INFLUENCE OF TEMPERATURE ON ASPHALT CONCRETE IN NIGERIA.

BY

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THESIS

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July/August, 1990

DECLARATION

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

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CERTIFICATION

This thesis entitled "Analysis of Performance of Asphaltic Concrete under the influence of Nigerian Climate", by FARUK, Abdullahi Umar meets the regulations governing the award of the degree of Master of Science (Civil Engineering) of Ahmadu Bello University, Zaria and it is approved for its contribution to knowledge and literary presentation.

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DEDICATION

This work is dedicated to the following people:-

1. Umma, Umar, Hadiza, Ladi, Ahmed, Amina and Haliru, (my parents, brothers and sisters).

2. Alh. Abubakar Zaki Gwandu
   (my maternal grandfather and Guardian).

3. Alh. Abdullahi Dalhatu
   (my boss and motivator).

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ABSTRACT

The Cause of Pavement Failure in Tropical countries like Nigeria and the factors responsible for best-performance asphaltic concrete surfacing are reviewed. Consequently appropriate laboratory tests consisting of both Marshall and Indirect tensile tests were conducted in order to determine the desirable optimum binder content, which maximises all the desirable properties of good-performance asphaltic concrete. To evaluate the effects of each factor, it's influence on the following performance parameters of asphaltic concrete were measured and monitored:- Marshall stability and stiffness, Indirect tensile strength and compressive modulus of the specimens and compacted density of the mix. The factors considered and varied to appropriate levels for this study included the:- Testing temperature, compactive effort, binder content; and Aggregate gradation; shape and surface texture. It was found out that testing temperature which simulate the ambient temperature have an adverse effect on the performance of asphaltic concrete. It was also found out that for Nigerian type of climate and traffic loading a high binder content of optimum value of 7% and stone content of Optimum range of (50-6590 are needed for best performance asphaltic concrete.

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LIST OF SYMBOLS

P	-	design binder content
a	-	width of loaded section
d	-	specimen diameter
E	-	Modulus of Elasticity
E_x	-	Tensile Modulus
E_y	-	Compressive Modulus
LD_H	-	Slope of the least squares regression line between load P, in kg, and the total horizontal deformation X, in mm, for loads up to the maximum (i.e $P/_X$).
LDV	-	Slope of the least squares regression line between P, in kg, and the total vertical deformation Y, in mm, for loads up to the maximum (i.e P/Y).
P	-	Applied load
P_f	-	Load at failure
t	-	Specimen thickness
x, y	-	Cartesian Coordinates.
ν	-	Poisson ratio (= 0.35)

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CHAPTER 1

INTRODUCTION

1.1 Preamble

Bituminous (flexible) pavements constitute over 99% of the paved road networks in Nigeria, hence the need for appraising and improving the flexible pavement Engineering techniques can not be over-emphasised. This becomes particularly necessary in view of the present deteriorating conditions of our roads as manifested by many elements of pavement failure such as cracking, pot-holes, deformation, bleeding etc. These result into huge financial and useful manpower losses by way of maintaince and road accidents thereby posing serious setback to our national development.

Presently the design of asphalt paving mixtures in this country is based solely on the marshall test procedure as contained in the Federal Ministry of Works General Specifications (For Roads and Bridges). This design technique has over the years shown sign of deficiency as it can not accommodate the ever increasing traffic growth, material and environmental influences. Therefore there is the need to study our pavement failure problems and evolve a more realistic design technique befitting our kind of weather, traffic pattern and materials characteristics.

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1.2 Statement of the Problem

The degeneration of bituminous roads in this country is not a new phenomenon. It has been a great burden and challenge to the authority and Pavement Engineers respectively in trying to find solutions to it. Previous studies undertaken on the pavement failure have associated the problem to a number of factors which include Excessive and Unregulated axle loads, improper and poor quality of construction materials, adverse effects of heat and moisture, inadequate supervision during construction and improper mix design. However not much effort has been put to investigate the extent to which these factors contribute to the degeneration of our roads more particularly the ambient temperature which is exceptionally high. Thus the task of this research work is to:- Investigate the influence of temperature, compaction, Aggregates texture and shape and material composition on the performance of Asphaltic concrete in respect of this country.

1.3 Aim and Objective of the Study

The main aim of this study is to determine through laboratory experiments the level of influence certain factors have on the performance of our bituminous mixes and consequently the cause of degeneration of our roads. Therefore the objective of the study include

of my state government (Sokoto State) to embark on direct labour roads construction and asphalt production, warrant the choice of this study.

The significance of this research work is that it will:-

1) Determine the influence of the ambient temperature, compaction, aggregate gradation and aggregates surface texture and shape, on the performance of asphalt concrete; and

2) Provide a guide for a high quality asphalt production and construction of asphalt paving roads.

1.5 Organisation of the Thesis

The review of the literature relevant to this study is presented in chapter 2, while discussión on the experimental programme is contained in Chapter 3. Analysis of Data and discussion on the results of the experiments carried out are presented in Chapter 4. Finally the Conclusions and Recommendations based on the findings from the study are contained in Chapter 5.

CHAPTER 2

LITERATURE REVIEW

2.1 Historical Background of Pavement Engineering

The development in Bituminous (Flexible) Pavements dates to 625 BC as reported by Oglesby (1982), when asphalt was first used with burnt bricks by the Romans to pave the streets in Babylon. Since then continuous progress is being recorded in pavement engineering. Early 20th century saw the transformation of the design techniques from purely empirical to semi-empirical design methods. The empirical methods were due to limited knowledge of the materials behaviour and limitations in the ability to solve boundary value problems. With the passage of time attempts were made to broaden the knowledge of pavement design techniques. As reported by Hudson et al (1969), the early Pavement Engineer simply chose his standard materials, and constructed his pavements to take care of the maximum estimated wheel load and observes the performance of these pavements. He will then repeat the construction of those which perform well while discontinuing and/or modifying for future use of the those pavements that perform poorly. Fig. 2.1.1. gives the illustration of the early pavement design methods, while the current design technique of pavement is illustrated by Fig. 2.1.2. The current

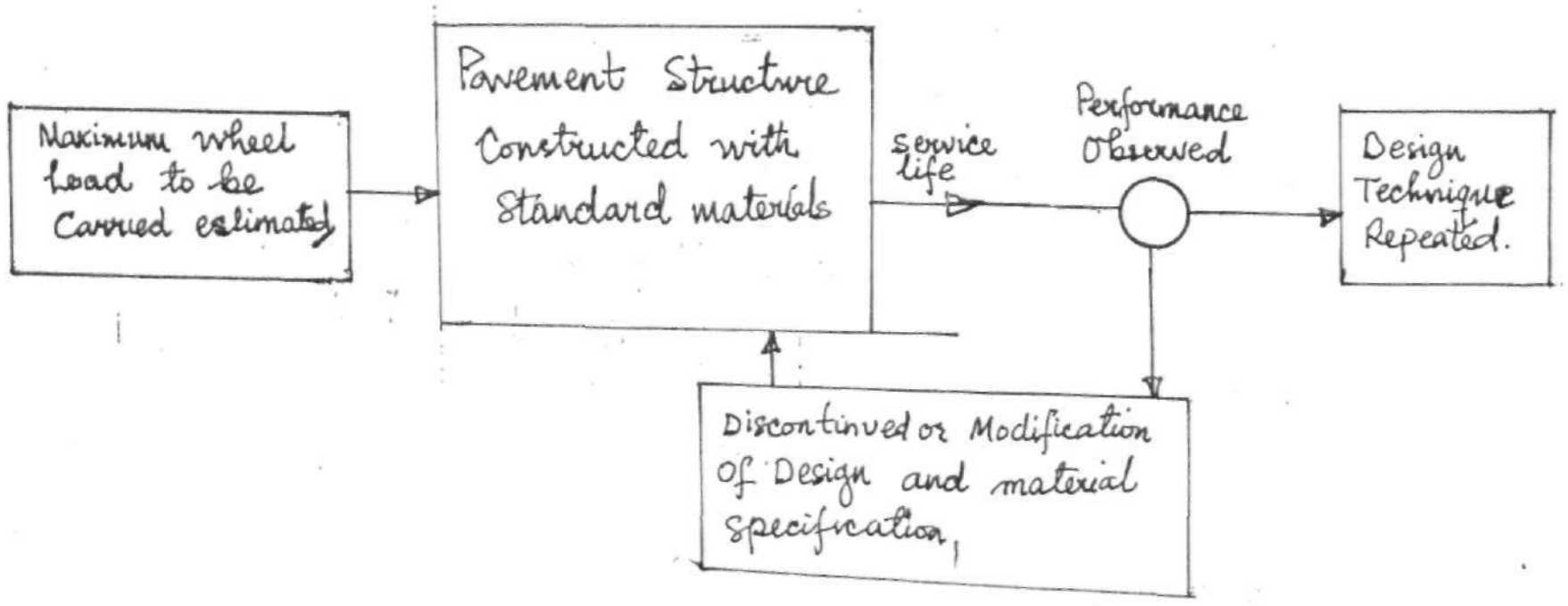


FIG. 2.1.1 FLOW DIAGRAM OF EARLY PAVEMENT DESIGN METHODS.

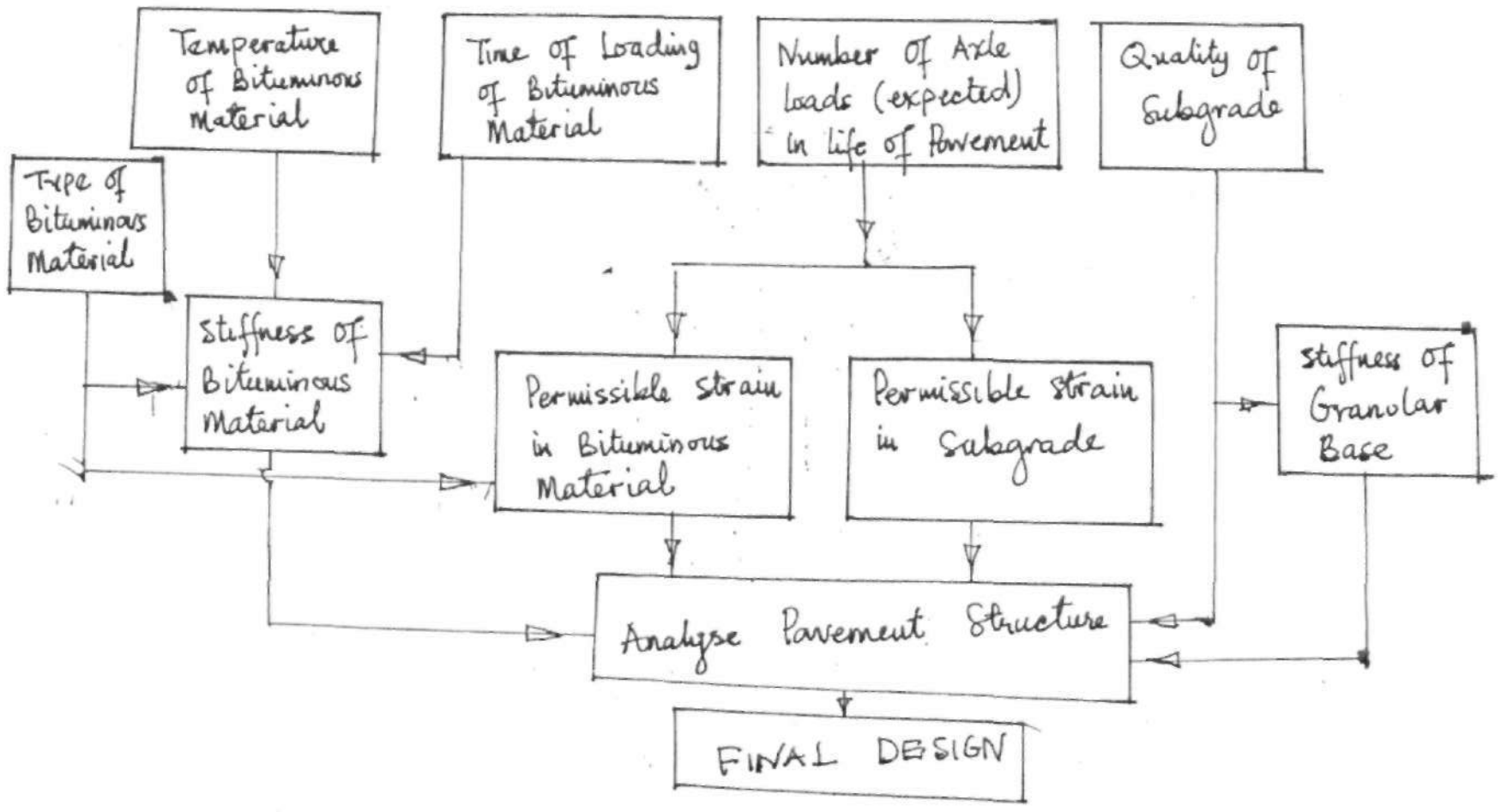


FIG. 2.1.2. FLOW CHART FOR CURRENT PAVEMENT SYSTEM (Adopted after Peattie K.R (1978)).

design technique evolved primarily through theoretical analysis, field experiments, observation of the prototype performances and laboratory experiments. This enabled the present day engineer to modify pavement behaviour evolve material specifications and test procedures.

However inspite of this break through in pavement engineering, to date no generally acceptable method is available for the design of pavement structure over the broad range of environments and materials. This is so due to the difference in environmental conditions which are dependent upon geographical locations and the variations in behaviour of materials under this environmental conditions. That is why design technique now focus on the understanding of climate conditions, loading pattern and material characteritics for a particular region.

2.2 FAILURE PROBLEM IN FLEXIBLE PAVEMENT

Premature failure problem of pavements had always been of great concern and challenge to the development of pavement engineering. Thus considerable work had been carried out to understand what causes the pavement failure with a view of finding a solution to it. One of such works was by Hudson et al (1969) who

INPUTS.

PAVEMENT SYSTEM.

OUTPUTS.

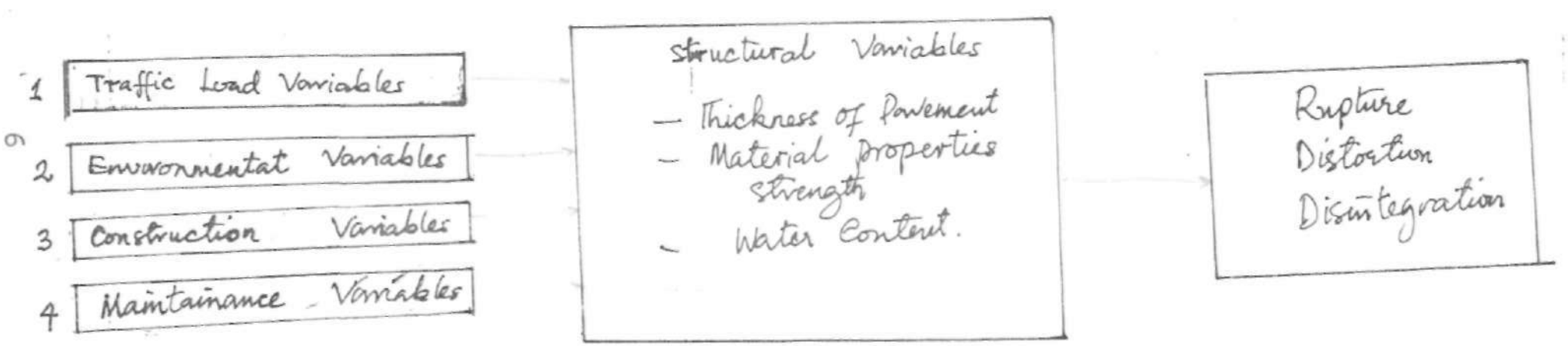


FIG. 2.2.1. PAVEMENT STRUCTURE AS A SYSTEM

considered the pavement structure as a system to which inputs are applied to produce desired outputs as illustrated by Fig. 2.2.1. The input to the system consists of traffic, environment and maintainance, while the outputs consists of the generation of mechanical states identified by deformation, rupture, distortion and disintegration. The pavement performance is thus a measure of the outputs which in turn is a function of both the inputs and the pavement system itself. Carey and Irick (1962) thus defined pavement failure as a condition when distress from the pavement structure has exceeded an acceptable levelof riding quality, economic efficiency and safety.

2.3 DISTRESS IN FLEXIBLE PAVEMENTS

From the foregoing discussion it is understood that a pevement is deemed to have failed when the distresses in the entire structure exceed some tolerable limit. These distresses occur in all layers of the pavement structures. Therefore pavement failure is the sum total of all the layer contributions. A study of the layer contributions of deformation developed in flexible pavement carried out independently by both AASHO and TRRL have indicated that bituminous surfacing and road bases contribute AASH - American Association of Societies of Highway Officials. TRRL - Transport and Road Research Laboratory.

about 50% of the total pavement deformation. The result of the study is presented in Table 2.3.1.

The commonly manifested distresses in surfacing are cracking, distortion and disintegration. Table 2.3.2 gives the summary of the usual distress manifestations and mechanism for various stress modes and causes. Distress could occur as a result of both traffic and non-traffic associated loads.

2.4 GENERAL CAUSES OF PAVEMENT DISTRESS

The distress in pavement structure is as a result of several factors acting on it which can be classified as Traffic associated and Non-traffic associated factors. The traffic-associated include the magnitude of the axle load, composition of traffic, frequency and speed. While the Non-traffic associated factors include environmental conditions, material characteristics and quality of production and laying of, the asphalt mix. The respective influences and contributions of these factors to the pavement distresses will be reviewed hereunder:

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2.4.1 Traffic-assoicated Factors

The essence of providing a pavement is to carry traffic load conveniently with safety assurance over it's design life. Therefore the effects the load have upon the components of the pavement structure must be of prior concern to the Engineer. Consequently

TABLE 2.3.1 CONTRIBUTION OF INDIVIDUAL LAYERS TO PERMANENT DEFORMATION IN FLEXIBLE PAVEMENTS

AASHO ROAD TEST		TRRL TRIAL	
LAYER	CONTRIBUTION (%)	LAYER	CONTRIBUTION (%)
Asphaltic Concrete	32		
Crushed stone roadbase	14	Rolled asphalt surfacing and Crushed stone road base	54
Sub-base	45	Sub-base	13
Sub grade	9	Sub grade	33
TOTAL	100		100

SOURCE: PEATTIE K.R. (1978)

NOTE:-

AASHO - American Association of Societies of Highway Officials.

TRRL - Transport and Road Research Laboratory

TABLE 2.3.2: CATEGORIES OF PAVEMENT DISTRESS

DISTRESS MODE	DISTRESS MANIFESTATION	EXAMPLES OF DISTRESS MECHANISM	GENERAL CAUSE
1. FRACTURE	CRACKING		
	Alligator cracks	Excessive loading	T.A.
	Edge cracks	Fatigue loading	T.A.
	Joint cracks	Moisture changes	N.T.A.
	Reflection cracks	Slippage (Horizontal forces)	T.A.
	Shrinkage cracks	Shrinkage	N.T.A.
	SPALLING		
	Breaking or chipping of Pavement at joints and Edges	Excessive loading	T.A.
		Fatigue loading	T.A.
		Thermal changes	N.T.A.
		Moisture changes	N.T.A.
2. DISTORTION	PERMANENT DEFORMATION		
	Channels or Ruts	Excessive loading	T.A.
	Grade Depression	Fatigue loading	T.A.
		Time-Dependent Deformation e.g. Creep,	T.A.
		Densification, e.g. compaction and consolidation	T.A.
		swelling	N.T.A.
	FAULTING		
	Difference in elevation of two slabs at a joint or a crack	Pumping caused by loading	T.A.
		Densification	T.A.
		Swelling	N.T.A.
		Erosion	N.T.A.

