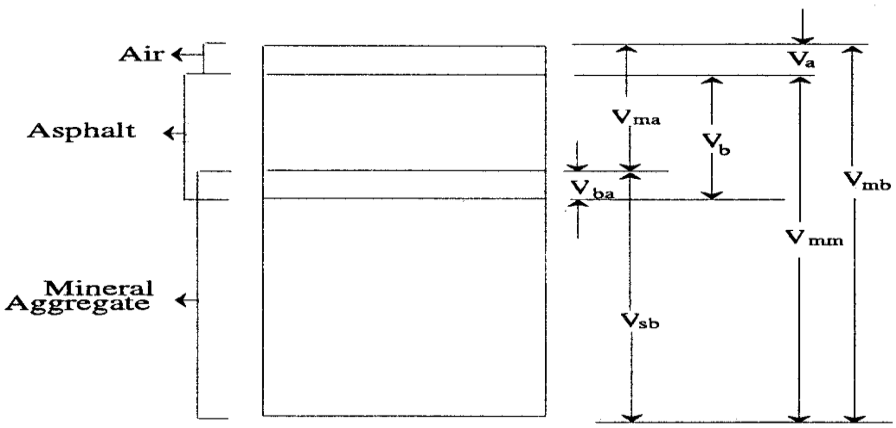

Figure 1.1: Representation of volumes in a compacted asphalt concrete specimen
Source: Krishnan and Rengaraju (1998) [56]

Where,
V_{ma}= vol. of voids in mineral aggregate.
V_{mb}= vol. of compacted mix,
V_a= vol. of air voids,
V_{mm}= void-less vol. of paving mix,
V_b= vol. of asphalt,
V_{ba}= vol. of absorbed asphalt,
V_{sb}= vol. of mineral aggregate

These two are separate phenomena and depending upon the temperature viscosity relationship of asphalt, the contribution of each of them to the overall consolidation varies. However, pavement performance studies [56] have shown that when the air voids (volume V_a) reduce to less than 3 percent, it is the plastic flow mechanism which predominates. The reason for this is the building up of pore pressure in the air void space due to which some amount of asphalt is forced to flow. The flow of asphalt into the voids and reduction in the asphalt film thickness will ultimately result in reduction of relative distance between aggregate particles. Therefore particle reorientation may be caused by the flow of asphalt into the voids. This relocation of particles can only occur after friction between the particles is overcome. In fact, some mixes that have adequate internal friction and cohesion at the time of construction can become over lubricated(through a reduction in air voids) due to additional consolidation. Due to this over lubrication, the strength and shear properties of the mix reduce greatly.

Air void, being one of the properties of asphalt concrete may change due to the washing out of the asphalt binder from the asphalt concrete due to moisture damage. The determination of the air void as the pavement ages is important to verify the damage or degradation of asphalt. Once this is ascertained, the issue can be remedied thereby ensuring sustainable pavement. The importance of air voids cannot be undermined, this is because as air void is a parameter in the design life calculation, any change in air void due to aging can incur wrong prediction of probable service life, and hence, sustainable pavement design may be inhibited. Therefore for sustainable pavement, determination of change in air void with aging is important. Also, the importance of air void is the dependency of coefficient of thermal contraction and expansion (CTC/CTE) on air void. Previous works shows that the CTC/CTE of asphalt concrete is dependent on air void [40]. Note that the CTC/CTE is used to determine the thermal stress under rapid cooling of asphalt concrete in raining season, which is subsequently used in thermal cracking prediction model. Therefore, changes in air void of asphalt content with aging may incur wrong prediction of thermal cracking in pavement. The emission of volatile components of asphalt concrete and chemical reaction of asphalt binder with oxygen may cause a change in the air void content of asphalt concrete too. In flexible highway pavement construction, asphalt concrete used has about 5–8 percent air voids immediately after laying of the roadway. Constitutive laws for asphalt concrete developed till now have modeled the mix as a linear elastic or viscoelastic material and have not taken into account the effect of void concentration on the mechanical behavior of the material.

This study investigated this issue of changing the air void of asphalt concrete with aging. Asphalt concrete is a composite material consisting of asphalt binder and aggregates. Asphalt binder contains hydrocarbon molecules. Molecules of hydrocarbon are likely to react with oxygen, water and vapour available in the environment. Oxygen reacts with asphalt binder at high temperature while mixing with aggregates in the mixing plant and during construction period, which is referred to as the short-term aging. After pavement construction, oxygen continuously reacts with asphalt binder which is considered as the long-term aging. Aging makes binder hard and brittle that result in an increase in the stiffness of asphalt concrete. This oxidation has a negative impact on pavement durability and stiffness. Effects of long-term aging on the rheological properties of asphalt binder are well explored in literatures [19, 39-40, 55]. The works cited above considered the change in the mechanical properties, shear modulus, and viscosity of aged binder due to laboratory oven aging. There is not a single study which deals with the air void of asphalt concrete due to aging as it affects strength, integrity and durability of the concrete. However, change in air void content causes changes in the mechanical properties and performances of asphalt concrete. Study of air void with aging is important as it will greatly enhance the sustainability of pavement when incorporated during design.

1.2. Volumetrics in Asphalt Mixtures

1.2.1. Theory of Air Voids

The dynamic modulus of asphalt mixture stiffness is a significant parameter that determines the ability of material to resist compressive deformation as it is subjected to cyclic compressive loading and unloading [54]. The material's ability to resist compressive deformation is a major contributor to pavement resistance to cracking due to cyclic or other types of loading and loading configurations. The cross sectional size of the material that can be utilized for load transmission could be reduced by air voids. It is therefore a reasonable argument to think that air voids play significant roles in the behaviour of fatigue related cracking experienced by road pavements during useful or design life since air voids in the pavement determines stiffness [38]. The Asphalt Institute developed a method for design in which the dynamic modulus is determined from the following equations, as presented in Huang's Pavement Analysis and Design textbook (1993):

$E^* = 100{,}000(10^{\beta_1})$

$\beta_1 = \beta_3 + 0.000005\beta_2 - 0.00189\beta_2 f^{-1.1}$

$\beta_2 = \beta_4^{0.5} T^{\beta_5}$

$\beta_3 = 0.553833 + 0.028829\,(P_{200} f^{-0.1703}) - 0.03476 V_a + 0.07037\lambda + 0.931757 f^{-0.02774}$

$\beta_4 = 0.483\, V_b$

$\beta_5 = 1.3 + 0.49825 \log f$

Where;

E* = dynamic modulus (psi)

F =loading frequency (Hz)

T = temperature (°F)
V_a = volume of air voids (%)
λ = asphalt viscosity at 77°F (10^6 poises)
P_{200} = percentage by weight of aggregates passing No. 200 (%)
V_b = volume of bitumen
$P_{77°F}$ = penetration at 77°F or 25°C

1.2.2. Percent Voids Filled with Asphalt (VFA)

The percentage of voids in the compacted aggregate mass that are filled with asphalt cement is known as the VFA. It is synonymous with the asphalt-void ratio. The VFA property is important not only as a measure of relative durability, but also because there is an excellent correlation between it and percent density. If the VFA is too low, it reflects inadequate asphalt to provide durability and to over-densify under traffic and bleed. In view of this, the VFA is a very important design property and requires proper attention. Most DOT specifications require 70-80 during the design phase; this requirement is intended for the mix during the design phase only and is typically not a production requirement. HMA designed for moderate to heavy traffic may not pass the VFA requirement with a relatively low percent of air voids in the field even though the amount of air voids is within the acceptable range. Because low air void contents may be very critical in terms of resisting permanent deformation, the VFA requirement helps to avoid those mixes that are susceptible to rutting in heavy traffic situations. The allowable air void content for HMA that are near the minimum VMA criteria is also restricted by VFA. HMA designed for lower traffic volumes may not pass the VFA requirement with a relatively high percent air voids in the field even though the air void requirement range is met. Adequate amount of VFA is necessary to avoid less durable HMA resulting from thin films of binder on the aggregate particles in light traffic situations.

1.2.3. Voids in the Mineral Aggregate (VMA)

The air-void spaces that exist between the aggregate particles in a compacted paving mixture, including spaces filled with asphalt are referred to as voids in the mineral aggregate (VMA). The space that is available to accommodate the asphalt and the volume of air voids necessary in the mixture is represented by VMA. The higher the quantity of VMA in the dry aggregate, the higher the space available for the film of asphalt. Based on the fact that the thicker the asphalt film on the aggregate particles the more durable the mix, specific minimum requirements for VMA are specified in most specifications. To achieve a durable asphalt film thickness, minimum VMA values should be adhered to. Increasing the density of gradation of the aggregate to a point where below minimum VMA values are obtained leads to thin films of asphalt and a dry looking, low durability mix. Therefore, economizing in asphalt content by lowering VMA is actually counter-productive and detrimental to pavement quality and should be avoided.

1.2.4. Asphalt Content

Of importance is the proportion of asphalt in the mixture and must be accurately determined in the laboratory and then precisely controlled on the job. The optimum asphalt content of a mix is largely dependent on aggregate characteristics such as gradation and absorptiveness. On the other hand, aggregate gradation is directly related to optimum asphalt content. The finer the mix gradation, the larger the total surface area of the aggregate and the greater the amount of asphalt required to uniformly coats the particles. Coarser mixes have less total aggregate surface area, hence they demand less asphalt.

1.2.5. Tensile Strength Ratio

The strength loss resulting from damage caused by "stripping" under laboratory controlled accelerated water conditioning is measured using the tensile strength test. Long-term susceptibility to stripping of an asphalt concrete can be predicted from results obtained in the test. To contend the effects of water damage, an anti-stripping additive is used in all asphalt mixes. The Contractor, in most cases is required to use 1.0 percent hydrated lime in the mixture.

1.3. Statement of Problem

Asphalt binders get stiffer and become brittle with age, which is referred to as asphalt aging or hardening. The performance of asphalt mixtures are affected by rheological properties that have a direct impact on them. Asphalt hardening often leads to fatigue cracking and eventually pavement failure with heavy and repeated traffic loading [9]. For design of sustainable pavement, accurate determination of the deterioration/change of properties of Asphalt Concrete (AC) with service life is essential. This is because, properties of asphalt concrete change with the service life due to the continuous aging. Aging, being one of the most important and dynamic characteristics of asphalt materials helps predict the life of a flexible pavement owing to the adhesive characteristics of binders with aggregate particles. Effects of aging on stiffness, fatigue life, etc. are well known; [19, 40], whereas, effect of aging on the air void of asphalt concrete as it affects strength, integrity (inherent properties) and durability of the concrete is still an unknown issue. This study investigated the effects of aging on air void of asphalt concrete sample.

1.4. Specific Objective

The specific objective of this research is to study the effect of air voids content in aging of bituminous mix.

1.4.1. Objectives of the Study

- To investigate how air voids affect the strength of asphalt concrete with age.
- To examine the effect of air voids on the inherent properties of the bituminous material.
- To study the influence of air voids on the performance of some parameters such as binder content, unit weight, voids filled with bitumen and stability as the pavement ages.

1.5. Scope of the Study

This research work studied the effect of air voids as it affects the strength, inherent properties of asphalt concrete and the performance of some parameters (binder content, voids filled with bitumen, unit weight of sample) as the pavement advance in age.

1.6. Justification of the Study

Age hardening of asphalt mixtures is an irreversible process, which contributes to a reduction of the durability of pavements and eventually increases the maintenance cost. The failure of highway pavements before its design life is on the increase, this is worrisome as the trend continues unabated and this affects the economic fortunes of Nigeria. Certain parameters are found to be essential in a comprehensive understanding of aging characteristics of asphalt binders such as air voids. Thus, there is a need to investigate the effect of these parameters on aging in order to predict pavement life over its design period. The determination of the air void with the age of the pavement is important to determine the damage or degradation of asphalt and this will greatly enhance the sustainability of pavement when incorporated during design.

1.7. Limitations of the Study

The work shall be limited to laboratory testing, and shall consider using 80-100 pen grade of bitumen for analysis. The research shall also be limited to asphalt-aggregate mixture aging procedures because these will inherently take air voids (permeability) and asphalt-aggregate chemical interaction into account.

