EFFECTS OF MINERAL FILLERS ON THE STABILITY AND VOLUMETRIC PROPERTIES OF ASPHALT

BY

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CERTIFICATION

This is to certify that the project topic titled “Effects of Mineral Fillers on the Stability and Volumetric Properties of Asphalt” was done by Chisom Daniel Chukwubuike with registration number (2017224005) in the Department of Civil Engineering, Nnamdi Azikiwe University, Awka, Anambra State.

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APPROVAL PAGE

This research work “Effects of Mineral Fillers on Stability and Volumetric Properties of Asphalt” has been assessed and approved by department of civil engineering Nnamdi Azikiwe University.

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DEDICATION

This work is dedicated to God Almighty, the giver of knowledge, wisdom and my protector.

ACKNOWLEDGEMENT

Special gratitude belongs to God Almighty for giving me the strength to complete this work and also for His guidance and protection throughout my stay in Nnamdi Azikiwe University.

My profound gratitude to my parents; Mr and Mrs Gabriel Chukwubuike for their constant prayers and financial support throughout my stay in school. I also want to thank my siblings Okenna, Chidozie, Chidinma & Ifeoma for their encouragement during trying times of my academic pursuit.

My immense gratitude also goes to my project supervisor in the person of Rev. Dr. C. M. Nwakaire for his time and guidance in the accomplishment of this project. May the lord enlarge your coast and also protect your family.

Special thanks goes to the Head of department in the person of Engr. Prof. C. A. Ezeagu and all the staff and lecturer in the department of civil engineering for their invaluable tutorship and professional guidance.

Finally, I will like to appreciate everyone who has in one way or the other contributed to making me a better person, may the Lord Almighty reward you all greatly.

ABSTRACT

The study was carried out to evaluate the effect of mineral fillers on the stability and volumetric properties of asphalt. The mineral fillers used were granite dust, Portland cement and lime. The optimum binder content was determined from asphalt mix produced with 100% granite dust at bitumen content ranging from 5%, 5.5% 6%, 6.5% and 7% by weight of the aggregate and the predetermined optimum binder content was used for production of asphalt mix with mineral fillers such as 100% lime, 100% cement, 50% granite dust: 50% cement, 50% granite dust: 50% lime, 50% cement: 50% lime. The bitumen aggregates and asphalt was subjected to various testing. Test conducted on the bitumen sample are: viscosity test, penetration test, fire and flash point test and specific gravity test. Test conducted on the aggregate were sieve analysis test, specific gravity test, aggregate impact and crushing value test and water absorption test. Test conducted on the asphalt mix was marshal stability test. Results obtained showed that the bitumen samples satisfied the specification given by Federal Ministry of Works and Housing, (1997) as the specific gravity was 1.03, flash and fire point was 287.7^0C and 316.7^0C, viscosity was $0.73Ns/m^2$ and penetration was 69.33mm, the aggregate samples also satisfied the specification given by Federal Ministry of Works and Housing, (1997) as the specific gravity ranged from 2.55-2.76, impact and crushing value was 6.5% and 4.3%, the water absorption of the aggregate was 0.9%, the specific gravity of the aggregate ranged from 2.55-2.76. The optimum binder content was 6%. Assessment of the stability and volumetric properties of asphalt produced with mineral fillers such as 100% granite dust, 100% lime, 100% cement, 50% granite dust: 50% cement, 50% granite dust: 50% lime, 50% cement: 50% lime revealed that the flow ranged from 2.67mm – 3.5mm, void in mineral aggregate ranged from 18.7% - 24.64%, void in total mix ranged from 4.8% - 11.75%, void filled by bitumen ranged from 52.6% - 74.9%, it was observed that the asphalt produced with the different mineral fillers and at different mix combination satisfied the specification given by Federal Ministry of Works and Housing, (1997) for stability, flow and void in mineral aggregate but failed to meet the requirement for void filled by bitumen and void in total mix. The study therefore concluded that Portland cement and granite dust has better effect on stability and volumetric properties of asphalt than hydrated lime.

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LIST OF SYMBOL & ABBREVIATION

G_S – Specific Gravity

OBC—Optimum Binder Content

GD—Granite Dust

GT—Granite

HL – Hydrated Lime

PC—Portland cement

AASHTO – American Association of State Highway and Transportation Officials

USCS – Unified Soil Classification System

ASTM – American Society for Testing and Material

BSL – British Standard Light

BSH – British Standard Heavy

D_{10} – Particle Size such that 10% is finer than the Size

D_{30}- Particle Size such that 30% is finer than the Size

D_{60}- Particle Size such that 60% is finer than the Size

C_U – Coefficient of Uniformity

C_C – Coefficient of Curvature

SC – Clayey Sand

SM – Silty Sand

GM – Silty Gravel

GC—Clayey Gravel

GW—Well Graded Gravel

GP—Poorly Graded Gravel

SP—Poorly Graded Sand

SW—Well Graded Sand

CL – Inorganic Clay of Low Plasticity (lean clay)

CH—Inorganic Clay of High Plasticity (fat clay)

ML- Silt of low Plasticity

MH – Silt of High Plasticity

CHAPTER ONE

INTRODUCTION

1.1 Background of study

In the top of the pavements on roads and airports, asphalt mixtures are employed. Typically, asphalt cements and aggregate make up the mix. Some asphalt mixture types are also utilized in base course. Similar to the design of other engineering materials, the design of other engineering materials, the design of an asphalt paving mix mostly involves choosing and balancing the component ingredients to produce the desired qualities in the finished pavement structure.

Typically, the term “mineral filler” refers to a mineral fine particle whose physical size passes through a filter with a mesh size of 200 (75 micron). By decreasing the natural temperature sensitivity of the binder, mineral filler is used and applied in asphalt mixtures to improve the qualities of the binder. By decreasing the natural temperature sensitivity of the binder, mineral filler is used and applied in asphalt mixtures to improve the qualities of the binder.

Mineral fillers play two roles in asphalt mixtures. Firstly, they act as a part of the mineral aggregate by filling the voids between the coarser particles in the mixtures and thereby strengthening the asphalt mixture, and secondly, when mixed with asphalt, fillers form mastic, a high-consistency binder or matrix that cements larger binder particles together; most likely, a major portion of the filler remains suspended in the binder while a smaller portion becomes part of the load bearing framework.

Being a thicker material than asphalt itself, mastic can bring more stiffness to the mixture. Additionally, it improves the adhesive qualities and provides greater thickness of asphalt binder which eventually helps to slow down the aging process. The filler material passing the number 200 standard mesh sieve, usually comprise a major amount of the total aggregate in SMA (Stone mastic asphalt) pavement. This large portion of the mineral filler with asphalt binder contributes to the interlocking of the coarse aggregate and mineral fillers which are of the same mineralogical composition as coarse aggregates but have variations in surface roughness and angularity; usually show different capacities to adsorb a given type of asphalt component; fillers particles absorb a portion of the oils in asphalt binder and the particles swell during the

interaction with asphalt binder; therefore increasing the viscosity and stiffness of the asphalt-filler mastic.

Whether a mix design is developed through a Marshall, Hveem, or Superpave mix design process there are basic volumetric requirements of all. Volumetric properties are properties of a defined material contained in a known volume. Asphalt mixture volumetric properties can include bulk specific gravity, theoretical maximum specific gravity, air void and voids in mineral aggregate.

Many agencies specify values of the volumetric properties to ensure optimum performance of the pavement. The asphalt mixture must be designed to meet these criteria. In production the asphalt mixture is evaluated to determine if the mix still meets the specifications and is consistent with the original mix design (JMF). The production asphalt mixture may vary from the mix design and may need to be modified to meet the specific volumetric criteria. To compare the in-production volumetric properties to agency specifications and the JMF a sample of loose asphalt mixture mix is obtained in accordance with FOP for AASHTO R 97. The sample is then compacted in a gyratory compactor to stimulate in-place asphalt mixture pavement after it has been placed, compacted and the volumetric properties of the compacted sample are determined. The volumetric proportions of the asphalt binder and aggregate components of an asphalt mixture and their relationship to the other components are considered. The mass of the components and their specific gravities are used to determine the volumes of each of the components in the mix. The volumetric properties of a compacted asphalt mixture: air voids (Va), voids in mineral aggregate (VMA), voids filled with asphalt binder (VFA), and effective asphalt binder content (Pbe) provide some indication of the mixture's probable performance.

1.2 Statement of Problems

One of the most complicated failures in asphalt pavement is the moisture damage, therefore asphalt pavement shows loss in the mixture structure in term of aggregate particles bounding. Since, filler contributes in asphalt durability, it was a major concern for highway engineers to find the effect of filler on asphalt durability. Airey et al (2008), examined the effect of filler on asphalt pavement moisture damage by AASHTO T283 test and Saturation Ageing Tensile

Stiffness (SATS) which was developed and later used to evaluate the moisture damage of asphalt in the UK. Granite filler and hydrated lime was used with conventional limestone filler. The result shown that the granite filler appears to have lesser performance than the conventional limestone filler. The result shown that the granite filler appears to have lesser performance than the conventional limestone filler. On the other hand, hydrated lime improved the resistance to moisture damages. Thus, filler type may have an impact on the asphalt durability. According to Cong and Zheng (2005), the performance of hot-mix asphalt could be affected by filler/asphalt ratio by using two filler types (Gabbro and limestone powder). Marshall method was used to find the optimum binder and the filler content which showed decreasing in the optimum binder content as the filler to the asphalt ratio was increased. Stability was increased and the strain decreased when the filler to asphalt ratio increased too. This was probably due to the proper asphalt binder film thickness around aggregate particles and stiffening the mix, however excess of filler in the mixture can make it susceptible to cracks and raveling.

1.3 Aim and Objectives

The primary aim of this study is to find out the influence of different filler types on the stability and the volumetric properties of Asphalt.

The specific objective of this study includes:

1. To evaluate the physical properties of selected aggregates and Bitumen in order to check their suitability for construction of pavement wearing course.
2. To conduct asphalt mix design for pavement wearing course using Marshall method of mix design.
3. To identify the effect of different mineral fillers on the stability and volumetric properties of asphalt

1.4 Significance of study

Quality and highly acceptable asphalt will be produced through this research. A poor or inappropriate Asphalt mix design can contribute to poor pavement performance. So it is important that a mix design be done properly. Good design procedures are based on sound research and many years of observing the performance of asphalt pavements. A good mix design procedure closely model the performance of the actual mix that will be produced, including

binder absorption, compaction during construction and under future traffic, moisture damage sensitivity, and rutting and fatigue properties.

The purpose of this research is to find the influence of fillers on the stability of asphalt while also determining how fillers affect the volumetric properties. It should be recognized that a mix design is just the starting point for achieving the desired asphalt pavement performance. Other factors, such as structural design, construction practices, and maintenance operations significantly influence pavement performance.

The benefits of this study helps the engineer in the construction of high way pavements since mineral fillers affect the bonding between asphalt and aggregates and improve the performance of the mix by increasing the cohesive performance of the mix under moisture exposure. Fillers are also improving resistance to deformation at high temperatures by stiffening the binder.

CHAPTER TWO

LITERATURE REVIEW

2.1 History of Asphalt Pavement

Hot mix asphalt (HMA) pavements have existed in their present form, as a mixture of angular aggregates and asphalt binder, since the beginning of the 20th century. However, HMA pavement can trace its roots back to ancient Roman roads and beyond.

The first recorded use of asphalt by humans was by the Sumerians around 3,000 B.C. Statues from that time period used asphalt as a binding substance for inlaying various shells, precious stones and pearls. Other common ancient asphalt uses were preservation (for mummies), waterproofing (pitch on ship hulls), and cementing (used to join together bricks in Babylonia). Around 1500 A.D., the Incas of Peru were using a composition similar to modern bituminous macadam to pave parts of their highway system. In fact, asphalt is mentioned several times in the Book of Genesis (Baird, 2002).

In more modern times, asphalt paving uses first began with foot paths in the 1830s and then progressed to actual asphalt roadways in the 1850s. The first asphalt roadways in the U.S. appeared in the early 1870s (Abraham, 1929).

2.1.1 Roman Roads

The oldest Roman road still in use today, Via Appia (Figure 2.1), dates back to 312 B.C. At its height, the Roman road network consisted of over 62,000 miles of roads. By law, all of the public was entitled to use Roman roads, but the maintenance of the roadway was the responsibility of the inhabitants of the district through which the road ran (the same basic system used in the U.S. today). Although Roman roads did not use asphalt as a binder, they did often use lime grout and other natural pozzolans as binders. Figure 2.2 shows a typical Roman road structure.

Figure 2.1: Roman Road Surface (Asphaltwa, 2010)

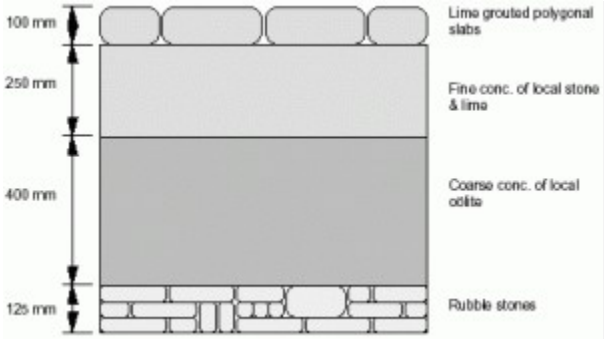


Figure 2.2: Roman Road Structure (Asphaltwa, 2010)

2.1.2 Telford Pavements

Skipping forward several thousand years, Telford pavements begin to show likeness to today's modern HMA pavements. Thomas Telford (born 1757) served his apprenticeship as a building mason (Smiles, 1904). Because of this, he extended his masonry knowledge to bridge building. During lean times, he carved grave-stones and other ornamental work (about 1780). Eventually, Telford became the "Surveyor of Public Works" for the county of Salop (Smiles, 1904), thus turning his attention more to roads. Telford attempted, where possible, to build roads on relatively flat grades (no more than a 1 in 30 slope) in order to reduce the number of horses needed to haul cargo. Telford's pavement section was about 14 to 18 inches in depth as shown in Figure 2.3. Telford pavements did not use any binding medium to hold the stones together.

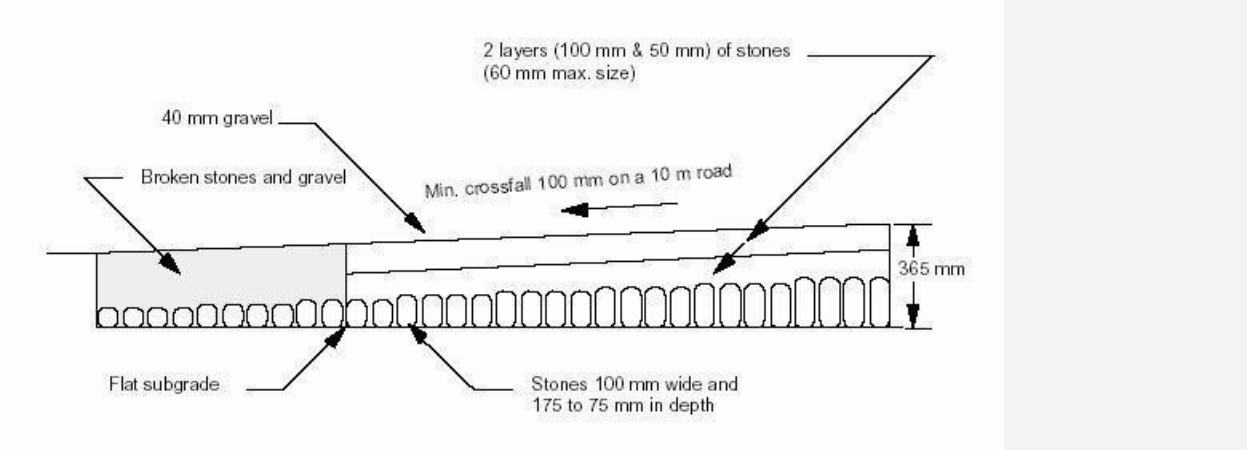


Figure 2.3: Typical Telford Road (Collins and Hart, 1936)

2.1.3 Macadam Pavements

Macadam pavements introduced the use of angular aggregates (Figure 2.4). John McAdam (born 1756 and sometimes spelled "Macadam") observed that most of the "paved" U.K. roads in early the 1800s were composed of rounded gravel (Smiles, 1904). He knew that angular aggregate over a well-compacted subgrade would perform substantially better. He used a sloped subgrade surface to improve drainage (unlike Telford who used a flat subgrade surface) onto which he placed angular aggregate (hand-broken, maximum size 3 inches) in two layers for a total depth of about 8 inches (Gillette, 1906). On top of this, the wearing course was placed (about 2 inches thick with a maximum aggregate size of 1 inch) (Collins and Hart, 1936). Macadam, who did not use any binding medium to hold the stones together, realized that the layers of broken stone would eventually become bound together by fines generated by traffic. The first macadam pavement in the U.S. was constructed in Maryland in 1823.

