ANALYSIS OF SELECTED TYPES OF ROAD SURFACINGS

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THESIS

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This thesis entitled "Analysis of Selected types of Road Surfacings" by AKPOMIE, B.S.O. meet with the regulations governing the award of the degree of Master of Science (Civil Engineering Construction Materials) of the Ahmadu Bello University, Zaria and is approved for its contribution to knowledge and literary presentation.

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I hereby declare that this thesis has been prepared by me and that it is a record of my own work. It has not been accepted in any previous application for a higher degree. All sources of information are specifically acknowledged by means of references.

AKPOMIE, B.S.O.
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DEDICATION

This Thesis is dedicated to my parents

Chief and Chief (Mrs) D.O. Akpomie.

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ABSTRACT

Various methods were used to design asphaltic concrete, hot rolled and mastic asphalts. Asphaltic concrete and mastic asphalt were designed principally using the Marshall procedure, while hot rolled asphalt was designed according to BS 594 and a method proposed for gap-graded mixes. A typical surface dressing design was also carried out for a lightly trafficked road suitable for Northern Nigeria. The optimum binder content and other design parameters were determined for the surfacings investigated. Some of the values obtained were 6.9%, 6.86% and 13.7% as binder content for asphaltic concrete, hot rolled and mastic asphalts respectively. 17.2kN, 8.4kN and 10.2kN for stability. 4.0%, 5.2% and 3.0% were obtained as voids in the mix. The analysis and cost estimates of test results showed that it would cost N3G5.17 to lay a m^3 of asphaltic concrete against N341.70 for hot rolled asphalt. It will cost 84.86 to apply surface dressing on a m^2 of road surface.

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1.1 PREAMBLE

Asphalt concrete can be defined as a mixture of coarse, fine aggregates, fillers and bitumen. The bitumen is added in measured quantities to produce a well coated, densely compacted mix.

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The aggregates can be close textured (densely graded or packed), medium textured or open textured surfacings. The coarse aggregates are those retained on sieve 2.36 mm (No. 7) B.S sieve while the fine aggregates are those passing sieve 2.36 mm and are retained on the 75 μm sieve (No. 200) B.S. sieve. The mineral filler may be a mineral dust having not less than 65 percent passing sieve 75 μm, other fillers include limestone dust, cement and hydrated lime.

A dense graded surfacing is that which is thoroughly mixed such that the voids in the coarse aggregates are filled by the immediate smaller aggregates until almost all voids are filled. A medium textured surfacing is produced by mixing aggregates in a way that it is not uniformly graded and contains more voids than the densely graded ones. The open textured surfacing as the name suggests contains more voids than the dense and medium textured surfacings. For surface dressings, single sized aggregates are used.

There are different types of road surfacing materials namely

a. Asphalt concrete

b. Hot rolled asphalt

c. Mastic asphalt

d. Sheet asphalt

e. Fine cold asphalt

f. Surface dressing etc.

The principal distinguishing feature separating the different types of asphalts lies in the gradation. Some asphalts have continuous gradation like asphaltic concrete while others are gap graded (Hot rolled asphalt), some do not even have any percentage of coarse aggregates. Different types of asphalts also make use of varying grades of bitumen. The more densely parked aggregates use harder grades of bitumen, this type of bitumen is usually mixed hot and laid hot. The medium and open textured ones could use from hard to very soft bitumens like the cut-backs and are laid hot or cold. Climatic conditions also play important roles in determining the grade of bitumen to use. In tropical countries where temperatures are high, the tendency is for harder grades of bitumen to be used while the reverse is the case in temperate climates.

The choice of asphalt is influenced by the location of facility (whether city or rural road), mode of construction to be adopted, the traffic volume as well as intentions of the authorities concerned.

Essentially, the design of asphalts, like other engineering designs, involves the selection, refining and proportioning of the constituent materials. By so doing, the desired properties and qualities of the finished product are obtained. The idea is to obtain a mix that is economical while still meeting most of these requirements. The surfacing should, in general,

i) Provide good running surface for vehicles
ii) Be highly resistant to surface wear and deformation
iii) Provide high resistance to skidding
iv) Improve night visibility
v) Contain enough bitumen to ensure durability
vi) Possess enough bitumen to permit good placement and rolling
vii) Contain sufficient voids in the total compacted mix to allow for slight additional compaction under traffic load.

Failure to meet any of the listed requirements can cause premature failure. The need therefore to design asphalt that will meet the above requirements and at the same time making sure that the underlying layers are not overstressed when loaded, cannot be overemphasized.

1.2 PROBLEM STATEMENT

Most wearing courses before long after construction crack, rut and even shove causing discomfort to users and at times accidents when they are located at akward places. The premature failure also causes the authorities concerned money that could have been used elsewhere to maintain the already constructed one, when resort could be made to alternative kinds of and even cheaper asphalts.

Flexible pavement failure could be due to a number of factors some of which are:

a. Subgrade or sub-base failure

b. Constructional errors

c. Underestimation of traffic load during design

d. Failure of the black top or asphalt.

The specific problem tackled in this work is how to arrest the premature failure of the asphalt surfacing. It is hoped that, the Project will highlight some of the major differences between the asphalts to be considered as regards design, durability and economy of production. Nigeria being an Oil Producing nation would find it attractive to have construction of flexible pavements as an acceptable strategy for road development for a long time. This indeed is the background assumed for the study.

1.3 OBJECTIVES OF THESIS

Asphalt surfaces play crucial roles in ensuring that the underlying layers of the pavement are not overstressed, at the same time preventing the ingress of water into the underlying layers. Thus this thesis investigated factors affecting the various design parameters of asphalt concrete and evaluated the relative performances of different asphalts namely

Hot rolled asphalt

Mastic asphalt

as it relates to the tropics. The study also looked into a typical surface dressing design as it affects Northern Nigeria.

In the course of achieving the study objectives, it was necessary to undertake design and specimen testing for the afore-mentioned asphalts. Laboratory tests as well as specimen preparation were undertaken in the Transportation Engineering Laboratory of the Department of Civil Engineering, ABU, Zaria. The asphaltic pavements were designed for a medium trafficked road (i.e. 150 - 300 comm vehicle/day).

1.4 SCOPE OF PROJECT

In section 1.1, Asphalt was defined as a dense mixture of bitumen and aggregate in which the composition is such that

the compacted mix obtains its strength by virtue of the combination of a strong stiff mortar and the interlocking of the coarse aggregates.

There are no basic differences in manufacture apart from that in the proportion of aggregates and the grades of bitumen used. This work therefore concentrated on preparing specimens according to the specifications of the Asphalt Institute and BS 594. The design parameters obtained for asphaltic concrete were compared with that given by the Asphalt Institute while others were compared with the parameters of the designed asphaltic concrete.

The cost estimates of production and laying were also examined. These cost were determined for all surfacings but mastic asphalt.

1.5 METHODOLOGY

Texts, standards and previous research efforts on different types of asphalts were studied. An experimental procedure with designs made strictly to specifications was drawn. A total of fifty one (51) samples of marshall stability specimens were cast, to enable the design parameters of the asphalt to be studied. The aggregates used were crushed granites with bitumen grades of 80/100, 40/60 10/20 and S125.

Discussion and analysis of the test results were made based on the criteria of the Asphalt Institute and Standards of the Federal Ministry of Works, Lagos, Nigeria. Conclusions and recommendations were made from the comparisons, discussion of the test results and cost estimates.

CHAPTER TWO

LITERATURE REVIEW

2.0 BACKGROUND

D. Brien (1876) defined asphalt as any dense mixture of bitumen and aggregate in which the composition is such that the mix, when compacted, obtains its strength by virtue of either a strong and stiff mortar or mainly from the aggregate interlock or both. The high quality bituminous material that derives its strength from aggregate interlock is known as asphaltic concrete while the other hot rolled asphalt derives its strength from the strong stiff mortar of bitumen and fine aggregates, Mastic asphalt derives its strength like hot rolled asphalt does.

Asphalt concrete which is a continuously graded type of mix is carefully graded such that the voids in the larger aggregates are filled by the immediate smaller ones, with the object of obtaining a mix of . voids (after adding bitumen) of about 4.0%. Hot rolled asphalt is gap graded (not voids controlled) that is, it consists of mortar of bitumen and fine aggregates together with single sized aggregates of considerably larger size. Mastic asphalt is continuously graded from fine aggregate to filler with voids overfilled with hard bitumen/ filler mortar.

This chapter summaries current literature on the mixture design procedure of hot rolled asphalt and asphaltic concrete as well as the material characterization.

2.1 MIXTURES DESIGN

Bituminous mixes when used in a road structure have a number of different functions to fulfil. They must, among others, do the following:

1. Resist deformation by traffic
2. Be impervious so as to protect the lower layers of the road structure, from moisture ingress and thereby preserve the structural integrity of the pavement.
3. Be durable
4. Give a skid-resistant surface.
5. Contribute to the strength of the complete road structure.

In addition to the listed properties, when laid, they must be tolerant to manufacturing variations that will inevitably occur. They must also be capable of being handled and laid in the conditions that are found on construction sites.

The mixture design problem composes of two parts. The first is to determine the optimum binder content for a chosen aggregate type and grading. The second is to assess whether the designed mix meets the specifications governing the job. Otherwise, the aggregate type and gradation are changed - with a resulting change in the optimum bitumen content. Heatherly and Leaver (1973) wrote that there is an optimal binder content that will produce a satisfactory mix for a given aggregate grading and deviations from it can lead to premature failure.

One of the methods of obtaining the optimum binder content is the Marshall test procedure. Some of the desirable characteristic to optimize mixture properties are presented in table 2.1.

Table 2.1 - Desirable Characteristics to Optimize mixture Properties

Mixture Property	Bitumen Content	Aggregate Gradation	Degree of compaction
Stability	Low	Dense	High
Durability	High	Dense	High
Workability	High	Open	-
Flexibility	High	Open	-
Fatique Resistance	High	Dense	High
Skid Resistance	Low	Open/ Dense	High
Imperviousness	High	Dense	High
Fracture Strength	High	Dense	High

Source: Asphalt Institute (1969, 1970)

2.2 MATERIALS CHARACTERISATION

Before describing the effects that the properties and proportions of the components can have on the performance of the complete mix, it is necessary to comment on the test methods that are suitable when investigating these effects.

Obviously, test methods which are to be used in the laboratory should be correlated with road performance. This however, requires trials that are lengthy and expensive. Mavais, C.P (1974) has shown that a good correlation exists between the results of Laboratory wheel tracking tests and Marshall Tests made on gap graded mixes especially if the Marshall Test results are expressed as 'Marshall quotient' (stiffness). If it is accepted that wheel tracking tests are simulations of road performance so far as deformation is concerned, then the Marshall test can also be used to study the deformation properties of mixes.

2.3 HOT ROLLED ASPHALT

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Coarse Aggregates –

The type, size and grading of coarse aggregate seem to have only a small effect on the properties of hot rolled asphalt. Duthie, J,L. (1974) was able to show this, when he used five different types of aggregates while the other materials were unchanged, that the Marshall Test results obtained were plus or minus 10% of the mean. He was able to show that there was little or no effect on the properties when the grading of the aggregates was changed from completely single size (14mm to 10mm) to a graded aggregate (20mm to 2.36 mm).

There is however, a fairly big effect caused by the quantity of coarse aggregate in a gap graded mix. Unless the coarse aggregate content is about 60% or more there is very

little stone to stone contact in the compacted mix. High coarse aggregate content has advantages as regards deformation, but its use may introduce other problems. At 55% coarse aggregate content, there is a danger that the mix will be permeable owing to difficulty of compaction. The limited permeability that can arise in these conditions is dangerous in that it allows water to seep into the mix. Under this conditions it is possible for stripping of the aggregates to occur. To prevent this, it is necessary to ensure that the voids in the mortar is restricted and 9.0% maximum has been suggested provisionally.

Fine Aggregates -

Of all the components that form hot rolled asphalt, the quantity of sand is the most important. The amount of variations caused by different natural sands has been investigated by D. Brien (1976) and Duthie, J.L. (1974) when at the same binder content, different asphalts were made with a number of sands. The results obtained varied by a factor of 4 from the best mix to the worst. This difference can be reduced, however, if instead of designing the mixes with some recipe binder content, the optimum binder content for each sand type is determined by the Marshall Test method. The reduction can be such as to limit the strength variation factor to about 2.5. BS 594 (1973) gives a procedure for selecting an optimum binder content for sand.

Crushed stone sands do not give the benefits that are expected from their angular shape and rough texture. This is because their continuous grading results in lower voids in the mortar at the optimum binder content and this in turn results in bigger flow values in the Marshall Test. Although mixes made from them have higher stability values than mixes with natural sands, the flow values are also increased by about the same amount and since Laboratory wheel tracking tests have shown that it is the Marshall quotient rather than stability that is related to deformation, there will appear to be little or no advantage in using them. One disadvantage of crushed rock fines is that mixes made with them are less tolerant to variation in bitumen content.

One way of characterising sand is by the 'dry viscosity test'. In this test, the time taken for a known weight of sand (usually 100gm) retained between sieves 300 μm and 150 μm to pass through a standard orifice of 6mm is measured. Sand having a dry viscosity of 15secs or less are those to be used if reasonable Marshall quotient values are to be obtained.