2. Research Methodology

2.1 Materials

The materials used for this study were bitumen, cement, coarse and fine aggregates. The bitumen was obtained from the Ministry of Works in AkwaIbom State, Nigeria. The cement used was the Portland Limestone Cement. After sampling of the materials, laboratory test such as sieve analysis of the aggregates, grading of bitumen, compaction, bitumen extraction and Marshall Stability test were conducted.

2.1.1. Bitumen/Binder

The grade of bitumen used in this study was 80-100 PEN bitumen with average penetration value of 81 PEN. The selection of bitumen was based on conventional bitumen grade suggested by the local authority, as stated in Standard Specification for Road Work (Public Works Department 2008). The bitumen was used as the binder material.

2.1.2. Aggregate and Gradation

The aggregates used in this study were collected from one local asphalt mixing plant in order to control the quality and properties throughout the study. Granite aggregates supplied by Ministry of works was used throughout this investigation. Prior to aggregate batching, the aggregates were washed, dried and sieved into their respective size ranges. Their specific gravity and water absorption rate were determine, and also ensured it is in conformity with the specifications.

2.2. Methods

2.2.1. Ageing with respect to pavement lifespan

To determine the effect of air voids as the pavement advances in age, test samples were prepared in batches. A fresh asphalt concrete was prepared with a mix ratio of 19.5% for aggregate sizes of 5-15mm(1/2 inch), 80.5 % for aggregate size of 0 – 5mm(fine aggregate and filler), and 4.73% bitumen content. These samples were subjected to Marshall Test, density determination, stability, void content and percentage binder content test. Samples were also collected from existing pavement with a life span of 5 years interval. These samples were subjected to the test mentioned above including bitumen extraction test to simulate the effect of air voids as the pavement ages towards its design life.

2.2.2. Marshall Stability Test

A standard Marshall mould with an average height of 65mm and diameter of 100mm was used in preparation of specimens using Portland Limestone Cement as filler in this investigation. The 80-100 PEN bitumen was used as binder for mix preparation. The aggregates were first mixed into batches according to the designated gradations and weight. Those batches were then heated in an oven at the designated mixing temperature for about 4 hours before the mixing process. A specified amount of bitumen was used to mix with the heated aggregate batches. The samples were prepared in 100mm diameter moulds which were fitted with a base and collar. The mixes were compacted (plate 2.1) with 75 blows on each side using a hammer consisting of a sliding weight which falls onto a circular foot held on a hardwood block which is rigidly fixed to a concrete base during compaction. After compaction, the specimens were removed from the molds and allowed to cool overnight as seen in plate 2.2. The samples were removed from the mould using an extraction plate and press and heated to the test temperature of 60°C in a water bath.

Plate 2.1: Compaction of specimen

Plate 2.2: Compacted fresh asphalt specimen

The samples were then inserted into the breaking head (breaking head consists of upper and lower cylindrical segments or test heads having an inside radius curvature of 5cm as seen in plate 2.3). The longer segment is mounted on a base having two perpendicular guide rods which facilitate insertion in the holes of upper test segment using the CBR tester for flow and stability test. This is illustrated in plate 2.4. The stability of the mix is defined as the maximum load carried by a compacted specimen at a standard test temperature of 60^0C. During stability test, flow is measured as the deformation in units of 0.25mm between zero load and maximum load carried by the specimen (flow value may also be measured by deformation units of 0.1 mm). This test ensures that the optimum binder content for the aggregate mix type and traffic intensity is gotten.

Plate 2.3: placement of specimen in mold

Plate 2.4: placement of specimen in breaking head (CBR tester machine)

2.2.3. Voids Content Determination

The air voids characteristics of asphaltic concrete was determined based on Marshall Mix design as outlined in ASTM D6927 (2015). The procedure was aimed at determining a number of parameters including voids in total mix, voids filled with bitumen, and voids in mineral aggregate. This was achieved through the water absorption method. The specimens were oven dried for a specified time and temperature and then allowed to cool in a desiccator. Immediately upon cooling the specimens were weighed. They were then submerged in water at 23°C for 24 hours. Specimens were removed, dried of moisture with a lint free cloth, and weighed. The result is as presented in section 3.

2.2.4. Density Determination

The compacted samples were weighed in air and recorded. The samples were also placed in water with the help of a string and bucket of water, their weights were recorded. The results are presented below.

2.2.5. Bitumen Extraction Test

Bitumen extraction is done to ascertain the quantity of bitumen present in an asphaltic concrete and if the bitumen content is within the required specification of 5 - 8% for wearing course and 4.5 – 6.5 % for binder course. Some portions of the compacted samples were heated to a temperature of about 120^{o}c to melt. About 1000g of the sample was weighed into the extractor (plate 2.5). It was covered with a filter paper and trichloroethylene solution was added as seen in plate 2.6, this is in accordance with AASHTO R 30 specification. The extractor machine was powered. The liquid content was collected through a collection plate. The procedure was repeated until the bitumen was completely extracted. The machine was stopped and the washed samples collected as shown in plate 2.7. It was then placed in an oven for drying. Then it was sieve to identify the aggregate size and to ascertain if it falls within the required grading envelop. The results are discussed below.

Plate 2.5: Before extraction

Plate 2.6: During extraction

Plate 2.7: After extraction

2.2.6. Asphalt Coring

Core testing is an extremely precise tool used to determine what method of pavement repair is needed. A sample of the pavement core is taken directly, giving insight into exactly what is going on with the subgrade layers of the pavement and identify potential problems before they occur. This also allows the drainage capacity of underlying soil to be tested, which will determine if and where drains are needed to maximize the lifespan of the pavement thereby developing the most accurate pavement management plan possible.

Core testing determines the following parameters: Pavement Thickness, Bond between pavement layers, Soil type under pavement, Drainage characteristics. Asphalt coring was conducted on some selected pavements in AkwaIbom State (plate 2.8). About 12 existing pavements were cored for test samples. Pavements were cored within the range of 5 years, 10 years, 15 years and 20 years respectively, representing the pavement lifespan of 20 years. Plate 2.10 shows some of the samples obtained through coring.

Plate 2.8: During asphalt coring

Plate 2.9: Asphalt cored hole

Plate 2.10: Cored specimens

The holes created during coring (as seen in plate 2.9) was filled with fresh bitumen at a laying temperature of 160^{o}c and compacted to avoid failure of the pavement through the hole (plate 2.11).

Plate 2.11: Replacement of cored hole with fresh aslphalt

Plate 2.12: After compaction of cored samples

After coring of the required samples, the same test conducted on the fresh asphalt specimen was conducted on them. The samples shown in plate 2.12 was gotten. The results obtained form a basis for comparison and discussion.

2.3. Data Collection

To aid in accurate comparison of the bituminous mix parameters obtained through experimental procedures, initial bituminous mix values (during production stage) was obtained in the AkwaIbom State Ministry of works. These values were compared to the obtained values after test.

2.4. Regression Analysis

In statistical modeling, regression analysis is a set of statistical processes for estimating the relationships between a predictor (dependent variable) and one or more criterion (independent variables). Regression analysis as a powerful statistical tool was used to examine the relationship between age of bituminous mix which served as the predictor and void content, binder content, unit weight of sample, voids filled with bitumen and stability, which served as criterion.

3. Results and Discussion

3.1. Mix proportion

The fresh asphaltic concrete had 19.5% for aggregate sizes of 5-15mm, 80.5% for aggregate size 0-5mm (made up of fine aggregate and filler material) and a bitumen content of 4.73%. The mix proportion for 5, 10, 15 and 20 years asphalt were 25%, 27%, 23% and 22% for aggregate size 5-15mm, 75%, 73%, 77% and 78% for aggregate size 0-5mm and 4.24%, 4.72%, 5.06% and 5.34% for bitumen content as shown in table 3.1. The variations in the mix proportion could have been due to the choice and availability of aggregate and the contractor's discretion. It is worthy of note that the mix proportions fell within the recommended range (BS EN 13043).

Table 3.1: Asphalt mix proportion for wearing course asphaltic concrete

Aggregate in mix	Age of asphaltic concrete				
	Fresh Asphalt	5 years	10 years	15 years	20 years
15 – 22 mm [3/4 inch]	-	-	-	-	-
5 – 15 mm [1/2 inch]	19.5 %	25 %	27%	23%	22%
0 – 5 mm [Fine aggt. and filler]	80.5%	75%	73%	77%	78%
Bitumen content	4.73%	4.24%	4.72%	5.06%	5.34%

3.2. Initial Bituminous Mix Properties

The initial bituminous mix properties of the asphalt pavement considered as obtained from the AkwaIbom State Ministry of Works is as presented in Table 3.2.

Table 3.2: Initial asphalt mix properties for wearing course

Properties	Age of Bituminous Mix			
	5 years	10 years	15 years	20 years
Bulk Density (g/cm^3)	1.81	2.18	1.91	2.10
Specific gravity	2.52	2.47	2.58	2.55
Porosity (%)	27.2	27.5	28.2	27.8
Void content (%)	6.5	5.8	6.2	7.0
Compressive strength (N/mm^2)	1.44	1.50	1.48	1.52
Indirect tensile stiffness modulus (N/mm^2)	2738	2840	2780	2795
Binder content	6.8	6.7	7.2	7.4
Unit weight	2.10	1.89	2.20	2.18
Abrasion loss (%)	12.8	13.1	12.6	12.5
Stability	4.50	5.80	8.70	8.50

Source: Ministry of Works, Aks (1999, 2004, 2009, 2014)

3.3. Asphalt Concrete Density/ Void Content Determination

The test conducted on the specimen for density and void content determination yielded the following results: Table 3.3a – 3.3c shows results of density and void content determination for fresh asphalt mix. The core specimen method was used for density determination which gave a unit weight of 2.42. The water adsorption method was used for void content determination which yields a void content of 7.0 (table 3.3a). Same procedure was used for other asphalt samples which yielded 2.28 g/cm^3 for 5 years (table 3.3b); 2.27 g/cm^3 for 10 years (table 3.3b); 2.29 g/cm^3 for 15 years (table 3.3c); and 2.31g/cm^3 for 20 years (table 3.3c). The void content determination yielded 5.3 for 5 years (table 3.3b); 4.2 for 10 years (table 3.3b); 3.5 for 15 years (table 3.3c); and 2.2 for 20 years (table 3.3c) respectively (ASTM D 2172 - 81).

Table 3.3a: Density and void content determination for fresh asphalt mix

Density Determination (Core Specimen Method)				**Void Content Determination (Water Absorption Method)**			
Age of Asphalt Sample	**Fresh Mix**			**Age of Asphalt Sample**	**Fresh Mix**		
Core Location	**N/A**			Core Location	**N/A**		
Sample Identification	**A**	**B**	**C**	Sample Identification	**A**	**B**	**C**
Weight of Sample in Air (g)	1199.2	1199.6	1199.4	Weight of Sample in Air (g)	1199.2	1199.6	*1199.4*
Sample Thickness (cm)	6.80	6.60	6.70	Weight of Sample After Soaking (g)	1207.7	1208.5	1207.5
Weight of Sample in Water (g)	704.0	702.0	703.0	Weight of Water Absorbed (g)	8.5	8.9	8.1
Volume of Sample (cm^3)	495.2	497.6	496.4	Void in Specimen (%)	7.0	7.4	6.7
Unit Weight (g/cm^3)	2.42	2.41	2.42	Av. Void in Specimen (%)	7.0		
AV. Unit Weight (g/cm^3)	2.42						

Table 3.3b: Density and void content determination for asphalt concrete of 5 and 10 years

Density Determination (Core Specimen Method)						
Age of Asphalt Sample	**(5) Years**			**(10) Years**		
Core Location	**1**	**2**	**3**	**1**	**2**	**3**
Sample Identification	**AR-1**	**AR-2**	**AR-3**	**IB-1**	**IB-2**	**IB-3**
Weight of Sample in Air (g)	812.0	654.5	750.2	719.5	1139.0	950.5
Sample Thickness (cm)	3.8	3.1	3.5	4.1	6.9	5.5
Weight of Sample in Water (g)	415.5	364.0	402.5	402.5	641.0	510.5
Volume of Sample (cm^3)	360.5	290.5	320.5	317.0	498.0	420.5
Unit Weight (g/cm^3)	2.25	2.25	2.34	2.27	2.29	2.26
AV. Unit Weight (g/cm^3)	2.28			2.27		