Figure 2.4: Macadam Pavement Core (Asphaltwa, 2010)

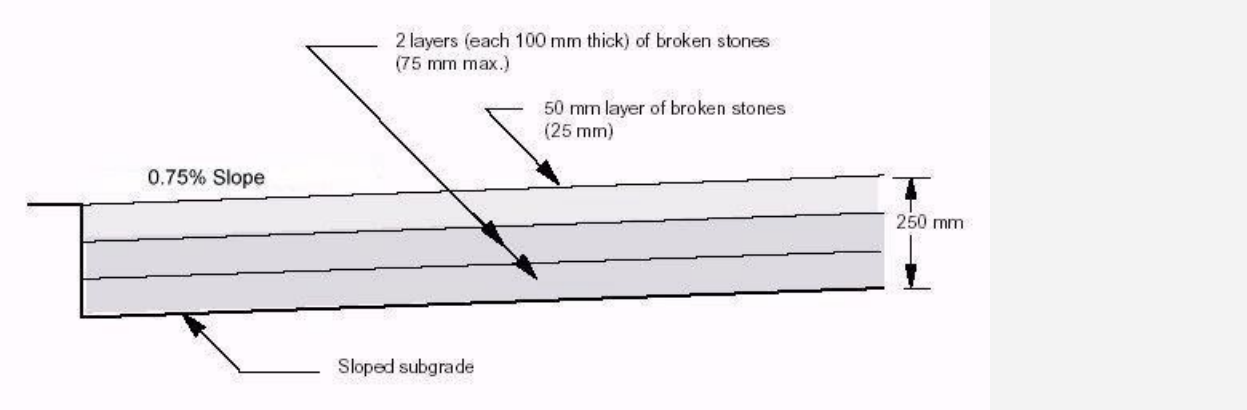


Figure 2.5: Typical Macadam Road (Collins and Hart, 1936)

2.1.4 Tar Macadam Pavements

A tar macadam road consists of a basic macadam road with a tar-bound surface. It appears that the first tar macadam pavement was placed outside of Nottingham (Lincoln Road) in 1848 (Hubbard, 1910; Collins and Hart, 1936). At that time, such pavements were considered suitable only for light traffic (i.e., not for urban streets). Coal tar, the binder, had been available in the U.K. from about 1800 as a residue from coal-gas lighting. Possibly this was one of the earlier efforts to recycle waste materials into a pavement.

As a side note, the term "Tarmac" was a proprietary product in the U.K. in the early 1900s (Hubbard, 1910). Actually, it was a plant mixed material, but was applied to the road surface "cold." Tarmac consisted of crushed blast furnace slag coated with tar, pitch, Portland cement and a resin. Today the term "tarmac" is generic and generally refers to airport pavements (however, inappropriately).

2.1.5 Sheet Asphalt Pavements

Sheet asphalt placed on a concrete base (foundation) became popular during the mid-1800s with the first one of this type being built in Paris in 1858. The first such pavement placed in the U.S. was in Newark, New Jersey, in 1870. Generally, the concrete layer was 4 inches thick for "light" traffic and 6 inches thick for "heavy" traffic (Baker, 1903). The final thickness was based on the weight of the traffic, the strength of the concrete and the soil support.

2.1.6 Bitulithic Pavements

HMA pavement began to take on its modern form around the beginning of the 20th century when Frederick J. Warren was issued patents for a “hot mix” asphalt paving material and process, which he called “bitulithic”. A typical bitulithic mix contained about 6 percent “bituminous cement” and graded aggregate proportioned for low air voids. The concept was to produce a mix which could use a more “fluid” binder than was used for sheet asphalt. Warren received eight patents in 1903. A review of the associated claims reveals that Warren, in effect, patented HMA, the asphalt binder, the construction of HMA surfaced streets and roads, and the overlay of “old” streets.

In 1910 in Topeka, Kansas, a court ruling stated that HMA mixes containing 0.5 inch maximum size aggregate did not infringe on Warren's patent (Steele and Himmelman, 1986). Thus, most U.S. hot mix asphalt (HMA) thereafter became oriented to the smaller maximum aggregate sizes. A typical “Topeka mix” consisted of 30 percent graded crushed rock or gravel (all passing the 0.5 inch sieve), about 58 to 62 percent sand (material passing the No. 10 sieve and retained on the No. 200 sieve), 8 to 12 percent filler (material passing the No. 200 sieve). This mixture required 7.5 to 9.5 percent asphalt cement. By 1920, Warren's original patents had expired in the U.S. (Oglesby and Hewes, 1962) but the legacy of the Topeka mix lived on as reflected by the U.S. tendency towards finer mixes.

2.2 Fillers and Other Types of Fillers

2.2.1 Fillers

The general category of filler for asphalt paving mixtures includes finely divided mineral particles which are either naturally present in the mineral aggregate or which are added to the mineral aggregate (Izatt, 1982). Filler particles are considered those passing the No. 200 (75 um) sieve. (One um is one-millionth of a meter.)

2.2.2 Mineral Filler (Added Filler)

This material is that part of the finely divided portion of a hot-mix aggregate gradation which is not present naturally and has to be added to beef the gradation up (Rigden, 1947). Particles sizes

of this material are smaller than the No. 200 (75 um) sieve. If this material contains particles larger, these are considered as “Fine Aggregate" in the preparation and testing of asphalt paving materials.

2.2.3 Special Fillers

These materials are finely divided mineral matter water passing the No. 200 (75 um) sieve which are usually by-products from the processing and production of products other than mineral aggregates and which normally require special handling methods and equipment (Izatt, 1982) . Included in this category are bag house fines, fly ashes, flue dusts and others.

2.2.4 Filler Additives

These are finely divided materials whose particle sizes are normally smaller than the No. 200 (75 um) sieve and have unusual or unique properties such as being rod-like particles (fibers), polymers (including rubber) or asphalt extenders such as carbon black and sulfur (Izatt, 1982). Hydrated lime is included in the group of filler additives since it is highly surface active and its use is normally restricted to two percent or less by weight of a hot-mix.

2.3 Importance of Fillers

Mineral filler, fine-grained mineral particles naturally present in or manufactured and added to aggregates play a significant role on the performance of asphalt mastic and asphalt mixtures (Chen and Xu, 2020). A clear understanding of effects of filler on the properties of asphalt paving mixtures is critical to ensure a good asphalt mixture design and its field performance. In this study, the basic properties of mineral filler, mainly including physical and chemical properties, were first reviewed and followed by the effects on mastic and mixture performances. This review finds that particle size distribution and specific surface area are the two most important physical characteristics for fillers in terms of its impact on mastic performances. Fillers, including diatomite, hydrated lime and cement, can improve the high temperature and durability performances of mastic and mixture. However, fillers, like glass powder, steel slag and bentonite, have a detrimental effect on the low-temperature performances. It can be concluded that it is important to carefully select the mineral filler type and proportion in order to improve the asphalt mixture performances.

2.4 Introduction to Asphalt

Asphalt is a mixture of aggregate (gravel and sand) and bitumen binder, and when mixed together at a high temperature it forms hot mix asphalt. This is the type of asphalt that we choose to use on all our asphalt projects including car parks, hardstands and large driveways. The asphalt mixture is a typically temperature-sensitive material; its mechanical characteristics and operational performance will also dramatically change with temperature variations. The asphalt mixture will perform differently under different temperature conditions. When the temperature is high, the viscosity of the binder will dramatically decrease, as will the adhesion among the aggregates. Meanwhile, the stiffness of the asphalt mixture also decreases, and there will be a large accumulated and permanent deformation under each repeat loading. Therefore, under high-temperature conditions, the asphalt pavement structure will have compression and rutting because of the viscosity of the asphalt mixture. When the temperature decreases, although the strength of the asphalt mixture will increase, the thermal stress in the asphalt mixture will also increase; sometimes, it will surpass the material strength and cause thermal cracking. Other distresses upon the asphalt pavement, such as fatigue cracking, reflective cracking, etc., are also directly or indirectly related to the temperature condition of the asphalt mixture. A full understanding of the characteristics and patterns of temperature fields in an asphalt pavement structure not only helps with accurately predicting the temperature distribution, but it can also can help with understanding the mechanism of pavement distress, and determine the strength of the asphalt pavement materials.

2.5 Benefits of Asphalt

According to Augustyn (2020), The use of asphalt is very old, dating back to its use as a water stop between brick walls of a reservoir at Mohenjo-Daro (about the 3rd millennium BC) in Pakistan. In the Middle East it was extensively used for paving roads and sealing waterworks, important applications even today. The Pitch Lake on the island of Trinidad was the first large commercial source, but natural sources have since declined in importance as petroleum became the major source. Gilsonite, Wurzilite, and similar vein asphalts have special uses in heat-resistant enamels; they are hard and are mined like coal. Petroleum asphalt is produced in all consistencies from light road oils to heavy, high-viscosity industrial types.

Asphalt softens when heated and is elastic under certain conditions. The mechanical properties of asphalt are of little significance except when it is used as a binder or adhesive. The principal application of asphalt is in road surfacing, which may be done in a variety of ways. Light oil "dust layer" treatments may be built up by repetition to form a hard surface, or a granular aggregate may be added to an asphalt coat, or earth materials from the road surface itself may be mixed with the asphalt. Other important applications include canal and reservoir linings, dam facings, and other harbor and sea works; asphalt so used may be a thin, sprayed membrane, covered with earth for protection against weathering and mechanical damage, or thicker surfaces, often including riprap (crushed rock). Asphalt is also used for roofs, coatings, floor tilings, soundproofing, waterproofing, and other building-construction elements and in a number of industrial products, such as batteries. For certain applications an asphaltic emulsion is prepared, in which fine globules of asphalt are suspended in water.

Asphalt not only offers superior resilience to ants, weeds and water, compared to other paving and construction materials, it's also one of the most cost-effective solutions for both commercial and residential projects. Compared to paving materials like bricks, slabs or pavers, asphalt has more flexibility to shape to curves and corners, allowing for a tight seal with and kerbing and reducing the gaps where water and debris can penetrate. Bricks, slabs and pavers are also venerable to movement and shifting causing separation between each piece.

2.6 Different Types of Asphalt Mix

According to (Bituminous Roadways .Inc) there are three (3) types of asphalt mixes.

2.6.1 Hot Mix Asphalt

Hot mix asphalt, commonly referred to as HMA, bitumen, or blacktop, is the most widely used type of asphalt in major paving projects. There are a few different types of HMA, each of which has unique performance properties and ideal usage applications. They include:

(A) Dense-Graded Mixes

Properly designed and constructed dense-graded mixes are, for the most part, impermeable, which makes them suitable for all traffic conditions and pavement layers.

There are two types of dense-graded mixes: fine-graded and coarse-graded, each of which is characterized by the size of the majority of the aggregate particles in the mix. These mixes can be used for surfaces, as well as asphalt repairs, and are the most commonly used and well-understood type of HMA in the U.S.

(B) Stone Matrix Asphalt

Also known as SMA or stone mastic asphalt, this type of mix is a gap-graded HMA designed to increase rutting resistance and durability. Because aggregates do not deform under load to the degree that asphalt binder does, SMA consists of a higher ratio of course stone aggregates to achieve stone-on-stone contact within the mix.

That contact, combined with a higher asphalt content than HMA, allows SMA to resist deformation, making it suitable for high-volume roadways. This type of mix also helps reduce tire noise and reflective cracking. Because SMA consists of highly durable aggregates, it's generally about 20%-25% more expensive than its other HMA counterparts.

(C) Porous Asphalt

This type of asphalt mix has a greater purpose, stormwater management. Porous asphalt is made to allow water to drain through the asphalt into a layer of aggregate, designed to naturally filter the water and reduce the amount of pollutants carried to a waterway. Filtration of runoff volume can reach nearly 80% with proper installation and maintenance. Porous asphalt consists of standard bituminous asphalt, but the void spaces are approximately 16% as opposed to two to three percent for conventional asphalt mixes. Developers for residential, commercial, and industrial sites might find this asphalt mix helpful to decrease the amount of stormwater runoff, especially when there is increased velocity and volume.

2.6.2 Warm mix asphalt

Like HMA, warm mix asphalt (WMA) is heated during the production process. But, unlike HMA, WMA contains either water or organic/chemical additives that reduce its required production temperature. Notably, WMA production requires less oil/fuel consumption and

produces fewer emissions than HMA production, making it more cost effective and environmentally friendly.

WMA's lower production temperature allows it to remain more workable as it cools as there's ultimately a smaller difference between the temperature of the mix and the surrounding air. Asphalt transport trucks can also haul warm mix farther distances from the plant to the project site given its lower temperature. And, since WMA has a lower viscosity than HMA, it can be compacted at a lower temperature than HMA.

Presently, WMA is relatively new in pavement application in the U.S., so long-term performance data is currently limited. However, WMA has experienced rapid adoption and it's growing in popularity due to its versatility, durability, and usability outside normal asphalt paving weather conditions. Warm mix asphalt is durable enough to withstand heavy traffic load and can be used in virtually all pavement applications.

2.6.3 Cold mix asphalt

As its name indicates, cold mix asphalt (CMA) is not heated at all during production, which allows it to be transported and laid while cold. But CMA isn't nearly as durable as HMA or WMA, so it's really only suitable for patching and other asphalt repairs. Ideally, CMA should serve as a temporary fix in low-traffic areas and should generally be replaced with a warm or hot mix when weather conditions allow.

2.7 Stability of Asphalt

The two principal modes of failure in pavements are fatigue cracking and permanent deformation. Engineers seek to hold these forms of failure to acceptable limits within a pavement design life. Fatigue resistance of an asphalt mixture is the ability of the mixture to withstand repeated bending without fracture. It is one of the common forms of distress in asphalt pavements and manifests itself in the form of cracking under repeated traffic loading or a series of temperature fluctuations/variation in the pavement. Fatigue cracking initiates at the bottom of asphalt base and appears on the pavement surface as interconnected tracks of different forms and it may also start at the surface and grow downwards as is the case for thermal (fatigue) cracking. Some forms of fatigue cracking include longitudinal cracking, transverse cracking, and block

cracking. The published studies on fatigue resistance indicate that hydrated lime improves the fatigue resistance of asphalt mixtures in 77% of the cases (EuLA, 2010). Permanent deformation and rutting are used interchangeably. Permanent deformation is caused by gradual buildup of irrecoverable strains under repeated loading which develop into a measurable rut (permanent depression along the wheel path). These strains are due to the visco-elastic response of bituminous materials to dynamic loading. Figure 2.6 shows the visco-elastic response to millions of wheel loadings. Rutting causes hydroplaning and safety concern for road users. It can develop into potholes/structural failure of the pavement if not corrected. In the past, subgrade deformation was considered to be the primary cause of rutting and many pavement design methods applied limiting criteria on vertical strain at the subgrade level also. However recent research indicates that most of the rutting occurs in the upper part of the asphalt surfacing layer. According to Brown (1997), a common misconception is that the subgrade strain criterion only refers to permanent deformation in the subgrade.

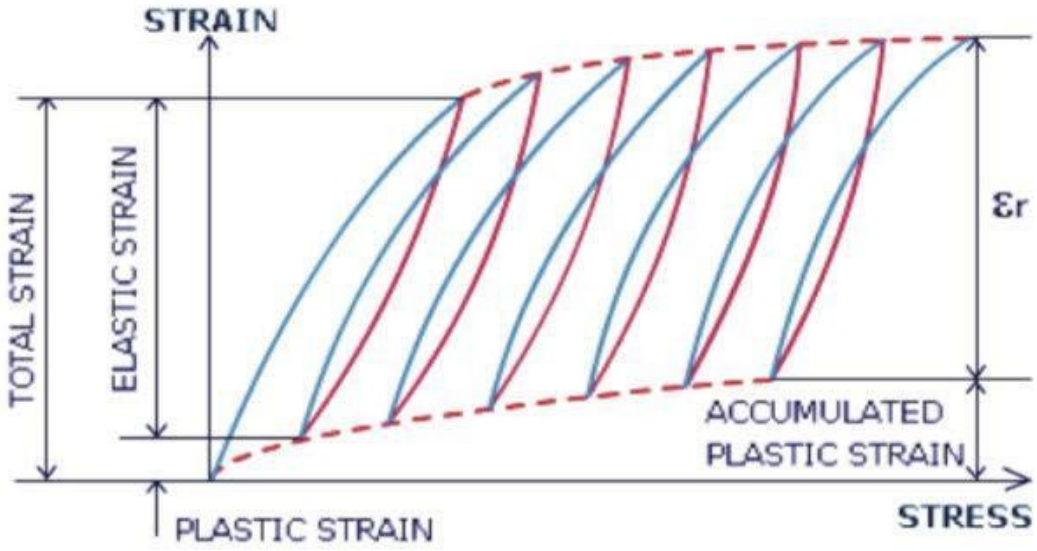


Fig. 2.6. Accumulated Plastic strains in Pavements (Asphalt Institute, 1996).