TABLE 2.3.2 : CATEGORIES OF PAVEMENT DISTRESS (CONTD)

	DISTRESS MODE	DISTRESS MANIFESTATION	EXAMPLES OF DISTRESS MECHANISM	GENERAL CAUSE
3.	DISINTERATION	STRIPPING Removal of asphalt from Aggregates	Adhesion	N.T.A.
			Moisture	N.T.A.
			Chemical Reactivity	N.T.A.
		RAVELLING Progressive separation of aggregates, fine aggregates coming out first	Inadequate compaction	N.T.A.
			Abrasion by Traffic	T.A.
			Degradation of Aggregate	N.T.A.
			Durability of Binder	N.T.A.
			Insufficient Asphalt	N.T.A.
			over heating of mixture	N.T.A.
		SCALLING -PCC Peeling away surface	Chemical Reactivity	N.T.A.
			Abrasion by Traffic	T.A.
			Freeze thaw action	N.T.A.
		POT HOLES Total removal of pavement materials at certain spots	Excessive loading	T.A.
			Fatigue loading	T.A.
			Moisture/Erosion	N.T.A.
			Adhesion	N.T.A.
			Degradation of Aggregate	T.A.
			Durability of Asphalt	T.A.
4.	SKID HAZARD	BLEEDING Flowing of binder on to the pavement surface	Low Viscosity asphalt	N.T.A.
			Too much Asphalt	N.T.A.
			Hot weather	N.T.A.
		POLISHED AGGREGATE Aggregate becoming clean and smooth	Traffic actions	T.A.
			Aggregate susceptible to polishing	N.T.A.
			Moisture	N.T.A.
		HYDRO PLANNING Retained water on Pavement surface	Poor drainage	N.T.A.
			Uneven surface	N.T.A.

Note: (1) Sources: (i) Asphalt Institute (1969)
(ii) Handbook of Highway Engineering, Baker (1975)

(2) N.T.A. = Non-Traffic Associated

T.A. = Traffic associated

through several years of research into pavement distresses, traffic load magnitude, composition, frequency and speed are found to have an adverse effect on the performance of the pavement. These factors affects the pavement performance as follows:-

a) Traffic Road magnitude: The tyre pressure affects primarily the surface wearing coarse and the composition of the materials in this layer is selected accordingly. The damaging effects of the wheel axle load on pavement is found to vary logarithmically with regards to the wheel load characteristics. Hence for structural design purposes only commercial vehicles need to be considered.

b) Composition of Traffic: Normal traffic on conventional roads is mixed in composition. Therefore a single parameter was introduced by AASHO (1962) for design purpose through the equivalence factor which relates the damage caused by an axle load to that caused by a standard axle load of 8.2 tonnes.

$$F = \frac{D_L}{D_{8.2}} = \left(\frac{L}{L_S}\right)^n \qquad (2)$$

where F = Equivalence factor or number of applications of the standard axle load

L_S = Standard axle load of magnitude of 8.2 tonnes

L = Wheel load

D_L = Damage caused by an axle load, L

$D_{8.2}$ = Damage caused by a standard axle load, L_S

n = a constant which depends on the strength and thickness of the pavement (= 4.5 for tropical

Peattie (1978) found that the damage caused by cars and other small axle loads is insignificant inspite of their large numbers, hence are ignored for the purposes of Pavement design.

c) Frequency of Traffic:- In addition to the damage caused by the individual wheel loads, the cumulative effect of traffic over a period of time also affect the performance of the pavement. However estimating and predicting future traffic is particularly difficult due to non uniformity of traffic growth, hence the pavement is usually over-stressed during it's design life.

d) Speed of Traffic:- The speed of the traffic affects the mechanical properties of the bituminous material. The critical factor here is the time which a load takes to pulse due to a moving vehicle. Studies on the effect of speed by Brown (1973) indicated that thickness of bituminous layers is dependent upon the average speed of the moving vehicle. He suggested that an approximate value for loading time in seconds is the reciprocal of the average speed of the vehicle in km/h.

$$\text{Loading time, } t = \frac{1}{\text{speed (km/h)}} \text{ secs} \qquad (2)$$

2.4.2 Non traffic associated Factors

The non traffic associated factors can be classified into three groups. Environmental factors which include the heat, cold and moisture, the material

characteristics factor which concern the pavement system itself and the factor of production and laying conditions of the Asphalt mixtures. These factors affect pavement performance as follows:-

a) Environmental effects:- The ambient temperature (heat or cold) and moisture have a great influence on the mechanical properties of the binder and granular material. A study by Enngr. Emina (1985) shows that moisture produces damaging effects on asphalt mixtures used for construction in Nigeria. Therefore only the effect of temperature will be reviewed and subsequently investigated.

Temperature has a strong influence on the stiffness and behaviour of bituminous material. If it is high (i.e heat) the effective stiffness modulus of the asphalt component will be reduced thereby resulting in decreasing it's load spreading properties and the stability of the pavement as a whole. While low temperature (i.e.cold) can cause cracking of the bituminous materials due to swelling. Previous works by Bissada (1972) have shown that the resistance of the mix to fatigue life decreases with increase in temperature. As the temperature increases, the viscosity of the binder decreases and hence the decrease in stiffness and more load transfer to the weaker lower part of the pavement consequently resulting in fatigue.

The concept of molecular chain theory can be used to explain the behaviour of bituminous material under high temperature. The concept explains that asphalt is composed of a long Polymer-like molecular chain structure. The mobilization of the chain and/or segments of the chain decreases at relatively low temperature, which reduces the ability of the asphalt to defo Therefore under load repetitions both total deformation and ability of the binder to recover after removal of the load is reduced. This therefore results in high modulus of resistance. However with high temperature reverse is the case.

b) Material characteristics:- The varying engineering properties and inadequate knowledge of predicting their behaviour under the influence of temperature, moisture and traffic loading is one of the problems encountered in pavement engineering. The materials used to produce asphalt mixes have their own individual properties and contribution to the strength of the asphaltmix. Hence to produce a good performance bituminous pavement, the varying individual components have to be assessed and maximised under the expected traffic and environmental conditions.

The typical components of the asphalt mixtures are coarse aggregates, fine aggregates, filler and the binder. The desirable qualities of these materials as reviewed by SHELL International ('962) are:-

(i) Coarse Aggregates: - These are the stone materials retained on BS Sieve number 7 mesh (or ASTM No 8 mesh). It gives the mix it's stability by interlocking of the aggregates particles and their frictional resistance to displacement. Therefore both shape and surface texture of the stone are essential to produce high stability mixes. Thus the ideal aggregate is hard angular stone with rough surface texture.

(ii) Fine Aggregates:- These are stone material between BS No. 7 mesh and No 200 mesh sieves. It adds to the stability too through the interlocking characteristics of the particles at the same time it reduces the voids in the coarse aggregates. It is required that the fines should have a rough texture to increase the stability. The coarse fines (from Mesh No. 7 to No. 30 mesh) are of great importance as they give the sandpaper texture to the mortar and therefore increase the non-skid properties of the asphalt mixes. While the finer fines (from mesh No. 30 to mesh No. 200) enhances the durability as they increase the surface area of the aggregates in the mix.

(iii) Filler: - This is the finer material passing the BS No 200 mesh sieve. The function of the filler is to act as a final void filling material as well as a means of stiffening the bitumen film on the aggregates particles. Materials such as limestone dust, cement and hydrated lime are often used as fillers.

(iv) Binder:- Petroleum bitumen is used as the binder. The binder is the most influencial component of the asphalt mixes. It is highly susceptible to temperature changes, that is the stiffness of the mixture is temperature dependent. The quality of the binder too is important to the stability and modulus of resilience of the binder.

c) Effects of Unsatisfactory mixes and laying Conditions:-

The production of unsatisfactory mixes and inadequate compaction during laying do contribute to the distress of the bituminous layers. The cause of unsatisfactory mixes could be due to a combination of the following:- excess bitumen, lack of sufficient bitumen, over-heating, mix too cold, excess moisture, poor mixing and poor grading. Also at the laying stage, the temperature of the mix must not be too cold as compaction is dependent on the temperature of the mix. While the level of compaction determines the stiffness and durability of the bituminous mixes to distribute the loads imposed by the traffic. Good compaction also increase the resistance and durability of the bituminous layers. The level of compaction is characterised by parameters such as density of the compacted mix (CDM), density of the aggregates and binder; and voids of the mix and mixed aggregates (VIM and VMA).

Cooper and pell (1974) found that the level of compaction is affected by several factors such as material temperature, binder content type; aggregates grading and the stiffness of the working platform.

2.5 ASPHALT MIX DESIGN

The inherent varying individual material properties of the mix components necessitate that their composition should be so proportionally balanced such that all the desirable properties of a good mix are achieved. Therefore to embark on a good mix design it is necessary to review the factors and the methods of the design.

2.5.1 The Factors affecting mix design

The purpose of mix design according to Monismith (1961) (1961) is to make an economical blend of asphalt and graded aggregates in order to produce a mixture that is durable, stable, flexible, impermeable, stiff and workable. These significant mixture properties are the factors considered in mix design and their requirements are as follows:-

a) Stability: This is the resistance to deformation under both static and dynamic loads. It is required to be high and is affected by aggregates, binder content and type and viscosity of bitumen and filler content.

b) Flexibility: - This is the resistance to cracking tendencies. Mixture is required to resist cracking due to:

(i) long term changes in support strength, e.g. weakness of subbase and subgrade.

(ii) repeated flexing under short duration of traffic loads.

(iii) volume change due to thermal changes. Flexibility is a function of binder content and type and aggregate gradation.

c) Durability: This is the resistance to ageing and external influences of water etc. It is a function of bitumen content and type and aggregate gradation.

d) Workability: This is the ability of the mix to be spread and compacted to the required density without difficulty. It is a function of bitumen content and aggregate shape and texture.

e) Safety: The mix should provide good skid resistance and must not disintegrate by fretting or ravelling especially the mix intended for wearing coarse. This is a function of binder content, aggregate type, shape and gradation and presence of foreing materials such as water, oil, etc.

f) Impermeability: This is the ability to prevent th ingress of water. Factors affecting impermeability are binder content, aggregate grading and compaction.

g) Stiffness: This is a measure of strength of the

mix. It must have high modulus to distribute loads more effectively. It is a function of aggregate gradation and binder type.

A summary of the desirable properties of mixes and their corresponding requirements for binder content, aggregates gradation and degree of compaction is presented in Table 2.5-1.

2.5.2 Mix design methods

The primary objective of a highway pavement surface is to provide an acceptable roadway surface that can withstand the deteriorating effects of traffic and environment for the service life of the facility. Therefore the mix design procedures must be able to optimise any or all of these desirable properties mentioned above in Table 2.5.1. Currently there are three design methods recongised by the Asphalt institute (9(1970). These are the Marshall stability, Hveem stabilometer and Hubbardfield methods.

Each of the design methods have it's own particular advantage and disadvantage in satisfying the requirements of a good mix design. A schematic description of the procedures of the three design methods is presented by the flow diagrams of figs 2.5.2 a, b and c respectively. Although neither of the three above mentioned methods satisfy all the requirements of a mix design in the full sense of the word, however on comparison of the three

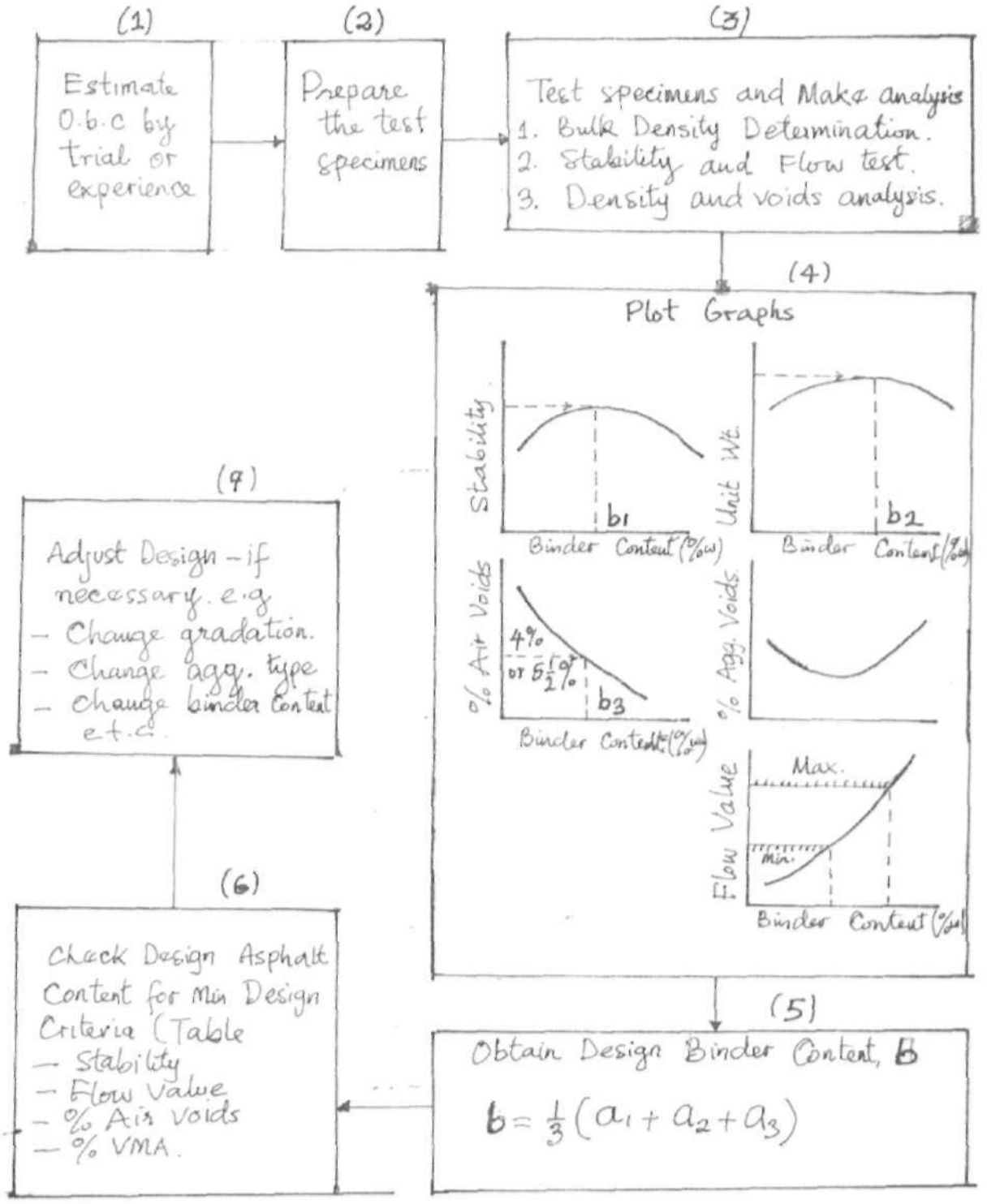


FIG 2·3·2 a FLOW DIAGRAM FOR MARSHALL METHOD.