Filler -

Filler can be thought of as acting in one of two ways. Firstly, it can be thought of as modifying the grading of the sand, thereby giving a denser mix and at the same time reducing the amount of bitumen. Secondly the filler and bitumen together form a paste that both lubricates and binds the sand to form a mortar. Some of the common filler materials used are cement, limestone dust etc.

Heukelom (1965), Rigden, , , . (1947) and others have shown that the ratio of the viscosity of a filler/bitumen mixture to the pure bitumen is a function of the effective volume concentration of the filler. The term 'effective' is important since different fillers have different effective volumes due to their different packing characteristics. The packing characteristics can be measured by the 'void' of dry compacted filler, a procedure now included in BS 812.

Bitumen -

The function of the bitumen in an asphalt mix is to act as a lubricant when the mix is being laid in order to assist compattion and thereafter to act as a high viscoelastic binder of high viscosity undertraffic. In order to have this high viscosity at service temperature a hard grade bitumen is required. Experience has shown the Lower limit of bitumen penetration to be about 30pen,, since asphalt made with harder bitumen may crack if the base is not sufficiently strong,, or may fret at the surface if they are not properly compacted. Depending on the climate, bitumen penetration of 40/50 or 60/70 are usually used.

Proportions of Sand, Filler and Bitumen

The effect that the amount of coarse aggregate has upon the properties of a mix has already been discussed. Its effects are independent of the properties of the mortar into

which it is put. When the proportions of sand, filler and bitumen are considered, Duthie, (1974) was able to show that;

> mixes made at a fixed filler content, as the bitumen content increases, the Marshall quotient recorded increases as well. This is due to the lubricating action of the bitumen which results in better compaction of the mix, and partly to the increased cohesion of the mix which contains more bitumen. This increase continues until a point is reached when increase in bitumen results in fall in the Marshall quotient, mainly due to bitumen pushing the sand particles apart so the contact between particles are lost. If the filler content is increased, the same effect is noticed but the optimum bitumen content observed is lower and the Marshall quotient recorded is higher. At the same time, the added filler has increased the viscosity of the bitumen.

Most conventional gap-graded mixes are composed of a mortar that contains 10 to 11% bitumen and 12 to 15% of filler. D. Brien (1976) has suggested a design method which begins with the BS 594 (1973) method but continues to study the effect of different percentages of coarse aggregates on the design parameters of the sand mortar.

2.4 ASPHALT CONCRETE

Coarse Aggregate -

Unlike hot rolled asphalt, the coarse aggregates used are carefully graded, the object being to produce a blend of

aggregates so proportioned that they will compact to give a dense interlock. The nature of the coarse aggregates has considerably more effect on the properties of asphaltic Concrete mix than it has on hot rolled asphalt. Angular crushed rock can give nearly double the stability of rounded uncrushed gravels. The size is important (larger aggregates giving higher stabilities) so also is the quantity of the coarse aggregates in the mix.

It is generally believed that aggregates grading which gives the greatest possible aggregate density will also give the highest stability when combined with the optimum amount of bitumen. The grading curve most commonly quoted is that due to Fuller, . and Thompson, (1907) and is of the form

$$P = 100\ (d/D)^n$$

where D = max. size of aggregate blend

d = size of aggregate in question

P ?= Percentage of aggregate in question

n = 0.5

Nij boer, . (1948) made mixes with aggregates blended to grading curves of this form, but with a range of n values and found out that the densest mixes, were those obtained when n lay between 0.4 and 0.5.

Fines -

As mentioned above, the shape and roughness of the fines have a big effect on the properties of the mix. Though the use of crushed stone fines makes it easier to follow the Fuller type of curve, the presence of all crushed material will lead to mixes that are difficult to compact at the time of laying. If sufficient bitumen is put into these mixes to give a suitable void content, there is the possibility of their bleeding when laid and subjected to traffic.

It is therefore necessary to include some natural sand in the fines to improve its workability, though this will have the effect of lowering the stability. The use of single sized sand will make it difficult to keep within the limits of a specified grading envelope.

Filler -

There is basically no difference between the filler material used for hot rolled asphalt and asphalt concrete. The only difference lies in the quantity. Since so much of the stability of asphalt concrete comes from the aggregate interlock, the need for a stiff binder is not as great as with hot rolled asphalt. It should be realised that though increase in filler content increases the recorded stability, it reduces the bitumen content. Such a change causes the mix to become very sensitive to further slight changes in bitumen content which is bad considering manufacturing tolerances. It is therefore preferred that the filler content be low.

Bitumen -

The grade of bitumen used for asphalt concrete is lower than for hot rolled asphalt because asphalt concrete does not depend on a stiff binder for its stability. Bitumen of between 70 to 100 penetration is used depending on the climate.

2.5 MASTIC ASPHALT

The materials Properties for mastic asphalt is not very different from that of hot rolled aspahlt. The main difference comes from the quantity of filler and the grade of bitumen used. While hot rolled asphalt uses bitumen grade range 40 to 70 penetration, mastic asphalt uses 10 to 30 penetration grade bitumen. The filler content content for mastic asphalt can be as high as 30% by weight of aggregate, while hot rolled asphalt is 9.0% Specification governing it is BS 1446 (1948).

2.6 SURFACE DRESSING

Oglesby, . (1975) defines surface dressing as a thin bituminous surface of binder covered by mineral aggregates, applied to an earth, gravel or water bound macadam surface or to a stabilized base.

On lightly travelled roads, surface dressing provides a relatively cheap pavement which solves the problems of dust control, formation of corrugations and loss of surfacing materials created by the abrasive action of vehicle tyres. Surface dressing has no structural strength of its own, so for heavily travelled roads it can only be used as a temporary pavement and as a protection for the base material pending the application of an asphalt base-course or wearing course.

O'Flaherty (1974) noted that surface dressing cannot be used to restore the riding qualities of an already mis-shapen road. To obtain good results, the surfacing being treated must be sound clean and have a uniform texture.

TRRL (1972) investigated and found out that surface dressing that have proved successful failed mainly through the aggregates being worn away by traffic. Where failure was premature, it was either by scabbing (i.e. early dislogdement of the chipping from the surfacing) or by bleeding or fatting up. (when the bitumen works its way up to the surface of the dressing). The problems to be solved in surface dressing consist primarily in designing to prevent scabbing and bleeding. Unfortunately, in many countries, the difficulty in achieving adequate control of the surface dressing process means that the lives of surface dressing are much shorter than they could be.

The binders most commonly used for surface dressing operation are S125, MC0, MC1 and some bitumen emulsions.

CHAPTER THREE

3.0 EXPERIMENTAL PROCEDURE

In accordance with the objectives of this project, an experimental procedure based on factors and levels was carried out dealing with the experimental design, materials, specimens and testing procedure as described in subsequent sections of this chapter.

3.1 EXPERIMENTAL DESIGN

In order to minimise the number of tests and facilitate interpretation of test results as recommended by Ostle (1966), Hadley et al (1967), Hudson (1970) Anderson (1970) and by Anderson and McClean (1974), the experimental design was based on factors involved and recommendations of previous research works, Jackson and O'Brien (1962) and Standard (FMW, 1970) in use.

Factors and Levels

Studies the world over (RRL, 1962) have shown that the stability, flow, compacted density and voids character of an asphaltic mixture are affected by such factors as the mixing and testing temperature, the amount and type of bitumen, amount and type of mineral filler, environmental conditions, type of aggregate gradation and the compactive efforts applied. This project, however, involved the casting of cores for the

asphalts being investigated and determining the optimum binder content. The other design parameters corresponding to the optimum binder content were analysed and compared.

3.2 MATERIALS

The materials used for this project are described and characterised as follows:

Aggregates

The aggregates used were crushed clean granite of recommended sizes, shapes and texture (FMW, 1970) obtained from the Transportation laboratory of the Department of Civil Engineering, Ahmadu Bello University, Zaria.

The specific gravity, Los Angles Abrasion value, Impact Value and crushing strength of the aggregates were determined and their mean values found. The specific gravities were 2.57 and 2.55 for coarse and fine aggregates respectively. While 2.88, 19 and 27 percents were obtained as Los Angeles Abrasion Value, impact value and crushing strength of the aggregates respectively. These tests were conducted in accordance with BS 812. Details of the test results are shown in Appendix C. The values obtained were compare compared with widely accepted and published values

TABLE 3.1: SUMMARY OF MEANS AND RANGES OF SOME ROADSTONE TESTS

Group Classification (BS 812)		Aggregate Crushing value	Aggregate impact value	Aggregate abrasion value	Specific gravity
Basalt	Mean	14	15	6.1	2.80
	Range	(7-25)	(7-25)	(2-12)	(2.6-3.0)
Granite	Mean	20	19	4.8	2.69
	Range	(9-35)	(9-35)	(3 - 9)	(2.6-3.0)
Flint	Mean	18	23	1.1	2.54
	Range	(7 -25)	(19-27)	(1-2)	(2.4-2.6)
Limestone	Mean	24	23	13.7	2.66
	Range	(11-37)	(17-33)	(7 - 26)	(2.5-2.9)

Source: (RRL) Road Note No. 24 (1959)

The aggregates were then graded according to the Asphalt Institute grading envelope as well as that of BS 594. The adopted gradings are shown in Figure 3.1 and marked 1, 2 and 3 for asphalt concrete, hot rolled and mastic asphalts. The specified grading envelopes are shown in table 3.2.

Filler

The filler material used was cement and it was obtained from the concrete Laboratory of the Civil Engineering Department of Ahmadu Bello University, Zaria. Typical test results of the properties (Physcial and chemical) of the cement obtained according to BS 812 are shown in Table 3.3.

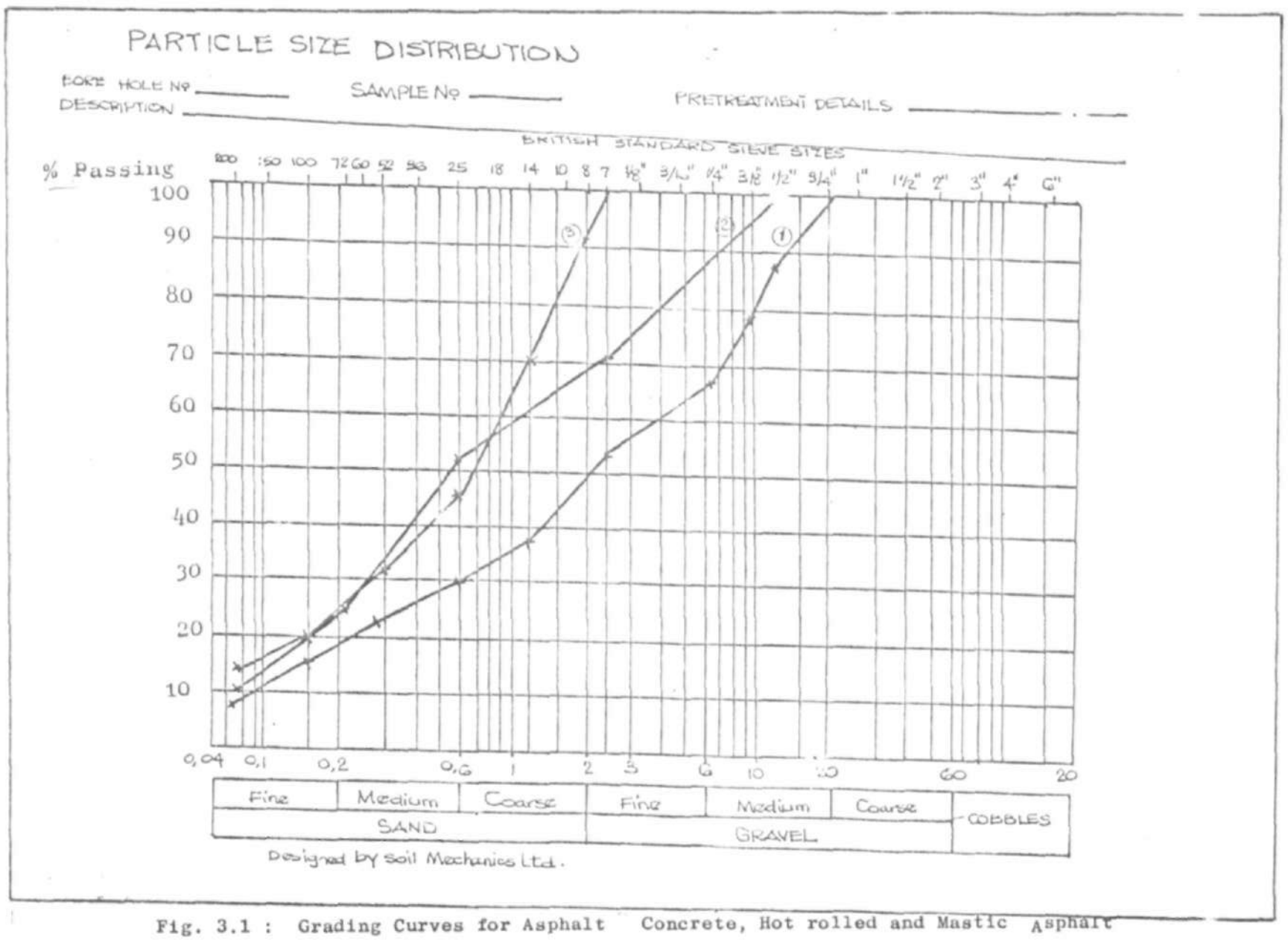


Fig. 3.1 : Grading Curves for Asphalt Concrete, Hot rolled and Mastic Asphalt Respectively.