Eisemann and Hilmar (1987) studied asphalt pavement deformation phenomenon using wheel tracking device. They measured the average rut depth as well as the volume of displaced materials below the tyres and in the upheaval zones adjacent to them and found that, at the initial

stages of trafficking, the increase of irreversible deformation below the tyre is distinctly greater than the increase in the upheaval zones and concluded that at the initial phase, traffic compaction or densification is the primary mechanism of rut development (See Figure 2.7a), while after the initial stage, the volume decrease below the tyre is approximately equal to the volume increase in the adjacent upheaval zones, which implies that most of the compaction under traffic is completed and further rutting is caused essentially by shear deformation, i.e., distortion without volume change (See Figure 2.7b). Thus, they concluded that, shear deformation is considered to be the primary mechanism of rutting for the greater part of the lifetime of the pavement.

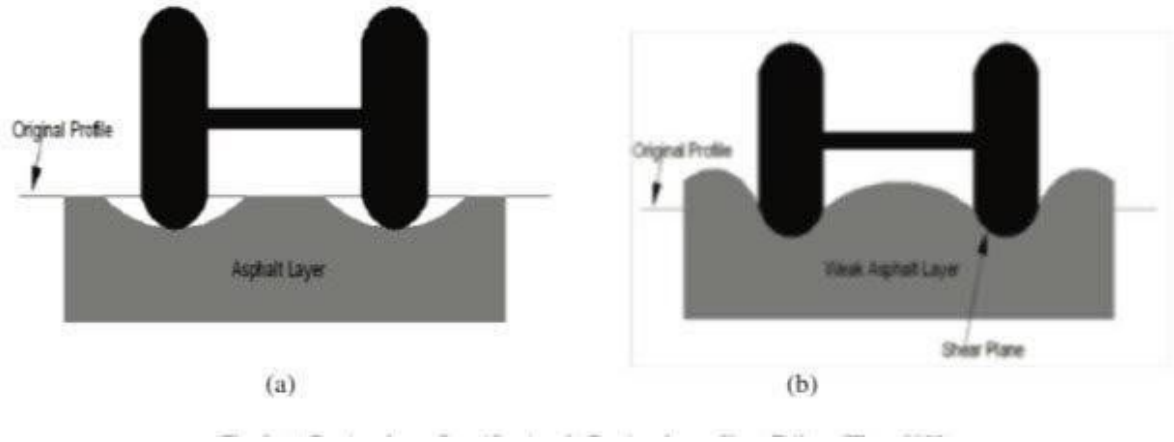


Fig. 2.7 (a) Rutting due to Densification (b) Rutting due to Shear Failure (Khan, 2008).

There are two major causes of permanent deformation, which are the use of weak asphalt and weak subgrade. The focus of this paper is on failure resulting from weak asphalt. Rutting resulting from accumulation of permanent deformation in the asphalt layer is now considered to be the principal component of flexible pavement rutting (Garba, 2002). In Nigeria, the trucks/tankers are relied on for the movement of freight (oil and gas products, finished products, raw materials etc). The increase in truck tyre pressures and axle loads put the asphalt mixtures under increasingly high stresses. Some of the factors that cause weak asphalt mixture include aggregates gradation, aggregate surface texture, voids in the asphalt mixture, air voids and the voids in the aggregate skeleton filled with bitumen, type of binder and temperature. The problems of fatigue cracking and permanent deformation have been addressed using different approaches. These include the use of harder bitumen, better design method like the superpave mix design, the use polymer modified asphalt, the use of grouted asphalt, and the use of lime-

modified asphalt, which is the main thrust of this paper. Lime acts as active filler, anti-oxidant, and as an additive that reacts with clay fines in asphalt. These mechanisms create multiple benefits for pavements:

(1) Hydrated lime acts as mineral filler, stiffening the asphalt binder and asphalt.

(2) It improves resistance to fracture growth at low temperatures.

(3) It favorably alters oxidation kinetics and interacts with products of oxidation to reduce their deleterious effects.

(4) It alters the plastic properties of clay fines to improve moisture stability and durability.

The filler effect of the lime in the asphalt reduces the potential of the asphalt to deform at high temperatures, especially during its early life when it is most susceptible to permanent deformation. The hydrated lime filler actually stiffens the asphalt film and reinforces it. It makes the asphalt less sensitive to moisture effects by improving the aggregate-asphalt bond, which improves rut resistance (Little and Epps, 2001). The results of laboratory wheel tracking tests conducted by Collins et al., (1997) indicate that hydrated lime increases resistance to rutting and permanent deformation. LMA (2004) carried out a test to determine the dynamic modulus of lime-modified and unmodified asphalt. The comparison of lime-modified and unmodified HMA mixtures indicates that the addition of lime increases the overall dynamic modulus by about 25% Higher modulus asphalt layers reduces the irrecoverable deformation in the foundation layers because of reduced vertical stresses, resulting in less rutting in the lower layers. Kok and Yilmaz (2009) studied the effects of SBS (Styrene-Butadiene-Styrene) and lime as mineral filler in hot mix asphalt. They found that the stability of unconditioned lime treated mixtures was approximately 8% higher than those of the unconditioned control mixture. However, this value increased up to 21% for the conditioned mixtures. According to retained Marshall stability, they concluded that the addition of only 2% lime had approximately same effect with addition of 6% SBS with regard to moisture damage. Mohan and Obaid (2014) investigated the effect of adding hydrated lime on the moisture damage resistance of asphalt concrete mixtures. In their study, they observed that the Marshall stability increased with increasing lime content, with rate equal

to (40%) for (2.5%) hydrated lime content. Also, they reported that the air voids decreased with increasing hydrated lime content, especially at the optimum content (2%) hydrated lime.

G.G. Al-khateeb et al, (2011) reported on the use of Ordinary Portland Cement (OPC) as a filler substitute to improve the rigidity of asphalt concrete mixes made with B60/70 bitumen and low-quality aggregates. This new mixture increases the pavement's stability and resistance to high temperatures. To establish the effects of OPC on the performance of asphalt mixtures in hot climates, four different percentages of OPC (0%, 2%, 4% and 6%) are used as filler substitutes in three different mixes. The performance of the three mixes are assessed using the Superpave Gyratory Compactor and the Asphalt Pavement Rutting Analyzer. Findings indicate that mixtures containing higher percentages of OPC as a filler are significantly more resistant to rutting. These experimental results show that Portland Cement Filler Asphalts (PCFA) represents a more stable alternative to conventional asphalt that also reduces thickness requirements, because of the higher resulting modulus of rigidity. This is equally important in hot rural areas like those found in Libya, where they have very few quarries and aggregates are very costly to transport.

Rashwan (2016) conducted an experiment on the effect of Limestone powder as a mineral filler on asphalt. The research showed a laboratory study of the effect of using different contents of limestone powder as mineral filler in hot mix asphalt HMA properties and performance. In this investigation limestone powder was used as mineral filler because it is the most common filler used in Egypt and its good evaluation in improving the HMA performance and properties, three percentages of limestone powder were used in this study (4, 6 and 8% of the total mix). Marshall Tests and indirect tensile test were performed to investigate the difference in behaviors of different samples with different parameters. The control mix used in this study contains crushed gravel, rough aggregate particles with medium gradation of aggregate. The results revealed that Marshall Stability (MS), Marshall Quotient (MQ) and air void (VTM) were decreased with increasing limestone powder content; However flow values and bulk density were increased with increasing limestone powder content. Also, the results indicated that the indirect tensile test (ITS) and stiffness modulus (SM) were decreased with increasing limestone powder content, while strain failure was increased with increasing the content of limestone powder. It can be

concluded that using of limestone as mineral filler in the HMA is proposed because it is common in Egypt and reduce the cost but shouldn't exceed 4% of the total hot asphalt mix.

CHAPTER THREE

MATERIALS AND METHODS

This chapter presents the materials and methods used to achieve the research goal. This process is stated below:

3.1 Collection and Preparation of Materials

3.1.1 Bitumen

The bitumen designated as BT was grade 60/70 penetration bitumen collected from Infrastructural Development Company located along Enugu-Onitsha Express way. The bitumen was collected in an empty ten (10) liters gallon and stored at the laboratory testing unit of the construction company. The properties of the bitumen used for the asphalt production were determined through penetration, flash and fire point and viscosity test.

3.1.2 Lime

Lime used for the experimental study was designated as LM was purchased from a vendor at Onitsha Market Anambra State Nigeria and was conveyed to the laboratory unit of Infrastructure Development Company located along Enugu-Onitsha Express way. The hydrated lime was in powdered form. It was observed to be very smooth and whitish in colour. The lime sample was kept in a safe location preparatory to laboratory testing. During testing, hydrated form of the lime sample mixed with Portland cement will be used to partially coarse aggregate (granite) in the asphalt mix.

Plate 3.0: Photograph of Hydrated Lime used for the Study

3.1.3 Portland Cement

Ordinary Portland cement (Dangote cement) was used for the experimental study. The cement was purchased at building material market in Onitsha Anambra State Nigeria. Upon purchase, the cement was conveyed to laboratory testing unit of Infrastructure Development Company where it was kept in a cool dry place preparatory for various laboratory testing. The cement sample satisfy the requirement for use as one of the major component of concrete in that, it was not caked or baked through visual inspection. The cement was used as fillers to partially replace the coarse aggregate in the asphalt mix.

Plate 3.1: Photograph of Portland cement used for the Study

3.1.4 Granite Dust

The quarry dust sample designated as GD was provided by the laboratory testing department of Infrastructure Development Company. The granite dust sample was relatively small in size (less than 4.75mm). The quarry dust was collected in one and half cement bag and was kept in a safe location preparatory to laboratory testing.

Plate 3.2: Photograph of Granite Dust used for the Study

3.1.5 Aggregate (fine and coarse aggregate)

Sand sample used in asphalt production was provided by the laboratory testing department of Infrastructure Development Company. The sand was sieved through 5.0mm test sieve to remove larger particles and then air-dried to a saturated state of an aggregate. The sample passed the necessary requirement for use as ingredient of concrete based on the fact that it is gritty with particle sizes visible to the naked eyes, physical properties of the sand samples were determined prior to its incorporation to the asphalt mix.

Similarly, Granite samples designated as GT was provided by the laboratory testing department of Infrastructure Development Company. The index properties of the aggregate were determined. The granite sample passed all the necessary physical test in that, it has high crushing strength; it is relatively large in size (within range of 4.75mm to 20mm). Granite sample used for the

experimental study will be partially admixed with fillers such as granite dust, hydrated lime and Portland cement.

Plate 3.3: Photographs of Granite used for the Study

3.2 Mix Design of Asphalt Fillers

Table 3.0: Mix Design Ratio of Aggregate used for the Asphalt Mix

Sieves sizes (mm)	**10/15mm**		**5/10mm**		**10/5mm**		**J.M.F**	**TOLLE**
		15%		**12%**		**73%**		
37.5								**100**
25.0								**100**
19.5	**100**	**15%**	**100**	**12.0%**	**100**	**73%**	**100.0**	**100**
12.5	**78.8**	**11.8%**	**100**	**12.0%**	**100**	**73%**	**96.8**	**85-100**
9.5	**8.9**	**1.3%**	**98.6**	**11.8%**	**100**	**73%**	**86.1**	**75-92**
6.4	**0.5**		**51.5**	**6.18%**	**98.9**	**72.2%**	**78.4**	**65-82**
2.36	**0.3**		**8.5**	**1.02%**	**79.2**	**57.8%**	**58.8**	**50-65**
1.18			**4.2**	**0.5%**	**51.9**	**37.9%**	**38.4**	**36-51**
0.600			**3.4**	**0.4%**	**40.5**	**29.6%**	**30.0**	**26-40**
0.300			**2.6**		**29.1**	**21.2%**	**23.8**	**18-30**
0.150			**2.0**		**20.0**	**14.6%**	**16.6**	**13-24**
0.075					**15.6**	**11.4%**	**11.4**	**7-14**
Optimum Bitumen Content								**5-8**

The percentages of fillers presented in Table 3.0 will be added to the blend of both coarse and fine aggregate. Bitumen content ranging from 5%, 5.5%, 6%, 6.5% and 7% by total weight of the

aggregate will be added to the asphalt mix containing 100% granite dust (1200g) which serves as the control. The optimum binder content of the control mix (100% granite dust) will be determined and used as basis for production of subsequent mix.

3.3 Laboratory Investigation

This phase will present the detailed procedures for laboratory testing of the materials used for the experimental study. Some of the tests are: sieve analysis, specific gravity, aggregate impact value and aggregate crushing value of both coarse and fine aggregate, penetration, viscosity and flash and fire point of bitumen and Marshall stability test of the asphalt mix. During the determination of stability and volumetric properties of the asphalt, the optimum bitumen content obtained from the conventional asphalt mix (containing 100% granite dust) will be used for production asphalt mix containing lime and cement as mineral fillers. Below is description of some of the laboratory test.

3.3.1 Bitumen Penetration Test

Penetration value is a measure of hardness or consistency of a bituminous material (TELM, 2013). It is the vertical distance traversed by the point of a standard needle in a bituminous material under specific condition of load, time and temperature (TELM, 2013). The distance is measured in one tenth of a millimeter and the test is employed for assessing the consistency of bitumen. This test is deemed unsuitable for testing of road tar due to the relatively high surface tension exhibited by this material and also the significant amount of carbon present.

Apparatus Employed

I. Container: A flat bottomed cylindrical metal dish, 55mm in diameter and 35mm in depth is required

II. Needle: A straight, highly polished, cylindrical hard steel rod as per standard dimension.

III. Water bath: A water bath maintained at $25^0 \pm 0.1^0$C containing not less than 10 litres of water, the sample being immersed to a depth not less than 100mm from the top and supported on a perforated shelf not less than 50mm from the bottom of the bath.

IV. Transfer dish or tray: Used to provide support to the container and must have capacity sufficient to immerse the container completely during the test.

V. Penetration apparatus: Provided to allow the needle penetrate without much friction and must be accurately calibrated to give result in one tenth of a millimeter.

VI. Thermometer: Ranging between 0-44^{0}C and readable up to 0.20C.

VII. Time Measuring Device: Must have accuracy of ±0.1 sec

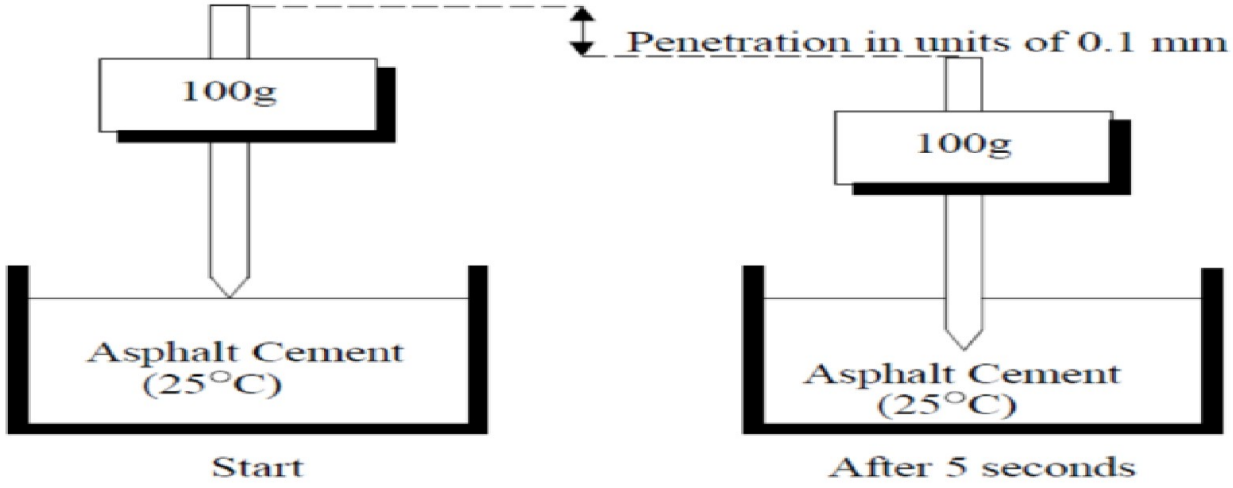


Figure 3.0: Apparatus set up of Bitumen Penetration Test (TELM, 2013).

Test Procedure

I. The sample was prepared by softening to a pouring consistency at a temperature not more than 90^{0}C.

II. The sample was thoroughly stirred until it became homogenous and free from air bubbles and water.

III. The melted sample was poured into a container, protected from dust and allowed to cool in an atmosphere between 15 to 30^0C for an hour and placed along with the transfer dish in the water bath at 25 ± 0.1^0C.

IV. The transfer dish was filled with water from the water bath to depth sufficient to cover the container completely; the bituminous sample was submerged in it, and then placed upon the stand of a penetration apparatus.

V. The needle was cleaned with benzene, dried and loaded with the required weight.

VI. The needle was adjusted to make contact with the surface of the sample; this was done by placing the needle point in contact with it image reflected by the surface of the bituminous material.

VII. The pointer of the dial gauge was adjusted to read zero and the needle was released for exactly five seconds.

VIII. The penetration machine was adjusted to measure the distance penetrated.

IX. At least three readings at points on the surface of the samples not less than 10mm apart and 10mm from the side of the dish were made and the average value was taken as bitumen penetration value.

Figure 3.1: Laboratory Determination of Bitumen Penetration Value (TELM, 2013).