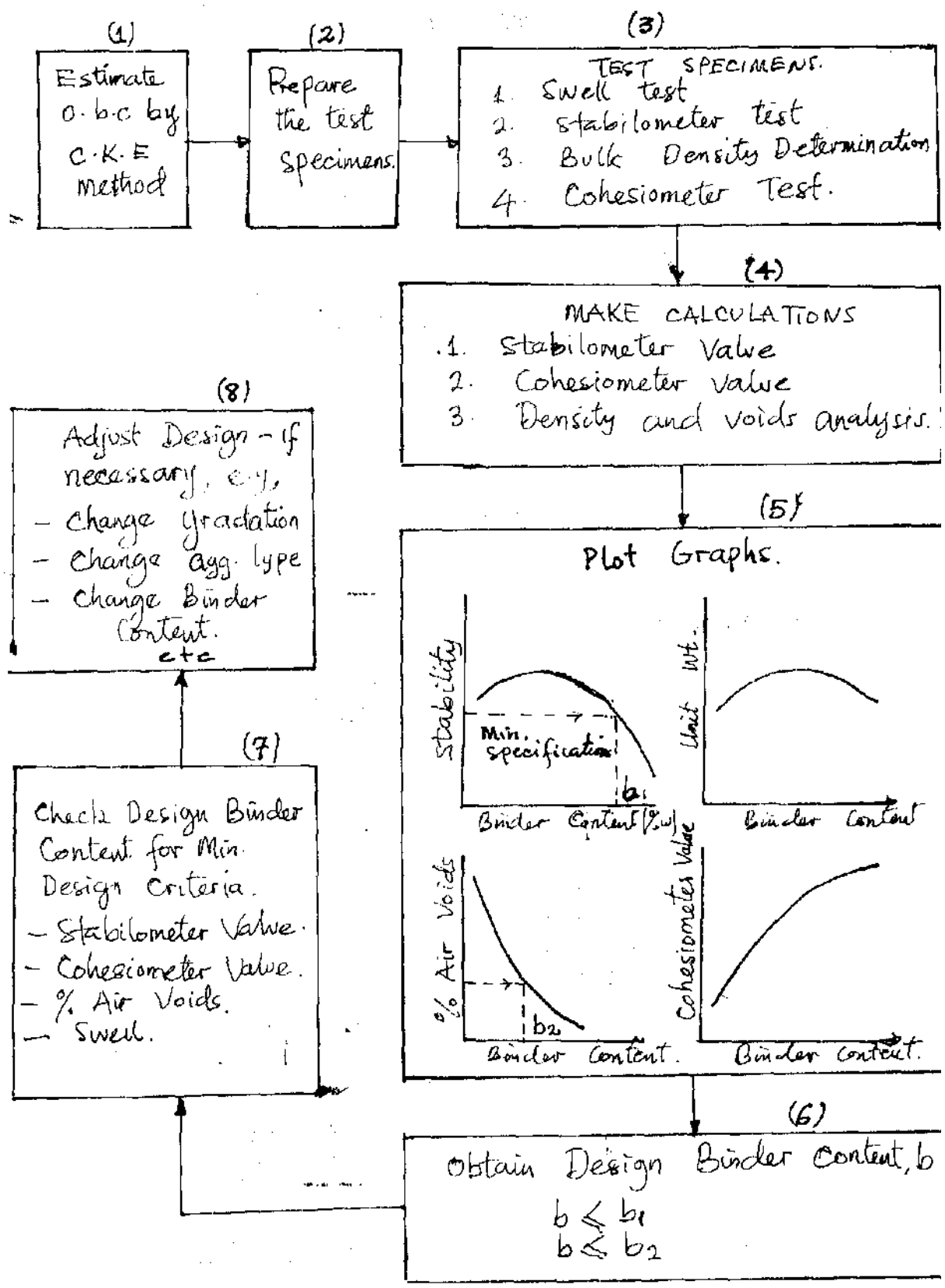


FIG. 2.3.2b. FLOW DIAGRAM FOR HVEEM METHOD

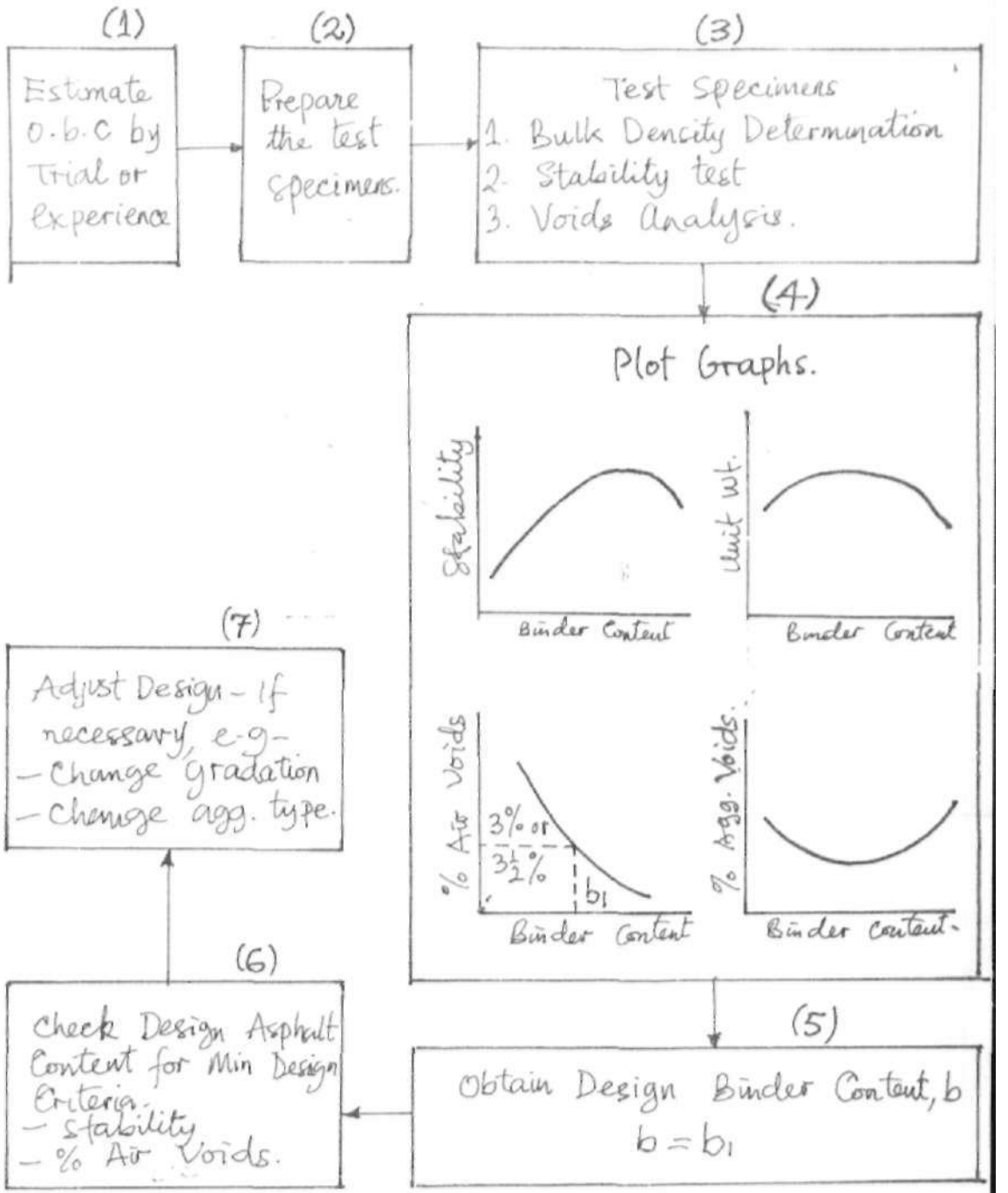


FIG. 2.3.2c. FLOW DIAGRAM FOR HUBBARD FEILD METHOD.

TABLE 2.5.1: DESIRABLE CHARACTERISTICS OF OPTIMISED MIXTURE PROPERTIES

MIX PROPERTY \ MIX VARIABLE	BINDER CONTENT	AGGREGATE GRADATION	DEGREE OF COMPACTION
Stability	Low	Dense	High
Durability	High	Dense	High
Flexibility	High	Open	-
Fatigue resistance	High	Dense	High
Skid Resistance	Low	Dense or Open[1]	High[2]
Imperviousness	High	Dense	High
Fraction Strength	High	Dense	High
Workability xtics	High	Open	-
Stress-Strain xtics	High	Dense	High
Thermal xtics	Low	Dense	High.

1. Skid resistance: What is important is the texture of the aggregate particles

2. Compaction:- Although compaction is not normally indicated for skid resistance. It is implied to ensure that aggregate particles will not dislodge under tractive forces applied to the surface.

Source: Robert F. Baker (1975), Handbook of Highway Engineering,.

TABLE 2.5.2: COMPARISON OF APPLICABILITY OF LABORATORY DESIGN METHODS

	FACTOR	MARSHALL	H EEM	HUBBARD-FIELD
1.	Use of Method	Lab design and field control	Lab design and field control	Lab design
2.	Type of asphalt	Pen grade	Pen and Liquid grades	Pen grade
3.	Initial of estimate of asphalt content	Trial	C.K.E	Trial
4.	Basis of Percent asphalt	Wt. of mix	Wt. of mix	Wt. of mix
5.	Type of Stability test	Compression	Triaxial	Extrusion
6.	Loading rate	50.8mm/min	1.27mm/min	-
7.	Tension test	-	Cohesionmeter	-
8	Compaction of specimen	impact	Kneading	Tampers
9.	Testing Temperature	60°C	60°C	60°C
10.	Specific gravity of aggregate	Bulk	Apparent	Bulk
11	Maximum size of aggregate	25.4mm	25.4mm	19.05mm
12.	Size of specimen	101.6mm dia by 63.5mm ht	101.6mm dia by 63.5mm ht.	50.8mm dia. by 25.4mm ht.
13.	Design asphalt content	Optimise all criteria	Minimum stability	3 or 3½% air voids

TABLE 2.5.2b: COMPARISON OF SUITABILITY OF LABORATORY DESIGN METHODS

PAVING MIXTURE / TEST TYPE	MARSHALL	HVEEN	HUBBARD-FIELD
Macadam	X	X	X
Open Graded	X	D	X
Coarse Graded	D	S	X
Dense Graded	S	S	X
Fine Graded	S	S	X
Stone Graded	S	S	
Sand sheet (sand Asphalt)	S	S	S
Fine Sheet (Sheet Asphalt)	S	S	S

S - Suitable

D - Doubtful

X - Unsuitable

Source: The Asphalt Institute (1969)

methods on the bases of suitability and application to the different type of asphalt paving mixtures, Hreem stabilometer design method seems to account for most desirable properties of the asphalt mixtures But the th Hveem method has inherent time and money costs compared to the other two, hence the reason why Marshall design method have greater acceptability due to it's advantage of easy adaptability.

2.5.3 Suggested mixture design Procedure

The essence of a mix design is to optimise any or all of the desired properties listed in Table 2.5.1. According to Brien (1978) a complete mix design should fulfil the following:-

(a) Measure the relevant characteristics of the material components that are available.

(b) Estimate the properties that are required in the completed compacted mix.

(c) Knowing (a) and (b) the procedure should then indicate the proportions in which the components should be mixed in order to produce the required properties as economically as possible. Therefore it can be seen from this that neither of the three procedures fulfil the requirements of mix design in the full sense of the word. However since the essence is to produce a procedure that can reflect nearly all the mix

requirements, a blend of the existing procedures can give a satisfactory result. In the light of this a design procedure proposed by Engineer Emna (1985) is adopted for this work with little modification. The suggested design procedure will estimate the design binder content as the mean value of the binder contents which optimise the marshall stifness, compacted density of the mix, tensile strength modulus of elasticity and 4% voids in the mix. The design binder content is therefore expressed as follows:-

$$B = 0.2\ (b_1 + b_2 + b_3 + b_4 + b_5)$$

where B = Design binder Content (% by wt of mix).

b_1 = Optimum binder content for maximum marshall stiffness, Qm.

b_2 = o.b.c. for maximum c.o.m

b_3 = O.b.c for maximum tensile strength, S_T

b_4 = O.b.c for maximum Compressive Modulus, E_y

and b_5 = O.b.c for maximum for 4% vim.

The design binder content B, should then be checked for minimum/maximum mixture design criteria according to marshall design criteria presented in table 2.5.3a. A flow diagram illustrating the suggested design procedure is presented in

TABLE 2.5.3: MIXTURE DESIGN CRITERIA

(a) MARSHALL DESIGN CRITERIA

Traffic Category	Heavy		Medium		Light	
No of Blows (each face)	75		50		35	
Test Property	min.	max.	min.	max.	min.	max
1. Stability, Kg	340		227		227	
2. Flow, mm	2.0	4.1	2.0	4.6	2.0	5.1
3. Percent Air voids, %						
- lavelling/surface course	3	5	3	5	3	5
Base course	3	8	3	8	3	8
4. Percent voids in mineral aggregate %						
- 25.4mm max size	13.3	-	13.3	-	13.3	-
- 19.1mm max size	14.0	-	14.0	-	14.0	-
- 12.7mm max size	15.0	-	15.0	-	15.0	-
- 9.5mm max size	18.5	-	18.5	-	18.5	-

(b) Hreem Design Criteria

Traffic Category						
Test Property	min.	max	min	max	min	max
1. Stabilometer value,%	37	-	35	-	30	-
2. Cohesiometer value (g/mm width corrected to 76.2 mm height)	1.97	-	1.97	-	1.97	-
3. Percent air voids, %	4	-	4	-	4	
4. Swell	Less than 0.76mm					

(c) HUBBARD - FIELD DESIGN CRITERIA

Traffic Category	Heavy		Medium	Light	
Test Property	min	Max	min	Min	Max
1. Stability, Kg	907	-	544		907
2. Percent Air voids, %	2	5	2		5

fig. 2.5.3, which is subject to modifications based on it's actual field performance. The justification of this suggested procedure lies in the following argument:-

1. Binder content b_1, optimises the marshall stiffness which is the ratio of marshall stability to flow. It had been established by Gadallah et al (1977) that marshall stiffness is a better measure of the mixture characteristics at failure than the marshall stability. Also Marias (1974) and Brien (1976) have shown that good correlation exists between results of the Marshall tests and laboratory wheel tracking made on gap-graded mixes when Marshall Stiffness (quotient) is used instead of stability.

2. Binder Content b_2, optimises the compacted density of the mix which is a measure of compactness of the mix. Brien (1978) stated that the aggregate grading which gives the greatest possible aggregate density will also give the highest stability when combined with the optimum amount of bitumen. Therefore the choice of b_2 is to enhance both mixture stiffness and pavement performance.

3. Binder contents b_3 and b_4, optimise the tensile properties of the mix. These according to

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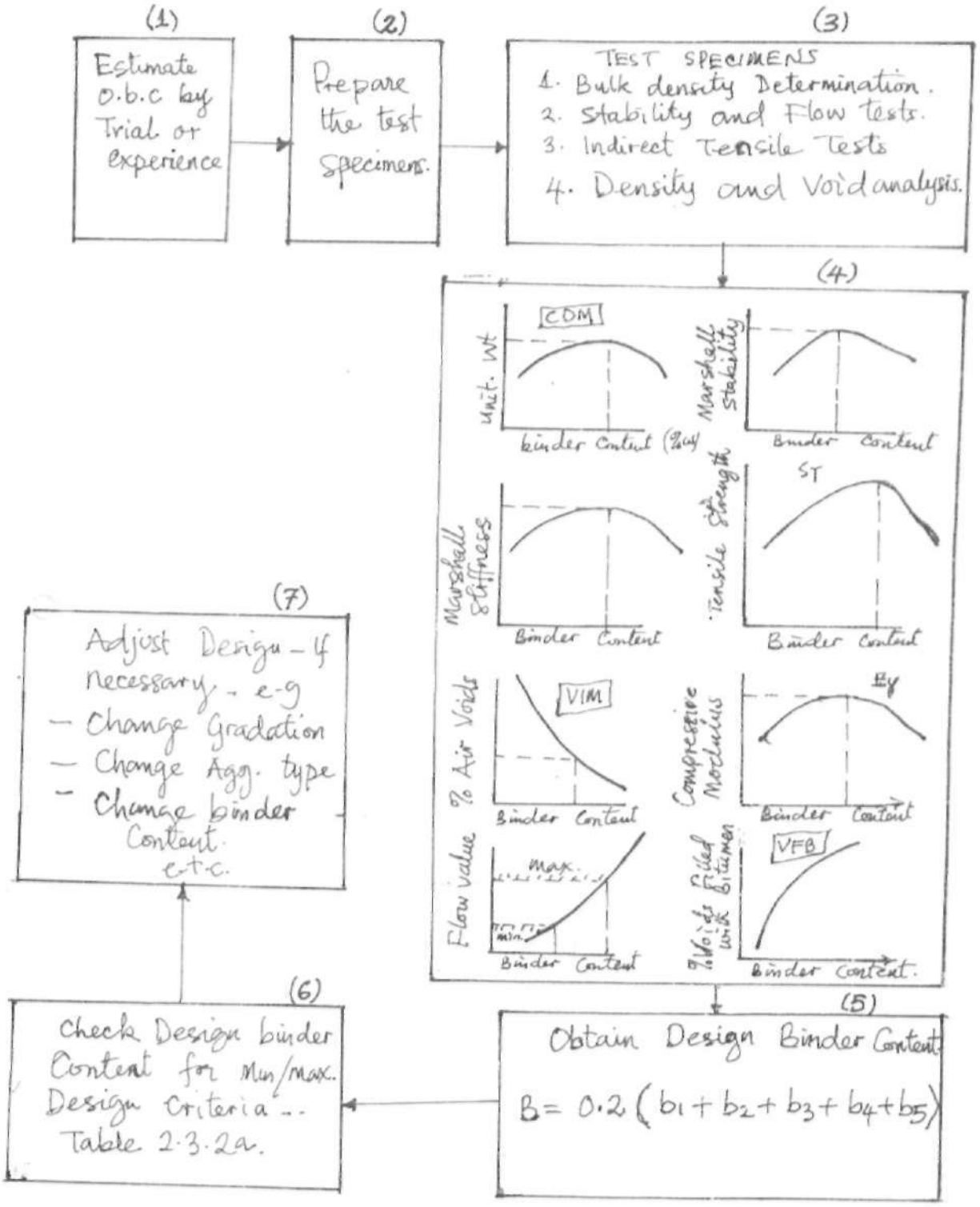


FIG. 2.5.3 FLOW DIAGRAM FOR SUGGESTED MIXTURE DESIGN METHOD.

Khandhal (1979) reflect the fracture strength of the mixture. These strength parameters are obtained through indirect tensile test which will be discussed later in this chapter.

4. Binder Content b_5, optimises the durability of the mixture. A study by Welborn (1970) showed that voids in the mix influences the durability of the mix. Too much voids in a mix makes the mix permeable to water ingress and too less voids lead to fattening. Both effects reduced the stability and durability of the mix. However, experience has shown that voids contents of 3% and 5% lies between the two extreme, hence the choice of 4% air void.

2.6. INDIRECT TENSILE TEST

2.6.1. Development of the test

The application of the indirect tensile test to bituminous mixes started around 1950, when it became necessary to check the fatigue behaviour of the mixes under the induced tensile forces developed on the road surface by the growing traffic. It was by then that the theory of concentrated or line loading v developed by Frocht (1948) was adopted and further modified

by Hondros (1959) to account for uniform loading across a curved loading strip with a view to apply it on softer materials, see Figs 2.6.1 Before the advent of this development there were problems encountered by the use of direct tensile test procedures. These include:-

a) difficulty in gripping procedure and application of purely tensile force.

b) the bending test has the disadvantage of an undefined stress distribution across the specimen and

c) influence of surface irrigularities to the result of the test.

Previous research findings on the use of the indirect tensile test by Hudson and Kennedy (1969) and Hadley et al (1969), had indicated that the test can be used to predict the tensile strength of asphaltic concrete with reasonable accuracy. P. S. Kandhal (1979) also concluded after adopting the test procedure that it can be used to determine the tensile strength, poisson ratio, tensile strain at failure and stiffness modulus of asphaltic concrete, which are data for designing pavement and predicting the potential of low temperature shrinkage cracking.

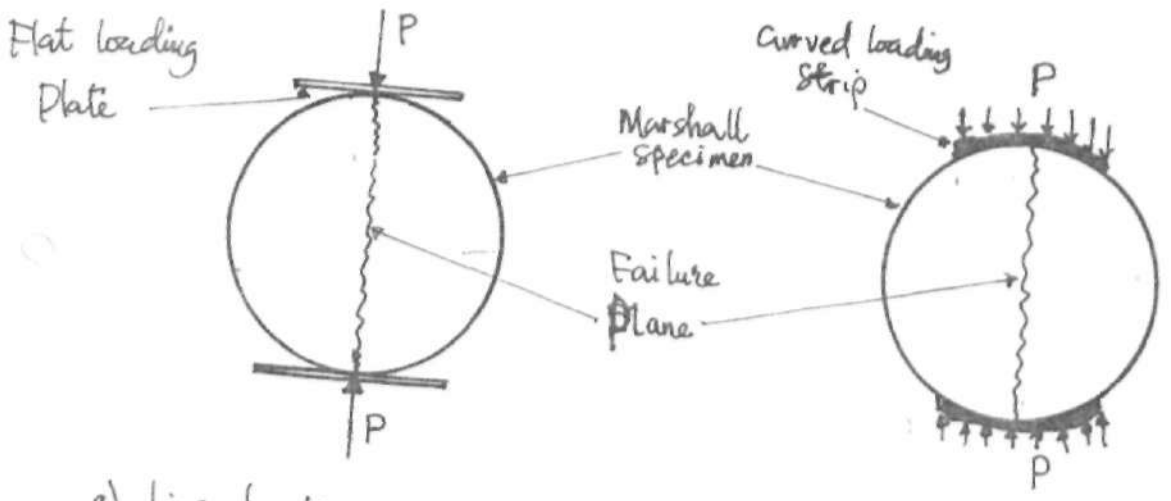


FIG 2.6.1. THEORY OF INDIRECT TENSILE TEST.

2.6.2. Advantages and shortcomings of the test

The advantages of the test lies in the following:-

a) It is simple to conduct

b) Marshall specimens may be used.

c) Surface irregularities do not seriously affect the result.

d) The coefficient of variation of the test is low.

e) The test can be used to test specimens under static and dynamic loading conditions.

f) Fatigue is initiated in a region of elatively uniform tensile stress.

While the disadvantages are mainly with the theory which assumes:-

1) That the test specimens are purely elastic, but asphalt concrete is visco-elastic at most environmental conditions, and

2. a line loading on the specimen whereas in practice the load is distributed with a loading strip.

However the gains of advantages of indirect tensile test outweigh the disadvantages, hence its prominance in current asphalt mix design.

2.6.3. Equipment and Testing Technique of Indirect tensile test

As reviewed above, the relevance of the test lies in its simplicity whereby any equipment capable of exerting compressive loads can be adopted for the test. Various types of equipments, ranging from those that use the sophisticated MTS loading system to those which are relatively simpler and cheaper, like the Triaxial Compression and CBR machines. Lottman (1978) and Maupin (1979) both have successfully used the triaxial compression machine. Here in Nigeria Engr. Emina (1985) had used the ordinary CBR machine at the University of Lagos.