Void Content Determination (Water Absorption Method)						
Age of Asphalt Sample	**(5) Years**			**(10) Years**		
Core Location	**1**	**2**	**3**	**1**	**2**	**3**
Sample Identification	**AR-1**	**AR-2**	**AR-3**	**IB-1**	**IB-2**	**IB-3**
Weight of Sample in Air (g)	676.0	654.5	665.5	974.5	719.5	847.0
Weight of Sample After Soaking (g)	679.5	658.0	669.0	978.5	722.5	850.5
Weight of Water Absorbed (g)	3.5	3.5	3.5	4.0	3.0	3.5
Void in Specimen (%)	5.2	5.3	5.3	4.1	4.2	4.2
Av. Void in Specimen (%)	5.3			4.2		

Table 3.3c: Density and void content determination for asphalt concrete of 15 and 20 years

Density Determination (Core Specimen Method)						
Age of Asphalt Sample	**(15) Years**			**(20) Years**		
Core Location	**1**	**2**	**3**	**1**	**2**	**3**
Sample Identification	**NR- 1**	**NR- 2**	**NR-3**	**Ab-1**	**Ab-2**	**Ab-2**
Weight of Sample in Air (g)	1671.5	1873.5	1780.5	1412.5	1433.0	1425.0
Sample Thickness (cm)	8.0	8.3	8.2	6.2	6.4	6.3
Weight of Sample in Water (g)	941.5	1063.5	985.5	800.0	812.5	806.5
Volume of Sample (cm^3)	730.0	810.0	780.0	612.5	620.5	615.5
Unit Weight (g/cm^3)	2.29	2.31	2.28	2.31	2.31	2.32
AV. Unit Weight (g/cm^3)	2.29			2.31		

Void Content Determination (Water Absorption Method)						
Age of Asphalt Sample	**(15) Years**			**(20) Years**		
Core Location	**1**	**2**	**3**	**1**	**2**	**3**
Sample Identification	**NR- 1**	**NR- 2**	**NR-3**	**Ab-1**	**Ab-2**	**Ab-3**
Weight of Sample in Air (g)	1671.5	1873.5	1779.3	1412.5	1433.0	1421.5

Weight of Sample After Soaking (g)	1677.9	1879.5	1785.5	1415.5	1436.0	1425.0
Weight of Water Absorbed (g)	6.4	6.0	6.2	3.0	3.0	3.5
Void in Specimen (%)	3.8	3.2	3.4	2.1	2.1	2.4
Av. Void in Specimen (%)	3.5			2.2		

3.4. Aggregate Grading Test Result

Proper gradation of coarse aggregate is one of the most important requirements in the production of workable asphaltic concrete. It ensures that a sample of asphalt concrete contains all standard fractions of aggregates in its required proportion resulting in minimum voids in the sample. In this research work, the grading of aggregate was done to ascertain if the aggregates used for the mix design fell within the aggregate curve envelop as recommended (ASTM C 136 – 06). The result is presented in fig. 3.1a – fig. 3.1e

From the graph of fig. 3.1a, it is obvious that in the fresh asphalt concrete sample, the percentage of aggregates passing the different sieve sizes were within the range, hence it falls within the distribution curve envelop.

AGGREGATE SIZE DISTRIBUTION

mm	Weight RTD	% Retained	% Passing	Allowable Limit
25.00	-	-	100	100
19.00	-	-	100	100
12.50	20 .54	2.14	97.86	85-100
9.50	46.04	4.79	93.07	75-92
6.30	147.18	15.31	77.76	65-82
2.80	164.85	17.14	60.62	50-65
1.25	140.84	14.65	45.97	36-51
0.600	113.76	11.83	34.14	26-40
0.300	106.95	1112	23.02	18-30
0.150	100.95	10.50	12.52	13-24
0.075	60.04	6.24	6.28	7-14
-0.075	46.95	4.88	-	-

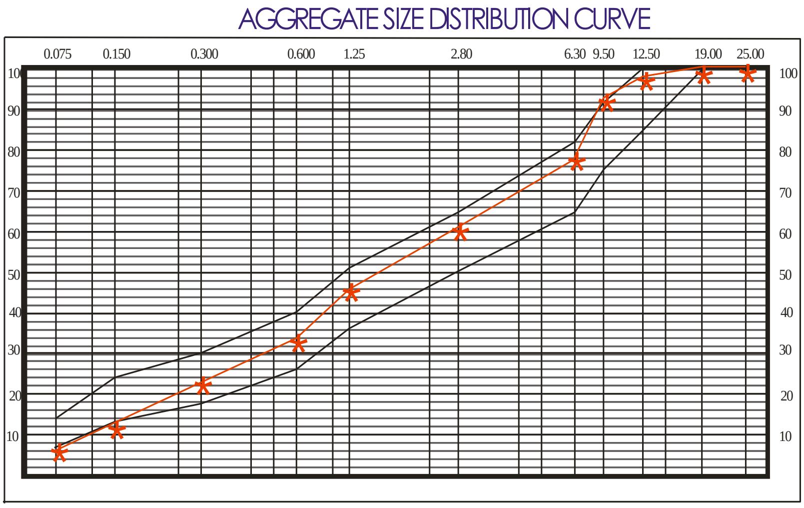

Fig. 3.1a: A graph showing the aggregate size distribution of fresh asphalt concrete

Unlike the graph of fig. 3.1a, the asphalt mix of 5 years has the percentage of aggregates passing the different sieve sizes outside the range, falling outside the distribution curve envelop as shown in fig. 3.1b

AGGREGATE SIZE DISTRIBUTION

mm	Weight RTD	% Retained	% Passing	Allowable Limit
25.00	-	-	100	100
19.00	-	-	100	100
12.50	60.38	5.31	94.69	85-100
9.50	142.42	12.53	82.16	75-92
6.30	106.38	9.36	72.80	65-82
2.80	208.97	18.39	54.41	50-65
1.25	176.91	15.57	38.84	36-51
0.600	123.39	10.86	27.98	26-40
0.300	108.50	9.55	18.43	18-30
0.150	88.90	7.82	10.61	13-24
0.075	57.50	5.06	5.55	7-14
-0.075	56.50	4.97	-	-

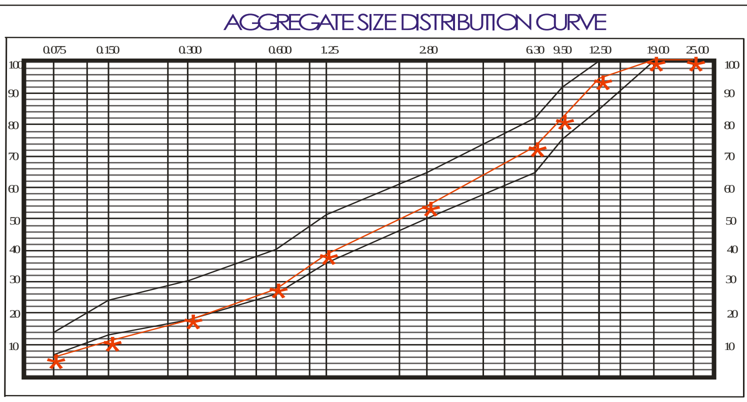

Figure 3.1b: A graph showing the aggregate size distribution of 5 years

Fig. 3.1c shows asphalt concrete of 10 years. The aggregate sizes were within the distribution curve envelop as most of the aggregates fell within the recommended range.

AGGREGATE SIZE DISTRIBUTION

mm	Weight RTD	% Retained	% Passing	Allowable Limit
25.00	-	-	100	100
19.00	-	-	100	100
12.50	58.00	5.05	94.95	85-100
9.50	126.84	11.04	83.91	75-92
6.30	104.45	9.09	74.82	65-82
2.80	214.75	18.68	56.14	50-65
1.25	135.20	11.76	44.38	36-51
0.600	94.03	8.18	36.20	26-40
0.300	123.66	10.76	25.44	18-30
0.150	141.13	12.28	13.16	13-24
0.075	75.26	6.55	6.61	7-14
-0.075	71.18	6.19	-	-

AGGREGATE SIZE DISTRIBUTION CURVE

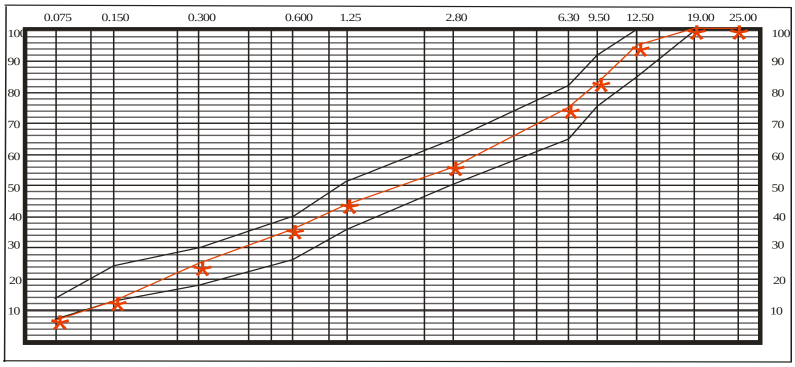

Fig. 3.1c: A graph showing the aggregate size distribution of 10 years

The trend found in asphalt mix of 10 years was replicated in that of 15 years, the aggregate sizes were within the distribution curve envelop but with some of the aggregates out of the recommended range as shown in fig. 3.1d

AGGREGATE SIZE DISTRIBUTION

mm	Weight RTD	% Retained	% Passing	Allowable Limit
25.00	-	-	100	100
19.00	-	-	100	100
12.50	39.38	4.13	95.87	85-100
9.50	80.69	8.47	87.40	75-92
6.30	87.92	9.23	78.17	65-82
2.80	195.87	20.56	57.61	50-65
1.25	123.30	12.95	44.66	36-51
0.600	87.70	9.21	35.45	26-40
0.300	112.22	11.78	23.67	18-30
0.150	120.67	12.67	11.0	13-24
0.075	46.60	4.89	6.11	7-14
-0.075	51.22	5.38	-	-

AGGREGATE SIZE DISTRIBUTION CURVE

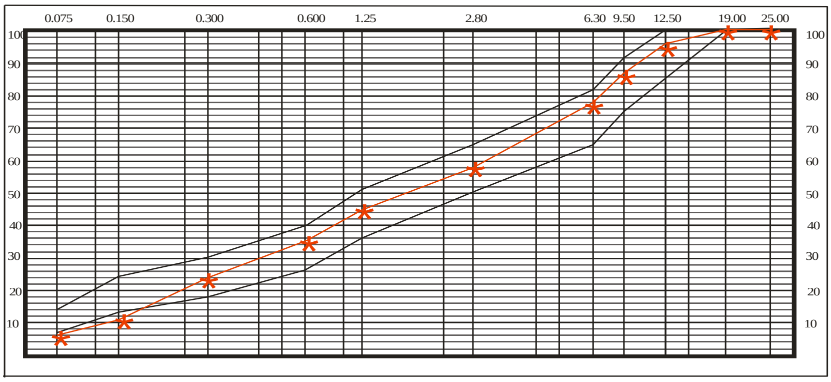

Figure 3.1d: A graph showing the aggregate size distribution of 15 years

As with the graph of fresh asphalt, 10 and 15 years asphalt concrete, the asphalt concrete of 20 years had aggregate sizes that fell within the recommended range, and as such were within the distribution curve envelop as shown in fig. 3.1e.