3.3.2 Bitumen Flash and Fire Point Test

The flash point of a bituminous material is the temperature at which the application of test flame causes the vapour from the material to momentarily catch fire in the form of a flash under specified condition of test (TELM. 2013). It is also regarded as the lowest temperature at which the application of test flame cause the material to ignite and burn for at least 5 seconds under specified condition of test (TELM. 2013).

Apparatus Used for Flash and Fire Point Test

I. Thermometer

II. A stove or heating device with provision to adjust the rate of heating.

III. Open cup tester with the modification that the cover of the cup is replaced by a clip which encircles the upper rim of the cup and carries a test flame.

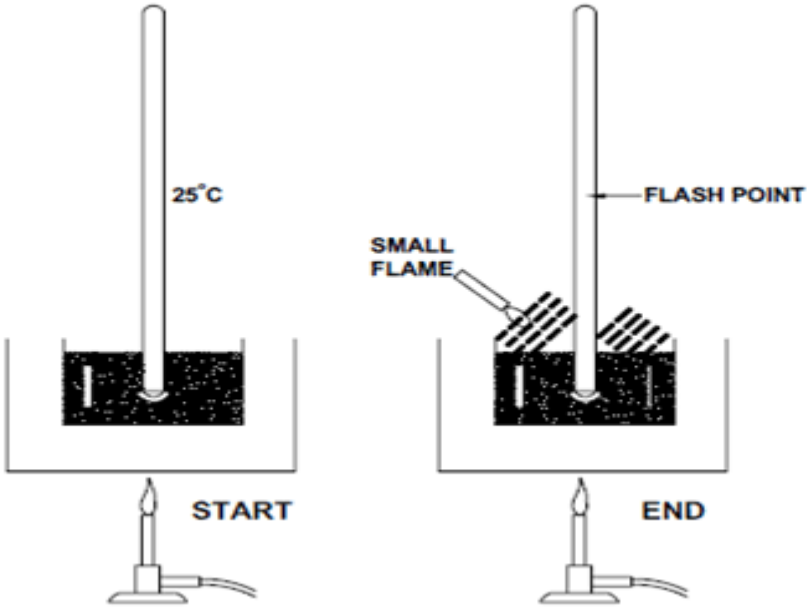


Figure 3.2: Apparatus Used for Bitumen Flash Point Test (Osunkunle, et al. 2016)

Figure 3.3: Apparatus Used for Bitumen Flash Point Test (Osunkunle, et al. 2016)

Test Procedure

I. The entire part of the open cup tester and the accessories are cleaned and dried.

II. The cup is filled with the bituminous binder up to the level of the filling mark.

III. The clip supporting the thermometer and test flame is placed in position on the cup.

IV. The thermometer is inserted and the open cup tester is placed on the stove.

V. The test flame is lighted and adjusted to size 4mm bead and fixed in the vertical axis of the cup, level with the upper edge of the cup.

VI. The bitumen sample in the tester is heated and the rate of heating is adjusted such that the temperature of the test specimen increases at the rate of 5^0C to 6^0C per minute.

VII. A burning match stick is placed at the binder surface from time to time and appearance of the flash, if any, is observed.

VIII. When the flash occurs the first time, the temperature at that instant is recorded as the flash point.

3.3.3 Bitumen Viscosity Test

Viscosity test of bitumen samples is one of the most important tests on bitumen to be conducted before road construction (TELM, 2013). Viscosity measures the degree of fluidity of the sample. It ensures the quality of bitumen used as a binder by giving a measure of fluidity at a particular temperature (TELM, 2013). If the bituminous binder used in the asphalt production has lower bituminous value, it will act as a lubricant only and may not be influential in binding of the asphalt constituents. A relatively viscous binder will restricts flow, thereby restricting the ability of the binder to spread and fill voids between the aggregate during pavement construction (TELM, 2013). Also, it will require more effort and there is a tendency of such binder forming a heterogeneous mix. Workability of the asphalt mix will be significantly affected (TELM, 2013). Therefore, it is essential to select a binder with appropriate viscosity so that it can form a uniform coat and fill up the voids between aggregates effectively.

Apparatus used for Viscosity Test are:

I. Viscometer Capillary Tube

II. Thermometer or Temperature Indicator

III. Vacuum System

IV. Thermostatically controlled oven

V. Stirrer

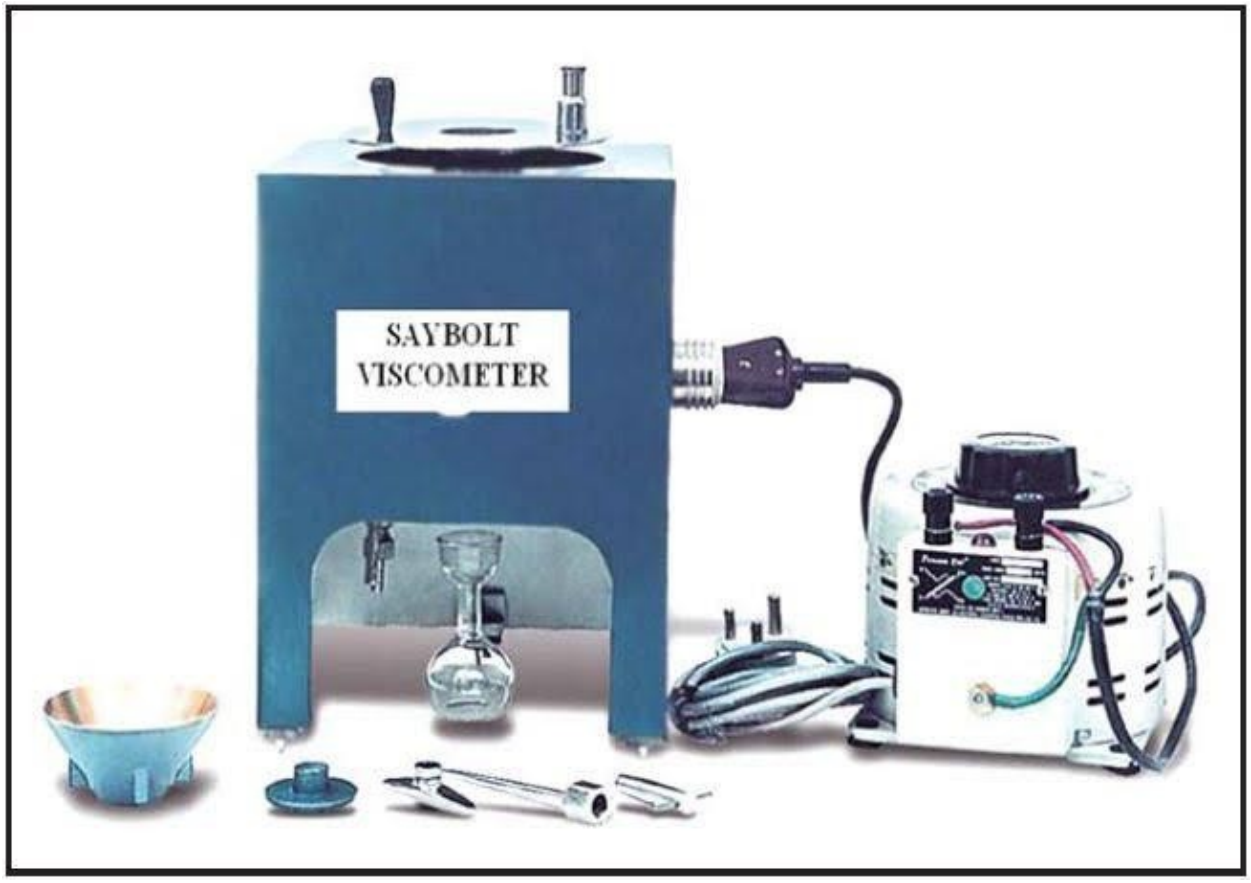


Figure 3.4: Apparatus for Viscosity Determination of Bitumen (TELM, 2013)

Figure 3.5: Apparatus for Viscosity Determination of Bitumen (TELM, 2013)

Test Procedure

I. The bitumen sample was placed in a beaker and heated to a specified temperature with the bitumen allowed to melt slightly above the softening point until pouring consistency was achieved.

II. About 30ml of melted bitumen was poured in a container and maintained to a specified temperature.

III. The bitumen was stirred to remove entrapped air and to also avoid local heating.

IV. The viscometer was charged by connecting to an electric source and thereafter placed in into bath or oven with the temperature of the bath maintained.

V. The vacuum system was connected to the viscometer and after establishing a vacuum of 30 ± 0.05cm, of mercury, the valve of the vacuum system was closed.

VI. The valve was opened and the bitumen was allowed to flow into the viscometer.

VII. The time required for the leading edge of the meniscus to pass between the two consecutive pairs of timing was measured and recorded in seconds.

VIII. The absolute viscosity of the sample was determined from the value of time recorded in the preceding step.

3.3.4 Aggregate Impact Value Test (AIV)

The property of a material to resist impact is referred to as toughness (TELM, 2013). Due to movement of vehicles to the road, the aggregate are subjected to impact resulting in their breaking down into small pieces (TELM, 2013). The aggregate should therefore have sufficient toughness to resist their disintegration due to impact. This characteristic is measured by impact value test. The aggregate impact value test is a measure of its resistance to sudden impact or shock which may differ from it resistance to gradually applied compressive load.

Test Apparatus

I. A cylindrical steel cup of internal diameter 102mm, depth 50mm and minimum thickness 6.3mm.

II. A metal hammer weighing 13.5 to 14kg with the lower end cylindrical in shape and 50mm long, 100mm in diameter with a 2mm chamfer at the lower edge and case hardened. The hammer should slide freely between vertical guides and be concentric with the cup.

III. A cylindrical metal mould having an internal diameter of 75mm and depth 50mm for measuring aggregates.

IV. Tamping rod 10mm in diameter and 230mm long and rounded at one end.

V. A weighing balance of capacity not less than 500g, readable and accurate up to 0.1g.

VI. A testing weighing machine 45 to 60kg, having a metal base with a plane lower surface not less than 30cm in diameter. It is supported on level and plane concrete floor of minimum 45mm thickness. The machine should have provision for fixing it base.

Plate 3.4: Apparatus Used for Aggregate Impact Value Test

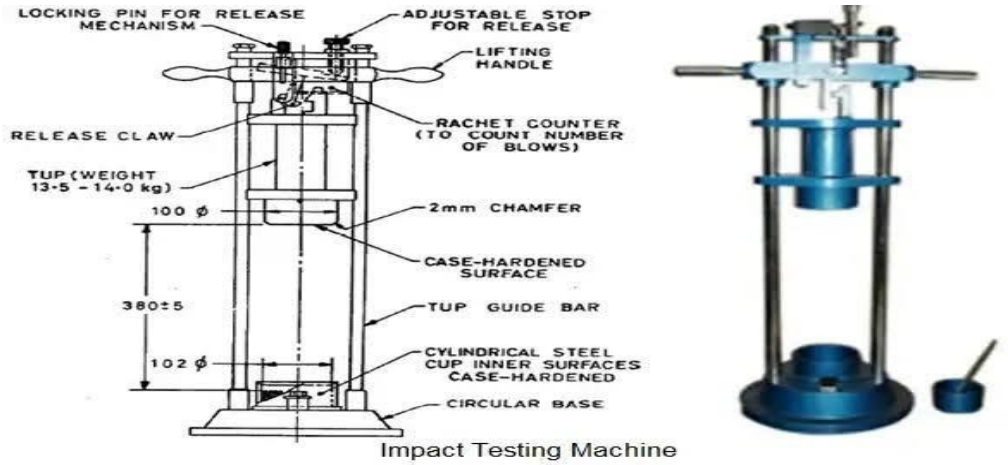


Figure 3.6: Apparatus Used for Aggregate Impact Value Test (Osunkunle, et al. 2016)

Test Procedure

I. The aggregate used for the experimental study was dried by heating at 100^0C-110^0C for a period of 4 hours and then allowed to cool.

II. The material was sieved through 12.5mm and 10mm sieve. The aggregate passing through 12.5mm and retained on 10mm sieve comprises the test material.

III. The aggregate passing through the 10mm sieve referred to as the test material was poured in a cylindrical metal mould about one- third the volume of the mould.

IV. The aggregate was compacted evenly by giving 25 no of blows with the aid of the tamping rod.

V. The remaining layers of the test aggregate were added into the mould until full and compacted evenly with 25 no of blows.

VI. The surplus aggregate was strike off using a spatula or straight edge.

VII. The net weight of the aggregate was determined to the nearest gram.

VIII. The impact machine was brought to rest without wedging so that it is rigid and the column guides are vertical.

IX. The cup was fixed firmly to the base of the machine; the test sample was placed and compacted evenly by giving 25 gentle strokes with the aid of a tamping rod.

X. The hammer was raised until its lower face is 380mm above the surface of the aggregate sample in the cup and allowed to fall freely on the aggregate sample. The aggregate sample was given 15 blows at an interval not less than one second between successive blows.

XI. The crushed aggregate was removed from the cup and sieved through 2.36mm sieve until no further significant amount passes in one minute. The fraction passing through the sieve was weighed to an accuracy of 1gm and the fraction retained on the sieve was also weighed.

XII. The test observation was noted down and the aggregate impact value was computed.

3.3.5 Aggregate Crushing Value Test (ACV)

The aggregate crushing value gives a relative measure of the resistance of an aggregate to crushing under gradually applied compressive load (TELM, 2013). Crushing value is a measure of the strength of the aggregate, aggregate with higher strength should have a minimum crushing value.

Apparatus Used

I. A weighing balance readable to an accuracy of 1gm

II. IS Sieve sizes of 2.36mm, 10mm and 12.5mm.

III. A compression testing machine capable of applying a load of 40 tonnes and which can be operated to give a uniform rate of loading so that the maximum load is reached in 10 minute.

IV. Cylindrical metal mould of sufficient rigidity to retain its form under rough usage having an internal diameter of 11.5cm and height of 18cm.

V. A tamping rod.

Plate 3.4: Apparatus Used for Aggregate Crushing Value Test.

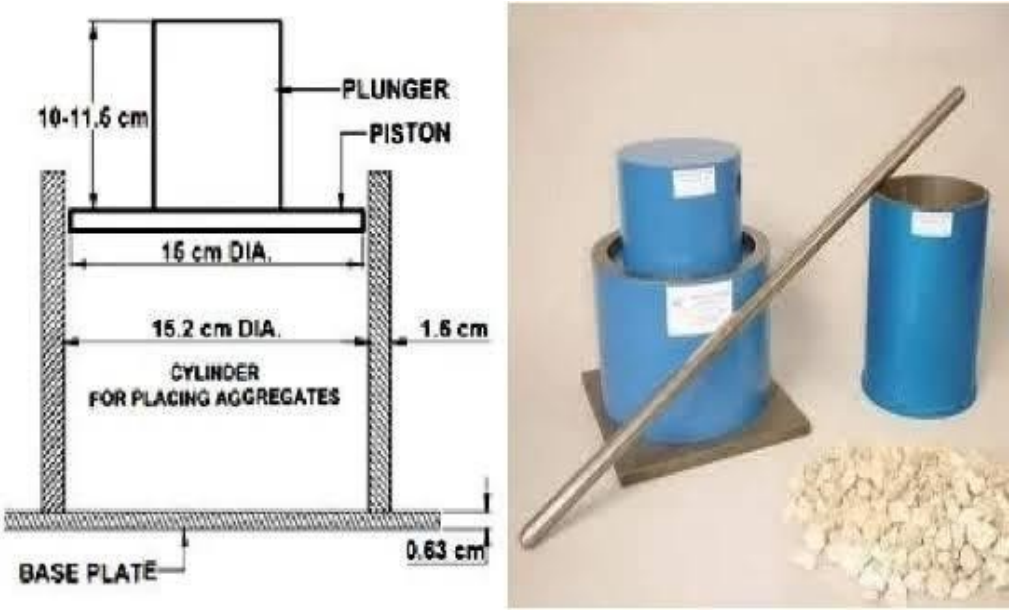


Figure 3.7: Apparatus Used for Aggregate Crushing Value Test (Osunkunle, et al. 2016)

Test Procedure

I. The aggregate used for the experimental study was dried by heating at 100^0C-110^0C for a period of 4 hours and then allowed to cool.

II. The material was sieved through 12.5mm and 10mm sieve. The aggregate passing through 12.5mm and retained on 10mm sieve comprises the test material.

III. The cylindrical mould was assembled into position and the test sample was added to about one-third the volume of the mould.

IV. The test aggregate was subjected to compaction by giving 25 no of blows with the aid of tamping rod.

V. The other layers was added and compacted evenly with the surface of the aggregate leveled to flush with the top surface of the mould using a straight edge.

VI. The plunger was inserted so that it rest horizontally on the surface of the test aggregate, care was exercised to ensure that there was no collision between the plunger and the cylinder.

VII. The apparatus with the test sample with the plunger in position was placed between the plates of the testing machine.