The testing technique involves the use of a curve loading strip (usually 0.5 in, (12.7mm) wide) made of material relatively harder than asphaltic concrete. Most researchers use stainless steel material. The strip is to be curved of the same radius with the cylindrical test specimens. The specimen is then loaded uniformly via the loading strip with compressive loads that act parallel to, and along the vertical diametrical plane. The loading configuration develops a relatively uniform tensile stress perpendicular and along the vertical diametrical plane that ultimately causes

TABLE 2.6.3: LIST OF PREVIOUSLY TESTED RATE OF LOADING FOR INDIRECT TENSILES TEST.

RESEARCHER	YEAR	ROOM TEMP		LOADING RATE	
		^{o}C	^{o}F	mm/min	in/min
Lottman	1978	13	55	1.60	0.065
Lottman	1978	23	73	3.80	0.150
Villa and Terrel	1975	13	55	1.60	0.065
Maupin Jr.	1979	12	54	1.60	0.065
Maupin Jr.	1979	25	77	51.0	2.0
Kandhal	1979	25	77	51.0	2.0
Emina (Nigeria)	1985	28	82	7.6	0.30

the specimen to fail by splitting or rupture along the vertical diameter. The load applications and vertical deformations of the specimen are continuously monitored until failure occurs. The readings will then be used to determine the tensile strength, S_T poisson ratio, μ ; and stiffness compressive modulus, E , through established formulae.

Although there is no standard specification for rate of loading, it is however related to the room temperature at the time of testing. Table 2.6.3 gives a list of various ranges of rates used previously by a number of researchers.

2.6.4. Working Equations

The theoretical equations used in calculations of the mix tensile properties are very complex and required integration of various mathematical functions. Therefore within limits of this research work the tested working equations developed by Adedimila (1980) will be used. The equations were calibrated based on the use of marshall specimen of 4in. (101.6mm) diameter and thickness t. The strip used was of stainless steel material of 0.5in (1207 mm) width.

The equations are summarised as follows:-

a) Tensile strength;

$$S_T = \frac{60.35\ P_f}{t} \quad - - - - (3)$$

b) Compressive modulus

$$E_y = \frac{LDU}{t}\ (35183 - 615\nu)$$

$$= 34967.75\ \frac{(LDV)}{t} \quad - - (4)$$

c) Tensile modulus

$$E_x = \frac{1DH}{t}\ (2647 + 9807\nu)$$

$$= 6079.45\ \frac{LDH}{t} \quad - - - - \quad (5)$$

The definitions of the various symbols used herein are given in the list of symbols. A value of 0.35 was adopted for the poisson ratio.

CHAPTER 3

EXPERIMENTAL PROGRAMME

The sole aim of any research is to study the system in question with a view of producing useful information for the successful operation, management and/or further study of the system. Therefore in line with the objectives of this study as stated in Chapter one, an experimental programme was designed to ensure that both efficiency and economical gains are achieved. This deals with the following:

1) Experimental design of factors and levels,
2) Materials selection and preparations.
3) Testing procedures and equipments selection.
4) Treatment of specimens.

The entire programme of the experimental study is described hereunder

3.1 Experimental design of factors and levels

To adequately achieve the goals of this study all the desirable properties of a good quality bituminous surfacing must be considered. From the various studies all over the world (RRL, 1962) it has been observed that parameters such as stability stiffness, flow, compacted density of the mix and air voids of

the mix are affected by such factors as mixing and testing temperature, binder content and type, Filler Content and type, and aggregate type and gradation and compaction.

These no doubt will require several number of tests and complexity of result interpretation according to Anderson and McClean (1974) if full randomised factorical design is to be considered. Thus in order to minimise the number of tests and facilitate intepretation of results as recommended by Hadley et al (1970), Anderson and Hudson (1970) and Anderson and McClean (1974); a fractional factoral design was adopted as presented in tables 3.1.1 through 3.1.4.

In the light of the above discussion and in line with the objective of this study the following factors were considered as tests variables:- binder content, testing temperature, compactive effort, stone Content, aggregate shape and surface texture and test type. The experimental procedure was conducted based on the following four phases in order to facilitate interpretation of the results.

TABLE 3.1.1a EXPERIMENTAL DESIGN FOR OPTIMISING BINDER CONTENT AND AGGREGATE GRADATION

COURSE	BINDER COURSE		SURFACE WEARING Course	
Stone Content(%)	60	70	35	
No of Specimen				
Binder Content%w				
3	4		-	
4	4	4	4	
5	4	4	4	
6	4	4	4	
7	4	4	4	
8	-	4	4	
TOTAL	20	20	20	60

TABLE 3.1.2 : EXPERIMENTAL DESIGN FOR OPTIMISING BINDER CONTENT AND AGGREGATE SHAPE

LAYER COURSE	SURFACE WEARING COURSE			
AGGREGATE SHAPE	ROUND AND SMOOTH	CRUSHED ANGULAR AND ROUGH		
TEST TYPE	MARSHALL	MARSHALL	INDIRECT TENSILE	
No of SPECIMEN				
Binder Content				
4	4	4	4	
5	4	4	4	
6	4	4	4	
7	4	4	4	
8	4	4	4	
TOTAL	20	20	20	60

TABLE 3.1.3 EXPERIMENTAL DESIGN FOR CHECKING THE EFFECT OF TESTING TEMPERATURE AND COMPACTIVE EFFORT

COMPACTIVE EFFORT (NO OF BLOWS TO EACH FACE)	75	50	35	
NO OF SPECIMENS				
TESTING TEMP ^{o}C				
20	4	4	4	
40	4	4	4	
60	4	4	4	
80	4	4	4	
TOTAL	16	16	16	48

TABLE 3.1.4 : EXPERIMENTAL DESIGN FOR CHECKING THE EFFECT OF STONE CONTENT

STONE CONTENT (%w)	15	35	50	60	70	90	TOTAL
No of SPECIMENS	4	-R-	-R-	-R-	-R-	4	8

R - Means the specimen had been cast and tested under the same levels and circumstances in previous tests.

TOTAL NUMBER OF SPECIMENS = 60+60+48+8

= 176

PHASE 1: Optimizing the binder content and the aggregate gradation to maximise stability, density and other desirable properties of the mix as contained in Table 2.5.1

PHASE 2: Optimising the binder contents and Aggregate shapes to maximise stability, density and other desirable properties as contained in Table 2.5.1.

PHASE 4: Optimising the stone content to maximise certain important properties.

PHASE 3: Determining the effects of Temperature properties and compactive efforts on the mix properties.

Factors and levels considered

As mentioned above about six factors were considered and varied at certain levels for this study depending on the test phase requirements and intention of study. Below is a brief account of how the variables were varied and/or kept constant:-

a) Binder Content: The binder content was varied at five (5) levels with 1% increment in phases 1 and 2 only, and kept constant at optimum level of 7% in phases 3 and 4.

b) Testing Temperature: The testing temperature was kept constant at 60°C in all the phases except one, that is phase 3 where it was varied at four levels with 20°C increment. (20°C, 40°C, 60°C and 80°C).

c) Compactive effort: The level of compaction was also kept constant in all but one phase at 50 number, of of blows to each face (end). In phase 3, compaction was varied at three levels of 35, 50 and 75 blows to each end.

d) Aggregate Gradation and stone Content: The stone content was varied at six different levels only. Three levels in phase one (i.e. 35, 60 and 70% Stone percentage by weight of the mixed aggregates). Another level of 50% in phases Two and Three and Two other levels of 15% and 90% in phase 4.

e) Aggregate shape. The shape, type and texture was varied at two levels in phase two only while kept constant in all other phases. Crushed Angular and rough aggregate were dominantly used in all phases while smooth round aggregate was used in phase two only.

f) Test Type: Two levels of test type were used. Marshall test was used in all phases while indirect tensile test was used in phase two only.

Tables 3.1. through 3.1.4. contain the details of the experimental design programme as per the phases described above.

Measurable Parameters

To measure the behaviour and performance of the mix under the influence of the mentioned factors some relevant parameters were measured and studied. Table 3.3.3a contains the list of the parameters measured under the different specimens tests.

Table 3.3.3a: MEASURABLE PARAMETERS

S/No	SPECIMEN TESTS	PARAMETERS
1	Bulk density and void analysis	CDM, CDMA, VIM, VMA, VFB and VIMO
2.	Stability and flow test	Marshall stability, Marshall Flow and Marshall Stiffness, Q_m.
3.	Indirect tensile test	Tensile strength, S_T and Compressive modulus, E_Y.

3.2 MATERIALS

The three basic components of Asphaltic Concrete are asphalt cement (binder), Aggregate (both Coarse and Fines) and Filler (cement etc). The particular materials used for this research work are described hereunder.

BINDER:- The bitumen used was of 80/100 Penetration grade, produced at the Kaduna refinery and obtained in the Ahmadu Bello University, Zaria, Civil Engineering Soil Laboratory. Table 3.2.1 contains both the Federal specifications and measured values of the various properties of the binder. It could be observed perhaps due to ageing that the binder is relatively more viscous than the normal 80/100 pens bitumen.

CEMENT:- The cement used was the ordinary Portland cement, type 1, produced at Ashaka cement factory in Bauchi State. The specific gravity of the cement used was 3.06 and about 90% by weight passed the B.S Sieve No. 200.

AGGREGATES: There are two types of aggregates used, the coarse aggregates and the fines.

(i) Coarse Aggregates: Two types of Coarse aggregates were used for the tests, both were found around Zaria environment. There was the crushed, rough and angular aggregate of granite origin and the round and relatively smoother aggregates of Conglomerate origin usually referred to as River Stones. The coarse were graded from BS Sieve No 1" mesh to No 7 mesh.

(ii) Fine Aggregates: The fine aggregates used were the natural, single size, round, and harsh sand found also around Samaru-Zaria. The Fines were graded from BS Sieve No 7 to 200 meshes.

PROPERTIES OF THE BINDER

S/No	PROPERTY	ASPHALT CEMENT (80-100pan)	
		FEDERAL SPECIFICATION	MEASURED (ACTUAL)
1	Specific Gravity @ 25°C	1.00-1.05	0.934
2	Softening Point R and B (°C)	45-52	67
3	Penetration @ 25°C .. 0.1mm	80-100	82
4	Ductility @ 25°C.. CM(minimum)	100	94.2
5	Loss on treating for 5 hours at 163°C by weight (max)	0.5	0.4
6	Solubility in cS_2 by wt (min)	99	99
7	Drop in penetration after heating (% original max)	20	20
8	Flash point (open cup °C min)	225	320
9	Ash % by wt max	0.5	0.5

Table.3.2.2 PROPERTIES OF THE AGGREGATES

S/No	PROPERTY	TYPE OF AGGREGATES	VALUES	
			SPECIFIC-ATION	MEASURED (DETAIL)
1	Specific gravity	Crushed, Coarse	-	2.74
		Round, Corase		2.63
		Fines	-	
		Cement	-	3.06
2	Crushing Value(%)	Crushed, Aggregate	30	28
		Round Aggregate	-	31
3	Impact Value (%)	Crushed Aggregate	-	25
		Round Aggregate		31

(iii) Aggregate Gradation:- About six different types of gradations were used because the research works also aims at finding the effect of the gradation. Two of the Aggregate gradations were within the grading envelope specified by Federal Ministry's specifications for wearing course and two within the specifications for binder course. While the other two gradations were theoretically assumed to correspond with lowest and highest stone contents outside the specified grading envelopes. Table 3.2.3 and Figs 3.2.2 a and b give the details on the aggregate distribution. Table 3.2.2 gives the properties of the aggregates used.

3.3 TESTING PROCEDURES

3.3.1 Test Methods:

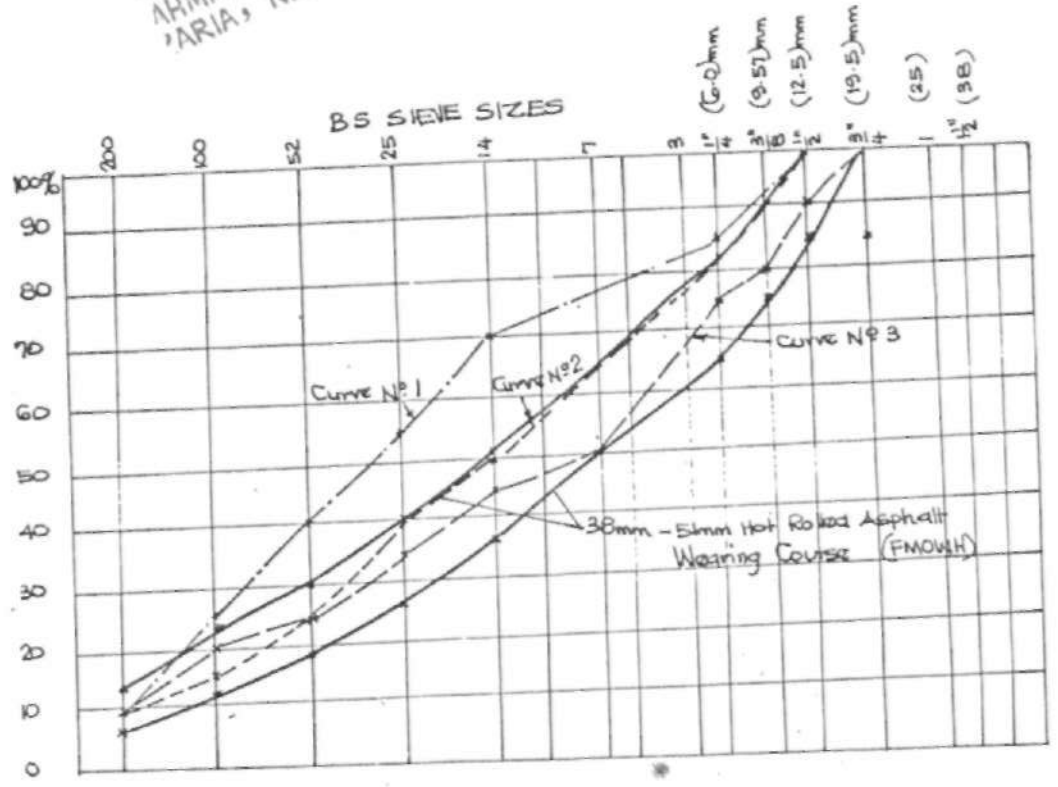
As reviewed earlier in Chapter 2, this work adopted both the Marshall and Indirect tensile tests methods for the preparation and testing of the test specimens. Below is a brief description of the test methods.

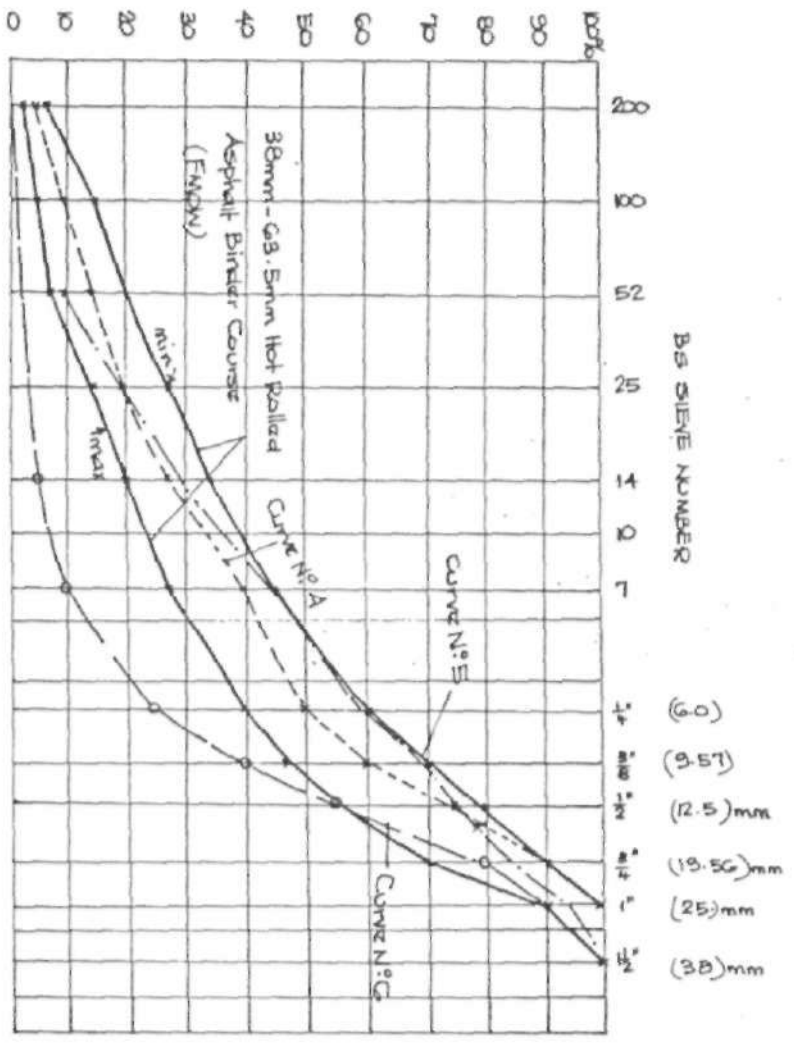
a) Marshall test:- The test was conducted according to ASTM Standards 1958, Part 4, D 1559, given by the Asphalt institute (1969) the method basically involved the preparation of test specimens, weighing bother

Table 3.2.4 FEDERAL MINISTRY OF WORKS SPECIFICATION FOR AGGREGATE SIZE DISTRIBUTION

	B.S SIEVE SIZE	% BY WT PASSING	
		1½" - 2½" (38.1mm) (63.5mm)	1½" - 2" (38.1mm) (50.8mm)
	COURSE	BINDER COURSE	WEARING COURSE
Coarse Agg.	1½" (38.1mm)	100	100
	1 (25.4mm)	90-100	100
	¾ (19.05mm)	70-90	100
	½ (12.7)	55-80	85-100
	⅜ (9.5mm)	47-70	75-92
	¼ (6.2mm)	40-60	65-82
Fine Agg.	No 7	27-45	50-65
	14	20-34	36-51
	25	24-27	26-40
	52	8-20	18-30
	100	5-15	13-24
Filler Cement	200	2-6	7-14
	Bitumen	4.5-6.5	5.0-8.0

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0
10
20
30
40
50
60
70
80
90
100%
38mm - 63.5mm Hot Rolled
Asphalt Binder Course
(Flexible)
min
max
Curve No A
Curve No E
Curve No G
200
100
52
25
14
10
7
BS SIEVE NUMBER
¼" (6.0)
⅜" (9.57)
½" (12.5) mm
¾" (19.56) mm
1" (25) mm
1½" (38) mm

in air and water, and immersing the specimens in water bath where they were kept for about 30-40 minutes. The water bath was kept at the desired testing temperature. Then the specimens were removed, carefully dried and placed between the upper and lower testing heads of the marshall apparatus (see plate 3.1 The assembled testing head was then positioned and centred in the loading device and load was applied at constant strain rate of 51mm/min until failure occurred. Both readings corresponding to maximum load (stability) and deformation (Marshall Flow) at failure were taken and recorded. The procedure was repeated until all the specimens were tested.

b) Indirect tesile test:- The Hondros (1959) version of indirect tensile test was adopted for this work according to the procedure suggested by Lottman (1978), Mawpin (1979) and Kandhal (1979). Marshall specimens were used for the tests and equipment used was the CBR machine as previously explained in chapter 2, the test involved the use of a curved loading strip of 1.3cm width, made of stainless steel material. The strip was placed and positioned on the marshall specimen such that the loading device of the CBR machine was centred on the curved strip. The specimen was then loaded at a constant strain rate of

2 rev/s (18.9mm/min) manually, via the loading strip. The compressive load was continuously increased until failure occurred. Both readings of Compressive load and Vertical deformation corresponding to the failure stage were recorded. It was however not possible to take readings of Horizontal deformation due to the fact that it was the first time test of this nature was conducted with the machine.