AGGREGATE SIZE DISTRIBUTION

mm	Weight RTD	% Retained	% Passing	Allowable Limit
25.00	-	-	100	100
19.00	-	-	100	100
12.50	0.00	0.00	100	85-100
9.50	62.43	6.60	93.4	75-92
6.30	122.38	12.8	80.6	65-82
2.80	247.86	26.0	54.6	50-65
1.25	138.68	14.6	40.0	36-51
0.600	76.96	8.1	31.9	26-40
0.300	60.59	6.4	25.5	18-30
0.150	76.66	8.0	17.5	13-24
0.075	79.53	8.4	9.1	7-14
-0.075	81.69	8.6	-	-

AGGREGATE SIZE DISTRIBUTION CURVE

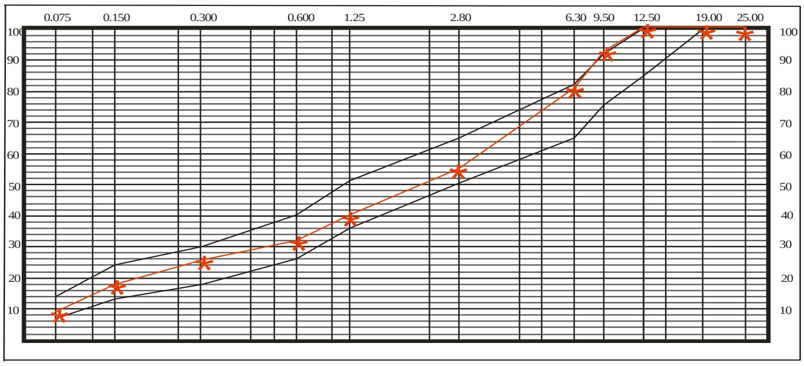

Figure 3.1e: A graph showing the aggregate size distribution of 20 years

3.5. Marshall Stability and Bitumen Extraction

The maximum load sustained by the bituminous material at a loading rate of 50.8mm/min was measured. The flow value which measures the vertical deformation when the maximum load is reach was noted. Their results are presented in table 3.4a – table 3.4e. The fresh asphalt concrete had a binder content of 4.73%, stability value of 3.78kN and a flow value of 1.81mm (table 3.4a). The asphalt concrete of 5 years had a binder content of 4.24%, stability value of 6.44kN and a flow value of 2.75mm (table 3.4b). The asphalt concrete of 10 years had a binder content of 4.72%, stability value of 8.50kN and a flow value of 3.4mm (table 3.4c). The asphalt concrete of 15 years had a binder content of 5.06%, stability value of 9.41kN and a flow value of 2.42mm (table 3.4d). The asphalt concrete of 20 years had a binder content of 5.34%, stability value of 10.61kN and a flow value of 2.86mm (table 3.4e) (ASTM D 2172 – 81).

Table 3.4a: Marshall Stability for fresh asphalt concrete

ASPHALT					Job Description:				Laboratory Testing of Asphalt Specimen										
MARSHALL TEST					WEARING COURSE			Assumed Gs of Bit.		pecific Gravity of Agg			Age of Asphalt Mix:			Fresh Asphalt			
								1.03		2.74			Date: 2nd Sept 2019						
REPORT																			
A	Temperature 0°c B	Bit. Content % C	Bit. Content c x 100 / 100 D	Weight in air (gm) E	Weight in air after 1 hr soak F	Weight in water (gm) G	Volume E - G (cm³) H	Bulk Density E/H (gm/cm³) I	SGM J	Volume of Bit D x I / Sg. Bit K	Vol. of Aggt (100 - D) x I / SG. Agg L	Voids in Mix Aggt 100 - L M	Void in Total Mix 100 - 100 I/J N	% Void filled 100 x K/M O	Stability mm P	Corr. Factor Q	Corr. Stability (KN) R	Flow 0.1mm S	Specification T
S1				1199.2	1200.0	704.0	495.2	2.42							30.6	1.09	4.89	1.88	
S2	60°C	P.H.C	% Wt	1199.6	1200.5	702.0	497.6	2.41	2.55	gm/ml	%	%	%	%	20.8	1.04	3.17	1.75	
S3				1199.8	1200.5	706.5	493.3	2.43							20.5	1.09	3.28	1.79	
Average		4.73	4.52	1199.5	1200.3	704.17	495.4	2.42		10.62	84.33	15.7	5.1	67.8	23.97		3.78	1.81	

Table 3.4b: Marshall Stability and extraction for asphalt concrete of 5 years

ASPHALT					Job Description:				Laboratory Testing of Asphalt Specimen										
MARSHAL TEST					WEARING COURSE			Assumed Gs of Bit.		pecific Gravity of Agg			Age of Asphalt Mix:			5 years			
REPORT								1.03		2.72			Date: 2nd Sept 2019						
A	Temperature 0°c B	Bit. Content % C	Bit. Content c x 100 / 100 D	Weight in air (gm) E	Weight in air after 1 hr soak F	Weight in water (gm) G	Volume E - G (cm³) H	Bulk Density E/H (gm/cm³) I	SGM J	Volume of Bit D x I / Sg. Bit K	Vol. of Aggt (100 - D) x I / SG. Agg L	Voids in Mix Aggt 100 - L M	Void in Total Mix 100 - 100 I/J N	% Void filled 100 x K/M O	Stability mm P	Corr. Factor Q	Corr. Stability (KN) R	Flow 0.1mm S	Specification T
S1				991.2	991.7	545.0	446.2	2.22							29.0	1.25	5.32	2.80	
S2	60°C	P.H.C	% Wt	992.7	993.0	552.0	440.7	2.25	2.55	gm/ml	%	%	%	%	39.0	1.32	7.55	2.70	
S3				991.8	992.0	549.0	443.5	2.24							33.0	1.23	6.28	2.75	
Average		4.24	4.07	991.90	992.23	548.67	443.47	2.24		8.85	79.00	21.00	12.16	42.14	33.67		6.38	2.75	

Extraction		
A	Asphalt (g)	1200
	Aggregate (g)	1143.06
B	Filter paper + Filler (g)	37.3
C	Filter paper alone (g)	31.25
E	Filler (g)	6.05
F	Total Aggregate	1149.11
G	Binder Content	50.89
H	% Binder Content	4.24

Table 3.4c: Marshall Stability and extraction for asphalt concrete of 10 years

ASPHALT					Job Description:				Laboratory Testing of Asphalt Specimen										
MARSHALL TEST					WEARING COURSE			Assumed Gs of Bit.		pecific Gravity of Agg			Age of Asphalt Mix:			10 Years			
								1.03		2.72			Date: 2nd Sept 2019						
REPORT																			
A	Temperature 0°c B	Bit. Content % C	Bit. Content c x 100 / 100 D	Weight in air (gm) E	Weight in air after 1 hr soak F	Weight in water (gm) G	Volume E - G (cm³) H	Bulk Density E/H (gm/cm³) I	SGM J	Volume of Bit D x I / Sg. Bit K	Vol. of Aggt (100 - D) x I / SG. Agg L	Voids in Mix Aggt 100 - L M	Void in Total Mix 100 - 100 I/J N	% Void filled 100 x K/M O	Stability mm P	Corr. Factor Q	Corr. Stability (KN) R	Flow 0.1mm S	Specification T
S1				974.2	975.7	545.5	428.7	2.27							40.9	1.39	8.34	2.95	
S2	60°C	P.H.C	% Wt	1137.3	1139.3	641.0	496.3	2.29	2.52	gm/ml	%	%	%	%	56.8	1.04	8.66	3.75	
S3				1080	1200.5	706.5	493.3	2.43							45.5	1.09	8.29	3.5	
Average		4.72	4.51	1063.8	1105.2	631	472.8	2.33		11.22	79.57	20.4	9.51	54.9	47.73		8.43	3.4	

Extraction		
A	Asphalt (g)	1000.00
B	Aggregate (g)	950.42
C	Filter paper + Filler (g)	34.70
D	Filter paper alone (g)	32.32
E	Filler (g)	2.38
F	Total Aggregate	952.85
G	Binder Content	47.20
H	% Binder Content	4.72

Table 3.4d: Marshall Stability and extraction for asphalt concrete of 15 years

ASPHALT					Job Description:				Laboratory Testing of Asphalt Specimen										
MARSHALL TEST					WEARING COURSE			Assumed Gs of Bit.		pecific Gravity of Agg			Age of Asphalt Mix:			15 Years			
REPORT								1.03		2.66			Date: 2nd Sept 2019						
A	Temperature 0°c B	Bit. Content % C	Bit. Content c x 100 / 100 D	Weight in air (gm) E	Weight in air after 1 hr soak F	Weight in water (gm) G	Volume E - G (cm³) H	Bulk Density E/H (gm/cm³) I	SGM J	Volume of Bit D x I / Sg. Bit K	Vol. of Aggt (100 - D) x I / SG. Agg L	Voids in Mix Aggt 100 - L M	Void in Total Mix 100 - 100 I/J N	% Void filled 100 x K/M O	Stability mm P	Corr. Factor Q	Corr. Stability (KN) R	Flow 0.1mm S	Specification T
S1				1194.8	1197.2	675.0	519.8	2.30							62.2	1.00	9.12	2.35	
S2	60°C	P.H.C	% Wt	1193.6	1195.7	669.0	524.6	2.28	2.49	gm/ml	%	%	%	%	68.9	0.96	9.70	2.4	
S3				1192.5	1190.5	660.5	512.5	2.25							65.5	1.02	9.50	2.5	
Average		5.06	4.82	1193.63	1194.47	668.17	519	2.28		11.22	79.57	20.4	9.51	54.9	65.53		9.44	2.42	

Extraction		
A	Asphalt (g)	1000.00
B	Aggregate (g)	940.40
C	Filter paper + Filler (g)	42.60
D	Filter paper alone (g)	33.60
E	Filler (g)	9.00
F	Total Aggregate	949.40
G	Binder Content	50.60
H	% Binder Content	5.06

Table 3.4e: Marshall Stability and extraction for asphalt concrete of 20 years

ASPHALT					Job Description:				Laboratory Testing of Asphalt Specimen										
MARSHALL TEST					WEARING COURSE			Assumed Gs of Bit.			pecific Gravity of Agg			Age of Asphalt Mix:			20 Years		
								1.03			2.59			Date: 2nd Sept 2019					
REPORT																			
	Temperature 0°c	Bit. Content %	Bit. Content c x 100 / 100	Weight in air (gm)	Weight in air after 1 hr soak	Weight in water (gm)	Volume E - G (cm³)	Bulk Density E/H (gm/cm³)	SGM	Volume of Bit D x I / Sg. Bit	Vol. of Aggt (100 - D) x I / SG. Agg	Voids in Mix Aggt 100 - L	Void in Total Mix 100 - 100 I/J	% Void filled 100 x K/M	Stability mm	Corr. Factor	Corr. Stability (KN)	Flow 0.1mm	Specification
A	B	C	D	E	F	G	H	I	J	K	L	M	N	O	P	Q	R	S	T
S1				1192.1	1193.3	662.5	529.6	2.25							80.1	0.96	11.3	2.58	
S2	60°C	P.H.C	% Wt	1195.9	1197.0	667.5	528.4	2.26	2.42	gm/ml	%	%	%	%	70.6	0.96	9.94	3.15	
S3				1194.5	1195.5	665.5	527.5	2.24							75.5	1.02	10.50	2.86	
Average		5.34	5.13	1194.17	1195.27	665.17	528.5	2.25		10.58	83.05	17	6.61	62.4	75.4		10.57	2.86	

Extraction		
A	Asphalt (g)	1200.00
B	Aggregate (g)	1126.35
C	Filter paper + Filler (g)	42.14
D	Filter paper alone (g)	32.60
E	Filler (g)	9.54
F	Total Aggregate	1135.89
G	Binder Content	64.11
H	% Binder Content	5.34

3.6. Analysis of Parameters

Parameters obtained from test such as void content, unit weight of specimen, binder content and stability for the different ages are shown in table 3.5. This forms the basis for comparison and discussion.