VIII. The load was applied at a uniform rate as possible so that the total load is reached in 10 minutes.

IX. The load was released and the whole of the material is removed from the cylinder and sieved on 2.36mm IS sieve.

X. The fraction passing through the sieve was weighed and recorded.

XI. The test observation was noted down and the aggregate crushing value was computed.

3.3.6 Sieve Analysis Test

Sieve analysis is a procedure used to assess the particle size distribution of a granular material Atkinson (2000). The size distribution is often of critical importance to the behavior of the material during use. Sieve analysis can performed on any type of non-organic or organic granular material including sand, crushed rock, clay, granite, feldspar and a wide range of manufactured

powders, grains and seed down to minimum size depending on the exact method. The standard grain size analysis test determines the relative proportion of different grain sizes as they are distributed among certain size ranges.

The apparatus needed for this experiment is listed below:

1. Stack of sieves including pan and cover.
2. Mechanical sieve shaker.
3. Weighing balance of 0.01g sensitivity.
4. Hand brush
5. Mortar and pestle (Used for crushing if the sample is conglomerated or lumped)
6. Thermostatically controlled Oven (With temperature of about 80^{O}C-110^{O}C).
7. Masking tape for identification of sample.
8. Exercise book and pen for recording of result.
9. The calculation for attaining Coefficient of uniformity and Coefficient of curvature are outlined below.

$$\text{Percentage retained (\%)} = \frac{mass of soil retained in the sieve (g)}{total mass of soil sample (g)} \times 100$$

$$\text{Cumulative percentage retained} = \sum \text{Percentage retained (\%)}$$

Cumulative Percentage Finer (%) = 100-Cummulative percentage retained.

$$\text{Coefficient of Curvature} = \frac{D60}{D10}$$

$$\text{Coefficient of Uniformity} = \frac{(D30)2}{D10 \times D60}$$

Where

D10= particle size such that 10% of the soil is finer than the size

D30= particle size such that 30% of the soil is finer than the size.

D60= particle size such that 60% of the soil is finer than the size**.**

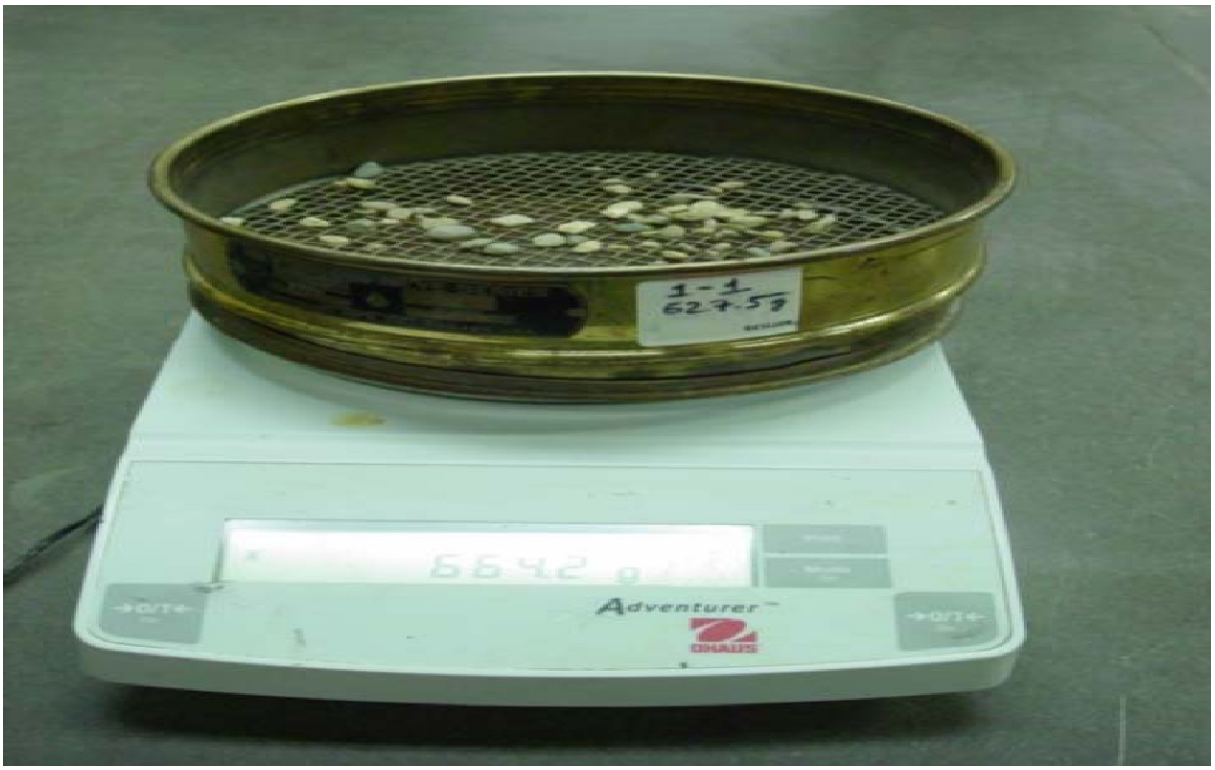


Figure 3.8: Apparatus for Particle Size Distribution Test (Braja, 2006)

Figure 3.9: Apparatus for Particle Size Distribution Test (Braja, 2006).

Test Procedure

I. The stack of sieves to be used for the experiment was properly cleaned using hand brush.

II. About 500g of air-dried soil sample was weighed with the aid of a weighing balance.

III. The weighed soil sample was poured into 75um sieve and wash under a steady supply of water until clear water start coming out from the sieve after passing through the soil sample.

IV. After washing pour the washed soil sample into a pre-weighed plate and dry it inside the thermostatically controlled oven at a controlled temperature of 80-110^{O}C for 16-24hrs.

V. The sample was removed from the oven and the weight was determine (net weight) by deducting the weight of plate from the weight of plate and soil.

VI. The stacks of sieve was arranged in the ascending order, placed in a mechanical sieve shaker, and thereafter the sample was poured and connected to the shaker for about 10-15 minute.

VII. The sieve shaker was disconnected and the mass retained on each of the sieve sizes was determined.

VIII. The percentage retained, Cumulative percentage retained and Cumulative percentage finer was determined.

IX. The graph of sieve Cumulative percentage finer against sieve sizes was plotted.

X. D10, D30 and D60 were determined from the plotted graph.

XI. The Coefficient of Curvature and Coefficient of Uniformity was determined and used to classify the soil adopting the American Association of State Highway and Transportation Official (AASHTO) and Unified Soil Classification System (USCS) respectively.

3.3.7 Specific Gravity Test of Fine Aggregate

Specific gravity is the ratio of mass of unit volume of soil at a stated temperature to mass of equal volume of gas-free distilled water at the same temperature (Krishna, 2002). Also as defined

by Braja, (2006), Specific gravity can be defined as the ratio of unit weight of a material to unit weight of water. The specific gravity of soil solids is often needed for various calculations in soil mechanics. It can be determined accurately in the soil laboratory.

The apparatus employed for this experiment includes:

I. Density bottle of 50ml capacity and a stopper.

II. Desiccator containing anhydrous silica gel.

III. Thermostatically controlled oven with temperature of about 80-110^{O}C.

IV. Weighing balance of 0.01g sensitivity.

V. Mantle heater.

VI. Plastic wash bottle.

VII. Distilled water.

VIII. Funnel

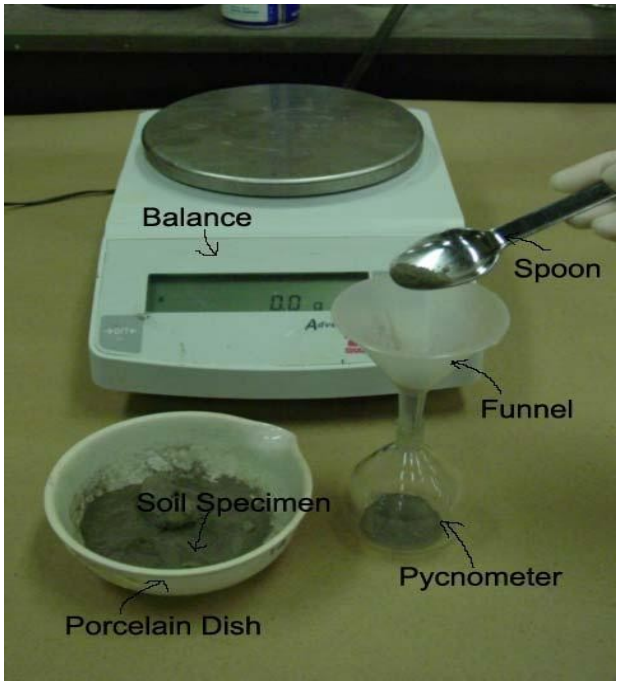
IX. Thin glass rod for stirring.

X. 425um Sieve.

XI. Dry piece of cloth for cleaning.

XII. Masking tape for identification of sample.

XIII. Exercise book and pen for recording of result.



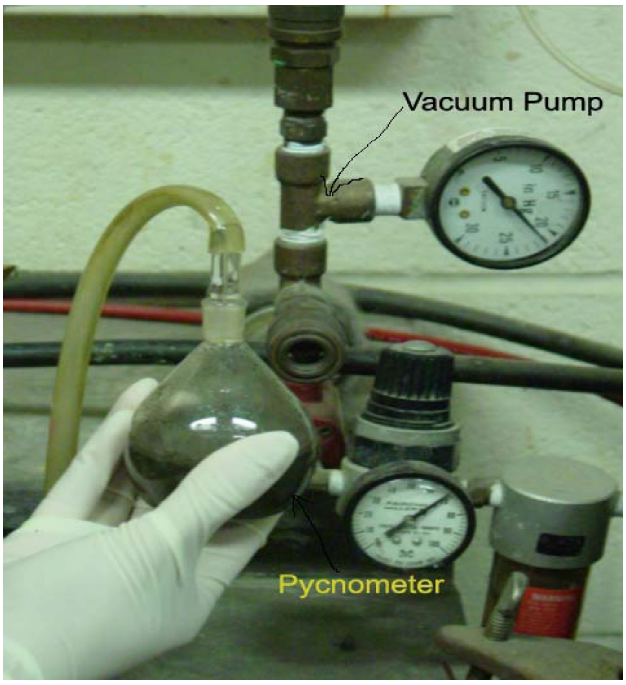


Figure 3.10: Apparatus used for Specific Gravity Test (Braja, 2006)

Test Procedure

I. The density bottle properly cleaned and rinsed with distilled water, thereafter oven- dried and then cooled it in a desiccator so as to remove any moisture present.

II. The empty clean and dry density bottle was weighed and recorded as (M_1).

III. About 10-15g of soil passing through 425um sieve was placed inside the density bottle, weigh and the weight of density bottle +dry soil + stopper was recorded as (M_2).

IV. Distilled water was added to fill about half to three-fourth of the density bottle, and then the sample was soaked for 24hrs (The time stated is to enable complete settlement of the soil particle which is evident when clear water appears above the submerged soil).

V. The density bottle was gently stirred using thin glass rod and thereafter connected to a mantle heater to de-air the sample, the sample was not allowed to boil over.

VI. After agitation, the sample was allowed to cool at room temperature and then filled with distilled water up to the specified mark (at lower meniscus level), the exterior surface of the density bottle was cleaned with a clean dry cloth and the weight of the density bottle + stopper +soil filled with water was determined and recorded as (M_3).

VII. The density bottle was emptied, cleaned and rinsed with distilled water, then filled with distilled water up to the same mark. The exterior surface of the density bottle was cleaned with a clean dry cloth and the weight of the density bottle filled with distilled water + stopper was determined and recorded as (M_4).

VIII. The test procedure was repeated for two more trials and the average specific gravity value was obtained from the total no of trial, the variation in the specific gravity result obtained for each trial must not exceed 2%, otherwise repeat the experiment.

The Procedure for Computation of result as follows:

Specific gravity (G_S) = $\frac{(M2-M1)}{(M2-M1)-(M3-M4)}$are

Where M_1= weight of density bottle + stopper

M_2= Weight of density bottle + air-dried soil + stopper.

3.3.8 Specific Gravity Test of Coarse Aggregate

The specific gravity of coarse aggregate is defined as the ratio of coarse aggregate to the weight of equal volume of water (Braja, 2006). The specific gravity of coarse aggregate is considered to be a measure of strength or quality of the material. Aggregates having low specific gravity are generally weaker than those with high specific gravity (Braja, 2006). This property helps in general identification of aggregate.

Apparatus Employed

I. Wires mesh Bucket or perforated container of convenient sizes with thin wire hangers for suspending it from a balance.

II. Pycnometer of 1000ml.

III. Set up consisting of container for filling water and suspending the wire basket in it and airtight container of capacity similar to that of a bucket, a shallow tray, two dry absorbent clothes.

Test Procedure

I. About 2 kg of aggregate sample is taken, washed to remove fines and then placed in the wire basket. The wire basket is then immersed in water, which is at a temperature of 22^0C to 32^0C.

II. Immediately after immersion the entrapped air is removed from the sample by lifting the basket 2mm above the base of the tank and allowing it to drop, 25 times at a rate of about one drop per second.

III. The basket, with aggregate are kept completely immersed in water for a period of 24 ± 0.5 hour.

IV. The basket and aggregate are weighed while suspended in water, which is at a temperature of 22^0C to 32^0C.

V. The basket and aggregates are removed from water and dried with dry absorbent cloth.

VI. The surface dried aggregates are also weighed.

VII. The aggregate is placed in a shallow tray and heated to about 110^0C in the oven for 24 hours. Later, it is cooled in an airtight container and weighed.

3.3.9 Marshall Stability Test of Asphalt Mix

The Marshall Stability test method is widely used for the design and control of asphalt concrete and hot rolled asphalt materials (TELM, 2013). It cannot be applied to open texture materials such as bitumen macadam. Marshall mix design method help engineers to select the optimum asphalt binder content at a density that meets stability and flow value requirements Olumide, (2013). The asphalt stability value (kg) is a measure of strength of asphalt mix and indicates the maximum load that the compacted specimen can carry at standard test temperature of 60^0C while the flow value (mm) is a measure of flexibility of asphalt mix and indicate the change in diameter of the sample in the direction of applied load between the start of loading and the time of maximum load (Olumide, 2013). The higher flow value, the higher the flexibility and the lesser the stiffness of asphalt.

Test Procedure

I. Cylindrical metal mold fitted with a base plate and extension collar

II. Filter paper placed at the bottom of the mold.

III. A tamping rod

IV. Marshall stability testing machine

V. Compaction pedestal

VI. Compaction hammer which satisfies BS 598, that it should have 4535g sliding weight with a free fall of 457mm.

VII. Sample extractor with suitable jack and frame arrangement.

VIII. Water bath

IX. Trowel

X. Spatula or straight edge for trimming of mix.

Figure 3.11: Apparatus set up for Marshall Stability Test (Source: TELM, 2013).

Test Procedure

I. About 1200g of dry blended aggregate and fillers was measured and heated at 170^0C. The heated aggregate was placed in a pan and mixed thoroughly.

II. A crater was formed in the aggregate and bitumen partially replaced with melted plastic waste bottles were heated to 170^0C was added.

III. The aggregate and bitumen were mixed thoroughly until the aggregate were well coated.

IV. The mould and compaction hammer were cleaned, assembled and heated to 170^0C.

V. A filter paper was placed at the bottom of the mould and the asphalt mix was placed in the mold and spade with a heated spatula around the perimeter.

VI. The extension collar was removed and the surface of the mix was smoothened with a trowel to a slightly rounded shape with the temperature of the mixture maintained at 170^0C immediately prior to compaction.

VII. The extension collar was replaced, and the mould assembly was placed on the compaction pedestal in the molder holder with the top of the specimen given 75 blows, enough care was exercised to ensure that the blows were evenly distributed.

VIII. The base plate and extension collar were removed and the sample inverted and the mould reassembled.

IX. The inverted face of the mould was also given 75 blows, after compaction, the base plate was removed and the mould containing the specimen was immersed in cool water for 2 minutes.

X. The specimen was removed from the mould by means of a sample extractor and suitable jack and frame arrangement, thereafter, the specimen as placed on a smooth, flat surface and allowed to cool at room temperature.

XI. The density of the specimen was determined by weighing the specimen in air and clean water at room temperature and the difference between the two weights in gram was used to determine the volume.

Stability and Flow Determination

I. On completion, the collar was removed and the mould containing the specimen was carefully transferred unto a smooth flat surface where the specimen (compacted asphalt) was extracted from the mould and allowed to stay overnight at room temperature.

II. The specimen was brought to room temperature by immersing in water bath for 20 to 40 minutes.

III. The guides rods and inside surface of the test head were thoroughly cleaned with the guide rods lubricated so that the upper test head slides freely over them.

IV. The specimen in the water bath was placed in the lower segment of the breaking head with the upper segment of the breaking head placed on the specimen.

V. The complete mould assembly was placed in position on the testing machine.

VI. The flow meter was placed in position over the guide rods and the sleeves were held firmly against the upper segment of the breaking head while the load was applied.

VII. The flow meter was adjusted to zero prior to the start of the test.

VIII. The load was applied to the specimen at a rate of 50mm per minute until the maximum load was reached and the load began to decrease.

IX. The maximum load was recorded and the flow meter was removed from its position over the guide rod at instant the load began to decrease.

X. The flow value was read and recorded with the time elapsed for the test for removal of sample from water bath to maximum load determination not exceeding 30 seconds.