3.3.2 TEST EQUIPMENTS

a) Marshall Tests:- The Standard Marshall testing machine was used. It consists of an electrically powered loading device designed to apply loads to test specimens through semi-circular Marshall stability testing heads. The machine operates at constant loading rate of 51mm/min, and is equipped with a calibrated proving ring which monitor the applied load. There is also another meter ring which monitor the amount of flow as the load was applied. Plate 3.1 illustrates the Marshall test equipments used.

b) Indirect tensile test: The ordinary CBR Compression machine shown on plate 3.6 was used. It has a proving ring having a loading capacity of 2.268 mg. The machine was capable of applying load at a maximum rate of 3 rev/s (28.3mm/min). However

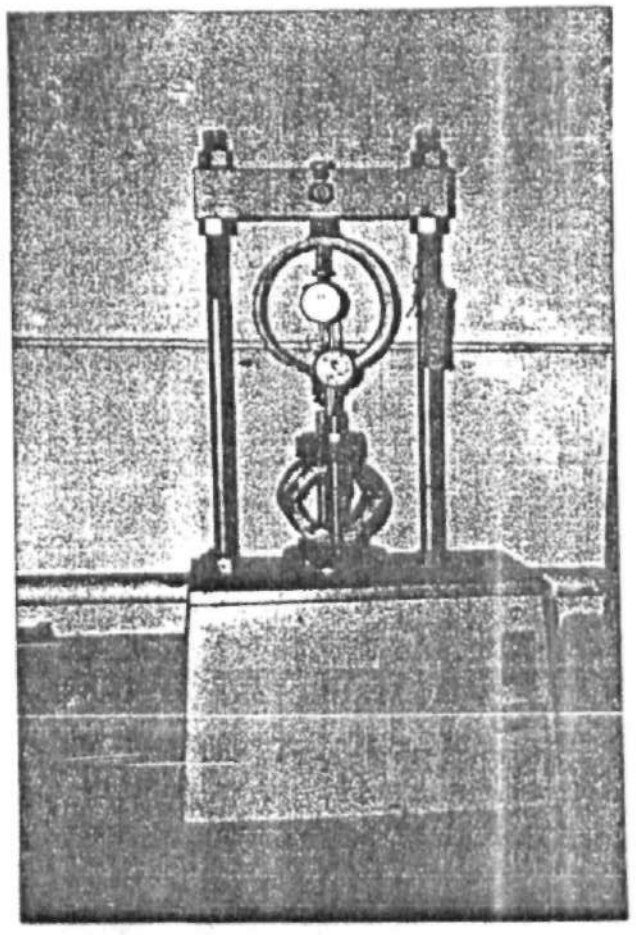


PLATE 3.1 - MARSHALL TESTING MACHINE

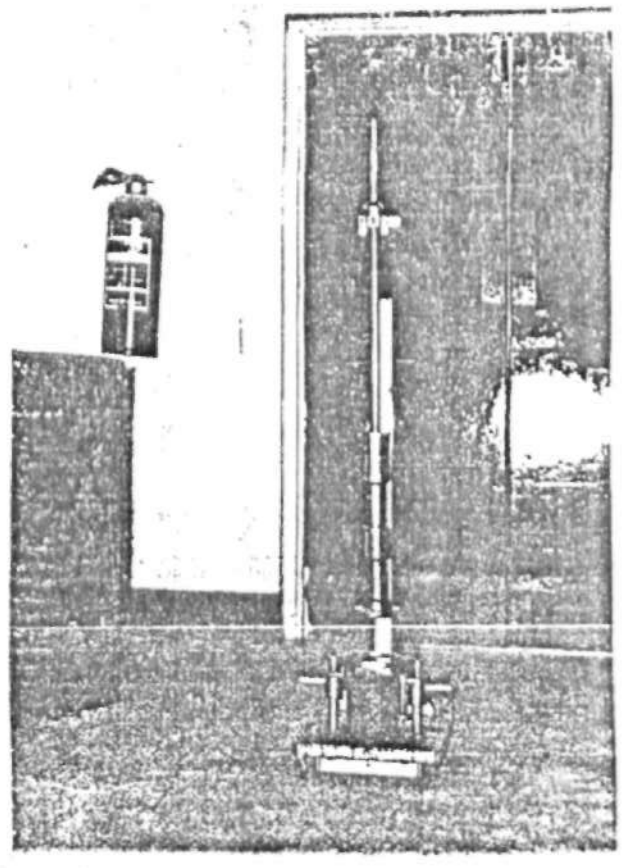


PLATE 3.2 - MARSHALL COMPACTOR

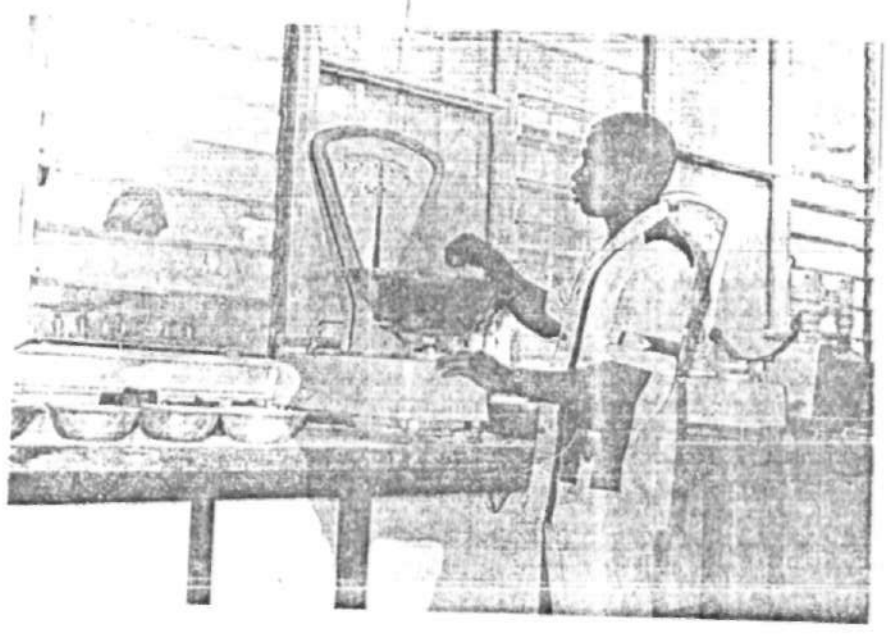
only a steady rate of 2 rev/s (18.8mm/min) was maintained. As per the requirement of the test two curved stainless steel strips of 1.3cm width were used. The machine had two attachable dial gauges for reading Compressive load and vertical deformation at failure.

3.4 SPECIMEN TREATMENTS

The preparation of tests specimens were done according to the Asphalt Institute (1969) recommended method for preparing marshall specimens. The stages involved were as follows:-

a) Preparation of Aggregates: The aggregates were first screened and seperated by dry sieving into the desired size fractions from BS Sieve No 38mm mesh down to No 200 mesh.

b) Preparation of Mixtures:- The various sizes of aggregates were weighed into 4 seperate pans according to the aggregate size distribution of the particular mix. Filler and binder (bitumen) were later weighed to the requirement of the mix and added to the contents of the pans. The pans with their contents were then taken into the oven which was already maintained at 150°C and kept for initial 10-15 mins. Then the pans were brought out one after the other and the contents were



3.3 Weighing Process (of materials)



3.4. Compaction of the mix with Marshall compactor.

mixed before taken back to the oven for another 10-15 mins. The aggregates and the bituminous materials were again thoroughly mixed before transfering the mixture into the marshall mould.

c) Compaction of the specimens:- After the final mixing of the bituminous mixture, the entire batch was then introduced into the mould previously cleared and assembled. Then the compacting hammer was mounted on the compaction device and the mixture given the desired number of blows on each face. The normal number of blows on each face was 50 except for specimens for which compaction was a variable and varied at 35, 50 and 75 blows. The compaction was done by manner of free fall from a height of 18 in (45.7cm). Then the compacted specimen was taken into a cold water bath for cooling the mould and it's contents. It was then extracted out of the mould and transferred to a smooth flat surface where it was allowed to stand overnight at room temperature. The samples were then levelled and weighed both in air and water after which they were then subjected to testing.

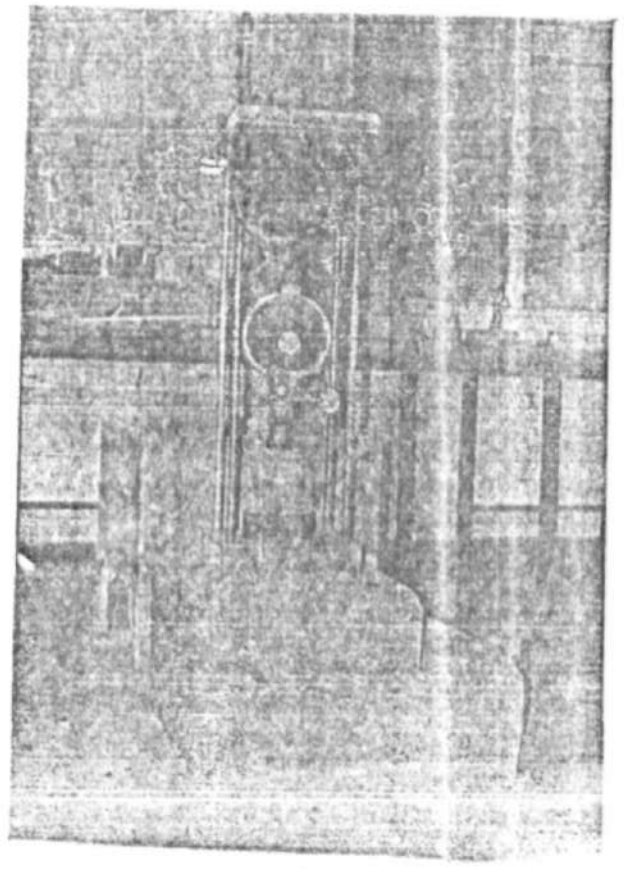


Plate 3.5 Combined mixture in an oven (open)

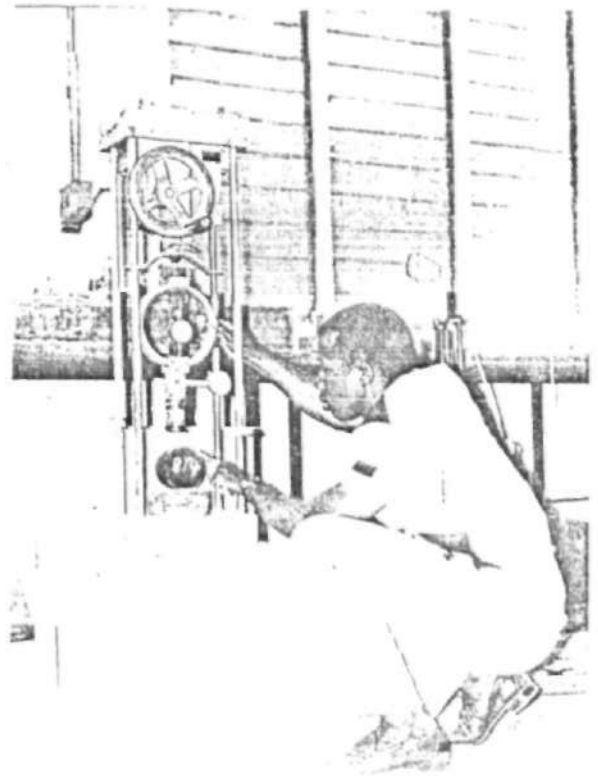


Plate 3.6 Heating at regular temperature (oven closed)

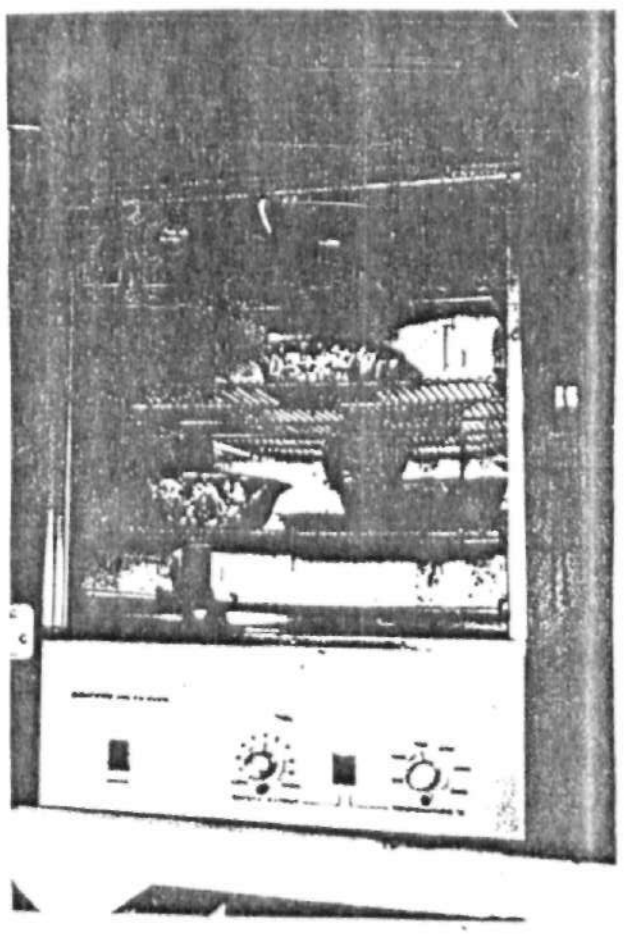


PLATE 3.7 - Ordinary CBR machine (used for indirect tensile test)

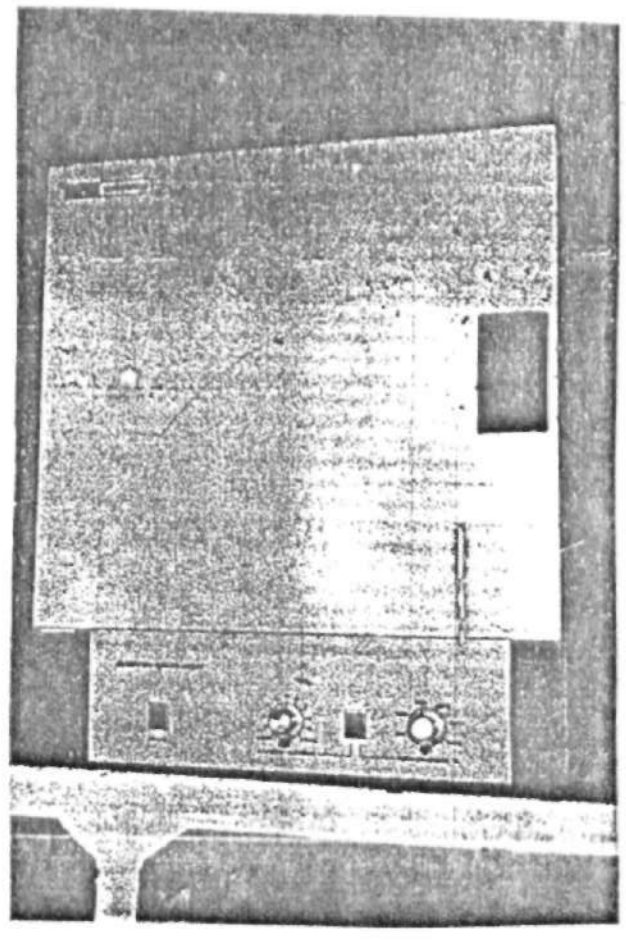
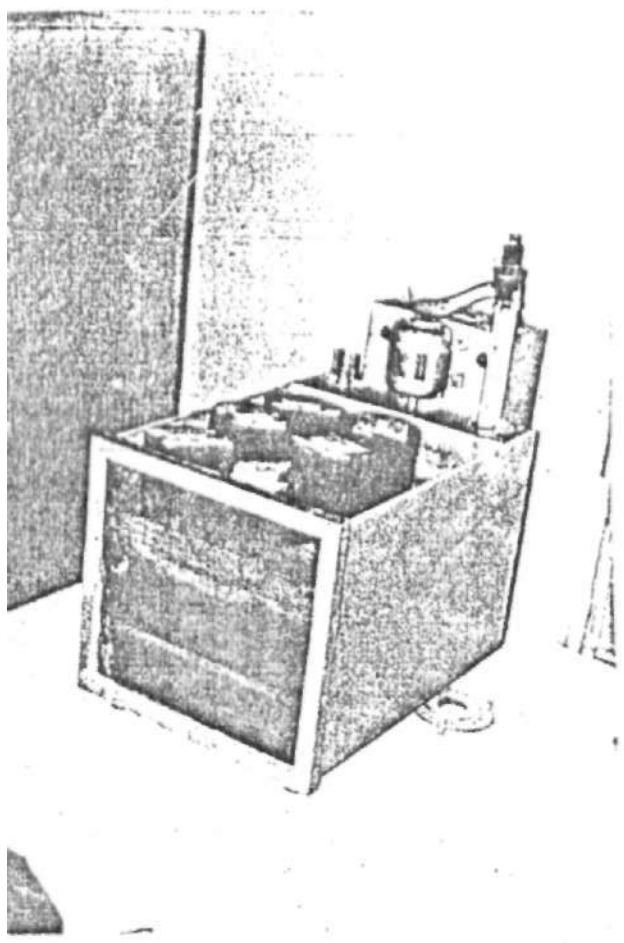


PLATE 3.8 - Centering of the specimen during loading



TE 3.9 - SPECIMENS IN WATER HEATER



Plate 3.10 Samples of the Marshall specimens.

3.5 SPECIMEN CHARACTERISTICS

Although the specimens were cast to have a predetermined marshall dimensions of 101.6mm (4in) diameter and height of 63.5mm (2½in), the result obtained showed that the specimen height ranged from 56mm to 74.8mm. Consequently the weight also varied from 1057g to 1204g in air and 548g to 678g in water. However for all the specimens the diameter was nominally at 101.6mm (4in). The result of bulk density determination also showed some variation from one specimen to another even for those of the same mix. The bulk density determination was done before the specimens were subjected to testing. The mean values of the compacted density of the mix, CDM; compacted density of the mixed aggregate CDMA; voids in the mix and mixed aggregates (VIM and VMA); and their respective coefficient of variation are summarised in table 3.7.1. The results of the individual specimens are presented in Appendices A through E.