Table 3.5: Analysis of parameters of bituminous mix for the different asphalt ages

Age of Bituminous Mix	Void Content [%]	Unit Weight of Sample [g/cm³]	Binder Content [%]	Stability [kN]
Fresh Asphalt	7.0	2.20	4.15	3.78
5 years	5.3	2.25	4.24	6.44
10 years	4.2	2.29	4.72	8.50
15 years	3.5	2.29	5.06	9.41
20 years	2.2	2.31	5.34	10.61

As can be seen in fig. 3.2, the voids content of the different ages considered fell between 7.0 – 2.2 % with the fresh asphalt having the highest void content and the asphalt concrete of 20 years having the lowest. Comparing these values to the initial values of void content as seen in table 3.2, there is a significant decrease in the void content as the pavement ages which could have been due to the viscoelastic behavior of the asphalt material, that is; its properties are a function of time (or frequency) and temperature. Due to traffic loading, consolidation occurs in the asphalt concrete as a result of densification and plastic flow occasioned by change in temperature. Pavement performance studies conducted by [56]shows that when air voids reduces, the plastic flow mechanism predominates. When there is an increase in temperature, the voids are filled with bitumen as a result of the expansion of the pavement, once weight is applied to the top layer and after contraction; the percentage of voids in the pavement is reduced. It was observed that over a period of time as this process continues, it results in low air voids content as the pavement ages. This is in line with a work done by [22] where the dynamic modulus decreased with the increase in testing temperature due to softening of asphalt mixtures at higher temperature. The unit weight of the specimens ranged from 2.20g/cm^3 – 2.31g/cm^3 with the fresh asphalt having the lowest unit weight and the asphalt mix of 20 years having the highest. Comparing these values to the initial unit weight values as seen in table 3.2, there is a significant increase. Density is the unit weight of the asphalt concrete achieved through the compaction process. Hence, the major objective of compaction is to produce a dense mass with high unit weight. However, it is also clear that it takes time to achieve a stable condition. Depending on the volume of traffic and initial density, it may require adequate time before the ultimate field density can be achieved. This gives rise to higher internal resistance of the asphaltic concrete to load and disintegration. The more the road pavement is used over a period of time, the asphaltic material being a flexible media tends to become denser due to increased compression from the axle load and the dissipation of voids and the aggregates become more compact. This gives rise to increased unit weight over gradual increase in the age and usage of the road pavement. The binder content of the specimens ranged from 4.15 – 5.34% with the fresh asphalt having the lowest and the asphalt mix of 20 years having the highest. Comparing these values to the initial values of binder content as seen in table 3.2, there is a significant decrease which could be as a result of oxidation and volatilization. Oxidation rate is affected by asphalt binder type and thickness, pavement air voids, aggregate type and ambient temperature. Asphalt binder content and aging mechanism is influenced either physically or chemically by the aggregates. According to [8] aggregates, depending on their mineral composition, can absorb oily components from asphalt binder. Also, aggregates may influence the asphalt binder aging by acting as a catalyst. Formation of the oxidation products can be advanced by aggregates in the low polar general fractions or may absorb the highly polar fractions and cause less oxidation in the asphalt binder. Aggregate with small pores had less potential to absorb asphalt binder, which results in accelerated aging. When volatilization appears on the components of the asphalt, the amount of asphaltenes increases. This increase causes the binder to behave like a solid, producing higher rigidity and deterioration of its condition. As this process is repeated over the years, there is a significant increase in the asphalt binder content as the pavement ages. This is in line with a work done by [15] where he noted that aggregate binder absorbance depended on the air voids and pore sizes of aggregate. The stability of these specimens can also be seen from the graph, ranging from 3.78kN – 10.61kN. Comparing these values to the initial stability values as seen in table 3.2, there is a significant increase. The fresh asphalt sample has the lowest stability because of the presence of air voids due to the viscoelastic behavior of the material while the asphalt mix of 20 years has the highest. The more dense and compressed the asphaltic material is, the higher its internal resistance to load and breakage, this gives rise to increased stability of the asphaltic material as the age of the mix increases. The stability of a mix is a function of internal friction and cohesion. Internal friction among the aggregate particles is related to aggregate characteristics such as shape and surface texture hence, cohesion results from the ability of the binder to bond. A proper degree of both internal friction and cohesion in asphalt mixture restrains the aggregate particles from moving over each other by the forces exerted by traffic.

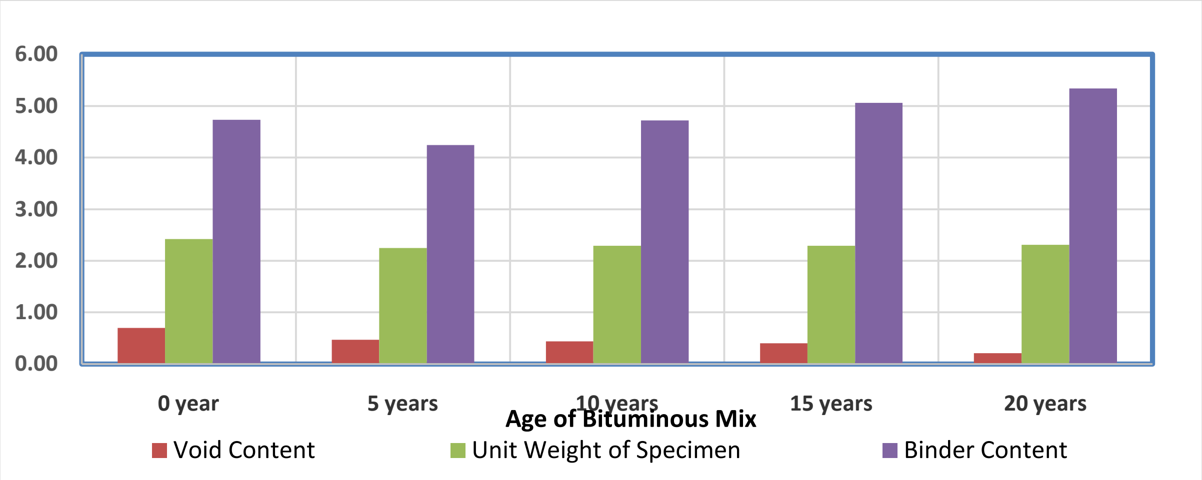

Figure 3.2: A bar chart showing the parameters of bituminous mix

Figure 3.3a – 3.3d shows a graph where age of bituminous mix is plotted against stability, unit weight, void content and binder content. It was observed that as the pavement advances in age, the voids content decreases from 7.0% for fresh asphalt mix to 2.2% for asphalt mix of 20 years (fig. 3.3a).

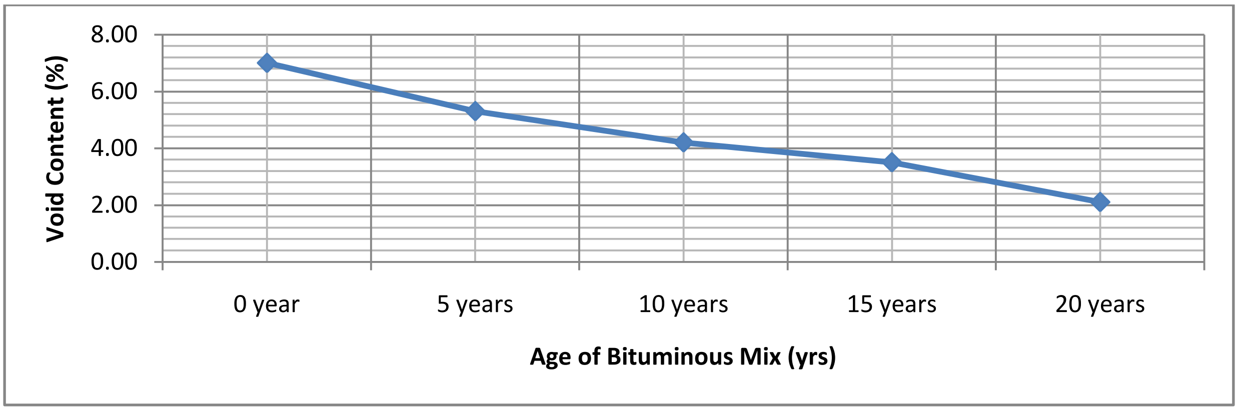

Figure 3.3a: Graph showing effect of aging on Void content

Comparing these values to the initial values of void content as seen in table 3.2, there is a significant decrease which could be due to traffic loading and consolidation in the asphalt concrete as a result of densification and plastic flow occasioned by change in temperature. Pavement performance studies conducted by [56] shows that when air voids reduces, the plastic flow mechanism predominates. The reason for this is the building up of pore pressure in the air void space due to which some amount of asphalt is forced to flow. The flow of asphalt into the voids and reduction in the asphalt film thickness will ultimately result in reduction of voids and relative distance between aggregate particles. Therefore particle reorientation may be caused by the flow of asphalt into the voids. The binder content increases from 4.15% for fresh asphalt mix to 5.34% for asphalt mix of 20 years (fig. 3.3b).

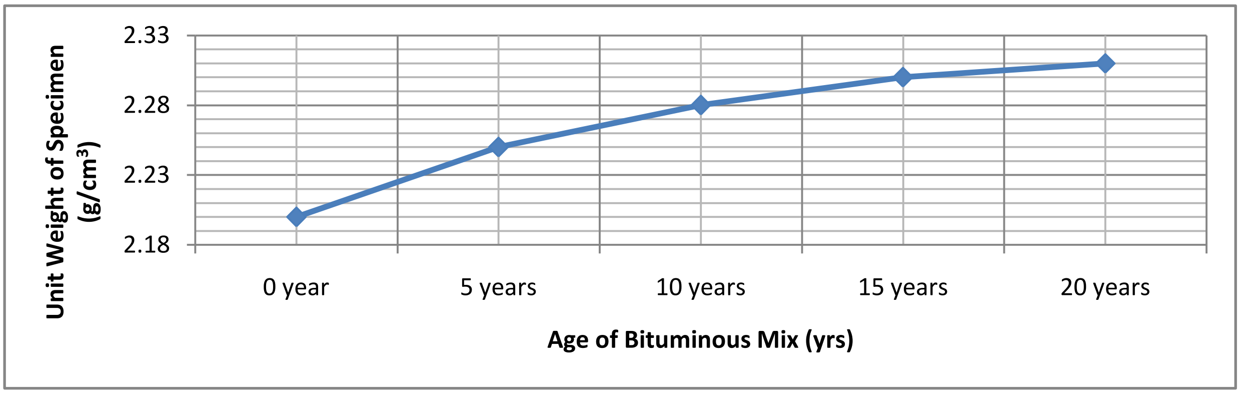

Figure 3.3b: Graph showing effect of aging on Unit Weight of Specimen

Comparing these values to the initial binder content values as seen in table 3.2, there is a significant decrease. As the quantity of binder (bitumen) in the mix increases, it fills the voids in total mix and also provides more cohesion and bonding between the different aggregates in the mix, thus reducing percentage voids in total mix and increasing percentage voids filled with bitumen. This gives rise to the trend presented in figure 3.3a, where the void content decreases with increase in the binder content. The unit weight increases from 2.20g/cm^3 for fresh asphalt mix to 2.31g/cm^3 for asphalt mix of 20 years (fig. 3.3c).

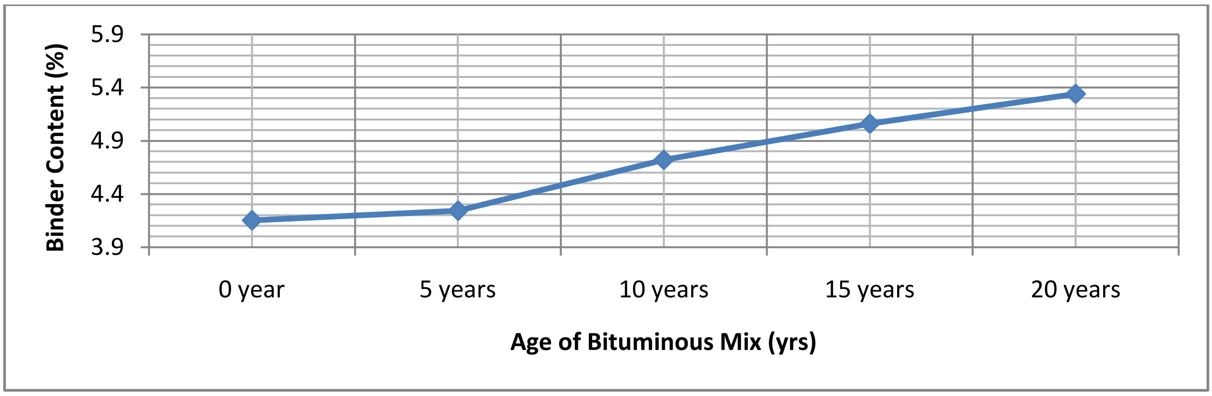

Figure 3.3c: Graph showing effect of aging on Binder Content

Comparing these values to the initial unit weight values as seen in table 3.2, there is a significant increase. Density is the unit weight of the asphalt concrete achieved through the compaction process. Hence, the major objective of compaction is to produce a dense mass with high unit weight. However, it is also clear that it takes time to achieve a stable condition. Depending on the volume of traffic and initial density, it may require adequate time before the ultimate field density can be achieved. This gives rise to higher internal resistance of the asphaltic concrete to load and disintegration. The more the road pavement is used over a period of time, the asphaltic material being a flexible media tends to become denser due to increased compression from the axle load and the dissipation of voids and the aggregates become more compact. This gives rise to increased unit weight over gradual increase in the age and usage of the road pavement. The stability also increases from 3.78kN for fresh asphalt mix to 10.61kN for asphalt mix of 20 years (fig. 3.3d).