XI. The stability value was thereafter determined through computation.

Computation of Test Parameters

Bulk Density of the mix (g/cm^3), determined during experimentation

$$\text{Specific Gravity of Mix} = \frac{100}{\%\frac{Bitumen}{SG\ of\ Bitumen} + \%\frac{Aggregate}{SG\ of\ Aggregate}}$$

$$\text{Volume of Bitumen (VB)} = Bitumen\ Content x\ Bulk\ \frac{Density}{Specific\ Gravity\ of\ Aggregate}$$

$$\text{Volume of Aggregate (VA)} = 100 - Bitumen\ Content\ x\ Bulk\ \frac{Density}{Specific\ Gravity\ of\ Aggregate}$$

$$\text{Voids in Mineral Aggregate (VMA)} = 100 - Volume\ of\ Aggregate\ (VA)$$

$$\text{Void in Total Mix (VTM)} = Specific\ Gravity\ (Gmm) - Bulk\ Density\ \frac{(Gmb)\ x\ 100}{Specific\ Gravity}$$

$$\text{Void Filled by Bitumen (VFB)} = VMA - \frac{VTM\ x\ 100}{VMA}$$

Where:

VTM = Volume of total air void in mix

VFB = Volume filled by bitumen

VMA = Volume in mineral aggregate

SG = Specific gravity of mix

VB = Volume of bitumen

Plate 3.5: Weighing of Coarse Aggregate used for Production of Asphalt Concrete

Plate 3.6: Weighing of cement sample used for Asphalt Production

Plate 3.7: Manual Mixing of Asphalt Aggregate Samples

Plate 3.8: Introduction of Binder Content and Weighing of Asphalt Mix

Plate 3.9: Sample of Asphalt Concrete Prepared with Different Mineral Fillers

CHAPTER FOUR

RESULTS AND DISCUSSION

During the of experimentation phase of the study, certain results were obtained which was useful in evaluating the effect of mineral fillers on the stability and volumetric properties of asphalt, these results are presented in Tables 4.0-4.5 below:

4.1 Evaluation of Bitumen Properties

Table 4.0: Properties of Bitumen used for the Research

Properties	**Specific Gravity**	**Flash point (^{0}C)**	**Fire Point (^{0}C)**	**Penetration (mm)**	**Viscosity (Ns/m^2)**
FMWH, (1997)	**>1.0**	**280^{0}C – 300^{0}C**	**300^{0}C-320^{0}C**	**60mm – 70mm**	**0.6 -1**
Bitumen Values	**1.03**	**287.7^{0}C**	**316.7^{0}C**	**69.33**	**0.73**

Table 4.0 depicts the properties of bitumen used for the study. The flash and fire point test was conducted to determine the temperature at which bitumen will give a flash and burns for a minimum of five (5) minute. The result obtained as shown in Table 4.3 revealed that the flash obtained for bitumen was 287.7^{0}C; this value is within the acceptable limit of the recommended range (280^{0}C – 300^{0}C) by Federal Ministry of Works and Housing, (1997). In the same vein, the average fire point obtained for bitumen was 316.7^{0}C. This value lies within the specified limit (300^{0}C – 320^{0}C) and it indicate that the bitumen can be used for the production of asphalt.

The specific gravity of bitumen was 1.03 which conforms to the standard set by Federal Ministry of Works and Housing, (1997) which state that the specific gravity of bitumen used for asphalt production must exceed 1.0. This result justifies the use of bitumen for the study.

Penetration test result of bitumen suggested that the penetration value of bitumen used for the experimental study was 69.33mm which conforms to 60/70 penetration grade of bitumen. Viscosity test of bitumen revealed that the viscosity of bitumen was 0.73 which lies within the range (0.6-1) recommended by Federal Ministry of Works and Housing, (1997) for viscosity of bitumen.

4.1.1 Aggregate Index Properties Assessment

Table 4.1: Physical Properties of Aggregate used for the Research

Properties	**SD**	**GT**	**GD**	**FMWH, (1997)**
Specific Gravity (Gs)	**2.55**	**2.61**	**2.76**	**2.5 – 2.9**
Water Absorption (%)	**-**	**0.92**	**-**	**0.2% - 1.5%**
Aggregate Impact Value (%)	**--**	**6.5**	**-**	**<30**
Aggregate Crushing Value (%)	**--**	**4.3**	**-**	**<30**
Percentage passing Sieve No 200 (0.075mm)	**8.03**	**7.11**	**19.73**	**-**
Coefficient of Uniformity (Cu)	**1.2**	**3.6**	**0**	**-**

Coefficient of Curvature (Cc)	**0.3**	**0.03**	**0**	**-**
Gradation	**SP**	**GP**	**-**	**-**
AASHTO Classification System	**A-2-4**	**A-1-b**	**A-2-4**	**-**
USCS Classification System	**SC**	**GC**	**SC**	**-**

Table 4.1 depicts the index properties of the aggregate samples used for the study. Assessment of the index properties of fine and coarse aggregate used for the asphalt production shows that the specific gravity of sand, granite and granite dust were 2.55, 2.61 and 2.76, this result suggest that the granite dust will yield relatively high bulk density when used for used for asphalt production. The percentage passing through sieve No 200 (0.075mm) for sand, granite and granite dust were 8.03, 7.11 and 19.73 and as a result, the sand, granite and granite dust were classified as A-2-4, A-1-b and A-2-4 according to AASHTO classification system SC (clay mixed with sand) GM (gravel mixed with clay) and SC according to Unified Soil Classification System. It was also observed that the fine content of granite dust was higher than that of sand and granite and as a result may produce asphalt with reduced flow and stability. Assessment of the gradation properties of the aggregate revealed that the aggregates were poorly graded.

Strength evaluation of the coarse aggregate used for the asphalt production showed that the impact and crushing strength of granite were 6.5% and 4.3% respectively. This value satisfied the requirement given by Federal Ministry of Works and Housing, (1997) which state that the impact and crushing strength of aggregate used for asphalt production must not exceed 30%. The conformity in index and strength properties of aggregate to relevant standard justified the use of aggregate for the study.

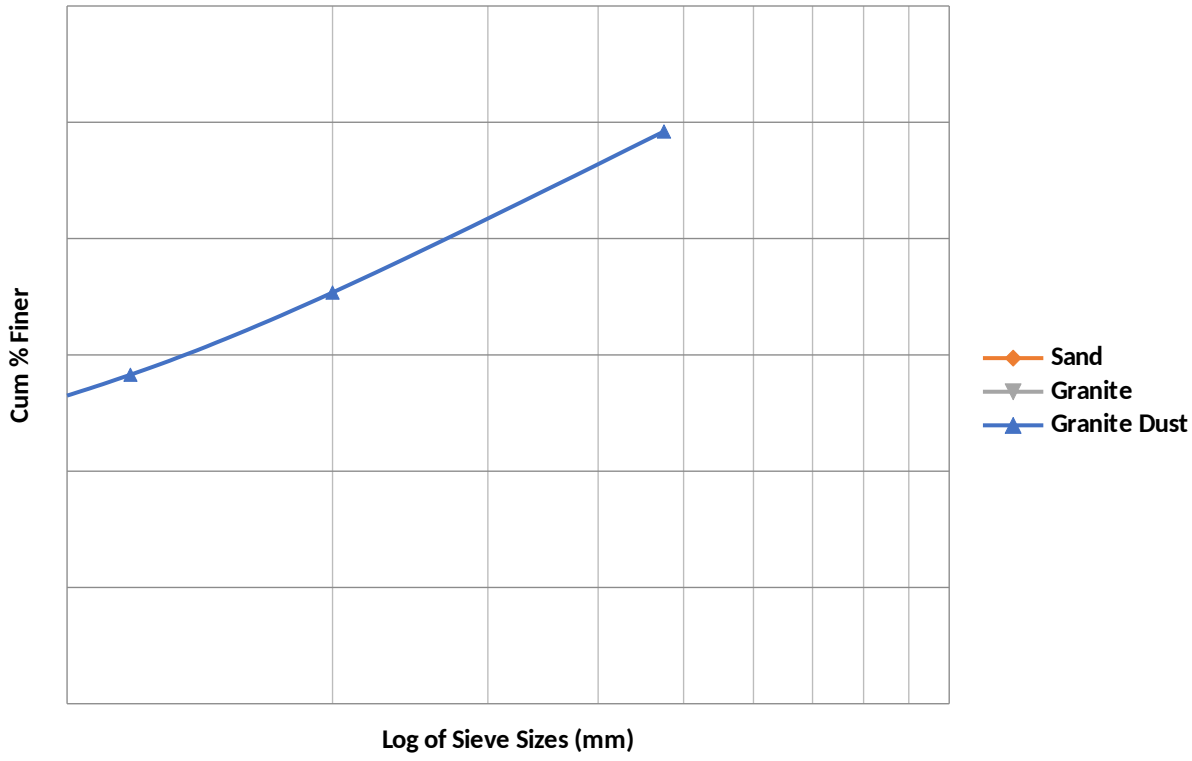


Figure 4.0: Particle Size Distribution Curve for Sand, Granite and Granite Dust

4.1.2 Optimum Binder Content Determination

Table 4.2: Data Summary for Determination of Optimum Binder Content

Binder Content/ Parameters	5	5.5	6	6.5	7
Bulk Density (gm/cm^3)	2.282	2.330	2.398	2.374	2.300
Stability (kN)	9.6	10.7	12.6	12.5	10.3
Void in Total Mix	10.8	8.2	4.8	5	7.30

(%)					
Void in Mineral Aggregate (%)	**21.90**	**20.60**	**18.70**	**19.90**	**22.90**
Specific Gravity	**2.557**	**2.538**	**2.518**	**2.499**	**2.480**
Void Filled by Bitumen	**50.1**	**60.3**	**74.9**	**75**	**68.3**

The optimum binder content corresponds to the bitumen content that gives the highest value of bulk density or stability of the asphalt mix (Ogundipe and Dada, 2016). Result obtained as shown in Figure 4.1-4.6 revealed that the optimum binder content of the asphalt mix prepared with 100% stone dust (control mix) was 6%. The optimum binder content obtained conforms to the specification given by the Federal Ministry of Works and Housing, (1997) which state that the optimum binder content of asphalt concrete must fall between 5 and 8% respectively. It was observed that the bulk density of the asphalt mix shares a direct relationship with the stability of the mix. The void in mineral aggregate, void in total mix and void filled by bitumen decreased with increasing percentages of bitumen up to the optimum binder content (6%). The flow of the asphalt mix increased with increasing percentages of bitumen. It was concluded that the properties of the control mix obeyed the specification given by the Federal Ministry of Works and Housing, (1997) apart from the void in the total mix which fell slightly above the standard.