TABLE 3.2: RECOMMENDED GRADING RANGES FOR ASPHALTIC CONCRETE, HOT ROLLED AND MASTIC ASPHALTS

BS Sieve sizes	Percentage passing		
	Asphaltic concrete	Hot rolled aspahlt	Mastic asphalt
20mm (3/4in)	100		
14mm (½in)	80-100		
10mm (3/8in)	-		
6.3mm (¼in)	-		
5mm (3/16in)	54-72	100	
3.35mm (1/8in)	-	-	
2.36mm(No.7)	42-58	95 -100	100
1.18mm (No.14)	34-48	-	
600 µm (N025)	26-38	75-100	92-100
300µm (No.52)	18-28	15-60	76-95
150µm (No.100)	12-20	-	
75µm (No.200)	6-12		56-85

Source: Asphalt Institute, (1969), BS594 (1973) and BS1446(1948)

TABLE 3.3 PROPERTIES OF CEMENT USED

Property	Number	Property	Number
a) Physical specific gravity	3.1	Initial set, min	180
Specific surface area (m2/kg)	320	Final set min	215
Fraction + 90µm	10	Bulk density in	-
Concrete cube (MN/m^2)		Benzene g/mL	0.66
at 3 days	0.9	b .Chemical. Free lime%	2.5
at 7 days	24	Loss on ignition %	1.4
at 28 days	34	SO_3 %	1.4

Bitumen

The bitumens used were 80/100, 40/60 and 10/20 penetration grades respectively for asphalt concrete, hot rolled and mastic asphalts while S125 was used for surface dressing. They were obtained from the Civil Engineering Department, Transportation Laboratory and the Maintenance and Services Department of the University. The properties of these bitumens are shown in Table 3.4.

Table 3.4: PROPERTIES OF BITUMEN USED

Property	Type of Surfacing			
	Asphaltic concrete	Hot rolled asphalt	Mastic asphalt	Surface dressing
Specific gravity	1.03	0.95	0.87	1.02
Softening point (R&B)	50°C	54°C	65°C	40°C
Penetration at 25°C	82	55	13	
Ductility at 25°C	100%	100%	100%	100%
Solubility in CCL_4	60%	64%	61%	62%
Viscosity (sec) at 60°C	1063	1126	1237	73
Flash point °C	220	230	250	50
Fire point °C	250	280	310	80

3.3 PREPARATION OF SPECIMEN

The respective gradings suitable for the different types of asphalt under investigation were first selected. Then the percentages by weight of aggregates retained on the different sieves were weighed. The capacity of the cylindrical core used was 1200gm (1.2kg) by weight so the percentage retained on each sieve weighed out was that percent of 1200gm, such that when the components were mixed, and weighed, it weighed 1200gm. The grading curve adopted is shown in Figure 3.1 while the table is included in appendix C.

The bitumens were added in different percentages within the stipulated limits of each of the asphalts considered. They ranged from 5 to 10%, 7 to 12% and 13 to 18% for asphalt concrete, hot rolled and mastic asphalts respectively. A typical design table is shown in Table 3.5.

TABLE 3.5: TYPICAL DESIGN TABLE

Sieve size	% Passing	% retained	5% bit.	6% bit.	7% bit.	8% bit.	9% bit.	10% bit.
200mm	100							
14mm	87	13	148.2	146.6	145.0	143.3	142	140.4
10mm	78	9	102.6	101.5	100.4	99.4	98.3	97.2
6.3mm	66	12	136.8	135.3	133.9	132.5	131.0	129.6
2.36mm	52	14	159.6	157.9	156.2	154.6	152.9	151.2
1.18mm	38	14	159.6	157.9	156.2	154.6	152.9	151.2
600 μm	30	8	91.2	90.2	89.3	88.3	87.4	86.4
300 μm	22	8	91,2	90.2	89.3	88.3	89.4	86.4
150 μm	16	6	68.4	67.7	67	66.2	65.5	64.8
75 μm	9	7	79.8	78.4	78.1	77.3	76.4	75.2
Filler	-	9	102.6	101.5	100.4	99.4	98.3	97.2
Bitumen	-		60	72	84	96	108	120

Taking the 5% bitumen content for instance, the percentage of 14mm aggregates retained was 13 therefore 13% of 1200gm equalled 148.2gm. 5% of 1200gm was 60gm hence the 60gm Bitumen that was added. This was done for the individual sieves before the aggregates were mixed. The bitumen was added and the mixture left in the oven until the bitumen was sufficiently fluid to coat the aggregates properly before the whole constituents were mixed together. The mixture was later put into the standard cylindrical mould and compacted with 50 blows of the rammer to both

faces. The mixture was allowed to cool in water before it was extruded from the mould.

3.4 TESTING PROCEDURE

The Marshall instrument was used to crush all the cores prepared, according to ASTM 0-1559, Asphalt Institute (1969) standards. Before then, the core samples had been weighed in air and water to determine its density. The cores were left in a water bath for about 30mins. The temperature of the water bath was maintained at $60^{o}C$.

The specimens were removed from the water bath, dried and taken to the Marshall instrument for crushing. Two readings were recorded from the instrument, namely, stability (i.e. resistance to deformation) and Flow value (corresponding deformation at failure).

A number of other design parameters were determined by calculations among which are:-

Compacted density of the mix (CDM), as given in eq. 3.1

$$CDM = \frac{\text{Wt in air}}{\text{Wt in air-wt in water}} \quad \ldots \quad (3.1)$$

Compacted density of mixed aggregates (CDMA) as given in eq.3.2

$$CDMA = CDM\ (1 - \frac{B}{100}) \quad \ldots \quad .. \quad (3.2)$$

where B = Percentage bitumen content by wt.

Specific gravity of the mixed aggregate (SGMA) as given in eq 3.3.

$$SGMA = \frac{100}{\sum_{i=1}^{n} \frac{w_i}{G_1}} \quad \ldots \quad .. \quad (3.3)$$

where w_i = % by wt of the i^{th} aggregate which has specific gravity G_1

n = number of constitutent aggregates making up the mix.

Specific gravity of the mix (SGM) as given in eq. 3.4

$$SGM = \frac{100}{\frac{100-B}{SGMA} + \frac{B}{G_B}} \quad \ldots \quad \ldots \quad (3.4)$$

where G_b = Specific gravity of bitumen.

Voids in the mix (VIM) given in eq. 3.5

$$Vlm = \frac{SGM - CDM}{SGM} \quad \ldots \quad \ldots \quad (3.5)$$

Voids in the aggregate is given in eq. 3.6

$$VMA = \frac{SGMA - CDMA}{SGMA} \quad \ldots \quad \ldots \quad (3.6)$$

Voids filled with bitumen (VFB) as given in eq. 3.7

$$VFB = \frac{VMA - VIM}{VMA} \quad \ldots \quad - \quad (3.7)$$

Voids in the mortar (VIMO) as given in eq. 3.8.

$$VIMO = \frac{VIM \times 100}{(100-S)} \quad \ldots \quad .. \quad (3.8)$$

where S = Percentage of coarse aggregates

A typical design calculation is included in appendix A

After calculating these parameters the results were displayed in graphical form as shown in Figures 3.2, 3.3 and 3.4 for asphalt concrete, hot rolled and mastic asphalts respectively. From these graphs, the binder contents corresponding to the crest of each of the graph in the figures were taken. These binder content values were taken and averaged to give the optimum binder content for the asphalts design. The value obtained for each asphalt was then substituted back into the appropriate graphs to obtain the corresponding design parameters.

For the hot rolled asphalt, however, the optimum binder content obtained was used to determine the design bitumen content for the different percentages of coarse aggregates added as can be seen in equation 3.9.

$$\text{design bitumen content} = \frac{\text{Optimum bitumen}(100-S)}{100} + \frac{2.3S}{100} \quad \ldots \quad \ldots \quad \ldots \quad (3.9)$$

where S = percentage of coarse aggregate added.