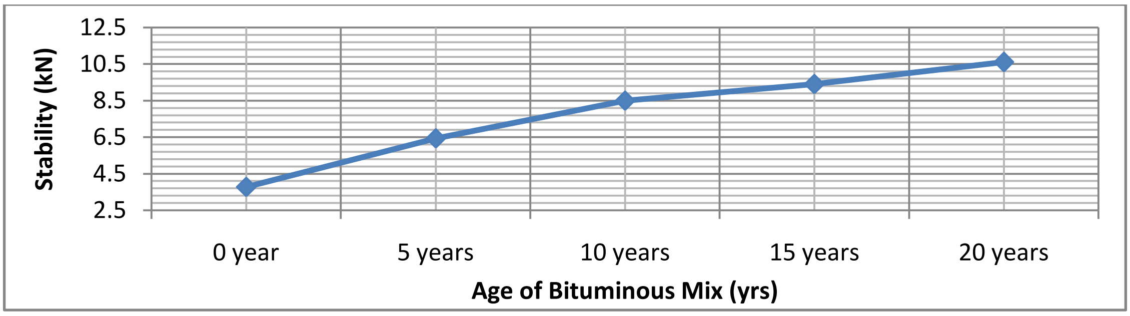

Figure 3.3d: Graph showing effect of aging on Stability

Comparing these values to the initial stability values as seen in table 3.2, there is a significant increase. As the age of the asphaltic concrete increases with a corresponding increase in the unit weight, the voids reduces, the asphaltic material becomes more dense and compressed as the age of the mix increases. The denser and compressed the asphaltic material is, the higher its internal resistance to load and breakage, this gives rise to increased stability of the material, hence increase in strength.

The summary of the bituminous mix properties (void content, binder content, unit weight of specimen, voids filled with bitumen, stability) with the age of the pavement is presented in table 3.5

Table 3.5: Summary of Bituminous Mix Properties with Age

Age of Bituminous Mix [Years]	Void Content [%]	Binder Content (%)	Unit Weight of Specimen [g/cm^3]	Voids Filled with Bitumen (%)	Stability [kN]
Fresh Asphalt	7.0	4.15	2.20	40.50	3.78
5	5.3	4.24	2.25	42.14	6.44
10	4.2	4.72	2.28	54.91	8.50
15	3.5	5.06	2.30	53.70	9.41
20	2.2	5.34	2.31	62.42	10.61

3.7. Regression Analysis

The age of the bituminous mix was considered as the predictor while the voids content, binder content, unit weight of specimen, voids filled with bitumen and stability were considered as the criterion. The variations in the predictor with the criterion are presented in fig.3.5a – fig.3.5e. The regression analysis shows the standard error, R-squared, adjusted R-squared and predicted R-squared values of the mix for each of the parameters considered. It is worthy of note that the trend shown in the regression analysis is in line with the experimental results gotten.

3.7.1. Regression Analysis: Void Content (%) versus Age of Bituminous Mix (yrs)

Regression Equation $V_v(\%) = 0.6300 - 0.10300\ A_b$ ..…………i

3.7.2. Polynomial Regression Analysis: Void Content (%) versus Age of Bituminous Mix (yrs)

The regression equation is:

$V_v(\%) = 0.6963 - 0.04755\ A_b + 0.002886\ A_b^2 - 0.000087\ A_b^3$ …..……..ii

S = 0.0089576 R-sq = 99.8% R-sq(adj) = 99.7% R-sq(pred) = 97.21%

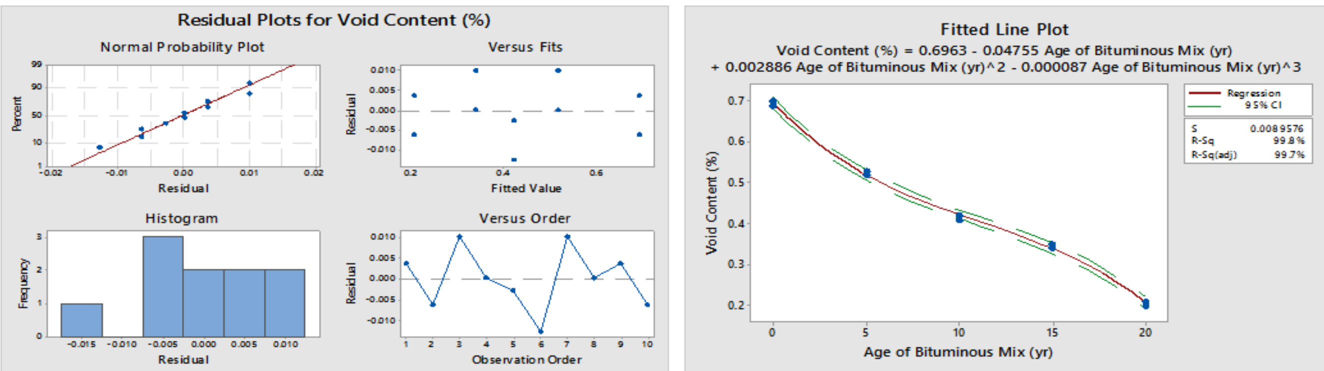

Figure 3.5a: Residual and fitted line plots for void content

3.7.3. Regression Analysis: Binder Content (%) versus Age of Bituminous Mix (yrs)

Regression Equation $B_c(\%) = 3.9250 + 0.3640\ A_b$ ………….iii

3.7.4. Polynomial Regression Analysis: Binder Content (%) versus Age of Bituminous Mix (yrs)

The regression equation is:

$B_c(\%) = 4.136 + 0.01421A_b - 0.009686{A_b}^2 + 0.000300{A_b}^3$ ...…………iv

S = 0.0425860   R-Sq = 97.6%   R-Sq(adj) = 96.9%

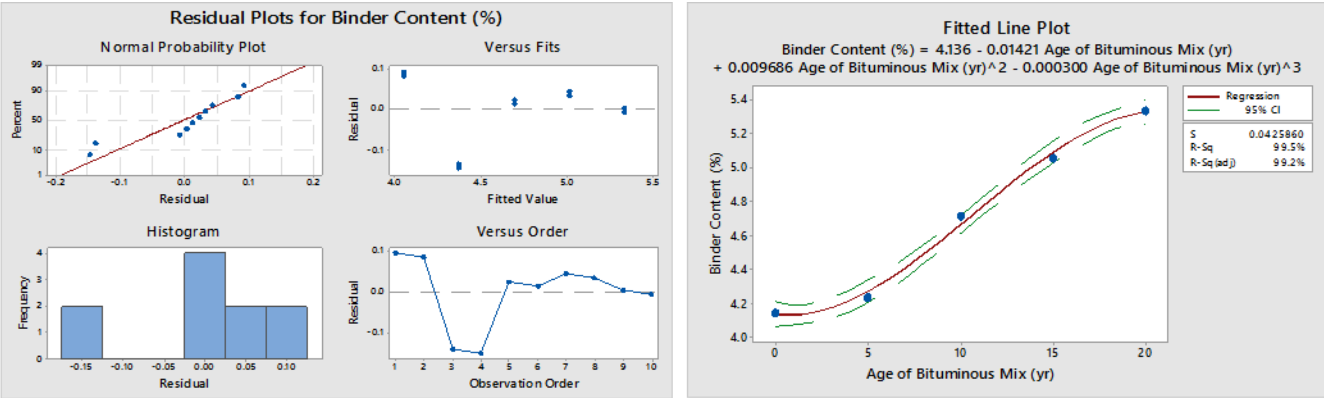

Figure 3.5b: Residual and fitted line plots for binder content

3.7.5. Regression Analysis: Unit Weight of Sample (g/cm³) versus Age of Bituminous Mix (yrs)

Regression Equation: $U_{ws}\left(\frac{g}{cm}^3\right) = 1.730 + 0.3200\ A_b$ ....………v

3.7.6. Polynomial Regression Analysis: UWS (g/cm³) versus Age of Bituminous Mix (yrs)

The regression equation is:

$U_{ws}\left(\frac{g}{cm}^3\right) = 2.196 - 0.01054A_b + 0.000257{A_b}^2$ …………vi

S = 0.0064918   R-sq = 98.4%   R-sq(adj) = 97.6%   R-sq (pred) = 97.11%

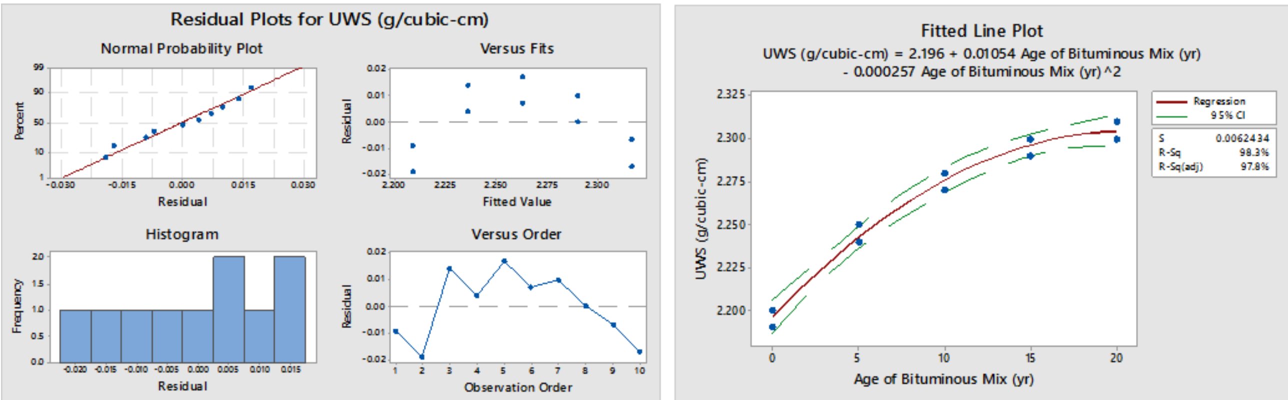

Figure 3.5c: Residual and fitted line plots for unit weight of specimen

3.7.7. Regression Analysis: Voids filled with Bitumen (g/cm³) versus Age of Bituminous mix (yrs)

Regression Equation: $V_{fb}(\%) = 38.38 + 5.96A_b$ ..………..vii

3.7.8. Polynomial Regression Analysis: VFB (%) versus Age of Bituminous Mix (yr)

The regression equation is

$V_{fb}(\%) = 39.67 + 1.098A_b + 0.00051{A_b}^2$ …………viii

S = 3.13834   R-Sq = 89.9%   R-Sq(adj) = 87.0%

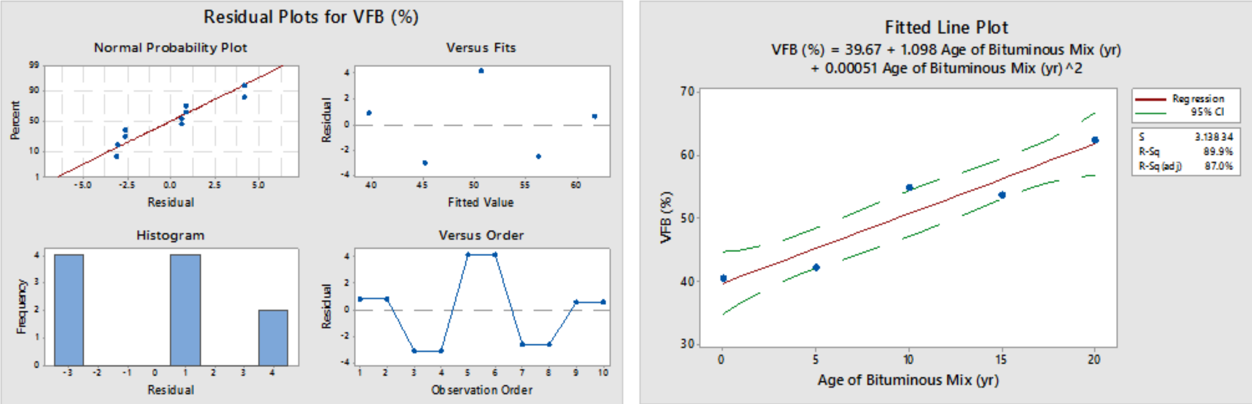

Figure 3.5d: Residual and fitted line plots for Voids Filled with Bitumen

3.7.9. Regression Analysis: Stability (kN) versus Age of Bituminous mix (yrs)

Regression Equation: $S_{ta}\,(kN) = 5.400 + 1.338A_b$ …………ix

3.7.10. Polynomial Regression Analysis: Stability (kN) versus Age of Bituminous mix (yrs)

The regression equation is:

$S_{ta}\,(kN) = 3.832 + 0.5697A_b - 0.01186{A_b}^2$ …………x

S = 0.207279   R-Sq = 99.5%   R-Sq(adj) = 99.3%

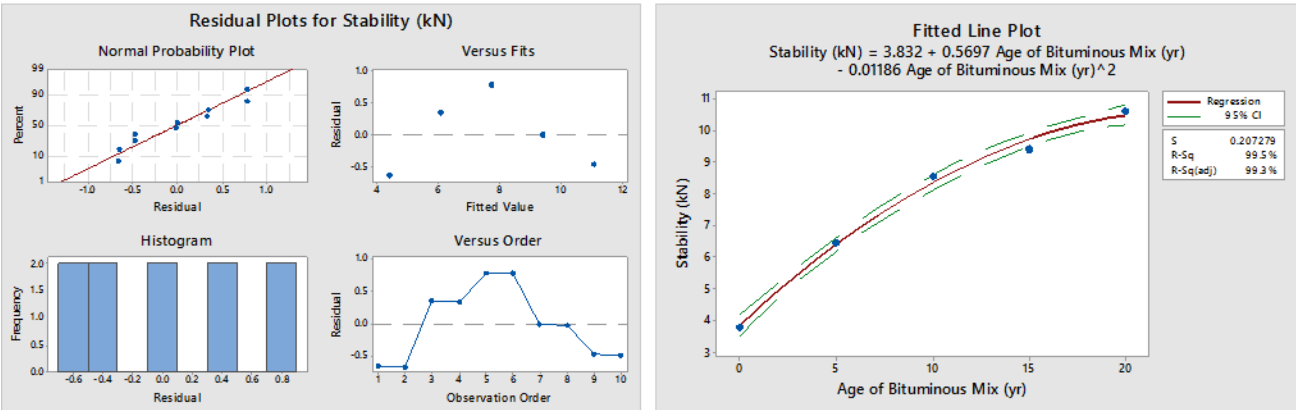

Figure 3.5e: Residual and fitted line plots for Stability

3.8. Fisher Pairwise Comparisons

A Pairwise Comparison is the process of comparing obtained values in pairs to judge which of each value is preferred overall. Each value is matched head-to-head (one-on-one) with each of the other values. This means that each comparison looks at the difference between the means of a pair of design conditions.

3.8.1. Comparisons for Void Content (%): Response = Void Content (%), Term = Age of Bituminous Mix (yrs)

Grouping Information Using Fisher LSD Method and 95% Confidence

Age of Bituminous Mix (yr)	**N**	**Mean**	**Grouping**
20	2	0.525	A
15	2	0.415	B
10	2	0.345	C
5	2	0.205	D

The comparison shows that at 95% confidence level, the groupings do not share a label, hence they are different. The Simultaneous Confidence Level = 84.70% showing that it is within tolerable limits.

3.8.2. Fisher Multiple Comparisons with a Control: Response = Void Content (%), Term = Age of Bituminous Mix (yrs)

Grouping Information Using Fisher LSD Method and 95% Confidence

Age of Bituminous Mix (yr)	**N**	**Mean**	**Grouping**
0.00 (Control)	2	0.695	A
20	2	0.525	
15	2	0.415	
10	2	0.345	
5	2	0.205	

The comparison shows that at 95% confidence level, the groupings do not share a label; hence they are significantly different from the control levels mean. The Simultaneous Confidence Level = 86.57%, showing the level of tolerance.

3.8.3. Comparisons for Binder content: Response = Binder Content (%), Term = Age of Bituminous Mix (yrs)

Grouping Information Using Fisher LSD Method and 95% Confidence

Age of Bituminous Mix (yr)	**N**	**Mean**	**Grouping**
20	2	5.335	A
15	2	5.055	B
10	2	4.715	C
5	2	4.235	D

The comparison shows that at 95% confidence level, the groupings do not share a label; hence they are different. The Simultaneous Confidence Level = 84.70%, showing the level of tolerance.

3.8.4. Fisher Multiple Comparisons with a Control: Response = Binder Content (%), Term = Age of Bituminous Mix (yrs)

Grouping Information Using Fisher LSD Method and 95% Confidence

Age of Bituminous Mix (yr)	**N**	**Mean**	**Grouping**
0.00 (Control)	2	4.145	A
20	2	5.335	
15	2	5.055	
10	2	4.715	
5	2	4.235	

The comparison shows that at 95% confidence level, the groupings do not share a label; hence they are significantly different from of the control. The Simultaneous Confidence Level = 86.57%, showing the level of tolerance.

3.8.5. Comparisons for Unit Weight Sample: Response = UWS (g/cm^3), Term = Age of Bituminous Mix (yrs)

Grouping Information Using Fisher LSD Method and 95% Confidence

Age of Bituminous Mix (yr)	**N**	**Mean**	**Grouping**
20	2	3.305	A
15	2	2.295	B
10	2	2.275	C
5	2	2.245	D

The comparison shows that at 95% confidence level, the groupings do not share a label; hence they are different. The Simultaneous Confidence Level = 84.70%, showing the level of tolerance.

3.8.6. Fisher Multiple Comparisons with a Control: Response = UWS (g/cm^3), Term = Age of Bituminous Mix (yrs)

Grouping Information Using Fisher LSD Method and 95% Confidence

Age of Bituminous Mix (yr)	N	Mean	Grouping
0.00 (Control)	2	2.195	A
20	2	3.305	
15	2	2.295	
10	2	2.275	
5	2	2.245	

The comparison shows that at 95% confidence level, the groupings do not share a label; hence they are significantly different from the control level mean. The Simultaneous Confidence Level = 86.57%, showing the level of tolerance.

3.8.7. Comparisons for Voids Filled with Bitumen: Response = VFB (%), Term = Age of Bituminous Mix (yrs)

Grouping Information Using Fisher LSD Method and 95% Confidence

Age of Bituminous Mix (yr)	N	Mean	Grouping
20	2	62.415	A
15	2	54.905	B
10	2	53.695	C
5	2	42.135	D

The comparison shows that at 95% confidence level, the groupings do not share a label; hence they are different. The Simultaneous Confidence Level = 84.70%, showing the level of tolerance.

3.8.8. Fisher Multiple Comparisons with a Control: Response = VFB (%), Term = Age of Bituminous Mix (yrs)

Grouping Information Using Fisher LSD Method and 95% Confidence

Age of Bituminous Mix (yr)	N	Mean	Grouping
0.00 (Control)	2	40.495	A
20	2	62.415	
15	2	54.905	
10	2	53.695	
5	2	42.135	

The comparison shows that at 95% confidence level, the groupings do not share a label; hence they are significantly different from the control level mean. The Simultaneous Confidence Level = 86.57%, showing the level of tolerance.

3.8.9. Comparisons for Stability: Response = Stability (kN), Term = Age of Bituminous Mix (yrs)

Grouping Information Using Fisher LSD Method and 95% Confidence

Age of Bituminous Mix (yr)	N	Mean	Grouping
20	2	10.605	A
15	2	9.405	B
10	2	8.535	C
5	2	6.435	D

The comparison shows that at 95% confidence level, the groupings do not share a label; hence they are different. The Simultaneous Confidence Level = 84.70%, showing the level of tolerance.

3.8.10. Fisher Multiple Comparisons for Stability: Response = Stability (kN), Term = Age of Bituminous Mix (yrs)

Grouping Information Using Fisher LSD Method and 95% Confidence

Age of Bituminous Mix (yr)	N	Mean	Grouping
0.00 (Control)	2	3.775	A
20	2	10.605	
15	2	9.405	
10	2	8.535	
5	2	6.435	

The comparison shows that at 95% confidence level, the groupings do not share a label; hence they are significantly different from the control level mean. The Simultaneous Confidence Level = 86.57%, showing the level of tolerance.

4. Conclusion and Recommendations

4.1. Conclusion

The following conclusions were made

- The difference in void content could have been due to the viscoelastic behavior of the asphalt material, that is; its properties are a function of time (or frequency) and temperature. Due to traffic loading, consolidation occurs in the asphalt concrete as a result of densification and plastic flow occasioned by change in temperature. Pavement performance studies conducted by [56] shows that when air voids reduces, the plastic flow mechanism predominates. The reason for this is the building up of pore pressure in the air void space due to which some amount of asphalt is forced to flow. The flow of asphalt into the voids and reduction in the asphalt film thickness will ultimately result in reduction of voids and relative distance between aggregate particles. When there is an increase in temperature, the voids are filled with bitumen as a result of the expansion of the pavement, once weight is applied to the top layer and after contraction; the percentage of voids in the pavement is reduced. It was observed that over a period of time as this process continues, it results in low air voids content as the pavement ages. This is in line with a work done by [22] where the dynamic modulus decreased with the increase in testing temperature due to softening of asphalt mixtures at higher temperature.
- Density is the unit weight of the asphalt concrete achieved through the compaction process. Hence, the major objective of compaction is to produce a dense mass with high unit weight. However, it is also clear that it takes time to achieve a stable condition. Depending on the volume of traffic and initial density, it may require adequate time before the ultimate field density can be achieved. This gives rise to higher internal resistance of the asphaltic concrete to load and disintegration. The more the road pavement is used over a period of time, the asphaltic material being a flexible media tends to become denser due to increased compression from the axle load and the dissipation of voids and the aggregates become more compact.
- The difference in the binder content is as a result of oxidation and volatilization. Oxidation rate is affected by asphalt binder type and thickness, pavement air voids, aggregate type and ambient temperature. Asphalt binder content and aging mechanism is influenced either physically or chemically by the aggregates. According to [8] aggregates, depending on their mineral composition, can absorb oily components from asphalt binder. Also, aggregates may influence the asphalt binder aging by acting as a catalyst. Formation of the oxidation products can be advanced by aggregates in the low polar general fractions or may absorb the highly polar fractions and cause less oxidation in the asphalt binder. Aggregate with small pores had less potential to absorb asphalt binder, which results in accelerated aging. When volatilization appears on the components of the asphalt, the amount of asphaltenes increases. This increase causes the binder to behave like a solid, producing higher rigidity and deterioration of its condition. As this process is repeated over the years, there is a significant increase in the asphalt binder content as the pavement ages. This is in line with a work done by [15] where he noted that aggregate binder absorbance depended on the air voids and pore sizes of aggregate.
- The difference in stability of these specimens could be attributed to the internal friction and cohesion of the material. The more dense and compressed the asphaltic material is, the higher its internal resistance

to load and breakage, this gives rise to increased stability of the asphaltic material as the age of the mix increases. Internal friction among the aggregate particles is related to aggregate characteristics such as shape and surface texture hence, cohesion results from the ability of the binder to bond. A proper degree of both internal friction and cohesion in asphalt mixture restrains the aggregate particles from moving over each other by the forces exerted by traffic.

4.2. Recommendation

The following recommendations are made

- Since air voids influence the performance of pavement during service life, it should be ensured that the air voids allowed in an asphalt mix is within the recommended standard at the production stage.
- At the execution stage, if it has been confirmed that the inherent air voids is above the required limit, the thickness of the asphalt should be increased to about 300mm.
- Further studies should be conducted to determine the rate of bitumen aging as a function of the depth from the pavement surface.