The result of Marshall and Indirect tensile tests are contained in table 3.7.2 and 3.7.3 respectively. From the result it can be seen that the Coefficient of variation of the Marshall stability ranged from 16.2%-26.9 that of flow value ranged from 2.23%-2.63% while that of Marshall stiffness from 8.5%-24.6%. These ranges of

TABLE 3.7.1 RESULTS OF BULK DENSITY DETERMINATION

STONE CONTENT (%)	BINDER CONTENT (%)	CDM		SGM		VIM		VMA		VFB	
		mean	CV	mean	CV	Mean	CV	Mean	CV	mean	CV
		Kg/m^3	%	-	%	%	%	%	%	%	%
35	4	2110	1.4	2.53	-	16.7	7.1	25.8	3.6	3.6	5.6
	5	2150	0.9	2.49	-	13.8	4.4	25.4	2.2	46	3.3
	6	2700	1.8	2.45	-	10.3	14.9	24.3	5.7	58	6.6
	7	2200	0.9	2.41	-	8.9	11.1	25.1	3.5	65	3.7
	8	2180	1.9	2.34	-	6.7	26.6	27.1	5.4	75	6.9
50	4	2180	1.4	2.56	-	15.0	8.1	24.6	6.0	39	3.2
	5	2240	1.2	2.51	-	10.9	9.7	22.9	4.2	53	4.8
	6	2280	0.6	2.47	-	7.6	6.7	22.4	3.0	66	2.5
	7	2300	1.1	2.43	-	5.4	19.8	22.5	4.2	76	4.9
	8	2280	0.9	2.39	-	4.7	18.3	24.0	3.1	81	3.7
60	3	2210	0.5	2.61	-	15.2	2.4	22.4	1.5	22	1.9
	4	2250	0.4	2.56	-	12.3	1.9	21.9	1.0	44	1.2
	5	2280	0.4	2.51	-	9.1	2.7	21.4	1.4	57	1.6
	6	2350	0.4	2.49	-	4.9	11.7	19.9	2.6	76	3.0
	7	2340	0.9	2.43	-	3.6	23.6	21.0	2.8	83	4.3
70	4	2340	0.7	2.58	-	9.1	7.6	18.9	3.3	52	4.0
	5	2330	0.7	2.53	-	7.8	8.8	20.9	2.4	62	4.3
	6	2350	0.6	2.49	-	5.7	10.5	20.6	2.7	72	2.9
	7	2300	0.6	2.44	-	5.8	10.7	23.0	2.3	75	2.5
	8	2340	0.3	2.40	-	2.5	13.1	21.6	1.3	88	1.4

3.72. MARSHAL TEST SPECIMEN CHARACTERISTICS

STONE CONTENT (%)	BINDER CONTENT %w	STABILITY		FLOON		MARSHALL	STIFFNESS
		Mean	CV	Mean	CV	Mean	CV
		Kg	%	mm	%	Kg/mm	%
35	4	775	6.8	2.7	6.4	291	8.6
	5	923	10.8	2.5	12.7	441	21.1
	6	865	31.3	2.2	4.3	393	34.4
	7	836	18.3	2.2	6.0	392	21.9
	8	676	16.4	2.2	4.4	311	16.1
50	4	550	18.1	2.4	5.3	231	13.4
	5	813	26.9	2.3	3.6	353	24.6
	6	981	18.1	2.2	4.3	489	15.1
	7	917	19.8	2.3	4.2	403	20.8
	8	733	16.2	2.6	12.2	272	8.50
60	3	709	18.7	2.3	0.3	309	11.4
	4	846	9.7	2.5	0.3	355	14.1
	5	1044	16.7	2.6	0.5	425	30.9
	6	1296	16.2	2.8	0.2	475	30.9
	7	1130	16.5	2.9	0.3	394	18.3
70	4	671	10.4	2.1	3.9	321	13.8
	5	805	10.6	2.0	4.7	398	9.3
	6	983	9.4	2.5	17.2	394	8.4
	7	841	7.1	2.8	7.9	307	6.3
	8	778	6.9	2.9	4.5	273	9.5

3.7.3 INDIRECT TENSILE TEST SPECIMEN CHARACTERISTICS

BINDER CONTENT (%)	APPLIED LOAD TO FAILURE KN		VERTICAL DEFORMATION TO FAILURE (mm)		TENSILE STRENGTH ST (KN/m^2)		COMPRESSIVE MODULUS ST (KN/m^2)	
	MEAN	CV	MEAN	CV	MEAN	CV	MEAN	CV
4	1.72	14.7	2.4	8.1	155	17.7	38.170	17.6
5	1.91	8.3	2.7	11.3	181	10.1	40.699	9.0
6	2.19	12.5	2.9	10.4	218	19.6	42.906	9.7
7	2.44	7.4	3.2	12.4	242	9.5	44.948	21.4
8	2.14	2.5	4.6	8.0	214	3.1	17.055	12.5

For number of specimens were cast for each binder content.

values are quite in order due to the inherent variations associated with asphalt mixtures. Moreso when they are quite comparable with the result of previous investigators who reported even high values. Similarly the Coefficient of variation of the tensile properties are u quite reasonable compared to the ones obtained by Kandhal (1979) and Emina (1985).

CHAPTER 4

DATA ANALYSIS AND DISCUSSION OF RESULTS

The discussion of the experimental test-results is basedon the observed effects of the considered variables on the Asphaltic concrete mixes prepared for this research work. The variables considered included the Aggregate gradation, aggregate shape and surface texture, binder content, testing temperature and Compaction. Below are the observations made:-

4.1 EFFECT OF AGGREGATE GRADATION (STONE CONTENT)

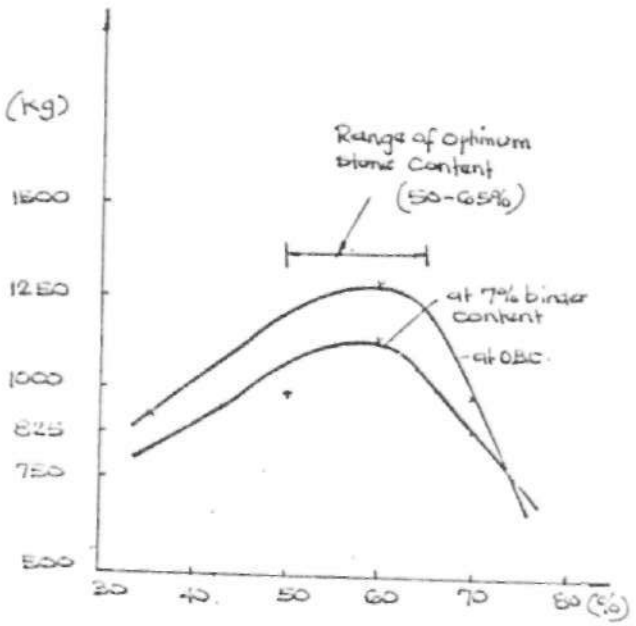
a) On Marshall Stability:- From the results contained in Table 4.1.1 and shown on Fig. 4.1.1, it can be seen that stability of Asphaltic concrete is dependent upon the stone content of the mixtures. Increasing the stone content from 35% to 60% increased the maximum stability of the asphalt concrete from 993 kg to 1296 kg respectively. However further increment to 70% resulted in decrease of the maximum stability value to about 983kg, about the same value obtained with 35% stone content. This shows that there is an optimum value of the Stone content for Maximum stability. This can easily be seen clearly in Fig. 4.1.2 where the range of optimum stone content is shown to be between 50% to 65%. This agrees with the results obtained by previous investigators according to Brien (1978), that the optimum stone Content will be around 55%. It can also be noted from Fig. 4.1.1 that

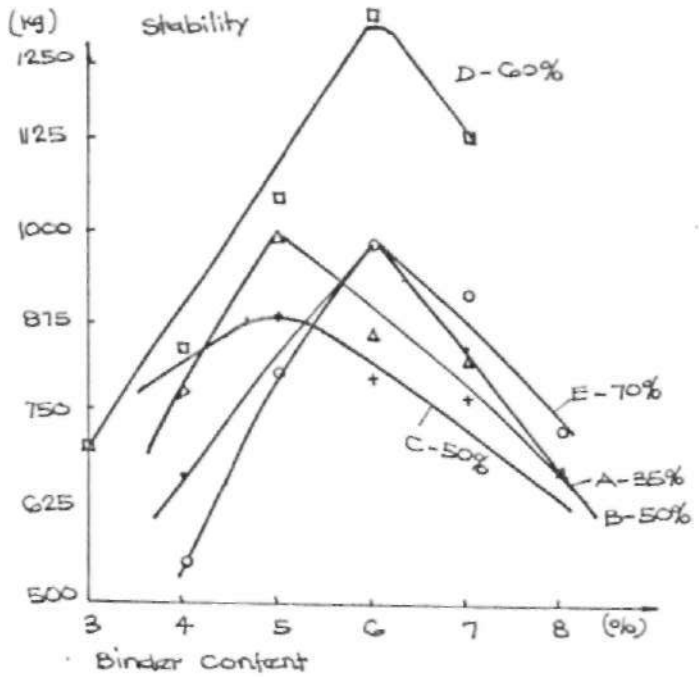
TABLE 4.1.1 RESULT OF MARSHALL STABILITY VALUES OF THE FOUR AGGREGATE GRADATIONS CONSIDE

Binder Content (% by wt of the mix)	Stone content (% by wt of the Aggregate)							
	35%		50%		60%		70%	
	Mean kg/m	Cv %	Mean kg/m	Cv %	Mean kg/m	Cv %	Mean kg/m	Cv %
3	-	-	-	-	709	18.8	-	-
4	775	6.8	550	18.1	846	9.7	671	10.4
5	995	10.8	813	26.9	1044	16.7	805	10.6
6	865	31.3	981	18.1	1296	16.2	983	9.4
7	856	10.7	917	19.8	1150	16.5	841	7.1
8	676	16.4	739	16.2	-	-	778	6.9

*Four number of specimen were cast for each mix.

Stability


(kg)
1500
1250
1000
875
750
500
Range of optimum Stone Content (50-65%)
at 7% binder content
at OBC
20
40
50
60
70
80 (%)
Stone Content


(kg)
Stability
1250
1125
1000
875
750
625
500
D-60%
E-70%
C-50%
A-35%
B-50%
3
4
5
6
7
8
(%)
Binder Content

optimum binder content varies with varying stone content. A value of about 5% was obtained as the optimum binder content for mixes made of 35% Stone Content while about 6% was obtained for mixes of higher Stone Contents of 50%, 60% and 78%.

b) On Marshall Stiffness:- From the results presented in table 4.1.2 and the details on Fig. 4.1.3, it can be concluded that the optimum Stone Content is about 60% giving the highest Marshall Stiffness Value of 480Kg/mm at a Binder Content of 6%. This observation is clearly shown in Fig. 4.1.4 where the range of optimum value is obtained as between 45% and 60% Stone Content.

Also the optimum binder content for the marshall stiffness varies as the stone Content varies. This shows the dependancy of the binder content on the aggregate gradation. For this particular research work a binder content of 6% is obtained to give maximum Marshall stiffness for mixes made of 45% to 60% Stone Contents, as demonstrated on Fig. 4.1.3.

c) On Compacted density of the mix, CDM:- From the results presented in table 4.1.3 and shown on Fig. 4.1.5, it is indicative that any increase in the stone Content from 35% will increase the Compacted density of the mix until an optimum value of about 60% is

TABLE 4.1.2: RESULT OF MARSHALL STIFFNESS OF THE FOUR CONSIDERED AGGREGATE GRADATIONS

Binder Content % by wt of the mix	Stone content (% by wt of the Aggregate)							
	35%		50%		60%		70%	
	Mean kg/m	Cv %	Mean kg/m	Cv %	Mean kg/m	Cv %	Mean kg/m	Cv %
3	-	-	-	-	309	11.4	-	-
4	291	8.6	231	13.4	355	14.1	321	13.8
5	414	21.1	353	24.6	425	30.9	398	9.3
6	393	34.4	439	15.1	475	19.8	394	8.4
7	392	21.9	403	20.8	394	18.3	307	6.3
8	311	16.1	279	8.5	-	-	273	9.3

*The mean value is for four no of specimen cast for each specimen

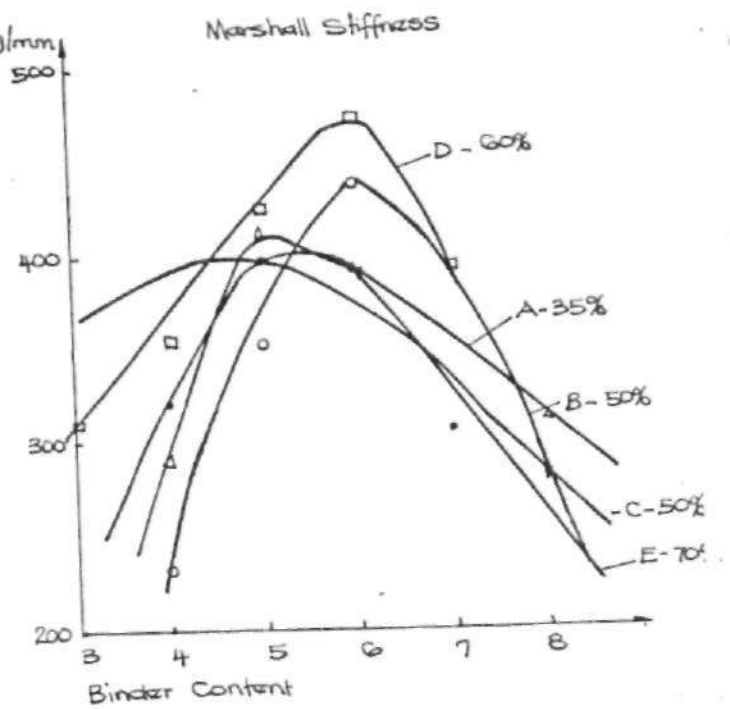


Fig. 4.1.3 - Relationship between Marshall stiffness, stone content and binder content.

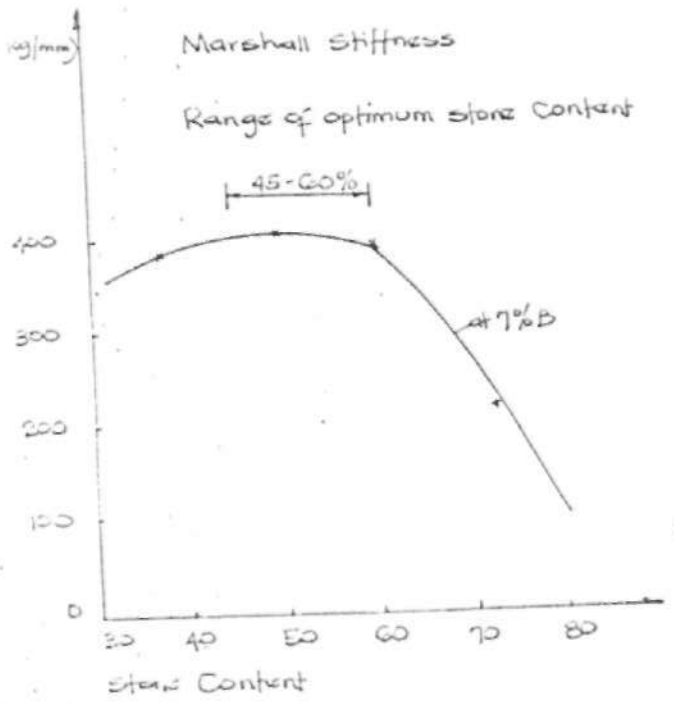


Fig. 4.1.4. - Relationship between Marshall stiffness and stone content

TABLE 4.1.3: RESULT OF COMPACTED DENSITIES OF THE MIXES OF THE FOUR AGGREGATE GRADATION CONSIDERED

Binder Content % by wt of the mix	Stone content (% by wt of the Aggregate)							
	35%		50%		60%		70%	
	Mean	Cv	Mean	Cv	Mean	Cv	Mean	Cv
	kg/m^3	%	kg/m^3	%	kg/m^3	%	kg/m^3	%
3	-	-	-	-	2210	0.5	-	-
4	2110	0.03	2180	1.4	2250	0.4	2340	0.7
5	2150	0.02	2240	1.2	2280	0.4	2330	0.7
6	2200	0.04	2280	0.6	2350	0.4	2350	0.6
7	2200	0.02	2300	1.1	2340	0.9	2300	0.6
8	2180	0.04	2280	0.9	-	-	2340	0.3

*The mean value is for four no of specimen cast for each mix.

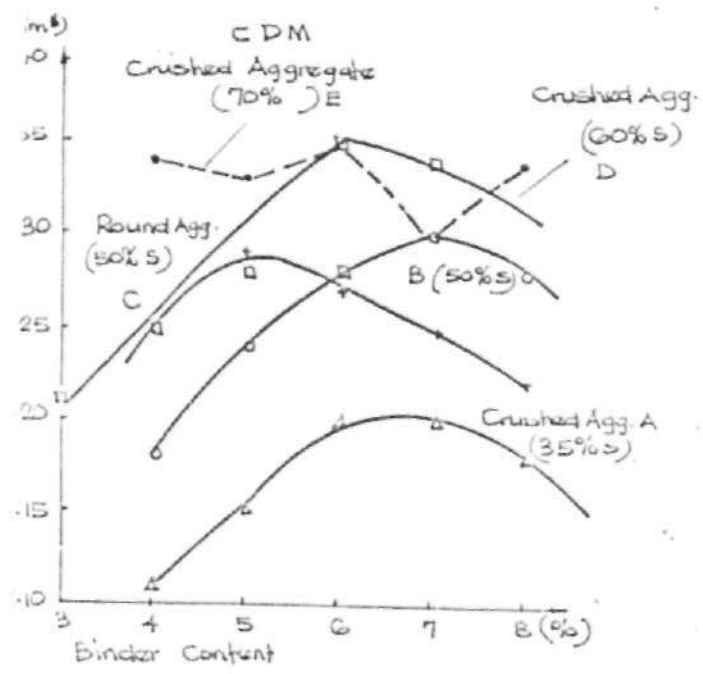


Fig. 4.1.5 - Relationship between compacted density of the mix, stone content and binder content.

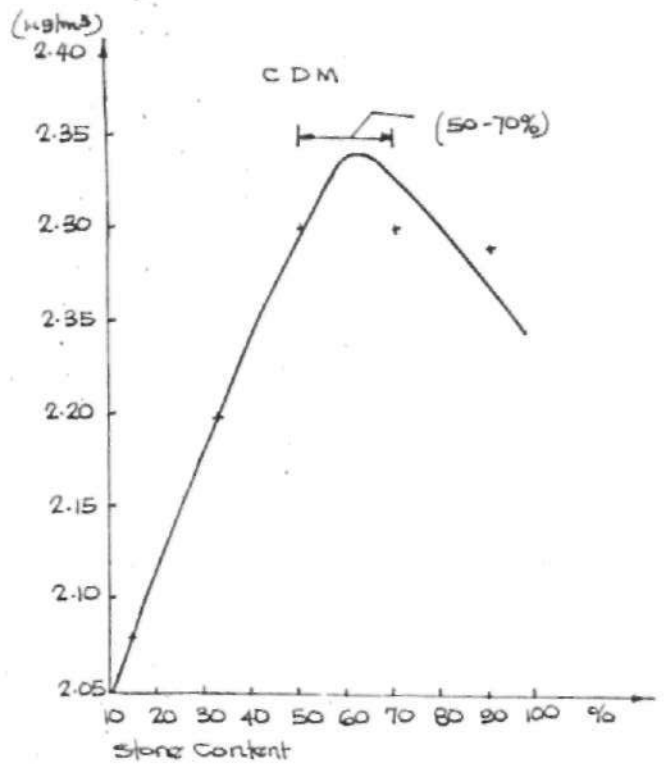


Fig. 4.1.6 - Relationship between compacted density of the mix and stone content.

reached. Also on Fig. 4.1.6 the range of optimum Stone Content is given as between 55% and 65% for maximum compacted density. It can also be noted in Fig 4.1.5 that with 70% Stone Content the value of CDM is irregular indicating (possibly) difficulty in compaction at higher stone Contents.

d) On Voids in the Mix, VIM:- The voids in the mix is obtained to decrease as the Stone Content increased as shown on Fig 4.1.7. This result is expected as VIM is dependent upon the Compacted density of the mix which in turn is dependent upon the Stone Content. From Fig 4.1.5 we observed that CDM increases with increase in Stone Content, while from Fig. 4.1.7 VIm decreases. Therefore VIM is inversely proportional to the CDM as indicated by the following standard formula:

$$VIM = \frac{SGM - CDM}{SGM}$$

However it was observed that beyond 60% Stone Content the VIM starts to increase with increase in Stone Content values corresponding to lowest VIM of about 50% to 65% as illustrated on Fig. 4.1.8.