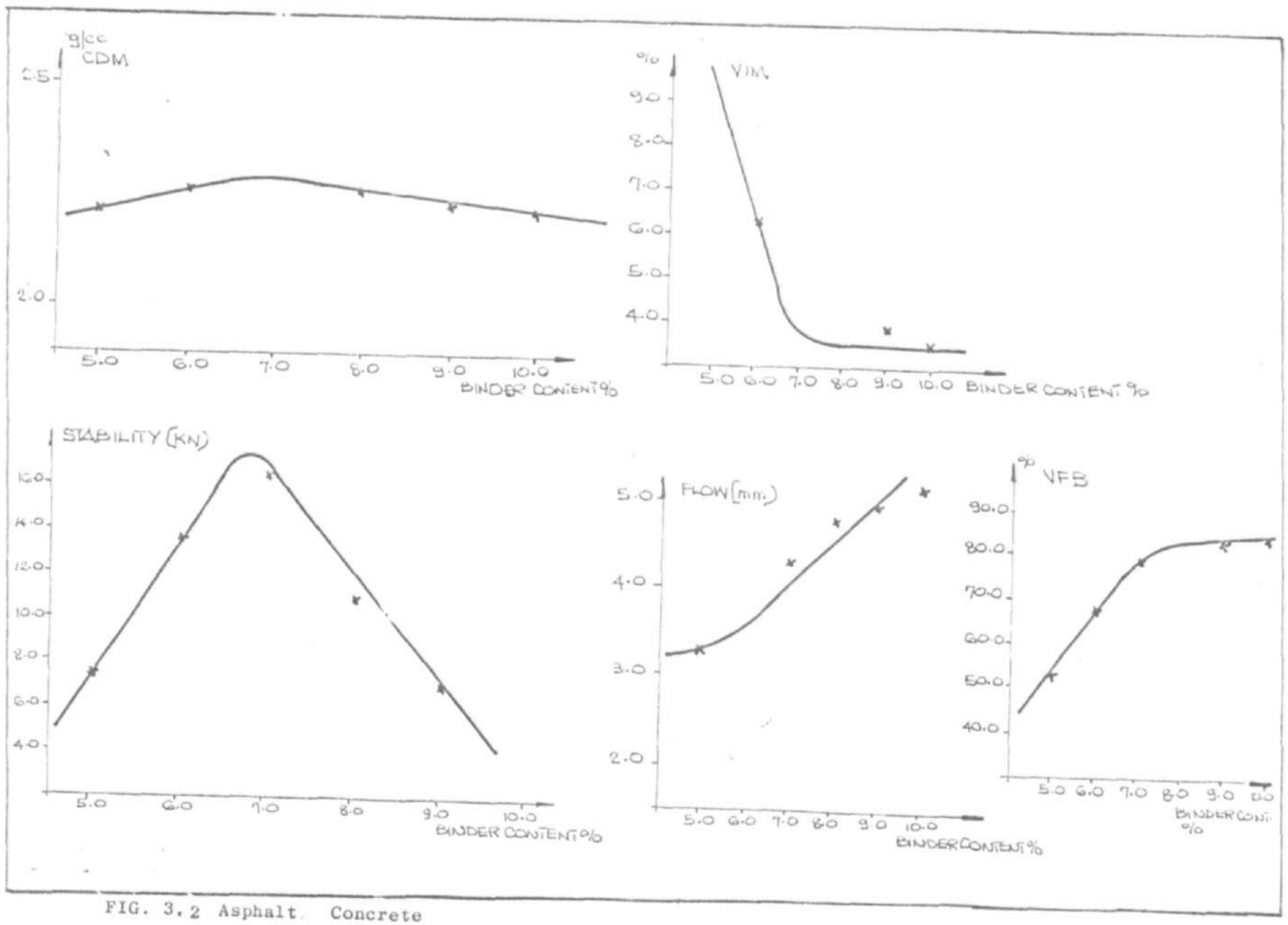


FIG. 3.2 Asphalt Concrete

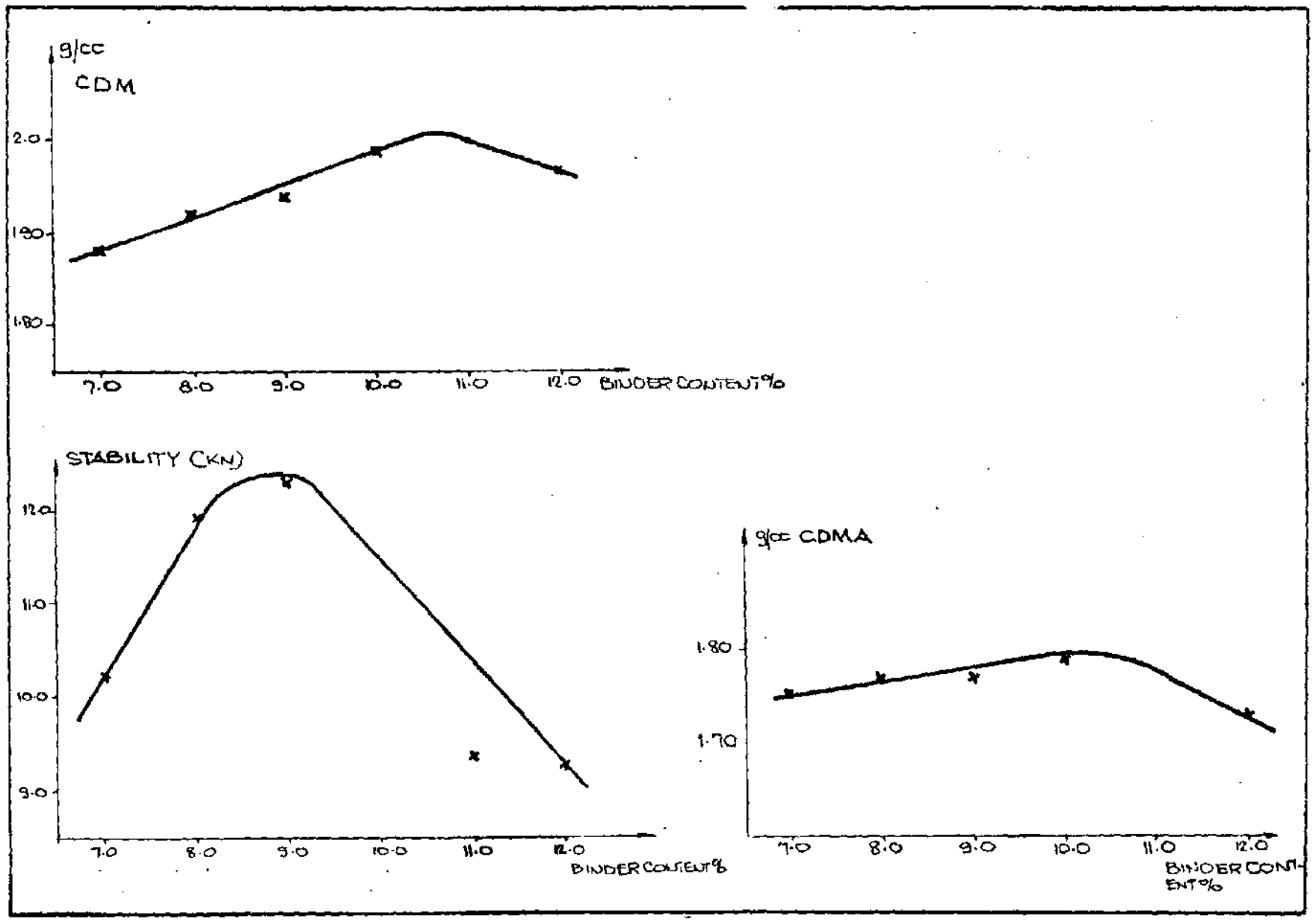


FIG. 3.3: Hot rolled asphalt

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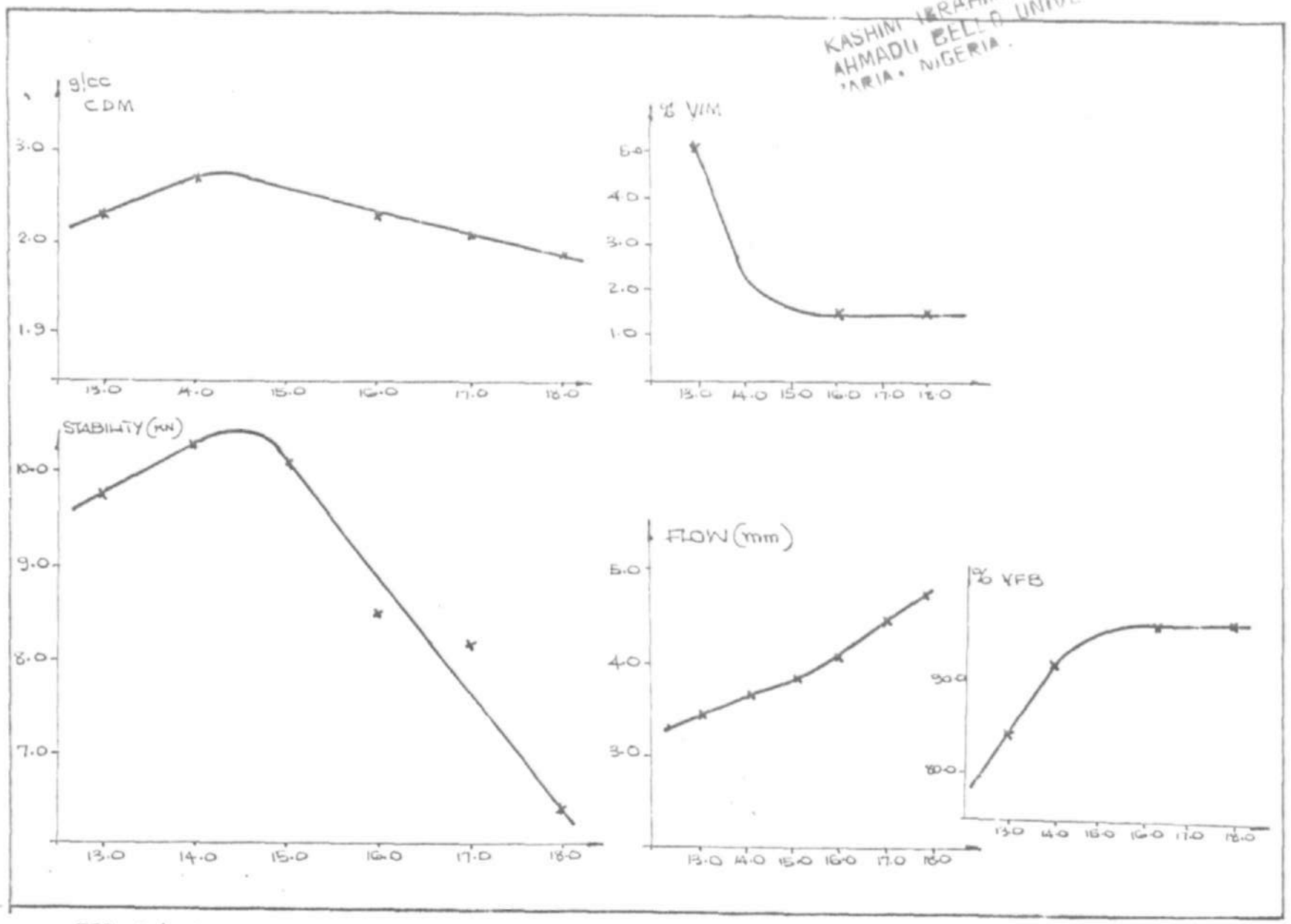


FIG. 3.4 : Mastic Asphalt

The design bitumen content at any coarse aggregate content was used to cast and crush cores and the design parameter compared with the zero coarse aggregate content.

The individual Marshall test specimen results are shown in tables 1, 2a & b and 3 in appendix A.

A typical surface dressing design was done for Northern Nigeria. The steps followed are discussed below.

The design of surface dressing parameters (that is rate of spread of chipping and rate of spread of binder) involves measuring the average least dimension (ALD) of the representative chippings. This is done by measuring the least dimension of about 150 to 200 aggregates and finding the average. The other factors that affect the design parameters of surface dressing are:

a) Traffic category (intensity of traffic)

b) Type of chipping

c) Existing surface

d) Climatic condition

The constants attached to these factors are shown in table 3.6.

Table 3.6 CONSTANTS FOR SURFACE DRESSING DESIGN

(a)

Traffic category	Vehicles/day	Constant
Very light	0 - 100	+3
Light	100 - 500	+1
Medium	500 - 1000	0
Medium heavy	1000 - 3000	-1
Heavy	3000 - 6000	-3
Very Heavy	6000 - and above	-5

Table 3.6 CONSTANTS FOR SURFACE DRESSING DESIGN (contd.)

(b) Type of chipping	Constant
Round/dusty	+2
Cubical	0
Flaky	-2
Pre-coated	-2
(c) Existing Surface	
Untreated/primed surface	+6
Very lean Bituminous	+4
Lean Bituminous	0
Average Bituminous	-1
(d) Climatic conditions	
North Europe (wet + cold)	+2
Tropical (wet +hot)	+1
Temperate	0
Semi-Arid (dry + hot)	-1

Source: Overseas Road Note 2 (2nd Edition) (1985)
"Maintenance techniques for district Engineers"

To determine the rate of spread of chippings, the ALD obtained from measurement, is located on the ALD-axis on Figure 3.5. A horizontal line is then drawn through it to

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Spray Applications

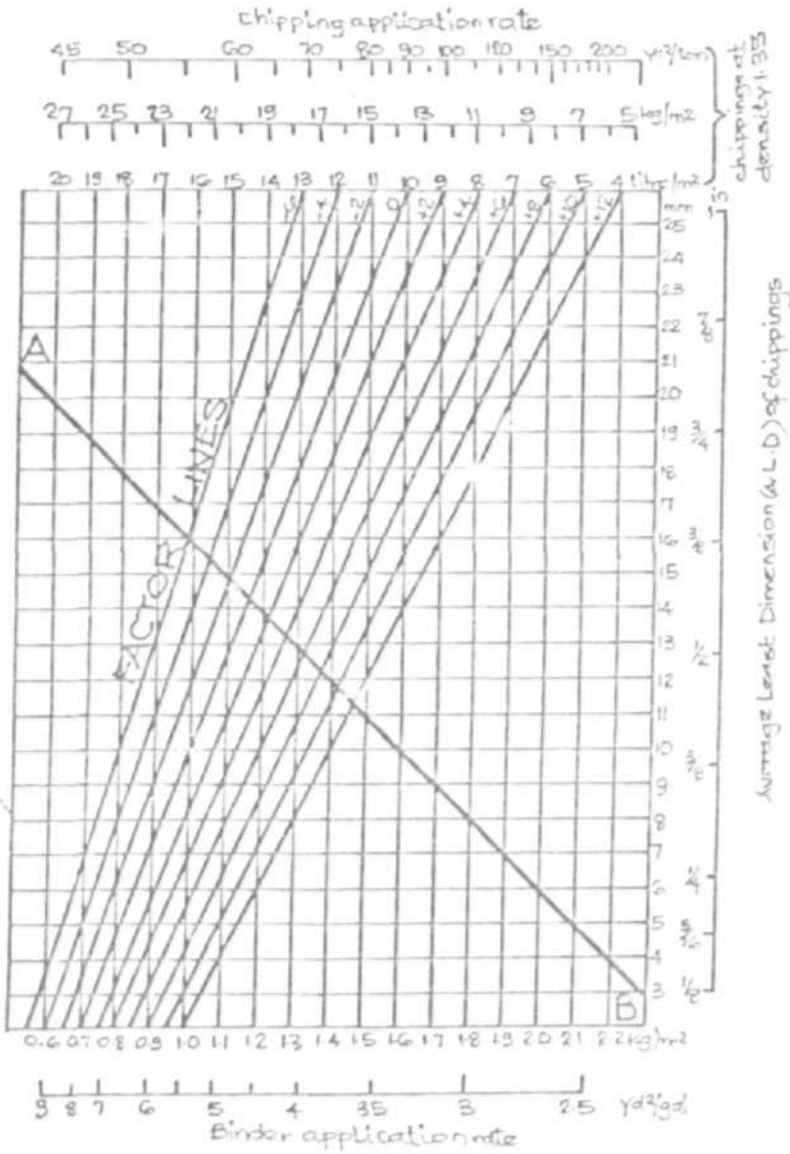


Fig. 3.5: Surface dressing Curves

Source: Overseas Road Note 2 (2nd edition) "Maintenance Techniques for District Engineers (1985)

intersect the inclined line AB. At the point of intersection, a vertical line is drawn upwards to intersect the rate of spread of chipping axis. This new point of intersection gives the appropriate rate of spread of chippings.

To determine the rate of spread of bitumen, the constants produced in Table 2.2 are considered. These are the factors that affect the rate of spread of binder. The traffic category, Type of chipping, Type of surface and location of road are already known so constants appropriate to them are taken from the table and added. The inclined line corresponding to the summed constant is noted. From the previous ALD point, a horizontal line is drawn again to intersect the noted inclined line. A vertical line is produced downwards from the point of intersection to the rate of spread of bitumen axis, the point of intersection gives the required rate of spread of binder.

The assumptions and values obtained are shown overleaf.

SURFACE DRESSING DESIGN

Design factors considered and assumptions made.

Volume of traffic per day - 100 - 500

Type of chippings - Round/Dusty

Existing surface - Untreated

Climatic conditions - Semi-Arid (dry & hot)

Average least dimension of aggregates = 10mm

Constants associated with road

	Constant
a. Traffic category - light	+1
b. Type of chippings - Round/Dusty	+2
c. Existing surface - Untreated/Primed	+6
d. Climatic conditions - Semi-Arid	-1

Total constant required for design = + 8

Rate of spread of bitumen = 1.26litres/m^2

1.26kg/m^2

Application rate of chippings = 13.4kg/m^2

Assume spraying width of bulk bitumen spreader to be 3 meters

and spray-bar output of bitumen 100 litres/min.

Speed of bitumen distributor

$$1.26 = \frac{100 \times 60}{3 \times 1000 \times \text{speed}}$$

$$\text{speed} = \frac{100 \times 60}{1.26 \times 3 \times 1000}$$

= 1.6 km/hr.

3.5 SPECIMEN CHARACTERISTICS

All specimens were 101.6mm (4in) nominally in diameter with height ranging from about 61mm to 75mm.

The mean value, standard deviation and the coefficient of variation of the design parameters are shown in tables 3.7, 3.8 and 3.9 for asphalt concrete, hot rolled and mastic asphalt respectively.