References

[1]. AASHTO Designation T-49-07. Standard Method of Test for Penetration of Bituminous Materials, 2007

[2]. AASHTO Designation T-240-09. Effect of Heat and Air on a Moving Film of Asphalt Binder (Rolling Thin-Film Oven Test), 2009

[3]. AASHTO Designation T-316-06. Standard Method of Test for Viscosity Determination of Asphalt Binder Using Rotational Viscometer, 2006 b

[4]. Airey, G. D., (2003). State of the Art Report on the Ageing Test Methods for Bituminous Pavement Materials, The International Journal of Pavement Engineering, Vol. 4

[5]. Ahmad, J., Yusoff, N. I. M., Hainin, M. R., Rahman, M. Y. A. (2014). Investigation into hot-mix asphalt moisture-induced damage under tropical climatic conditions. Construction and Building Materials. 50: 567-576

[6]. Al-Azri N, Bullin J, Richard R, Ferry A, Glover CJ, Jung S et al (2006) Binder oxidative aging in Texas pavements: hardening rates, hardening susceptibilities, and impact of pavement depth, Transp Res Rec 1962 (1): 12-20.

[7]. Allen RG, Little DN, Bhasin A (2012) Structural Characterization of Micromechanical Properties in Asphalt Using Atomic Force Microscopy, J Mater Civil Eng 24(10): 1317-27.

[8]. Anderson, D. A., Christensen, D. W., Bahia, H. U., Dongre, R., Sharma, M., Antle, C. E., and Button, J. (1994). "Binder Characterization and Evaluation. Volume 3: Physical Characterization".

[9]. Apeagyei, A. K. “Laboratory Evaluation of Antioxidants for Asphalt Binders,” Construction and Building Materials, Vol. 25, pp. 47–53, 2011.

[10]. Aschenbrener, T. (2002). “Survey on Moisture Damage of Hot Mix Asphalt Pavements,” Colorado DOT, Denver, CO.

[11]. Bari J. and Witczak, M.W, (2005). “Evaluation of the Effect of Lime Modification on the Dynamic Modulus Stiffness of Hot-Mix Asphalt Use with the New Mechanistic- Empirical Pavement Design Guide,” Transportation Research Record No. 1929, pp. 10-19.

[12]. Betenson, W.B., (1998). “Quality - The Word for the 21st Century,” presentation at Workshop on Lime in Hot Mix Asphalt, Sacramento, California.

[13]. Brett Neal (2007 -2017). Pavement defects and failures. Pavement pro.

[14]. Brown, E.R. and Cross, S.A. (1992), A national study of rutting in hot mix asphalt (HMA) pavements, National Center for Asphalt Technology, USA

[15]. Chadbourn, B. A., SkokJr, E. L., Newcomb, D. E., Crow, B. L., and Spindler, S. (1999). "The Effect of Voids in Mineral Aggregate (VMA) on Hot-Mix Asphalt Pavements."

[16]. Chang, George; Xu, Qinwu; Rutledge, Jennifer; and Gerber, Sabrina. (2014) “A Study on Intelligent Compaction and In-Place Density.” FHWA-HIF-14-017, National Technical Information Service.

[17]. Christensen, Donald W. and Bonaquist, Ramon F. (2006) "Volumetric Requirements of Superpave Mix Design." National Cooperative Highway Research Program (NCHRP) Report 567: Transportation Research Board.
[18]. Chipps, G. D., Davison, R., and Glover, C.(2001). A Model for Oxidative Aging of Rubber-Modified Asphalts and Implications to Performance Analysis, Energy & Fuels, Vol. 15,
[19]. Colbert, B., and Z. You (2012). The Properties of Asphalt Binder Blended with Variable Quantities of RecycledAsphalt Using Short Term and Long Term Aging Simulations. *Construction and Building Materials*, Vol. 26, pp. 552–557.
[20]. Cross, S.A., Adu-Osei, A., and Hainin, M.R. (1999), Effect of gradation on performance of asphalt mixtures, Paper presented at 78 th annual meeting of the Transportation Research Board, Washington D.C.
[21]. Eisenmann, J., and Hilmer, A. (1997) "Influence of Wheel Load and Inflation Pressure on the Rutting Effect at Asphalt-Pavements - Experiments and Theoretical Investigations", Proceedings, SixthInternational Conference on the Structural Design of Asphalt Pavements, Vol. I, Ann Arbor, 392-403.
[22]. Dalim Kumar, OkanSirin, EmadKassem. (2016). Laboratory Investigation of asphalt mixture aging. Civil and Architectural Engineering, Qatar University Doha, Qatar – Digital object identifier (DOI): dx.doi.org/10.14311/EE.251
[23]. Das PK, Birgisson B, Jelagin D, Kringos N (2013a). Investigation of the asphalt mixture morphology influence on its ageing susceptibility. Materials and Structures.
[24]. Das PK, Kringos N, Wallqvist V, Birgisson B (2013b) Micromechanical investigation of phase separation in bitumen by combining AFM with DSC results, Road Mater Pavement Des 14(S1):25-37.
[25]. Dickinson EJ (2000) Prediction of the hardening of the bitumen in pavement surfacings by reaction with atmospheric oxygen. Int J Road Mater Pavement Des, 1(3):255–80.
[26]. deMoraes MB, Pereira RB, Simao RA, Leite LFM (2010) High temperature AFM study of CAP 30/45 pen grade bitumen. Journal of Microscopy, 239(1), 46–53.
[27]. Durrieu F, Farcas F, Mouillet V (2007) The influence of UV aging of a tyrene/butadiene/styrene modified bitumen: comparison between laboratory and on site aging, Fuel 86:1446-51.
[28]. El-Basyouny, M.M. and Mamlouk, M.S (1999), Effect of aggregate gradation on rutting potential of Superpave mixes, Paper presented at 78th annual meeting of the Transportation Research Board, Washington D.C.
[29]. Epps, J. A.; Seeds, S. B.; Monismith, C. L.; Harvey, J. T.; Alavi, S. H.; Scholz, T. V.; Leahy, R. B.; andAshmore, S. C. (2002) "Recommended Performance-Related Specifications for Hot-Mix Asphalt Construction: Results of the WestTrack Project." NCHRP Report 455, National Cooperative Highway Research Program, Transportation Research Board, Washington, D.C.
[30]. Estakhri, Cindy; Cao, Rongji; Alvarez-Lugo, Allex; and Button, Joe. (2009) "Production, Placement, and Performance Evaluation of Warm Mix Asphalt in Texas." Geotechnical Special Publication No. 193, American Society of Civil Engineers.
[31]. European Commission (1999), COST 333 Development of new pavement design method - final report of the action, Brussels, Belgium.
[32]. Exxon Company (1997). Petroleum Encyclopedia for the Users of Petroleum Products. Exxon web site. http://www.exxondist.com. Accessed 28 August 2001.
[33]. Farrar MJ, Harnsberger PM, Thomas KP, Wiser W (2006) Evaluation of Oxidation in Asphalt Pavement Test Sections After Four Years of Service. Proceedings of the International Conference on Perpetual Pavement, Columbus, Ohio.
[34]. Hicks, R.G. and Scholz, T.V., "Life Cycle Costs For Lime in Hot Mix Asphalt," Final Report Volumes 1, 2, and 3, National Lime Association, Arlington VA, (April2003) (http://www.lime.org/publications.html#asphalt).

[35]. Huang, S., Petersen, J.C., Robertson, R.E., and Branthaver, J.F., (2002). "Effect of Hydrated lime on Long- Term Oxidative Aging Characteristics of Asphalt," Transportation Research Board No. 1810, pp. 17-24.

[36]. Huang, S., and Robertson, R.E., (June 8-9, 2004). "Effect of Hydrated Lime on the Rheological Properties of Compatible and incompatible Asphalts After Long-Term Oxidative Aging," International Symposium on Design and Construction of Long Lasting Asphalt Pavements, Auburn, AL.

[37]. Idham, M.K., Hainin, M. R., Yaacob, H., Warid, M. N. M. and Abdullah, M. E. (2013). Effect of aging on resilient modulus of hot mix asphalt mixtures, Advanced Materials Research. 723:291-297.

[38]. Igwe, E. A., (2015) "Effects of Air Voids Variation on Stiffness property of HMA Concrete Modified with Rubber Latex" International Journal of Emerging Trends in Engineering Research, Vol. 3,N0. 9. Available online @ http://www.warse.org/IJETER/static/pdf/file/ijeter02392015.pdf

[39]. Islam, M. R., Hossain, M. I. and Tarefder, R. A. (2015). "A Study of Asphalt Aging Using Indirect Tensile Strength Test." *Construction and Building Materials*, Vol. 95, pp. 218-223.

[40]. Islam, M. R., and Tarefder, R. A. (2015). "A Study of Asphalt Aging Through Beam Fatigue Test."*Transportation Research Record: Journal of the Transportation Research Board*, Vol. 2505, pp. 115-120

[41]. Jung, Sung Hoon, (2006). The Effects of Asphalt Binder Oxidation on Hot Mix Asphalt Concrete Mixture Rheology and Fatigue Performance, Texas A&M University

[42]. Kaloush, K.E., Witczak, M.W., Way, G.B., Zborowski, A., Abojaradeh, M., Sotil, A., (2002). Performance Evaluation of Arizona Asphalt Rubber Mixes Using Advanced Dynamic Material Characterization Tests,Final Report, Arizona State University

[43]. Kaloush, K.E., Zborowski, A., Sotil, A., Way, G.B., (2007). Material Characteristics of Asphalt Rubber Mixtures, Proceedings of the Asphalt Rubber 2003 Conference, p.129-145, December 2003 Lee, Soon-Jae, Characterization of Recycled Aged CRM Binders, Clemson University

[44]. KhannaS.k. and Justo C.E.G. (2015). Highway Engineering textbook

[45]. Luminari, M. and Fidato, A. (1998), A state of the art report on mix design, in Franken, L. (ed), Bituminous Binders and Mixes, Report of RILEM technical Committee 152-PBM.

[46]. McGennis, R.B., Anderseon, R.M., Kennedy, T.W., and Solaimanian, M. (1994), Introduction to Superpaveasphalt mixture design, Federal Highway Administration, Office of Technology Applications, Washington, DC.Mix Desism Methods for Asphalt Concrete and Other Hot Mix Tvesl The Asphalt Institute, MS-2. Sixth Edition, (1993).

[47]. Maupin, G. W. (2007). "Preliminary Field Investigation of Intelligent Compaction of Hot-Mix Asphalt." VTRC-08-R7, Virginia Department of Transportation.

[48]. Minchin, R.; Thomas, H. R.; and Swanson, David. (2001) "Theory Behind Vibratory Based Onboard Asphalt Density Measuring System." Transportation Research Record 1761, Transportation Research Board, National Research Council, Washington, D.C.

[49]. Nodes, Julie E. (2006) "Impact of Incentives on In-Place Air Voids." Transportation Research Circular E-C105. Factors Affecting Compaction of Asphalt Pavements. Transportation Research Board, 163-168.

[50]. Nose, Yukinori. (2006) "Vibratory Pneumatic Tire Roller." Transportation Research Circular EC105.Factors Affecting Compaction of Asphalt Pavements. Transportation Research Board, 54-68.

[51]. Oluwasola, E.A., Hainin, M. R., and Aziz, M.M.A. (2015). Evaluation of asphalt mixtures incorporating electric arc furnace steel slag and copper mine tailings for road construction. Transportation Geotechnics.2: 47-55.

[52]. Pellinen, T.K. (2001), Investigation on the use of dynamic modulus as an indicator of hot mix asphalt performance, Doctoral thesis, Arizona State University.

[53]. Roberts, F.L.; Kandhal, P.S.; Brown, E.R.; Lee, D.Y. and Kennedy, T.W. (1996). *Hot Mix Asphalt Materials, Mixture Design, and Construction*. National Asphalt Pavement Association Education Foundation. Lanham, MD

[54]. Rowe,G. M.; Hakimzadeh, S. and Phillip Blakenship, P. E. (2008) "Evaluation of Aspects of E* Test Using HMA Specimens with Varying Void Contents, Abatech Inc.
[55]. Singh, D., M. Zaman, and S. Commuri. (2011). Effect of Long Term Aging on Dynamic Modulus of Hot MixAsphalt. *Geotechnical Special Publication*, Vol. 211, pp. 4773–4781.
[56]. J. MuraliKrishan and V. R. Rengaraju (1998). Transportation Engineering Division, Department of Civil Engineering, Indian Institute of Technology Madras
[57]. Yusoff, N.I.M., Jakarni, F.M., Nguyen, V.H., Hainin, M. R., and Airey, G.D. (2013). Modelling the rheological properties of bituminous binders using mathematical equations. Construction and Building Materials. 40: 174-188.