TABLE 4.1.4: RESULT OF AIR VOIDS (VOIDS IN THE MIX) OF THE FOUR AGGREGATE GRADATION CONSIDERED

Binder Content % by wt of the mix	Stone content (% by wt of the Aggregate)							
	35%		50%		60%		70%	
	Mean	Cv	Mean	Cv	Mean	Cv	Mean	Cv
	%	%	%	%	%	%	%	%
3	-	-	-	-	15.2	2.4	-	-
4	16.7	7.1	15.0	8.1	12.3	1.9	9.1	7.6
5	13.8	4.4	10.9	9.7	9.1	2.7	7.8	8.0
6	10.3	14.9	7.6	6.7	4.9	11.7	5.7	10.5
7	8.9	11.1	5.4	19.8	3.6	23.6	5.8	10.7
8	6.7	26.6	4.7	18.3	-	-	2.5	13.1

*The mean value is taken for four number of specimen for each mix.

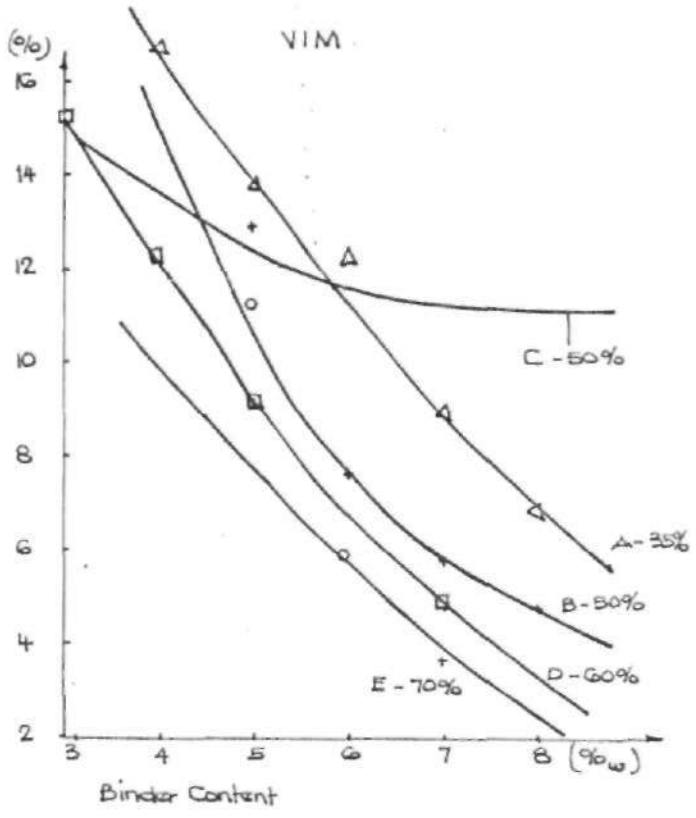


Fig. 4.1.7 - Relationship between VIM, stone content and binder content.

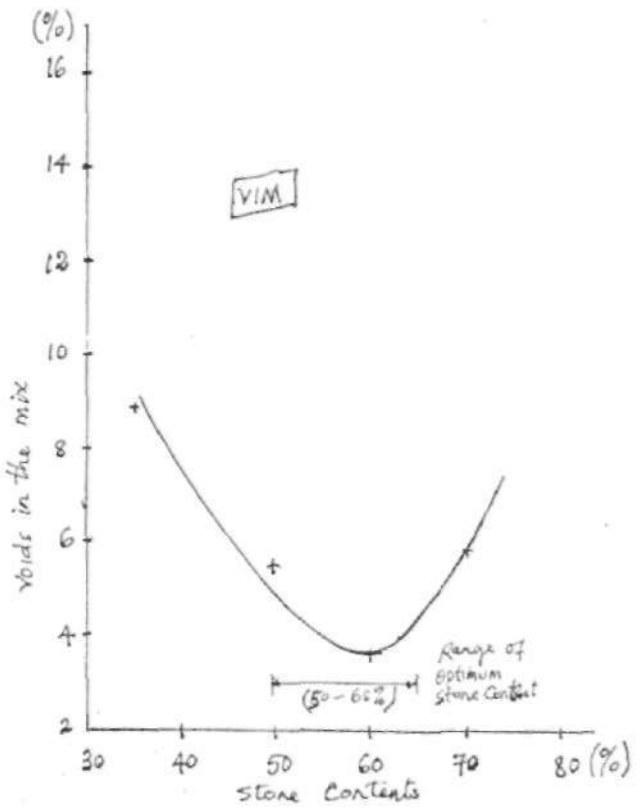


Fig. 4.1.8 - Relationship between VIM and stone content

e) On Voids in the Mixed Aggregate. VMA:- From the results contained in table 4.1.5 and shown on fig. 4.1.9, it can be noted that an initial decrease in Stone Content Causes decrease in VMA, until a minimum value is obtained at a range between 50% to 65% Stone Contents. The VMA started to rise again with further increase in Stone Content. This agrees with the findings of Jackson and Brien (1962) that adding sand to coarse aggregate reduces the voids in the mixed aggregates until so much sand is added that it pushes the stone apart and causes the voids to rise again.

f) On Voids filled with bitumen (VFB): From the results contained in table 4.1.6 and shown on Fig. 4.1.10, it can be seen that, increasing the stone content from 35% to 60% results in more voids being filled with bitumen. However further increase of stone content to 78% saw the decrease in the VFB. This indicates that there is also an optimum value for stone content at which the VFB will be maximum. A range between 55% to 65% have been obtained in Fig. 4.1.11 for the optimum stone content corresponding to max. VFB.

4.2 EFFECT OF AGGREGATE SURFACE TEXTURE AND SHAPE

a) On marshall stability: From the results presented in table 4.2.1 and shown on fig. 4.1.1, it can be seen

TABLE 4.1.5: RESULTS OF AIR VOIDS IN THE MIXED AGGREGATES OF THE FOUR AGGREGATE GRADATIONS CONSIDERED (VMA)

Binder Content % by wt of the mix	Store content (% by wt of the Aggregate)							
	35%		50%		60%		70%	
	Mean	Cv	Mean	Cv	Mean	Cv	Mean	Cv
	%	%	%	%	%	%	%	%
3	-	-	-	-	22	1.5	-	-
4	26	3.6	25	6.0	22	1.0	19	3.3
5	25	2.2	23	4.2	22	1.4	20	2.4
6	24	5.7	22	3.0	20	2.6	21	2.7
7	25	3.5	23	4.2	21	2.8	23	2.3
8	27	5.4	24	3.1	-	-	22	1.3

*The mean value is for four number of specimen for each mix.

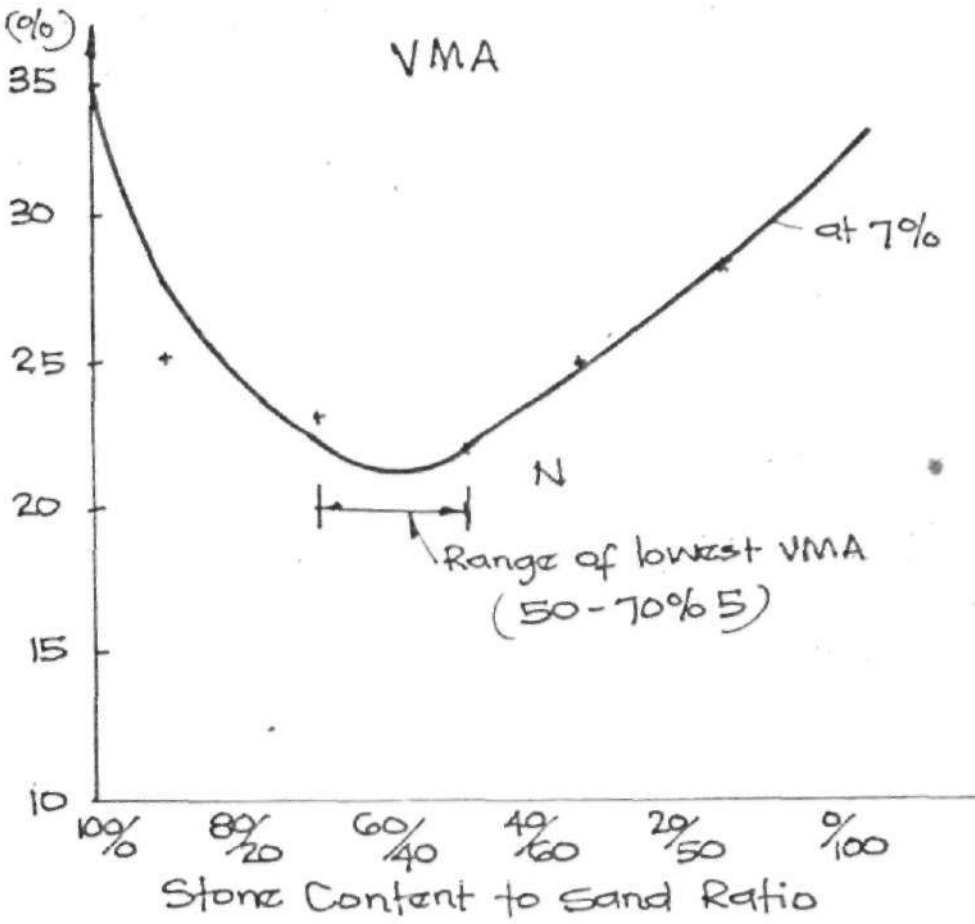


Fig. 4.1.9 - Relationship between VMA and stone content.

TABLE 4.1.6: RESULTS OF VOIDS FILLED WITH BITUMEN OF THE FOUR AGGREGATE GRADATION CONSIDERED

Binder Content % by wt of the mix	Store content (% by wt of the Aggregate)							
	35%		50%		60%		70%	
	Mean %	Cv %	Mean %	Cv %	Mean %	Cv %	Mean %	Cv %
3	-	-	-	-	32	1.9	-	-
4	36	5.6	39	3.2	44	1.2	52	4.0
5	46	3.3	52	4.8	57	1.6	62	4.3
6	58	6.6	66	2.5	76	3.0	72	2.9
7	65	3.7	76	4.9	83	4.3	75	2.5
8	75	6.9	81	3.8	-	-	88	1.4

*The mean values are for four number of specimen for each mix.

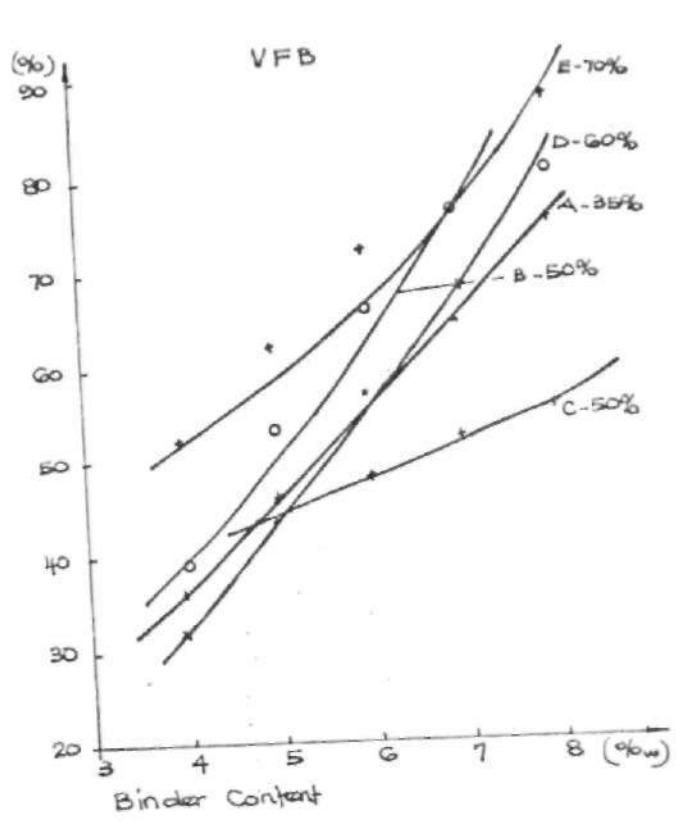


Fig. 4.1.10 - Relationship between VFB, stone content and binder content.

Voids Filled with Bitumen.

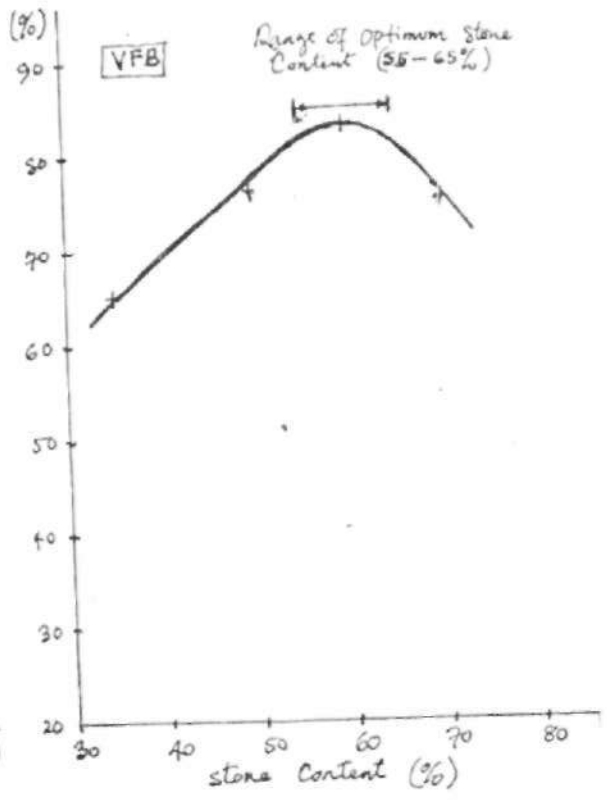


Fig. 4.1.11 - Relationship between VFB and stone content

TABLE 4.2.1 COMPARISON OF EFFECTS OF AGGREGATE SHAPE AND TEXTURE BETWEEN ROUGH TEXTURED CRUSHED AGGREGATE AND SMOOTH TEXTURED ROUND AGGREGATE

AGGREGATE TYPE SHAPE TYPE	PERFORMANCE PARAMETERS								
	maximum Marshall stability	Maximum marshall stiffness	Flow value @ O.B.C	Optimum binder content (O.B.C)	CDM @ O.B.C	CDMA @ O.B.C	VIM @ O.B.C.	VMA @ O.B.C.	VFB @ O.B.C
	(kg)	kg/mm	mm	%	kg/m^3	kg/m^3	%	%	%
CRUSHED AND ANGULAR	981	439	2.28	7.0	2300	2140	5.4	22.5	76.3
UNCRUSHED AND ROUND	890	398	2.25	5.0	2290	2180	11.3	20.0	43.5

Note:- For both mixes have the same Aggregate Gradation Curve No. 3, (Fig. 3.4.1 and 2) of 50% stone content.

that the maximum stability obtained for rough textured crushed aggregate (curve B-50%) is higher than that obtained for smooth textured round aggregate (curve C-50%). About 981 kg was obtained as the maximum stability for the rough textured aggregate mixes against 890 kg for the mixes made with round aggregate of the same aggregate gradation and Stone content. This agrees with the findings of Jimenez (1962) that as long as there is sufficient asphalt (binder) in the mixture for the particular aggregate, rough textured aggregates tend to perform better than smooth textured aggregates for identical asphalt contents.

b) On marshall stiffness:- From table 4.2.1 and the details shown in Fig 4.1.3 it can be seen that the maximum marshall stability values are 439kg/mm and 398 kg/mm for mixes of crushed angular and round aggregates respectively. This is due to the interlocking properties of the rough textured aggregates.

c) On Marshall Flow:- From table 4.2.1 and fig. 4.1.12, it can be seen that the Flow values of the mixes made with the crushed aggregates have a smooth drought curve while that of round aggregate mixes is irregular. The irregularity of the flow of the round aggregate mixes can be due to the presence of some crushed aggregate mixed together with the round aggregates. The flow values of the smooth aggregates are higher than those

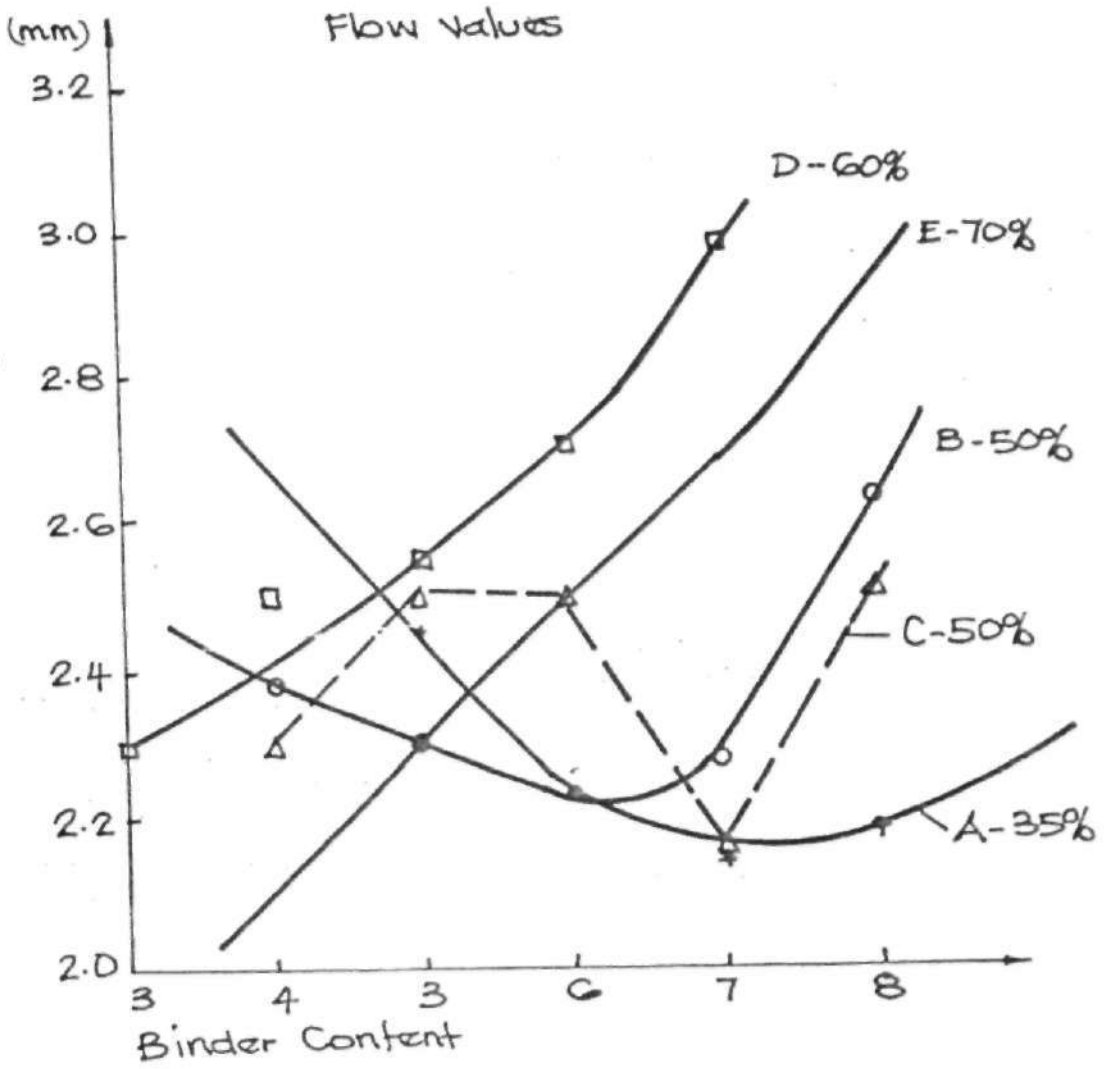


Fig. 4.1.12 - Relationship between Marshall flow values, stone content and binder content.

observed for rough textured aggregates mixes for binder contents upto 7%. This is because the aggregate-bitumen bond is much more stronger for rough surface than smooth textured aggregates. While smooth aggregates can slide over the bitumen on load application with ease, while the rough textured can not do so.