TABLE 3.7 STATISTICAL REDUCTIONS OF TEST RESULTS

Asphalt type	Binder content % wt.	No Specimen	CDM Mean	CDM Standard deviation	CDM C_v %	VIM Mean	VIM Standard deviation	VIM C_v %	VMA Mean	VMA Standard deviation	VMA C_v %	STABILITY Mean	STABILITY Stand-ard dev.	STABILITY C_v %
	5	3	2.22	0.2	9	9.4	0.8	8.5	19.8	0.71	3.6	7.4	4.1	55
Asphalt concrete	6	3	2.27	0.006	0.26	6.2	0.2	3.2	18.6	0.17	0.91	13.6	1.85	13.6
	7	3	2.30	0	0	3.8	0	0	18.3	0	0.0	16.5	1.12	6.7
	8	3	2.23	0.02	0.88	3.8	0.7	1.8	20.2	0.47	2.3	10.9	1.90	17.4
	9	3	2.24	0.006	0.27	3.9	0.17	4.3	21.2	0.17	0.80	7.2	0.78	10.8
	10	3	2.23	0.006	0.27	3.6	0.23	6.3	22.8	0.17	0.75	4.9	1.02	20.8

TABLE 3.8A & B STATISTICAL REDUCTIONS OF TEST RESULTS

Asphalt type	Binder content % wt.	No Specimen	CDM			VIM			VMA			STABILITY		
			Mean g/cc	Standard deviation σ	C_v %	Mean	Standard deviation	C_v %	Mean	Standard dev. σ	C_v	Mean kN	Standard dev. σ	C_v %
	7	2	1.88	0.04	2.1	20	1.83	9.2	33.2	1.63	4.9	10.2	3.4	33.3
	8	"	1.92	0.007	0.36	17.2	0.35	2.0	32.4	0.28	0.86	11.9	0.07	0.59
Sand	9	"	1.94	0.028	1.4	15.3	1.20	7.8	32.4	0.99	3.1	12.3	1.48	12.0
mortar	10	"	1.99	0.021	1.1	12.0	0.92	7.7	31.7	0.71	2.2	11.6	1.56	13.5
	11	"	2.0	0.02	1.0	10.3	0.92	8.9	32.1	0.71	2.2	9.4	1.34	14.3
	12	"	1.97	0.28	1.4	10.9	1.27	11.7	34.0	0.99	2.9	9.3	0.92	9.8
	7.6	3	2.12	0.01	0.47	8.7	0.46	5.3	24.6	0.41	1.7	8.4	1.67	19.8
Rolled	6.86	2	2.20	0.0	0.0	5.2	0.0	0.0	20.6	0.0	0.0	8.4	0.35	42.0
asphalt	6.10	"	2.25	0.0	0.0	44.7	0.0	0.0	18.5	0.0	0.0	9.0	0.28	3.1
	5.72	"	2.26	0.007	0.31	4.6	0.57	12.3	17.5	0.28	1.6	9.1	0.42	4.6

TABLE 3.9 STATISTICAL REDUCTIONS OF TEST RESULTS

Asphalt type	Binder content % wt.	No Specimen	CTM			VIM			VMA			STABILITY		
			Mean G/cc	Standard deviation σ	C_v %	Mean	Standard deviation	C_v %	Mean	Standard dev. σ	C_v	Mean kN	Standard dev. σ	C_v %
Mastic asphalt	13	2	2.03	0.007	0.35	5.4	0.35	6.5	32.3	0.21	0.65	9.5	0.78	8.2
	14	"	2.07	0.0	0.0	2.40	0.0	0.0	31.6	0.0	0.0	10.3	0.14	1.4
	15	"	2.06	0.0	0.0	1.40	0.0	0.0	32.7	0.0	0.0	10.1	0.78	7.7
	16	"	2.03	0.007	0.35	1.6	0.32	20.0	36.1	0.21	0.58	2.5	0.21	2.4
	17	"	2.01	0.007	0.35	1.6	0.35	21.9	37.5	0.21	0.56	8.2	0.64	7.8
	18	"	1.99	0.007	0.35	1.6	0.35	21.9	38.8	0.14	0.36	6.4	0.28	4.3

CHAPTER FOUR

DISCUSSION OF RESULTS

4.0 GENERAL

The Asphalt Institute design criteria for roads is presented in Table 4.1, while the results of the Marshall tests of the designed asphalts are presented in Tables 4.2a & b, 4.3 a and b and 4.4 a & b for asphalt concrete, hot rolled asphalt and mastic asphalt respectively.

TABLE 4.1: ASPHALT INSTITUTE DESIGN CRITERIA - ROADS

Property	Traffic category		
	Heavy	Medium	Light
No of blows	2x75	2x50	2x35
Stability (N)min.	6672	3336	2224
Flow (mm) Max. (0.25mm)	2.0-4.1 8 - 16	2.0-4.6 8 - 18	2.0-5.1 8 - 20
Voids in the mix (VIM) %			
Surface	3 - 5	3-5	3-5
Base-course	3-8	3-8	3-8
Void filled with bitumen (VFB)%	80	80	80
Voids in mineral agg. (VMA)(%)min			
agg. < 10mm	16	16	16
agg. < 14mm	14	14	14
agg. < 20mm	13	13	13

Source: Asphalt Institute (1969)

4.1 OPTIMUM BINDER CONTENT

The optimum binder content obtained for asphalt concrete was 6.9 percent, 13.7 percent was obtained for mastic asphalts while 9.9, 7.6, 6.86, 6.10 and 5.72 percents were obtained for hot rolled asphalt. The values are quoted in the order of quantity of coarse aggregate content present in the mortar of hot rolled asphalt, namely 0, 20, 40, 50 and 55 percent respectively.

The values obtained are indicative of the percentage voids in the various mixes. Asphalt concrete has the smallest voids followed by hot rolled and then mastic asphalt. The reduction in the bitumen content obtained with higher percentages of coarse aggregate, in the mix of hot rolled asphalt is due to the fact that as the coarse aggregate content increases, the voids reduce and hence a corresponding reduction in bitumen content, since bitumen acts as a voids filler while lubricating and binding the aggregates together.

4.2 MARSHALL STABILITY

The stability value obtained for asphalt concrete was 17.2kN. This value when compared with 8.34kN in table 4.1 is very satisfactory. The value got for mastic asphalt was 10.20kN, hot rolled asphalt had 11.6kN, 8.4kN, 8.4kN, 9.0kN and 9.1kN as its recorded stabilities. All these value are far above the minimum recommended. Asphalt concrete because of its

dependence on aggregate interlock for strength is expected to have a higher stability than the other two. Mastic and hot rolled asphalts depend on a strong and stiff mortar for their strength hence their lower stabilities. The initial reduction in the measured stability of hot rolled asphalt with increase in coarse aggregate content is due to the reduction in the quantity of mortar as coarse aggregate increases. The resistance to deformation of hot rolled asphalt is mainly the resistance to deformation of the mortar. However, as coarse aggregates are added, the mixture starts enjoying some aggregate interlock hence the increase at even higher coarse aggregate contents.

The one over-riding property of asphalts is the Marshall quotient (stiffness). This is given as the ratio of stability to flow. This parameter is used to compare the relative resistance to deformation by different mixes. Asphalt Institute (1969) has suggested that the stiffness must be greater than 1.2 times the highest tyre pressure expected. The stiffness of the designed

TABLE 4.2 a &b DESIGN RESULTS FOR ASPHALT CONCRETE

Factor	Binder Content (%)
Unit weight	6.9
Stability	6.8
4% VIM	6.9
80% VFB	7.0

(a) Optimum Binder content for different factors.

Property	Value
Stability (kN)	17.2
Flow(mm)	4.0
VIM (%)	3.8
VFB (%)	78

(b) Test results at overall optimum binder content.

Overall optimum binder content(%)

$$= \frac{6.9 \times 6.8 + 6.9 + 7.0}{4}$$

$$= 6.9\%$$

Marshall quotient (stiffness)

$$= \frac{17.2}{4.0}$$

$$= 4300\ kNm^{-1}$$

asphalt concrete was 4300 kNm^{-1} as compared to the Asphalt Institute's recommended value of 656.5kNm^{-1}. The values for hot rolled asphalt were 2168kNm^{-1}, 3575kNm^{-1}, 2903kNm^{-1} and 2600 kNm^{-1}, these values showed increment of 29, 65, 33 and 20 percents over the zero percent coarse aggregate mixture. Mastic asphalt recorded 2833 kNm^{-1}. All these values are above the minimum recommended. It also showed that asphalt concrete resists deformation more than hot rolled and mastic asphalts.

Experience has shown, however, that asphalt concrete, despite having a high resistance to deformation, is prone to cracking and -difficult to compact. It is now recognised that even for high temperature service conditions, high stability is not all-important. Rather attention be paid to designing mixes to the properties of flexibility, durability and workability.

4.3 Voids in Mineral aggregates (VMA)

This parameter is directly related to the quantity of bitumen needed as high bitumen content usually correspond to high VMA. On the other hand, the larger the aggregates the smaller the quantity of bitumen required. In this instance VMA value obtained for asphaltic concrete was 20•20 percent. The values were 32.1, 24.6,20.6, 18.5 and 17.8 percent for

bot rolled asphalt while mastic asphalt gave 33.8 percent. The findings obviously confirm that the higher the VMA, the higher the bitumen content. Table 4.1 gives the values below which if the VMA of a mix falls, it will be deficient in bitumen or air voids. For the largest size of aggregate (passing 14mm sieve) the value is 14% which is well below all the values obtained. The importance of VMA is to ensure that there is enough room in the thoroughly compacted mixture to allow for some compaction under traffic and to check for crack and brittleness in early life as the surface may ravel. It is generally taken that the durability correlates highly with VMA values. Going from the VMA values obtained, it follows that mastic and hot rolled asphalts are more durable than the asphalt concrete designed.

TABLE 4.3a: Design results for hot rolled asphalt

Factor	Binder content(%)
Unit weight	10.7
Stability (KN)	8.9
CDMA	10.2

(a) *Optimum binder content for different factors*

Overall optimum Binder content

$$= \frac{10.7 + 8.9 + 10.2}{3}$$

$$= 9.9\%.$$

TABLE 4.3b: Design results for hot rolled asphalt

Coarse aggregate content (%)	Properly Binder content (%)	Stability (kN)	Flow (mm)	VIM (%)	VFB (%)
0	9.9	11.6	5.4 by Inter-polation	12.0 by inter-polation	62.2
30	7.6	8.4	3.30	8.4	65.9
40	6.86	8.4	2.35	5.2	74.8
50	6.10	9.0	3.10	4.7	74.6
55	5.72	9.1	3.5	4.6	74.2

4.4 Voids is the mix (VIM)

As related to VMA, voids in the mix (VIM) also plays very important role in determining a durable mix. The idea is that if the VIM is low, the tendency is for the mix to bleed in service (the aggregates will be buried totally and the surface becomes very slippery when wet). On the other hand, if the voids exceed a desirable maximum, the surface will not be durable as stripping will occur as well as water will seep through into the sub-base or subgrade thereby causing failure of the pavement. Table 4.1 shows a range of between 3 and 5 percent with the higher boundary aimed at when designing for a heavier trafficked road. The values obtained for the asphalts were 4 percent for asphalt concrete, 12.0, 8.4, 5.2

4.7 and 4.6% for hot rolled asphalt while mastic gave 3.0%. Though BS594 leaves the percentage coarse aggregate content to be added to the sand mortar to choice, a method proposed by D. Brien stipulates that another parameter, voids is the mortar (VIMO) be used to choose the percentage. He specified VIMO of not more than 9.0%. The VIMO (which is determined by the ratio of the voids in the mix (VIM) to the value obtained by subtracting the coarse aggregate content from hundred) obtained were 12, 12, 8.7, 9.4 and 9.2 percent. Going by D. Brien, the ideal mix should have a coarse aggregate content of 40%. The VIM for 40% coarse aggregate was 5.2% which is very close to the upper limit given in Table 4.1. Mastic asphalt though supposed to be voidless had 3.0% which is okay.

TABLE 4.4: DESIGN RESULTS FOR MASTIC ASPHALT

Factor	Binder Content(%)
Unit wt	14.3
Stability	14.5
4%VIM	13.5
80% VFB	12.4

(a) Optimum binder content for different factors

Overall Optimum Binder content

$$= \frac{14.3+14.5+13.4+12.4}{4}$$

$$= 13.7\%$$

Property	Value
Stability(kN)	10.2
Flow (mm)	3.6
VIM (%)	3.0
VFB(%)	82.0

(b) Test results at overall Optimum binder content.

Marshall quotient

$$= \frac{10.3}{3.0}$$

$$= 3.433 \text{ kNmm}^{-1}$$

4.5 Compacted density of the mix (CDM)

This parameter is used as a guide during construction. During laying operation, the compacted or laid asphalt must have a relative density of about 95% of the Laboratory mix design. The value obtained for asphaltic concrete was 2300kg/m^3, the value for hot rolled asphalt was calculated to be 2130kg/m^3, 2120kg/m^3, 2200kg/m^3, 2250kg/m^3 and 2260kg/m^3. Mastic asphalt's CDM was 2030kg/m^3. The very high value obtained for asphalt concrete shows that it is the most densely packed of the three asphalts. It also means that more compactive effort is needed for asphalt concrete in the field. Mastic asphalt does not lend itself to mechanization, it has to be laid by trowelling.