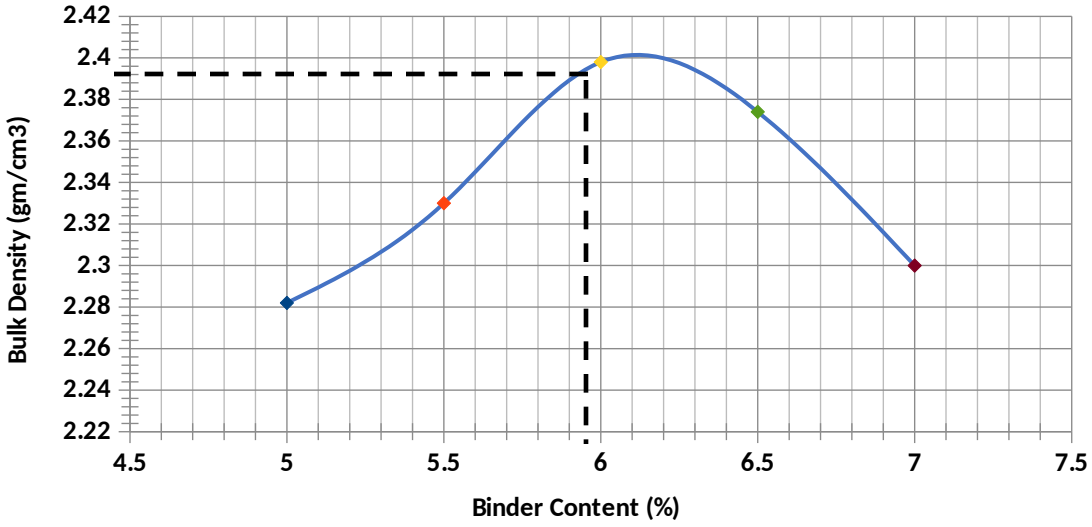


Figure 4.1: Graph of Bulk Density against Binder Content

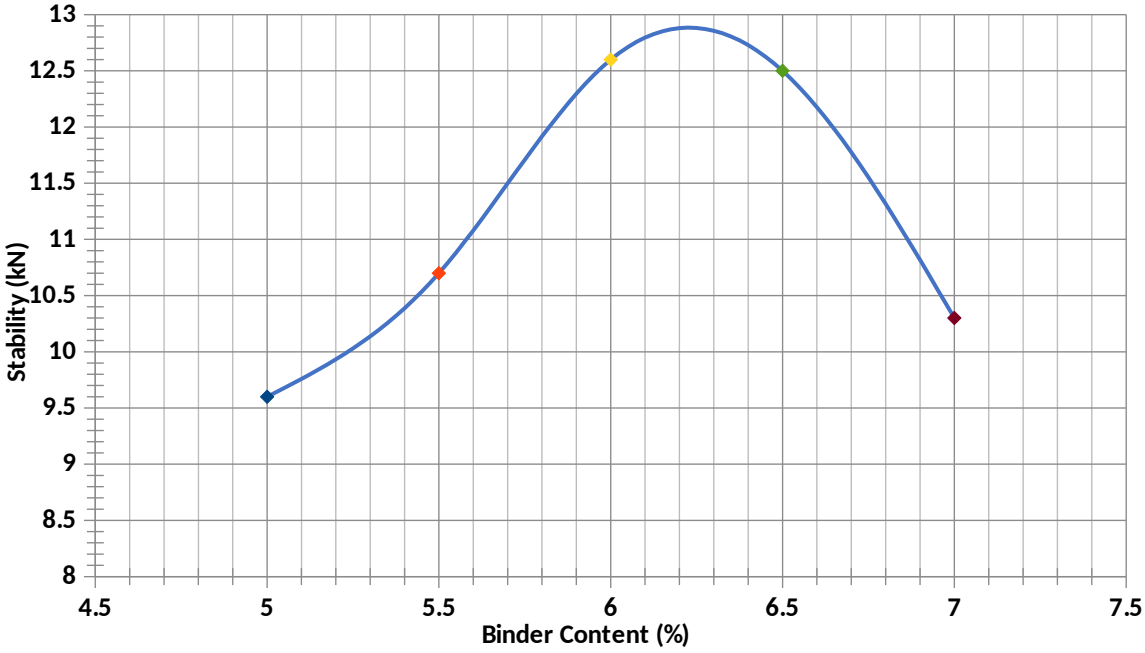


Figure 4.2: Graph of Stability against Binder Content

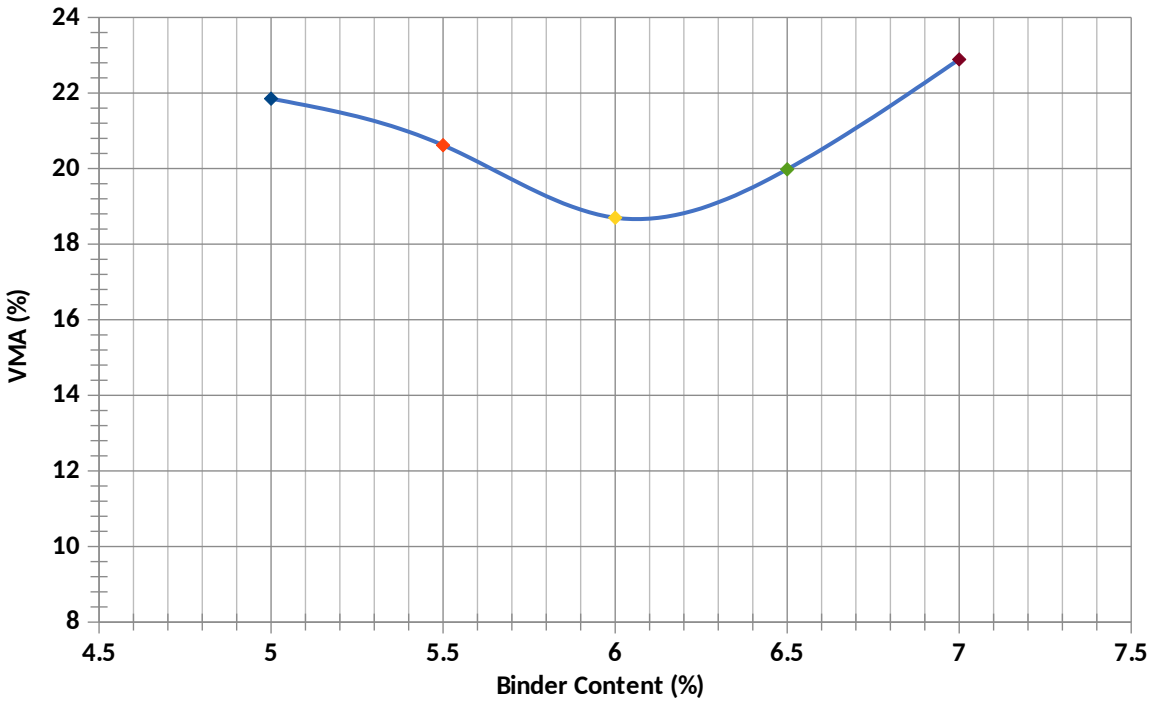


Figure 4.3: Graph of Void in Mineral Aggregate against Binder Content

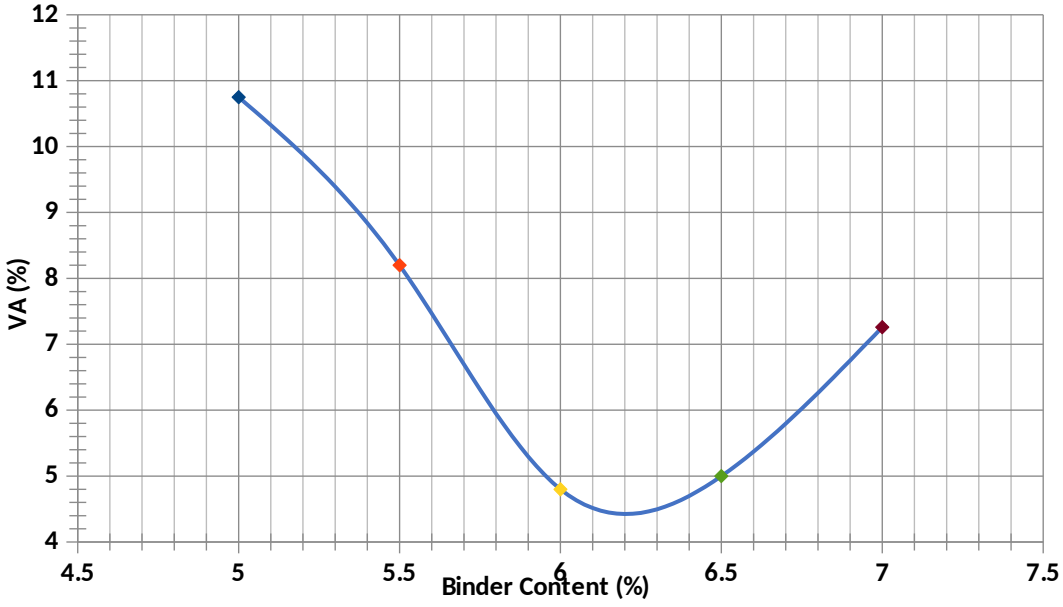


Figure 4.4: Graph of Void in Total Mix against Binder Content

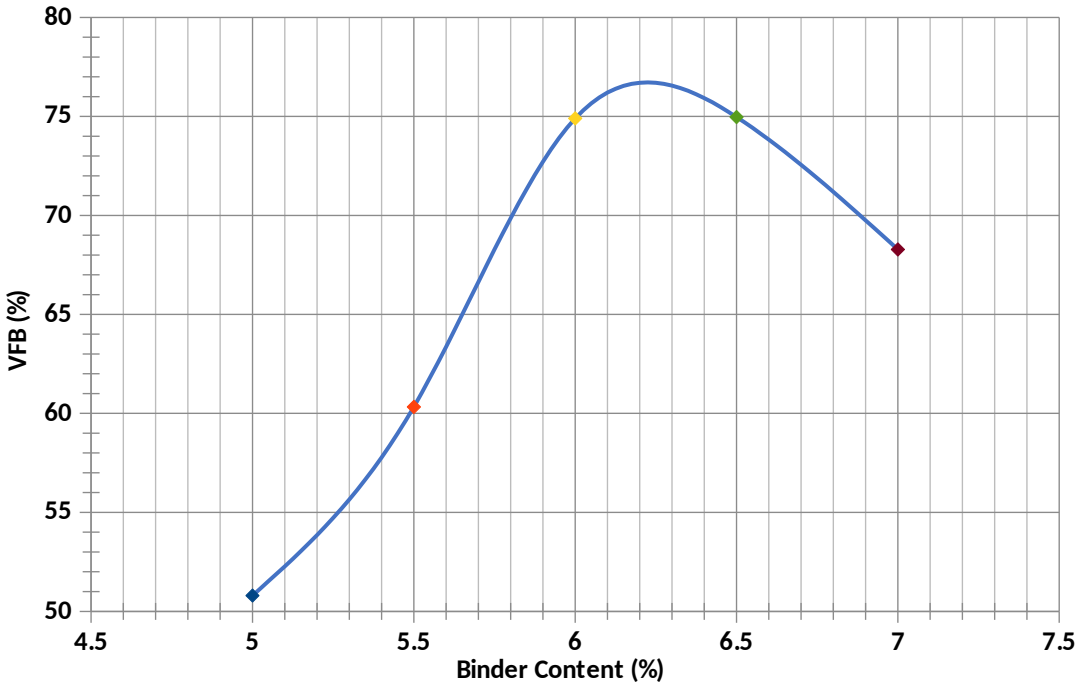


Figure 4.5: Graph of Void Filled by Bitumen against Binder Content

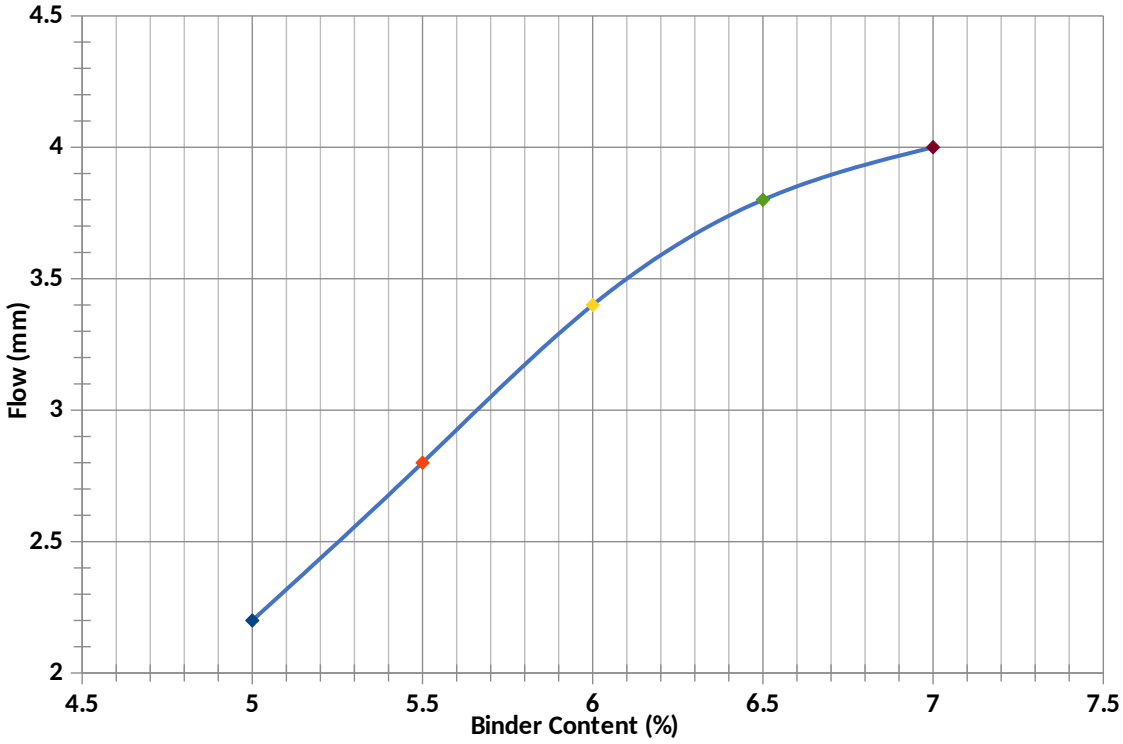


Figure 4.6: Graph of Flow against Binder Content

4.2 Evaluation of Effect of Mineral Fillers on Stability and Volumetric Properties of Asphalt

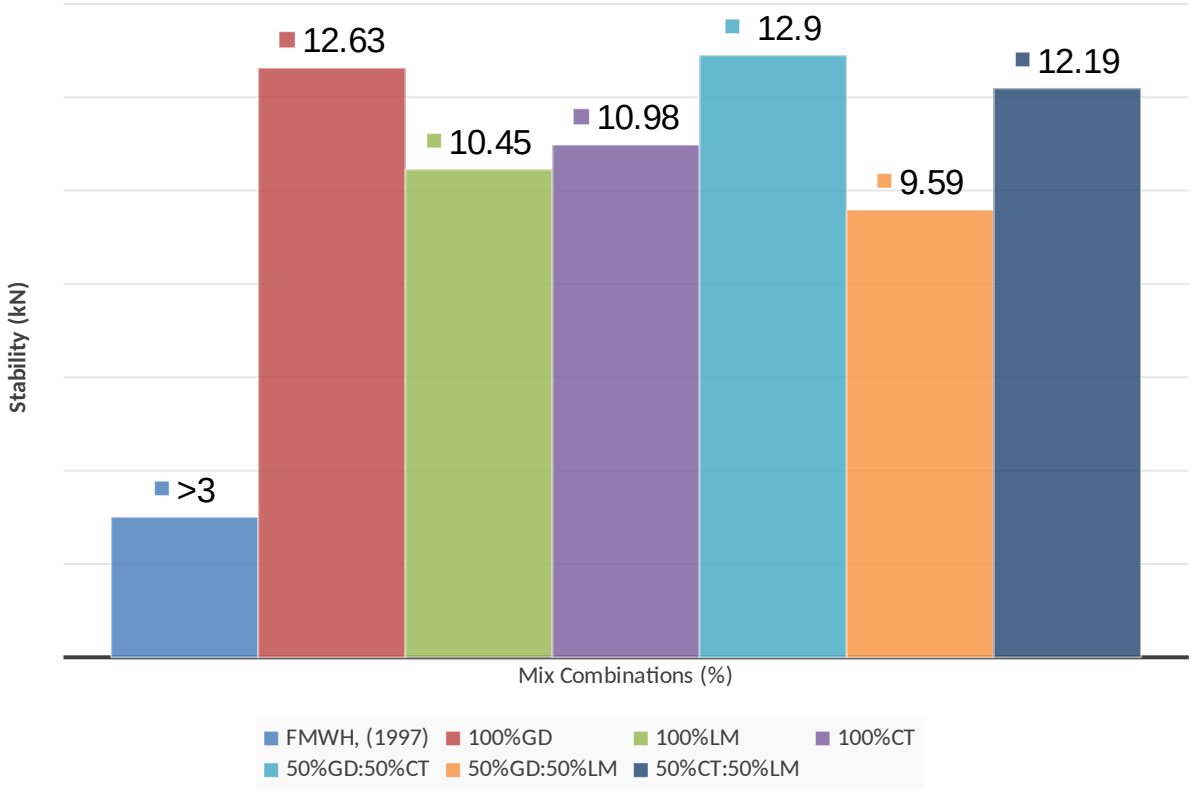
Table 4.3: Volumetric Properties of Mineral Fillers

Volumetric Properties	Flow (mm)	Void in Mineral Aggregate (%)	Void in Total Mix (%)	Void Filled by Bitumen
FMWH, (1997)	**2mm - 4mm**	**10% - 30%**	**3% - 5%**	**75% - 82%**
100%GD	**3.4**	**18.7**	**4.8**	**74.9**
100%LM	**3.5**	**26.3**	**13.7**	**48.18**
100%CT	**2.8**	**21.86**	**8.49**	**61.44**
50%GD: 50%CT	**4.0**	**22.10**	**8.77**	**60.59**
50%GD: 50%LM	**2.67**	**24.26**	**11.31**	**53.67**
50%CT: 50%LM	**3.03**	**24.64**	**11.75**	**52.60**

4.2.1 Stability

Figure 4.7 depicts the stability of the asphalt produced with different mineral fillers and at different mix proportions. The peak stability value was recorded for a mixture of granite dust and cement while the lowest stability value was recorded for a mixture of granite dust and lime. Assessment of the stability of asphalt produced with different mineral fillers revealed that granite dust has better effect on the stability of asphalt than cement and lime. It was observed that on addition of cement to granite dust, the stability of the asphalt mix increased with respect to that of the control mix while on partial addition of lime to granite dust, it was observed that the stability of the asphalt mix decreased with respect to that of the control mix. The stability of asphalt produced with a mixture of lime and cement was relatively lower than that of the control

mix. The increase in stability of asphalt produced with addition of cement to granite dust could be attributed to the bonding strength and fineness of cement. Therefore, it can be inferred that cement has better effect on the stability of asphalt than lime. Generally, the stability obtained for the different mineral fillers and at different mix proportion satisfied the requirement given by



Federal Ministry of Works and Housing, (1997).

Figure 4.7: Graph Chart Showing the Stability of Asphalt Mix Produced with Different Mineral Fillers

4.2.2 Flow

Table 4.3 shows the flow of the asphalt produced with different mineral fillers and at different mix combinations. The peak flow for the asphalt was recorded for asphalt produced with a mixture of granite dust and cement while the lowest flow was recorded for asphalt produced with a mixture of granite dust and lime. On partial addition of cement to lime, it was observed that the flow of the asphalt mix increased while on addition of lime to granite dust, it was observed that the flow of the asphalt mix decreased. The flow obtained for a blend of lime and cement was

lower than that of the control mix. The low flow of the asphalt produced with a mixture of granite dust and lime can be attributed to the fineness of lime as material with relatively high fineness will require high amount of bitumen to ensure flow of the asphalt than material with lesser amount fines. It can therefore be deduced that cement yields better flow for asphalt mix when used as mineral fillers than lime. In all circumstance, asphalt produced with the different mineral fillers and at different mix proportion obeyed the specification given by Federal Ministry of Works and Housing, (1997).

4.2.3 Void in Total Mix

Table 4.3 shows the voids in total mix present in asphalt produced with different mineral fillers and at different mix combinations. Apart from the control mix, asphalt produced with lime, cement, blend of lime and cement, blend of granite dust and cement, blend of granite dust and lime failed to meet the specification given by Federal Ministry of Works and Housing, (1997). The highest void in total mix was recorded for asphalt produced with 100% lime while the lowest void was recorded for the control mix. It was observed that asphalt produced with a blend of granite dust and cement has lesser void in total mix compared to asphalt produced with a blend of granite dust and lime. It can therefore be deduced that cement is more effective in filling voids when used as mineral fillers in asphalt than lime.

4.2.4 Void in Mineral Aggregate

The void in mineral aggregate for the asphalt concrete produced with different mineral fillers and at different mix combinations are shown in Table 4.3. The void in the mineral aggregate for the asphalt produced with different mineral fillers and at different mix combinations fall within the specification given by Federal Ministry of Works and Housing, (1997) which state that void in mineral aggregate should fall within 10% - 30%. The lowest void in the mineral aggregate was recorded for asphalt produced with 100% granite dust while the highest void in mineral aggregate was recorded for asphalt produced with 100% lime as mineral fillers. It was also observed that the void in mineral aggregate for asphalt produced with a blend of granite dust and cement was relatively lower compared to asphalt mix produced with a blend of granite dust and lime as mineral fillers. Therefore, it can be inferred that cement is more effective in reducing the void in the mineral aggregate when used as mineral fillers in asphalt mix. The void in mineral

aggregate produced with the blend of cement and lime as mineral fillers was higher than that of the control mix.

4.2.5 Void Filled by Bitumen

Table 4.2 depicts the void filled by bitumen for the asphalt mix produced with different mineral fillers and at different mix combinations. The highest void filled by bitumen was recorded for asphalt produced with 100% granite dust (control mix) while the lowest void filled by bitumen was recorded for asphalt with 100% lime. It was observed that the void filled by bitumen for asphalt produced with a blend of granite dust and cement as mineral fillers was higher than that produced with a blend of granite dust and lime as mineral fillers. The low value of void filled by bitumen recorded for asphalt produced with lime, blend of granite dust and lime and a blend of cement and lime as mineral fillers could be attributed to the fineness of cement and lime. Material with high amount of fineness requires more bitumen than material with less amount of fineness and fineness reduces the void filling ability of bitumen. It was observed that only asphalt produced with 100% granite dust as mineral fillers conforms to the specification given by Federal Ministry of Works and Housing, (1997).

CHAPTER FIVE

CONCLUSION AND RECOMMENDATION

5.1 Conclusion

From the findings obtained on the effect of mineral fillers on the stability and volumetric properties of asphalt, the following conclusion can be drawn.

1. Preliminary testing of bitumen used for asphalt production suggests that the bitumen satisfied the requirements given by Federal Ministry of Works and Housing, (1997) as the flash and fire point was 287.7^{0}C and 316.7^{0}C, penetration and viscosity was 69.33mm and 0.73Ns/m^2 and specific gravity was 1.02. This result justified the use of bitumen for the production of asphalt mix.

2. The aggregate samples also met the specification given by Federal Ministry of Works and Housing, (1997) as their impact and crushing strength was 6.3% and 4.5%, specific gravity of sand, granite and granite dust was 2.55, 2.61 and 2.76 respectively.