Another property of the CDM is in estimation. It comes very handy when estimates of material are being made. Since asphaltic concrete is the densest of all designed asphalts, it will require more materials during construction.

4.6 Flow Value

The flow relates to the stability as the amount of deformation at failure. The importance of this property is to guide against brittleness. If the stability is high and flow value low, the tendency is for the the mixture to be brittle (i.e. high Marshall quotient) so permitting the seepage of water into the pavement materials thereby causing failure. On the other hand if the flow is high and stability is low, the mix will deform easily under traffic (shoving).

The flow values obtained were 4.0mm for asphalt - concrete, 5.4mm, 3.30mm, 2.35mm, 3.10mm and 3.5mm for hot rolled asphalt while 3.0 was obtained for mastic asphalt. The drop in flow value recorded for hot rolled asphalt is mainly due to the reduction in mortar quantity as the quantity of coarse aggregates increased and since deformation of hot rolled asphalt is the deformation of the mortar therefore the drop in flow value with increase in coarse aggregate content.

The values obtained, when compared with that of table 4.1 are within the range specified. The flow value can also be used as an indicator of durability since it is also related to the VMA and VIM.

4.7 Voids filled with bitumen (VFB)

This is a parameter that indicates the deficiency of a mixture in binder content. It is the difference between voids in the mineral aggregate and voids in the mix expressed as a percentage of the former. From table 4.2, it can be seen that the VFB for asphalt concrete was 79% while hot rolled asphalt had values of 62.2, 65.9, 74.8, 74.6 and 74.2% and mastic had 82 percent. Though Asphalt institute specifies 80% VFB, BS594 is silent on it. It is expected that since higher VMA correlates higher binder content and therefore higher VFB, it is possible that the high value of VMA coupled with an equally high value of VIM is responsible for the low value calculated for hot rolled asphalt. The 82 percent obtained for mastic asphalt is expected since the mixture

is supposed to be voidless.

4.8 Superimposed graphs

On the superimposed graphs of figure 4.1a, 4.2a and 4.3a representing the parameters CDM, CDMA and SGM of hot rolled asphalt on asphalt concrete, it will be observed that the hot rolled asphalt graphs are below those of asphalt concrete. This clearly shows that asphalt concrete is more densely graded and require more compactive efforts than hot rolled asphalt.

The shapes of the graphs in fig. 4.1b, 4.2b and 4.3b for hot rolled asphalt are directly opposite to that of asphalt concrete, this is because the points plotted for hot rolled asphalt correspond to different coarse aggregate contents therefore different bitumen content. There is however, a coarse aggregate content for this project between 40 and 50% that the design parameters are same. This percentage is approximately 46.7 and binder content 6.35%.

It can also be seen in fig 4.1a that increments in coarse aggregate content had very little effect on the CDM of hot rolled asphalt. It means that hot rolled asphalt is more tolerant of mix variation than asphalt concrete.

4.9 Surface dressing

In designing a typical surface dressing road for Northern Nigeria, as shown in chapter 3, the rate of spread of bitumen was determined to be 1.26 litres/m^2. Rate of spread of

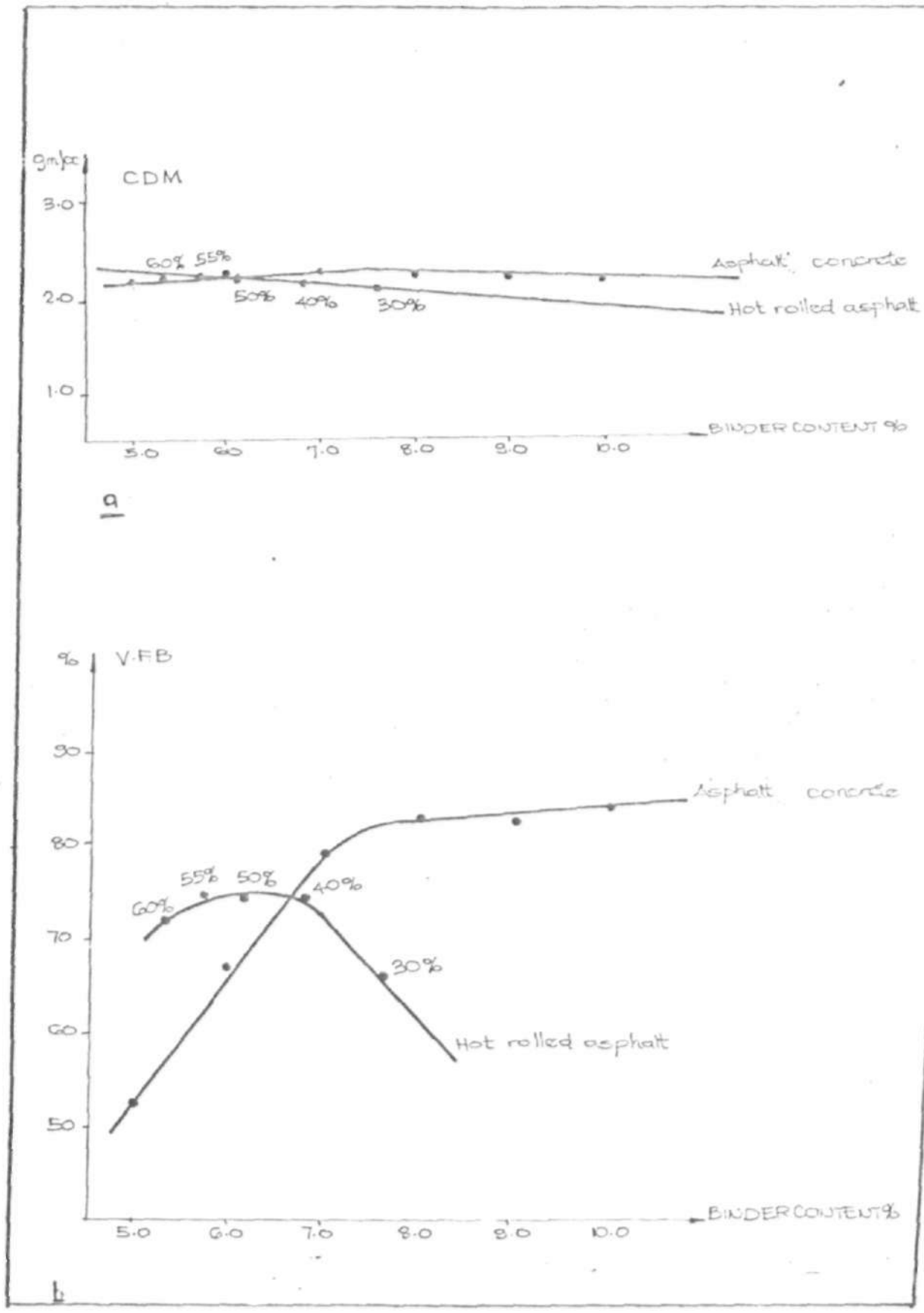
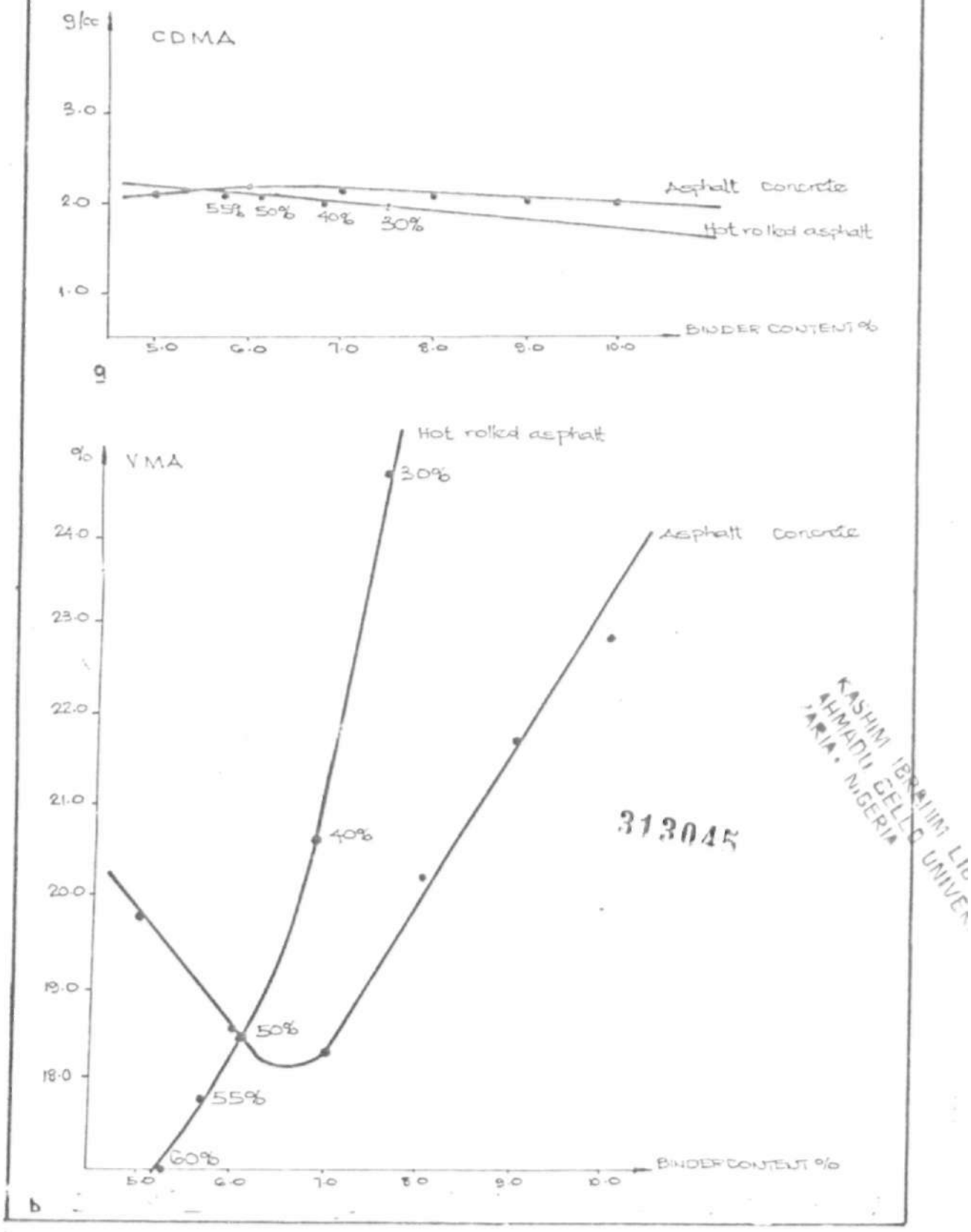


Fig. 4.1: Superimposed graph of hot rolled asphalt on asphalt concrete



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FIG. 4.2: Superimposed graphof hot rolled asphalt on asphalt concrete.

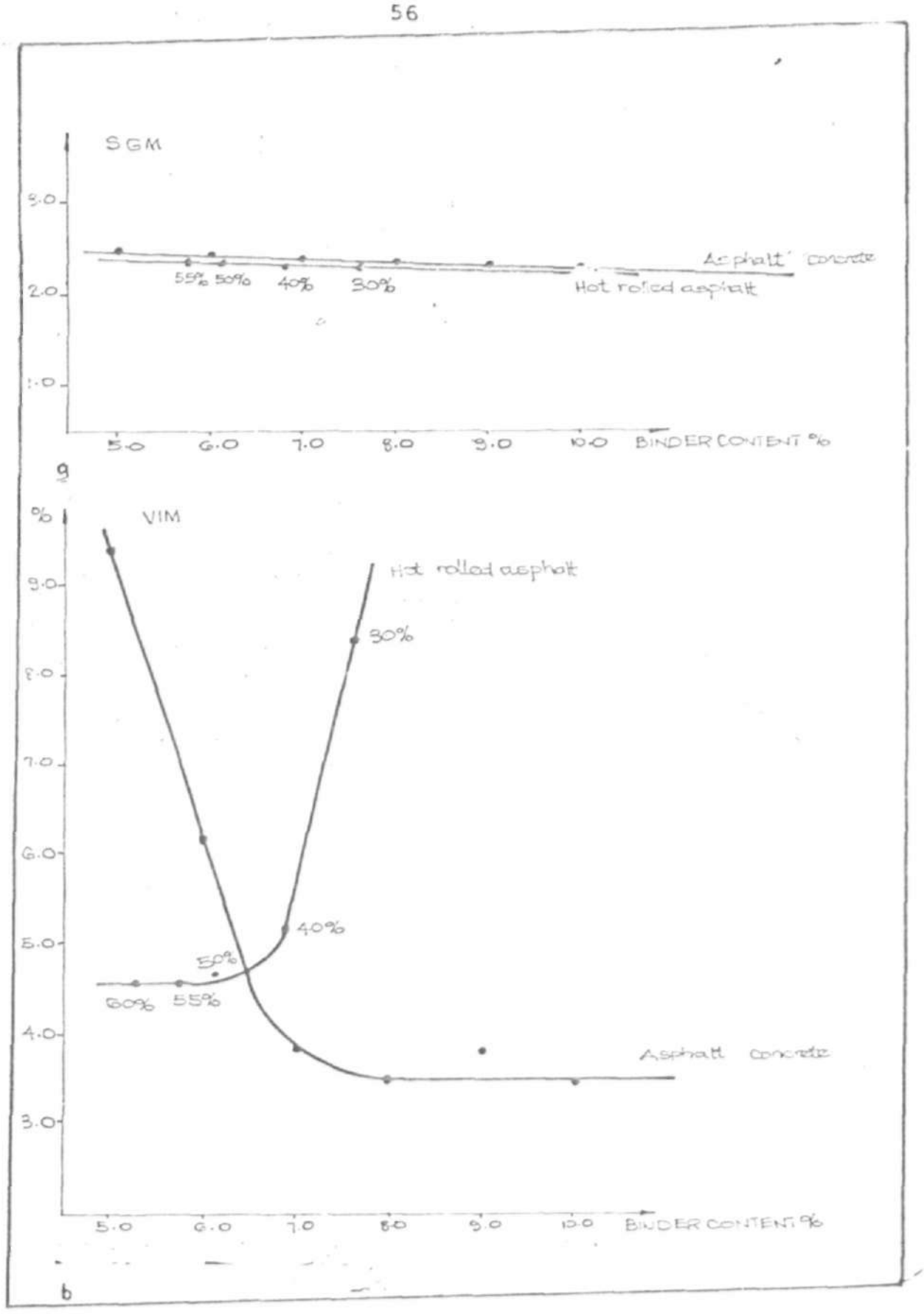


Fig. 4.3: Superimposed graphs of hot rolled asphalt on asphalt Concrete.

chipping was 13.4kg/m^2. The speed of the bitumen distributor was also calculated to be 1.6km/hr.

The most important aspect in surface dressing design is the rate of spread of bitumen. If the rate is too low, there will be scabbing. On the other hand if the rate is too high, there will be fatting up or bleeding of the bitumen. The idea, therefore, is to have a rate that when applied will just coat the aggregates and also be enough to hold the aggregates in position. The bitumen must also be fluid enough to wet the surface of the road.

Some of the above dificiencies can be minimise by techniques like the pretreatment of the chippings with light coating of tar, creosote or diesel to suppress dust and to aid adhesion. Deficiencies in bitumen distribution are virtually impossible to counter, though they can be reduced by the application of a light 'fog-spray' of bitumen and a sprinkling of sand or rock dust after the chippings have been spread on the bitumen film.

4.11 Cost Estimates

In preparing the cost estimates for the asphalts, a length of road of 1km was taken with width of 7.3meter and the thickness of asphalt of 0.05 meters. The same was assumed for surface dressing with double surfacing.. The costs were deterimined as follows:

1. Asphalt concrete

Description	Quantity	Unit	Rate ₦ k	Amount ₦ k
Coarse aggregates	146.0	m^3	34.30	5007.80
Fine aggregates	131.8	"	14.28	1882.10
Filler(cement)	70.35	Tonne	800.00	56280.00
Bitumen	57.93	"	265.00	15351.45
				₦78521.35

Labour, profit, overheads and plant and equipment = 70% of cost of materials

Cost of laying = 1.7 x 78521.35

= 133486.30

Rate per m^3 of asphalt = $\frac{133486.30}{365}$

= ₦365.70

2. Hot rolled asphalt

Description	Quantity	Unit	Rate	Amount
Coarse aggregates	116.4	m^3	₦34.30	₦3992.52
Fine aggregate s	150.75	"	14.28	2152.71
Filler (cement)	64.33	tonne	800.00	51464.00
Bitumen	55.08	tonne	265.00	14596.20
				₦72205.43

Cost of laying = 1.7 x 72205.43

= 122749.23

Rate per m^3 of asphalt = $\frac{122749.23}{365}$

≈ ₦336.30

3. *Surface dressing*

Description	Quantity	Unit	Rate ₦ k	Amount ₦ k
Prime coat				
bitumen (mc 30)	5.84	tonnes	730.00	4263. 20
sand	38.36	m^3	14.20	547. 78
Surface dressing				
S125	18.40	tonnes	730.00	13432.00
Coarse aggregates	76.12	m^3	34.30	2610.92
				₦20853.90

Cost of laying = 1.70 x 20853.90

= ₦35450.10

Rate per m^2 = $\frac{35410.10}{7300}$

= ₦4.86

Going by this project, the cost of laying a m^3 of asphaltic concrete is ₦365.71 as against ₦336.30 for hot rolled asphalt. Hot rolled asphalt is more economical than asphaltic concrete. It will cost ₦4.86 to lay a m^2 of surface dressing. The complete calculation is included in the Appendix B.

4.11 Variability of Results

Variations in measured values were calculated (Table 3.7, 3.8a & b and 3.9) by estimating values of standard deviation σ

coefficient of variation C_v and mean of compacted density of the mix (CDM), stability, voids in the mix (VIM) and void in the mineral aggregate (VMA).

The C_v ranged from 0 to 9.0%, 0 to 2.1% and 0 to 35% for the CDM for asphaltic concrete, hot rolled and mastic asphalts respectively. The VIM ranged from 0 to 8.5, 0 0 to 11.7 and 0 to 21.9%. The stability and VMA values ranged from 6.7 to 55%, 0.59 to 33.3% and 1.4 to 8.2% then 0 to 3.6, 0 to 4.9 and 0 to 0.65 percent in the order earlier enumerated.

The C_v values are very much in order, considering the inherent variations usually associated with asphalt mixture properties. Most previous investigators including Schmidt (1972), Marshall and Kennedy (1974) have reported similar and even higher C_v values for various asphalt mixture design parameters.

CHAPTER FIVE

5.0 CONCLUSIONS AND RECOMMENDATIONS

5.1 CONCLUSIONS

From the results discussed and analysed in chapter 4, asphalt concrete was found to withstand stability more than hot rolled and mastic asphalts, as evidenced by its very high stability of 17.2kN. This was also correlated by the Marshall quotient obtained for it. Its value was 4300kNm^{-1} as compared with the highest for hot rolled asphalt of 3575kNm^{-1} and mastic asphalt's 2833kNm^{-1}. It has been reported, though, that high stability values are not the all-important criteria in the design of asphalt surfacings. They have to meet other criteria like durability, flexibility and workability.

The flow value, apart from being related to stability plays very important roles in the life of the asphalt in service. It relates very much to the quantity of bitumen in the mixture. If the bitumen content is high, the flow value is equally high. An asphalt with low flow value and high stability will not deform easily but will be brittle, while that with low stability and high flow will shove easily under-traffic. The results of 3.06mm, 3.0mm and 4.0mm obtained for hot rolled asphalt, mastic asphalt and asphalt concrete respectively showed that hot rolled and mastic asphalts will not shove as easily as asphalt concrete.

The voids in the mix is critical as regards durability. The closer the VIM is to the upper limit specified by standards, the more likely it is for the mixture to be durable, while the closer it is to the lower limit, the more the tendency for bleeding to occur. The values obtained from this project showed that hot rolled asphalt with 40% coarse aggregate content will be more durable considering the level of uncertainty associated with vehicular loading in developing countries. The values were 5.2% for hot rolled asphalt, 4.0% for asphalt concrete and 3.0% for mastic asphalt.

The voids in the mineral aggregate is also related to durability. High VMA values correlate sufficiency in binder content and air voids, hence durability. The larger the aggregates, the smaller the VMA. For the surfacings designed, the maximum size of aggregates were 14mm ($\frac{1}{2}$in). The results calculated were 20.20%, 20.60% and 34.2% for asphaltic concrete, hot rolled and mastic asphalts respectively. It followed that mastic was the most durable and asphaltic concrete the least.

The compacted density of the mix which is used as a control measure during laying and compaction was lowest for mastic asphalt. The values obtained were 1990kg/m^3 for mastic asphalt, 2207 kg/m^3 and 2300kg/m^3 for hot rolled asphalt and asphalt concrete respectively. It showed that mastic and hot rolled asphalts were more workable and flexible than asphalt concrete since they required less compactive efforts.

From the superimposed graphs, it was also evident that hot rolled asphalt required lesser compactive efforts as the graphs of hot rolled asphalt were below those of - asphalt concrete. There is a coarse aggregate content of 46.7% and binder content of 6.35% that the design parameters of the designed hot rolled asphalt and asphalt concrete are the same.

Going by the method proposed by D. Brien, the 40% coarse aggregate content should be chosen as the appropriate mix for the designed hot rolled asphalt. The improvement of this coarse aggregate content parameters over that of sand mortar were obvious. It improved the Marshall quotient from 2168 kNm^{-1} to 3375kNm^{-1}, the highest of all percentage coarse aggregates. The VIMO was improved from 12.8% to 8.7, the only one below the 9.0% suggested. The VIM was also lowered to 5.2% from 12.0%. It also gave the highest VFB of 74.8 percent from 62.2 percent for sand mortar.

It was estimated that it would cost ₦365.71 to lay a m^3 of asphalt concrete as against ₦336.30 for hot rolled asphalt.

5.2 RECOMMENDATIONS

(1) From the superimposed graphs, the CDM of hot rolled asphalt was found to vary slightly with increase in coarse aggregate content. It means that it is more tolerant of manufacturing variations. One problem of asphalt concrete is its sensitivity to manufacturing variation and hence disparity in wearing surface texture and subsequent failure of the black top. To arrest this failure, hot rolled aspahlt should be used.

(2) Dense bituminous surfacings depend on compaction for their strength and durability. Asphalt concrete is difficult to compact therefore its durability is suspect. Hot rolled asphalt by virtue of its lower CDM is easier to compact and hence likely to be more durable than asphalt concrete.

(3) The cost estimate of manufacturing and laying made showed that it will be cheaper to use hot rolled asphalt as against asphalt concrete. Whether it will be cheaper to maintain, this thesis did not look into it but literatures say it is dependable.

(4). The method proposed for designing hot rolled asphalt by D. Brien should be adopted because it gives the designer the opportunity of knowing the effects of adding different percentages of coarse aggregates on the parameters of the sand mortar. It also enables a wide variety of materials not covered by BS 594 to be used.

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APPENDIX A

TYPICAL CALCULATIONS OF ASPHALT PARAMETERS

Raw data: Binder content 7% from table 2a

wt in air,	wt in H_2O	Vol. of sample
1) 1165gm	555gm	610cc
2) 1162gm	535gm	627cc

1) $CDM = \frac{\text{wt in air}}{\text{wt in air-wt in } H_2O} = \frac{1165}{610} = 1.9$

2) $= \frac{1162}{627} = 1.85$

Average CDM $= \frac{1.85 + 1.91}{2} = 1.88$

CDMA $= 1.88 \times (1 - \frac{7}{100})$

$= 1.75$

$$SGMA = \frac{100}{\frac{86}{2.55} + \frac{14}{3.1}}$$

$$= \frac{100}{33.73 + 4.516}$$

$= 2.62$

$$SGM = \frac{100 + 7}{\frac{100}{2.62} + \frac{7}{0.95}} = \frac{107}{38.168 + 7.368}$$

$= 2.35$

Volids in the Mix (VIM) $= \frac{2.35 - 1.83}{2.35}$

$= 20\%$

Voids in the mineral aggregate (VMA $= \frac{2.62 - 1.75}{2.62}$

$= 33.2\%$

Voids filled with bitumen $= \frac{33.2 - 20}{33.2}$

$= 39.8\%$

TABLE 1

ASPHALTIC CONCRETE

SPECIMEN NO.	Wt (g) in air	Wt (g) in H_2O	Vol	OBV	BITUMEN %	SGV	Vbm	GMA	SGMA	VMA	VFB	STABILITY (kN)	CORR FACTOR	ADJUSTED STAB.	FLOW (mm)	DIAL READING	Marshal $\frac{S}{F}$
1	1190	651	539	2.21			9.8	2		20		5.3	0.93	4.9	3.10	118	
2	1179	653	526	2.24			8.6			19.1		12.6	0.96	12.1	3.90	283	
3	1152	629	523	2.20			10.2			20.5		5.3	0.96	5.1	3.00	118	
			AV.	2.22	5.0	2.45	9.4	2.11	2.63	19.8	52.5			7.4	3.30		2.29
4	1160	650	510	2.27			6.2			18.9		15.0	1.00	15.0	3.00	334	
5	1138	636	502	2.26			6.6			19.2		11.4	1.04	11.9	3.60	253	
6	1156	647	509	2.27			6.2			18.9		13.8	1.00	13.8	4.10	308	
			AV.	2.27	6.0	2.42	6.2	2.14	2.63	19.0	66.7			13.6	3.57		3.81
7	1148	642	499	2.30			3.8					17.0	1.04	17.7	4.30	384	
8	1187	670	517	2.30			3.8					16.2	1.00	16.2	4.30	376	
9	1160	655	505	2.30			3.8					14.9	1.04	15.5	4.20	331	
			AV.	2.30	7.0	2.39	3.8	2.15	2.63	18.3	79.2			16.5	4.30		3.84
10	1143	641	502	2.27			3.8			20.6		12.6	1.04	13.1	5.40	281	
11	1110	624	486	2.28			3.4			20.4		8.3	1.09	9.7	4.80	185	
12	1150	640	510	2.25			4.2			21.3		9.9	1.00	9.9	4.30	221	
			AV	2.27	8.0	2.36	3.8	2.10	2.63	20.8	82.7			10.9	4.80		2.27
13	1140	629	511	2.24			3.9			22.5		8.1	1.00	8.1	4.40	178	
14	1158	639	519	2.23			4.2			22.2		0.0	1.00	7.0	5.50	153	
15	1185	657	528	2.24			3.9			22.5		6.9	0.96	6.6	4.90	152	
			AV	2.24	9.0	2.33	3.9	2.06	2.63	22.4	82.5			7.2	4.93		1.46
16	1150	634	516	2.23			3.5			23.7		5.10	1.00	5.1	4.70	113	
17	1148	631	517	2.22			3.9			24.0		3.8	1.00	3.8	4.90	84	
18	1172	646	526	2.23			3.5			23.7		6.0	0.96	5.8	5.80	132	
			AV.	2.23	10.0	2.31	3.6	2.03	2.63	22.8	84.6			4.9	5.13		0.96