3. The stability of asphalt produced with granite dust, Portland cement, hydrated lime, blend of granite dust with cement, blend of granite dust with hydrated lime and blend of Portland cement with hydrated lime met the criteria given by Federal Ministry of Works and Housing, (1997) as their stability was greater than 3kN.

4. Asphalt mix produced granite dust admixed with cement yielded higher value of bulk density and stability than asphalt produced with granite dust admixed with hydrated lime and cement mixed with hydrated lime.

5. Asphalt produced with granite dust partially mixed with cement yielded the highest flow than asphalt produced with granite dust partially mixed with hydrated lime and cement mixed with hydrated lime.

6. The void in mineral aggregate from asphalt produced when granite dust was blended with hydrated lime was relatively higher than asphalt produced when granite dust was admixed with cement.

7. The total air void from asphalt produced when granite dust was blended with cement was relatively lower than asphalt produced when granite dust was admixed with hydrated lime.

8. The void filled by bitumen from asphalt produced when granite dust was blended with hydrated lime was relatively lower than asphalt produced when granite dust was admixed with cement. The low value obtained for granite dust to hydrated lime mixture could be attributed to the stiffening property of hydrated lime.

9. Portland cement has better effect on the stability of asphalt concrete than hydrated lime as the stability of asphalt concrete produced with cement was relatively higher than that of hydrated lime.

10. Portland cement have better effect on the volumetric properties of asphalt concrete than hydrated lime as asphalt concrete produced with a blend of granite dust and cement had better flow, void in mineral aggregate, void in total mix and void filled by bitumen than hydrated lime.

5.2 Recommendations

The recommendation on the effect of mineral fillers on the stability and volumetric properties of asphalt concrete is as follows:

1. The study recommends the use of Portland Cement and Hydrated Lime as mineral fillers when or if there is a shortage in granite dust during asphalt production.

2. The study recommends the use of Portland Cement as fillers for asphalt production than hydrated lime as cement yield better effect on stability and volumetric properties of asphalt mix than hydrated lime.

3. The study recommends the use of Granite Dust, Hydrated Lime and Portland Cement in Asphalt Mix Design as they all pass the FMWH stability requirement.

4. The study recommends Granite Dust as the best and suitable filler at the optimum binder content of 6% because its stability and volumetric properties all fall within the FMWH specifications.

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APPENDICES

APPENDIX A

Marshall Stability Test

Table A1: Marshall Stability Test Results for 100% Granite Dust at 5% Bitumen Content

Bulk Density (g/cm^3)	Specific Gravity (Gs)	Volume of Bitumen	Volume of Aggregate	Void in Mineral Aggregate (%)	Void in Total Mix	Void Filled by Bitumen	Stability (kN)	Flow (mm)
2.282	2.557	11.10	78.15	21.85	10.75	50.80	9.6	2.2

Table A2: Marshall Stability Test Results for 100% Granite Dust at 5.5% Bitumen Content

Bulk Density (g/cm^3)	Specific Gravity (Gs)	Volume of Bitumen	Volume of Aggregate	Void in Mineral Aggregate (%)	Void in Total Mix	Void Filled by Bitumen	Stability (kN)	Flow (mm)
2.330	2.532	12.44	79.38	20.62	8.20	60.33	10.67	2.8

Table A3: Marshall Stability Test Results for 100% Granite Dust at 6% Bitumen Content

Bulk Density (g/cm^3)	Specific Gravity (Gs)	Volume of Bitumen	Volume of Aggregate	Void in Mineral Aggregate (%)	Void in Total Mix	Void Filled by Bitumen	Stability (kN)	Flow (mm)
2.398	2.518	14.0	81.3	18.7	4.8	74.9	12.63	3.4

Table A4: Marshall Stability Test Results for 100% Granite Dust at 6.5% Bitumen Content

Bulk Density (g/cm^3)	Specific Gravity (Gs)	Volume of Bitumen	Volume of Aggregate	Void in Mineral Aggregate	Void in Total	Void Filled by Bitumen	Stability	Flow

				(%)	Mix		(kN)	(mm)
2.374	**2.499**	**14.98**	**80.02**	**19.98**	**5.00**	**74.97**	**13.22**	**3.8**

Table A5: Marshall Stability Test Results for 100% Granite Dust at 7% Bitumen Content

Bulk Density (g/cm^3)	**Specific Gravity (Gs)**	**Volume of Bitumen**	**Volume of Aggregate**	**Void in Mineral Aggregate (%)**	**Void in Total Mix**	**Void Filled by Bitumen**	**Stability (kN)**	**Flow (mm)**
2.300	**2.480**	**15.63**	**77.11**	**22.89**	**7.26**	**68.28**	**11.91**	**4.0**

Table A6: Marshall Stability Test Results for 100% Lime

Bulk Density (g/cm^3)	**Specific Gravity (Gs)**	**Volume of Bitumen**	**Volume of Aggregate**	**Void in Mineral Aggregate (%)**	**Void in Total Mix**	**Void Filled by Bitumen**	**Stability (KN)**	**Flow (mm)**
2.175	**2.52**	**12.67**	**73.70**	**26.30**	**13.70**	**48.18**	**10.45**	**3.5**

Table A7: Marshall Stability Test Results for 100% Cement

	Specific Gravity (Gs)	**Volume of Bitumen**	**Volume of Aggregate**	**Void in Mineral Aggregate**	**Void in Total**	**Void Filled by**	**Stability**	**Flow**

Bulk Density (g/cm^3)				**(%)**	**Mix**	**Bitumen**	**(KN)**	**(mm)**
2.306	**2.52**	**13.43**	**78.14**	**21.86**	**8.49**	**61.44**	**10.98**	**2.8**

Table A8: Marshall Stability Test Results for 50% Granite Dust: 50% Cement

Bulk Density (g/cm^3)	**Specific Gravity (Gs)**	**Volume of Bitumen**	**Volume of Aggregate**	**Void in Mineral Aggregate (%)**	**Void in Total Mix**	**Void Filled by Bitumen**	**Stability (KN)**	**Flow (mm)**
2.299	**2.52**	**13.39**	**77.90**	**22.10**	**8.77**	**60.59**	**12.90**	**4.0**

Table A9: Marshall Stability Test Results for 50% Granite Dust: 50% Lime

Bulk Density (g/cm^3)	**Specific Gravity (Gs)**	**Volume of Bitumen**	**Volume of Aggregate**	**Void in Mineral Aggregate (%)**	**Void in Total Mix**	**Void Filled by Bitumen**	**Stability (KN)**	**Flow (mm)**
2.235	**2.52**	**13.02**	**75.74**	**24.26**	**11.31**	**53.67**	**9.59**	**2.67**

Table A10: Marshall Stability Test Results for 50% Cement: 50% Lime

Bulk Density (g/cm^3)	Specific Gravity (Gs)	Volume of Bitumen	Volume of Aggregate	Void in Mineral Aggregate (%)	Void in Total Mix	Void Filled by Bitumen	Stability (KN)	Flow (mm)
2.224	2.52	12.96	75.36	24.64	11.75	52.60	12.19	3.03

APPENDIX B

Aggregate Impact and Crushing Value Test

Table B1: Aggregate Impact Value Test Result

Sieve Sizes (mm)	Weight Retained (g)	Weight Passing (g)	Total
2.0	547.5	99.3	646.8
Aggregate Impact Value (%)	6.5		

Table B2: Aggregate Crushing Value Test Result

Sieve Sizes (mm)	Weight Retained (g)	Weight Passing (g)	Total
2.0	1750.2	526.0	2276.2
Aggregate Crushing Value (%)	4.3		

APPENDIX C

Sieve Analysis Test

Table C1: Particle Size Distribution Test Result for Sand

SIEVES SIZES (mm)	Mass Retained	% mass Retained	cumulative retained	cumulative % finer
2	7.97	2.6566667	2.6566667	97.343333

1.18	**9.98**	**3.3266667**	**5.9833333**	**94.016667**
0.86	**11.73**	**3.91**	**9.8933333**	**90.106667**
0.6	**27.95**	**9.3166667**	**19.21**	**80.79**
0.425	**44.02**	**14.673333**	**33.883333**	**66.116667**
0.3	**53.69**	**17.89667**	**51.78**	**48.22**
0.15	**112.1**	**37.36667**	**89.146667**	**10.853333**
0.075	**8.46**	**2.82**	**91.966667**	**8.0333333**
Tray	**2.3**	**0.766667**	**92.733333**	**0**

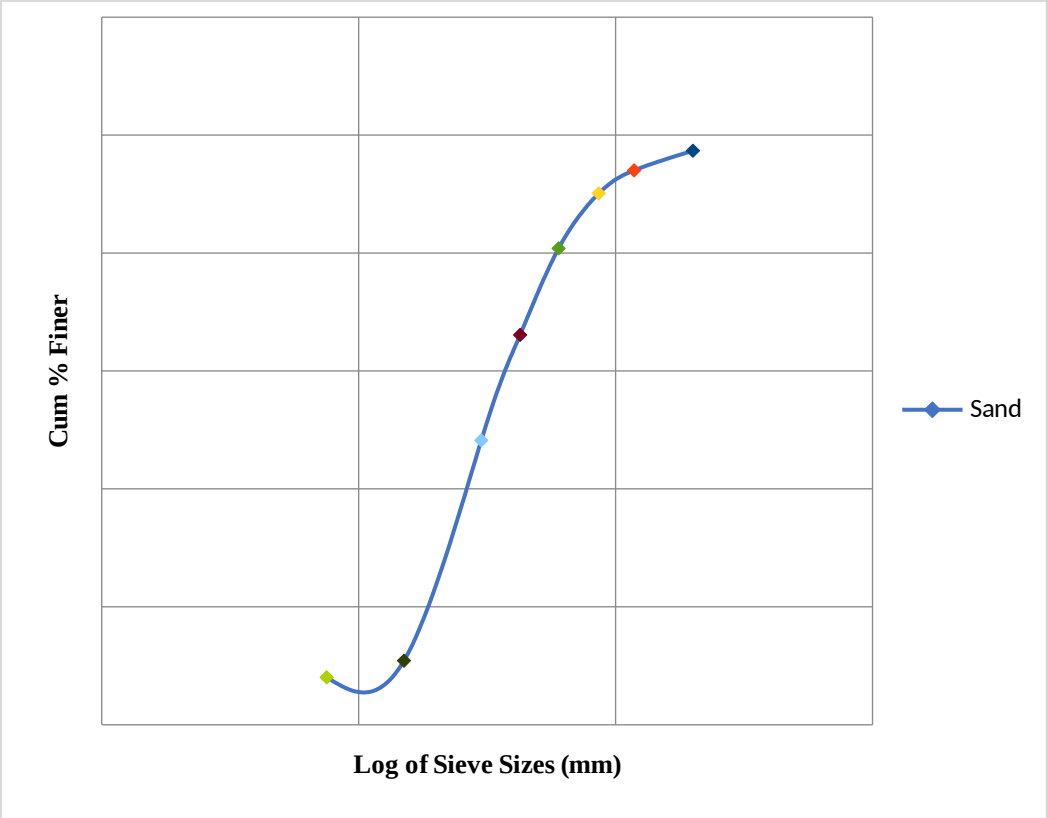

Figure C1: Particle Size Distribution Curve for Sand

Table C2: Particle Size Distribution Test Results for Granite

SIEVES SIZES (mm)	Mass Retained	% mass Retained	cumulative retained	cumulative % finer
31.25	0.15	0.01	0.01	99.99
25	88.5	8.05	8.06	91.94
20	364.11	33.10	41.16	58.84
12.5	508	46.18	87.34	12.66
9.5	15.67	1.42	88.76	11.24
6.3	30.55	2.78	91.54	8.46
4.75	14.87	1.35	92.89	7.11
Tray	12.71	1.16	94.05	5.95

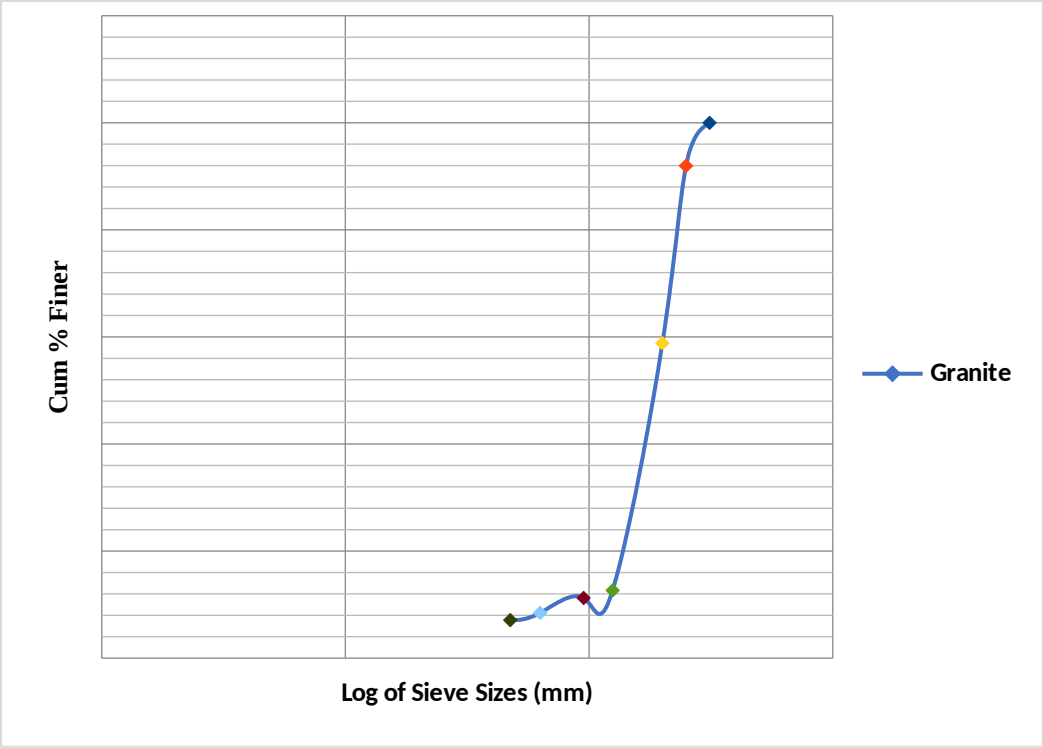

Figure C2: Particle Size Distribution Curve for Granite

Table C3: Particle Size Distribution Test Results for Granite Dust

Sieve Sizes (mm)	Mass Retained	% Mass Retained	Cum % Retained	Cum % Finer
4.75	7.96	1.592	1.592	98.408
2	138.46	27.692	29.284	70.716
1.18	70.72	14.144	43.428	56.572
0.85	33.56	6.712	50.14	49.86
0.6	30.85	6.17	56.31	43.69
0.425	25.78	5.156	61.466	38.534
0.3	20.9	4.18	65.646	34.354
0.15	38.7	7.74	73.386	26.614
0.075	34.44	6.888	80.274	19.726
Tray	40.4	8.08	88.354	11.646

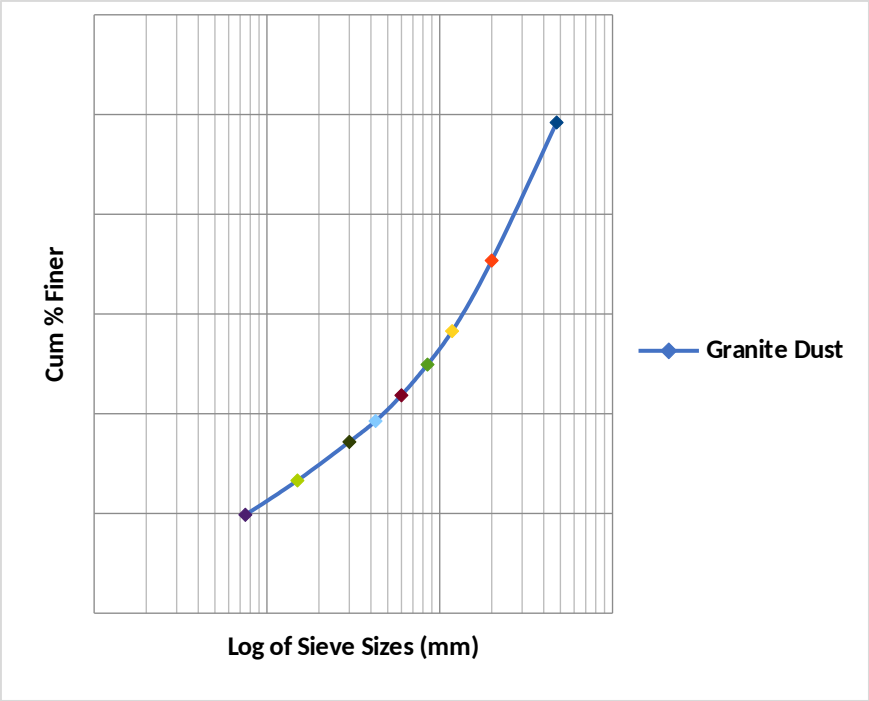

Figure C3: Particle Size Distribution Curve for Granite Dust