TABLE 2a

SAND MORTAR

SPECIMEN NO.	Wt in Air (gm)	Wt in H_2O (gm)	Vol(CC)	CDM	SPECIMEN NO.	SGM	VIM%	CDMA	SGMA	VMA%	VFB%	STABILITY: DIAL GAUGE	STABILITY: KN	STABILITY: CORRECTION FACTOR	STABILITY: ADJUSTED STABILITY	FLOW (mm)	Qm
1	1165	555	610	1.91						32.2		360	16.1	0.78	12.6	4.80	
2	1162	535	627	1.85						34.5		228	10.3	0.76	7.8	4.60	
			AV	1.88	7	2.35	20.0	1.75	2.62	33.4	39.8				10.2	4.70	2.17
3	1165	558	607	1.92						32.5		337	15.1	0.78	11.8	5.70	
4	1148	548	600	1.91						32.9		342	15.3	0.78	11.9	4.50	
			AV	1.92	8	2.32	17.5	1.77	2.62	32.4	46.9				11.9	5.10	2.33
5	1145	550	595	1.92						33.3		307	13.8	0.81	11.2	5.40	
6	1148	563	585	1.96						31.9		359	16.0	0.83	13.3	5.20	
			AV	1.94	9	2.29	15.3	1.77	2.62	32.4	52.8				12.3	5.30	2.32
7	1135	560	575	1.97						32.3		282	12.7	0.83	10.5	5.70	
8	1142	570	572	2.0						31.3		329	14.8	0.86	12.7	5.00	
			AV	1.99	10	2.26	12.2	1.79	2.62	31.7	62.2				11.6	5.35	2.16
9	1154	580	574	2.01						31.7		275	12.4	0.83	10.3	5.50	
10	1150	570	580	1.98						32.7		225	10.1	0.83	8.4	5.10	
			AV	2.0	11	2.23	10.5	1.78	2.62	32.1	67.9				9.4	5.30	1.77
11	1129	550	579	1.95						34.5		228	10.3	0.83	8.6	5.20	
12	1115	555	560	1.99						33.1		256	11.5	0.86	9.9	5.75	
			AV	1.97	12	2.21	10.9	1.73	2.62	34.0	67.9				9.3	5.45	1.70

TABLE 2b

HOT ROLLED ASPHALT

SPECIMEN NO	Wt in Air (gm)	Wt in H_2O (gm)	Vol(CC)	GSM	BINDER %	SGM	VIM%	CIMA	SGMA	VMA%	VFB%	STABILITY				FLOW (mm)	Qm
												DIAL GAUGE	KN	CORRECTION FACTOR	ADJUSTED STABILITY		
13	1174	625	549	2.13						24.3		234	10.5	0.89	9.4	3.1	
14	1169	605	544	2.11						25.0		156	7.03	0.93	6.5	2.9	
15	1179	625	554	2.13						24.3		236	10.6	0.89	9.4	3.8	
			AV	2.12	2.12	7.6	2.32	8.7	1.96	1.60	24.6	65.9			8.4	3.3	2.55
16	1144	625	519	2.20								189	8.1	1.00	8.1	2.40	
17	1175	640	535	2.20								192		0.96	8.6	2.30	
			AV	2.20	6.86	2.32	5.2	2.04	2.57	20.6	74.8				8.4	2.35	3.58
18	1165	640	517	2.25								212	9.6	0.96	9.2	2.90	
19	1165	648	517	2.25								203	9.2	0.96	8.8	3.30	
			AV	2.25	6.10	2.36	4.7	2.11	2.59	18.5	74.6				9.0	3.10	2.9
20	1168	650	518	2.26						17.7		196	8.8	1.00	8.8	3.80	
21	1165	647	518	2.25						18.1		207	9.4	1.00	9.4	3.20	
			AV	2.26	5.72	2.37	4.6	2.13	2.59	17.8	74.2				9.1	3.5	2.6

MASTIC ASPHALT

TABLE 3

SPECIMEN NO	Wt in Air (gm)	Wt in H_2O	Vol(CC)	GBM	BINDER %	SGM	VIM%	GMA	SGMA	VMA %	VFB %	STABILITY DIAL GUAGE	STABILITY KN	STABILITY CORRECTED FACTOR	STABILITY ADJUSTED STABILITY	FLOW (mm)	%
1	1193	605	588	2.03		.						274	12.3	0.81	10.0	3.50	
2	1180	597	583	2.02								238	10.7	0.83	8.9	3.30	
			AV	2.03	13	2.14	5.1	1.80	2.66	33.8	84				9.5	3.40	2.79
3	1136	588	548	2.07								260	11.7	0.89	10.4	3.90	
4	1162	601	561	2.07								265	11.9	0.86	10.2	3.20	
			AV	2.07	14	2.12	2.4	1.82	2.66	31.6	92				10.3	3.6	2.56
5	1152	593	559	2.06								244	11.0	0.86	9.5	3.50	
6	1142	588	554	2.06								275	12.3	0.86	10.6	3.90	
			AV	2.06	15	2.09	1.4	1.79	2.66	32.7	96				10.1	3.8	2.66
7	1155	584	571	2.02								222	10	0.86	8.6	3.95	
8	1145	581	564	2.03								212	9.6	0.86	8.3	4.10	
			AV	2.03	16	2.06	1.5	1.75	2.66	36.1	96				8.5	4.00	2.13
9	1075	541	534	2.01								202	9.2	0.93	8.6	4.20	
10	1070	535	535	2.00								196	8.8	0.87	7.7	4.60	
			AV	2.01	17	2.04	1.5	1.72	2.66	37.5	95				8.2	4.4	1.86
11	1145	567	578	1.98						38.9		170	7.6	0.81	6.2	4.30	
12	1130	563	567	1.99						38.7		180	8.1	0.81	6.6	5.10	
			AV	1.99	18	2.02	1.5	1.69	2.66	38.8	96				6.4	4.70	1.36

APPENDIX B

COST ESTIMATE OF ASPHALT CONCRETE

Assume a road of the following parameters

Length = 1000meters

Width = 7.3meters

thickness of asphalt = 0.05meters

Quantity of asphalt = 1000 x 7.3 x 0.05

$= 365m^3$

Mass of asphaltic concrete = 365 x 2300 kg

= 839300kg

Since CDM of asphaltic concrete = $2.3gm/cm^3$

But optimum binder content = 6.9%

Weight of bitumen required = 0.069x 839500

= 57925.5kg

Specific gravity of bitumen = 1.03

Volume of bitumen reqd. = $\frac{57925.5}{1030}$

$= 56.24m^3$

= 56240 litres.

From grading, we have

48% coarse aggregate

43% fine aggregate

9% filler material (cement)

Quantity of coarse aggregates = 0.48x 781574.5 kg

= 375155.76kg

Quantity of fine aggregates = 0.43 x 781574.5

= 336077.04kg

Quantity of filler material (cement) = 0.09x781574.5

= 70341.7kg.

A tonne of 80/100 sales ₦265.00

A litre of 80/100 pen. bitumen sells for 27k

56240litres will sell for ₦0.27 x 56240

= ₦15184.8

Vol. of coarse aggregate = 375155.76 x $\frac{1}{2570}$ m^3

= 145.97 m^3 x $\frac{1}{3.5}$

= 42 trips of tipper

A trip of coarse sells for ₦120.00

Cost = ₦120 x 42

= ₦5040.00

Vol. of fine aggregate = 336077.04 x $\frac{1}{2550}$

= 131.8 m^3 x $^{1}/_{3.5}$

= 38 trips

A trip sells for ₦50.00

Cost = 38 x 50 = ₦1900.00

Cement = $\frac{70341.7}{50}$ = 1406.8 bags

= 1407 bags

Cement sells for ₦40.00 per bag

cost = 1407 x 40

= ₦56280.00

Therefore total cost of production = ₦78404.80

Total cost of laying = ₦78404.80 x 1.7

= 133288.16

Unit cost of asphaltic concrete = $\frac{133288.16}{365}$

COST ESTIMATE FOR HOT ROLLED ASPHALT

Volume of asphalt = 365 m^3

CDM of hot rolled asphalt = 2.2gm/cm^3 = 2200kg/m^3

Mass of asphalt = 365 x 2200

= 803000kg

Optimum binder content = 6.86%

Mass of bitumen reqd. = 0.0686 x 803000kg

= 55085.8kg

Specific gravity of bitumen = 0.95

Vol. of bitumen = $\frac{55085.8}{950}$

= 57.990 m^3

= 57985 litres x 0.27

= ₦15657.3

From grading, we have

40% coarse aggregates

51.4% fine aggregates

8.6% filler material (cement)

Quantity of coarse aggregates = 0.4 x 747914.2

= 299165.68kg

Quantity of fine aggregates = 0.514 x 747914.2

= 384427.9kg

Quantity of filler material = 0.086 x 747914.2

= 64330.62kg

A litre of 40/60 pen. bitumen sells for 27k

Cost of bitumen = ₦0.27 x 57990

= ₦15657.30

Vol. of coarse aggregates = $\frac{299165.68}{2570}$ = 116.4m^3

No of trips = $\frac{116.4}{3.6}$ = 34 trips

Cost of coarse aggregates = 34 x ₦120.00

= ₦4080.00

Vol. of fine aggregates = $\frac{384427.9}{2550}$

= $150.75m^3$

No of trips = 43

Cost = 43 x ₦50

= ₦2150.00

No of bags of cement = $\frac{64330.62}{50}$

= 1286.6 = 1287bags

Cost = 1287 x ₦40

=₦51480.00

Total cost of production therefore = ₦73367.30

Total cost of laying = ₦73367.30 x 1.7

= ₦124724.41

Unit cost of Hot rolled asphalt = $\frac{₦124724.41}{365}$

= $₦341.7/m^3$

But a tonne of S125 sells for ₦730.00

Cost of bitumen = 9.198 x 730

= ₦6714.54

Rate of application of chipping = 13.4kg/m^2

Total wt of aggregate = 13.4 x 7300kg

= 97820kg

Specific gravity of chipping = 2.570

Vol. of aggregates = $\frac{97820}{2570}$

= 38.06m^3

No of trips required = $\frac{38.06}{3.5}$ = 10.8 = 11

But a trip of coarse aggregate sells for ₦120.0

Cost = ₦(11 x 120)

=₦1320.00

Total cost for double surface dressing = 2(1320+6714.54)

= ₦16069.08

Total cost of priming and surface dressing

= ₦(16069.08 + 4813.20)

= ₦20882.28

Cost including overhead = 1.7 x ₦20882.28

= 33499.88

Rate therefore = $\frac{35499.88}{7300}$ ₦/m^2

= ₦4.86/m^2

APPENDIX C

TABLE 4

	Asphalt concrete	Hot rolled asphalt	Mastic asphalt
Sieve No	% Passing	% Passing	% Passing
200mm	100		
14mm	87		
10mm	78		
6.3mm	66		
2.36mm	52	100	100
1.18mm	38	-	70
600μm	30	75	45
300μm	22	35	31
150μm	16	-	20
75μm	9	14	13

ADOPTED GRADINGS FOR ASPHALT , HOT ROLLED AND MASTIC ASPHALTS.

SPECIFIC GRAVITY DETERMINATION FOR COARSE AGGREGATE

TEST 1

wt of Empty bottle (W_1) = 615gms

wt of bottle + sample (W_2) = 740gm

wt of bottle + sample+water(W_3) = 1755gm

wt of bottle + water (W_4) = 1675gm

$$S.G. = \frac{W_2 - W_1}{(W_4-W_1)-(W_3-W_2)} = \frac{(740 - 615)}{(1675-615)-(1755- 740)}$$

$$= 2.78$$

TEST 2

W_1 = 615gms

W_2 = 722.5gm

W_3 = 1736.8gm

W_4 = 1675gm

$$S.G = \frac{722.5-615}{(1675-615)-(1736.8-722.5)}$$

$$= 2.35$$

$$\text{Average Specific gravity} = \frac{2.35 + 2.78}{2}$$

$$= 2.57$$

SPECIFIC GRAVITY DETERMINATION FOR FINE AGGREGATES

TEST 1

Wt of bottle (W_1) = 615gm

wt of bottle + sample (W_2) = 765gm

wt of bottle + sample+ water (W_3) = 1765gms

wt of bottle + water (W_4) = 1675gm

$$S.G. = \frac{W_2 - W_1}{(W_4-W_1)(W_3-W_2)} = \frac{765 - 615}{(1675-615)-(1765-765)}$$

S.G = 2.5

TEST 2

W_1 = 598.2gm

W_2 = 674.3gm

W_3 = 1706.9gm

W_4 = 1660.5gm

$$S.G = \frac{674.3 - 598.2}{(1660-615)-(1706.9-674.3)}$$

= 2.59

$$\text{Average specific gravity} = \frac{2.5 + 2.59}{2}$$

= 2.55

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