(d) On CDM:- From table 4.2.1 and Fig. 4.1.5 it can be seen that CDM of mixes made of crushed aggregate (2300kg/m^3) is only slightly higher than that made of round aggregate (2290 kg/m^3). Which suggest that though the density could be the same for to mixes of the two aggregates, the rough surface textured may perform better.

(e) On VIM:- From the results presented in table 4.2.1 and shown on fig. 4.1.7, it can be noted that the round smooth aggregate provided more air voids in the mix. This could be due to the interparticle arrangement which provided more larger spaces and tight contect which are not easily penetrated by bitumen in the case of round aggregates than the crushed angular type. It could be seen that even at 8% binder content the VIM of the round aggregates mixes is higher than 11% well above the Marshall design criteria of 2-5%.

f) On VFB: From results contained in table 4.2.1 and shown on fig 4.1.10 it can be observed that above 4% binder content, more air voids are filled with the bitumen for crushed aggregates mixes than those for round aggregates. This could be due to the interparticle arrangement which gives wider voids and tight inter particle-contact in the case of round aggregates while smaller voids which are easily penetrated by bitumen are obtained in the case of the crushed angular aggregates.

g) On Optimum binder content B: From the results contained in Table 4.2.1 and shown in Figs 4.1.1, 4.1.3 and 4.1.7 it can be seen that the optimum binder contents obtained for round aggregates mixes are much lower than those for crushed aggregate mixes. The average optimum binder content obtained were 5% and 7% for round aggregate and crushed aggregate mixes respectively. This agrees with the the observation made by Jimenez (1962) that Optimum asphalt content for fatigue resistance is of the order of 6% binder content for smooth round aggregates whereas that for the rough textured aggregate is 7.5%.

4.3 EFFECT OF BINDER CONTENT

a) On marshall stability:- From the results contained in table 4.3.1 and shown on fig. 4.3.1, it could be seen that the stability increases with increase of the binder content initially, until til it reaches a maximum value of about 981 kg at a binder content of about 6.2%. Further increase of binder content saw the decrease of stability values. This indicates that there is an optimum value of the binder content for the stability values.

b) On Marshall Flow Value:- The Flow value initially decreases with increasing binder content from 4% to 6% and then started increasing with further increase of binder content. From Fig. 4.3.2 it can be noted that there is an optimum binder content of about 6% for the minimum flow value.

c) On Marshall Stiffness: From the results contained in table 4.3.1, it can be observed that a similar behaviour is obtained as was the case with stability. An optimum value of 6.2% was obtained for maximum Marshall stiffness. Fig 4.3.3

d) On tensile Properties of the Mix: From the results contained in table 4.3.2 and shown on figs 4.3.4 and 4.3.5, it can be seen that the tensile strength and

TABLE 4.3.1: MARSHALL TEST RESULTS FOR MIXTURES OF SURFACE WEARING COURSE AGGREGATE GRADATION HAVING 50% STONE CONTENT

Binder Content % by WT. of the mix	Number of specimens cast.	PERFORMANCE PARAMETER					
		Marshall Stability (*)		Marshall Flow Value		Marshall Stiffness (Q_m)	
		Mean (kg)	CV (%)	Mean (mm)	CV (%)	Mean (Kg/mm)	CV (%)
4	4	550	18.1	2.38	3.3	231	13.4
5	4	813	26.9	2.30	3.6	353	24.6
6	4	981	18.1	2.23	4.3	439	15.1
7	4	917	19.8	2.28	4.2	403	20.8
8	4	733	16.2	2.63	12.2	279	8.5

*Corrected for thickness of specimen to give stability for a 63.5mm (2.5in) thick specimen.

TABLE 4.3.2 BULK DENSITY TEST FOR MIXTURES OF SURFACE WEARING COURSE AGGREGATE GRADATION HAVING 50% STONE CEMENT

Binder Content % by WT of the mix	Number of specimens cast*	PERFORMANCE PARAMETER					
		Compacted density of the mm, CDM		Voids in the mix (VIM)		Voids filled with bitumen (VFB)	
		Mean kg/m^3	CV %	Mean %	CV %	Mean %	CV %
4	4	2.18	1.4	15.0	8.1	39.0	3.2
5	4	2.24	1.2	10.9	9.7	52.6	4.8
6	4	2.28	0.6	7.6	6.7	65.1	2.5
7	4	2.30	1.1	5.4	19.8	76.3	4.9
8	4	2.28	0.9	4.7	15.3	80.5	3.7

*Each specimen was given 50 blows of Compaction hammer on each face.

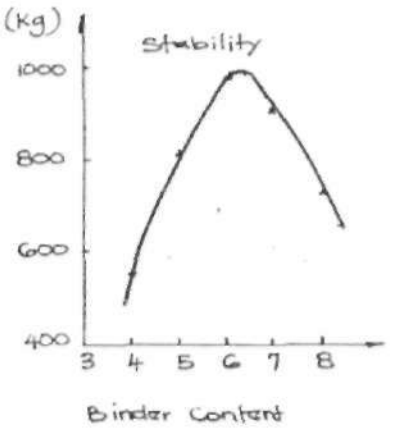


Fig.4.3.1- Relationship between stability and binder content.

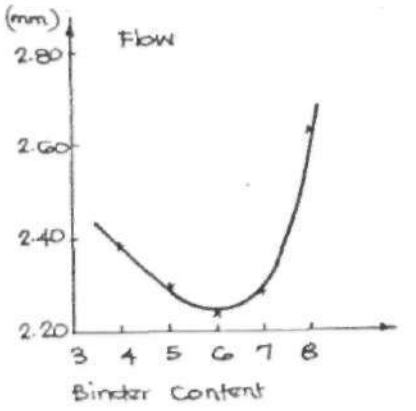
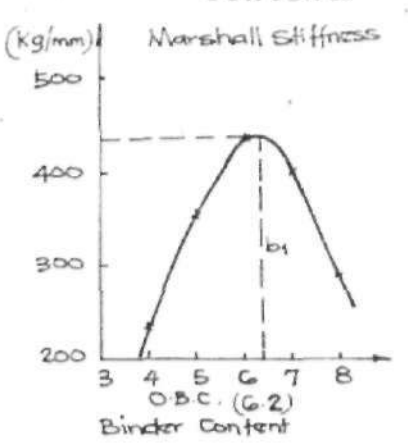


Fig.4.3.2.- Relationship between flow valves and binder content.



GFiv. 4.3.3- Relationship between marshall stiff-ness and binder content.

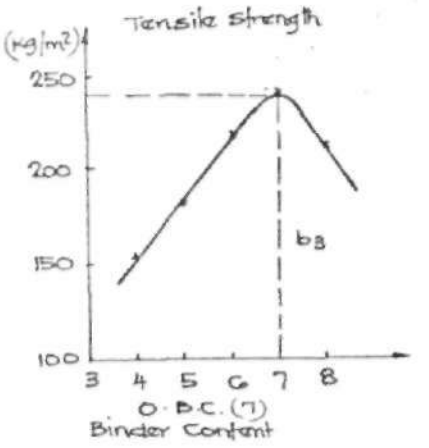


Fig.4.3.4- Relationship bt between Tensity strength of the mix and binder content

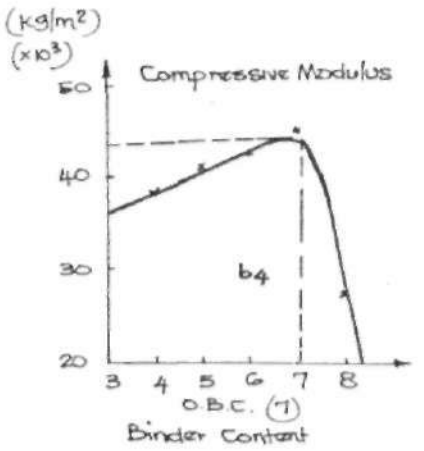


Fig. 4.3.5 - Relationship between compressive modulus and binder content.

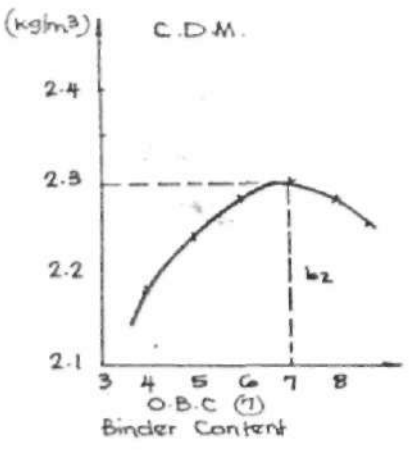


Fig. 4.3.6 - Relationship between CDM of the mix and binder content

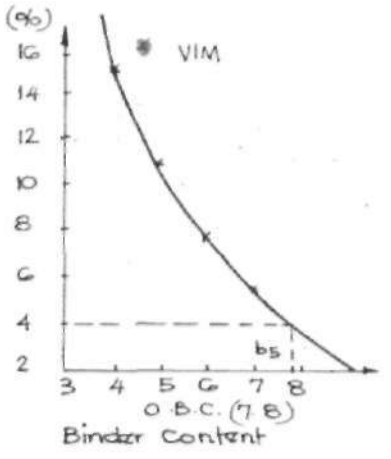


Fig. 4.3.7 - Relationship between VIM and binder content

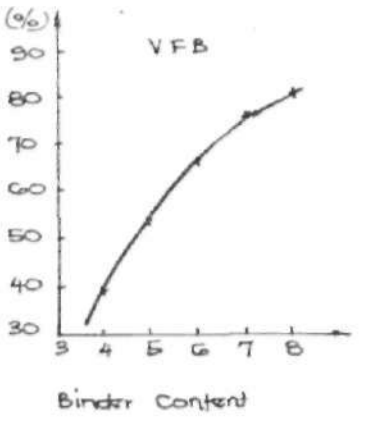


Fig. 4.3.8 - Relationship between VFB and binder content

Compressive Modulus of the asphaltic concrete initially increases and reaches maximum values at binder content of 7%. It then started deminishing with further increase of bitumen. It could be noted that higher binder content of 7% was obtained for tensile properties of the asphalt mixes than for marshall properties which was only 6.2%.

e) On CDM:- From the results contained in Table 4.3.3 and shown in Fig 4.3.6, it can be observed that compacted density initially increased with increasing binder content until it reaches a maximum value at an optimum binder content of about 7%. It then starts decreasing with further increase in binder content.

f) On VIM:- As observed from the results tabulated on table 4.3.3 and illustrated on Fig. 4.3.7, it can be seen that binder content increment reduce the amount of VIM. A value of about 7.5% binder content was obtained to corresponding with 4% VIM.

g) On VFB:- As expected the VFB increased with increase of binder content. About 76% of air voids was filled with bitumen at binder content of 7%. This can be seen on fig. 4.3.8.

h) ON VMA: From the results shown in Fig. 4.3.9, it can be observed that VMA reduces with increase in

TABLE 4.3.3: INDIRECT TENSILE TEST RESULTS FOR MIXTURES OF SURFACE WEARING COURSE AGGREGATE GRADATION HAVING 50% STONE CONTENT

Binder Content (% by wt of the mix)	Number of specimen test	PERFORMANCE PARAMETERS			
		Tensile Strength		Compressive modulus	
		Mean Kg/m^2	CV %	Mean Kg/m^2	%
4	4	155	17.7	38,270	17.6
5	4	181	10.1	40,699	9.0
6	4	218	19.6	42,906	9.7
7	4	242	9.5	44,948	21.4
8	4	214	3.1	27,055	12.5

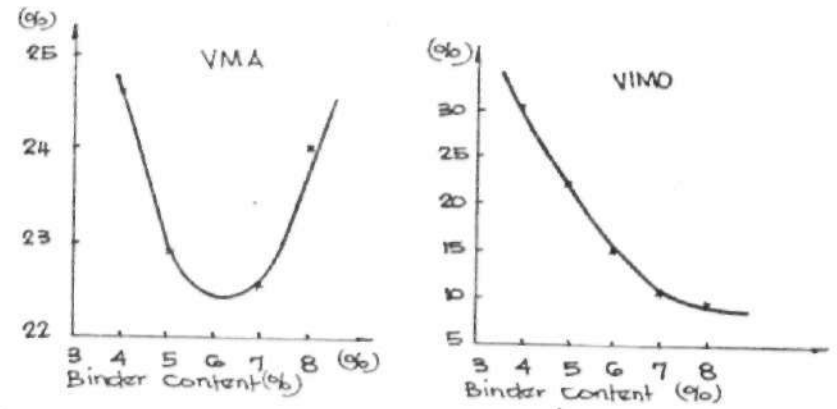
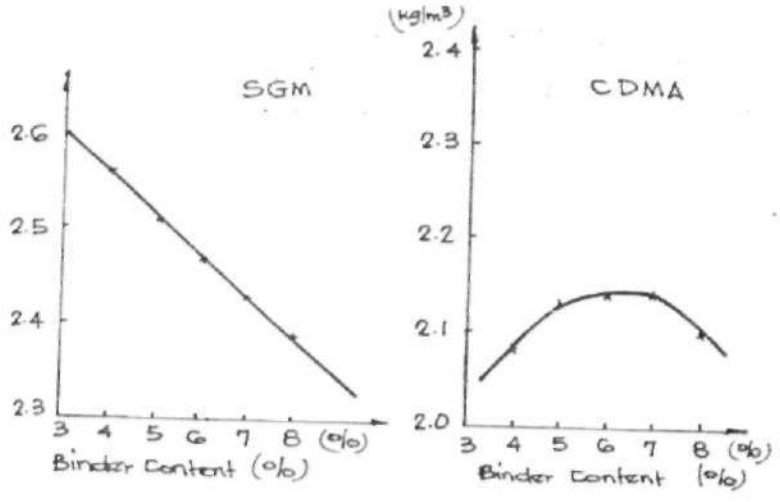


Fig.4.3.9- Relationship between VMA and binder content.



binder content initially until it reaches a lowest possible value at an optimum binder content of about 6.2%. It then increases with further increase of binder when the particles are pushed away by the bitumen to create more voids in the aggregates.

4.4 EFFECT OF TESTING TEMPERATURE AND COMPACTION

a) On Marshall Stability:- From the results contained in able 4.4.1 it can be noted that the higher the testing temperature the lower the stability will be, as exhibited by the downward trend of the temperatures curves on fig. 4.4.1. However increasing compactive effort resulted in increasing the stability values as shown on Fig. 4.4.1.

b) On Marshall Flow: From the results shown in Fig. 4.4.2, it can be seen that flow decreases as the testing temperature increases. It could also be observed that at low temperature of about 20°C, there is a linear relationship between the flow values and Compactive effort and that the Flow increases as the amount of compactive effort increases. However at a higher temperature of 80°C, there seems to be an optimum value of compactive effort of about 50 blows each face.

TABLE 4.4.1 EFFECT OF TEMPERATURE VARIATION AND COMPACTIVE EFFORT.

TESTING TEMPERATURE °C	NUMBER OF BLOWS TO EACH FACE (COMPACTION)	STABILITY		FLOW		MARSHALL STIGRNESS		Comm		VIM		VMA	VFB		
		mean	CV%	mean	CV	mean	CV	Mean	CV	mean	CV	mean	CV	mean	CV
		Kg	%	mm	%	Kg/mm	%	Kg/m^3	%	%	%	mean %	%	%	%
20	75	4470	0.4	6.20	1.6	724	1.9	2340	0.6	3.8	17.6	21	2.4	82	2.8
	50	4055	3.0	5.10	6.0	802	8.5	2310	0.2	4.8	4.2	22	0.0	79	1.2%
	35	3198	8.7	4.40	4.9	726	4.1	2320	1.0	4.8	21.0	22	5.0	79	4.0
40	75	2161	6.4	4.20	3.5	494	5.7	2340	0.2	3.8	5.2	21	0	82	0.6
	50	1804	9.0	3.8	5.0	481	9,0	2330	0.7	4.3	17.0	22	5.0	80	3.0
	35	1150	5.7	2.70	4.9	434	2.3	2300	0.7	5.5	12.0	23	3.0	76	3.0
60	75	1046	5.0	3.00	12.5	358	10.1	2330	0.2	4.0	5.0	21	0	82	0.6
	50	979	14.0	2.30	4.0	432	17.0	2300	1.1	5.4	20.0	23	6.0	77	5.0
	35	725	15.9	3.00	4.2	239	12.6	2280	0.6	6.4	8.0	24	3.0	73	2.0
80	75	656	10.0	2.20	5.8	301	6.8	2340	0.7	3.8	19.7	21	2.4	83	3.5
	50	376	18.0	2.4	9.0	156	19.0	2300	1.2	5.5	20.0	23	4.0	77	5.0
	35	265	17.6	2.20	6.0	124	18.6	2290	0.8	6.0	12	23	4.0	74	3.0

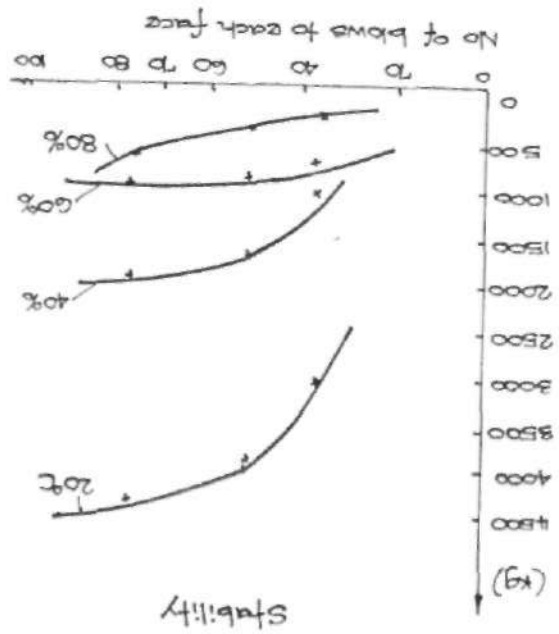


Fig. 4.4.1- Relationship between stability testing temperature and compactive effort.

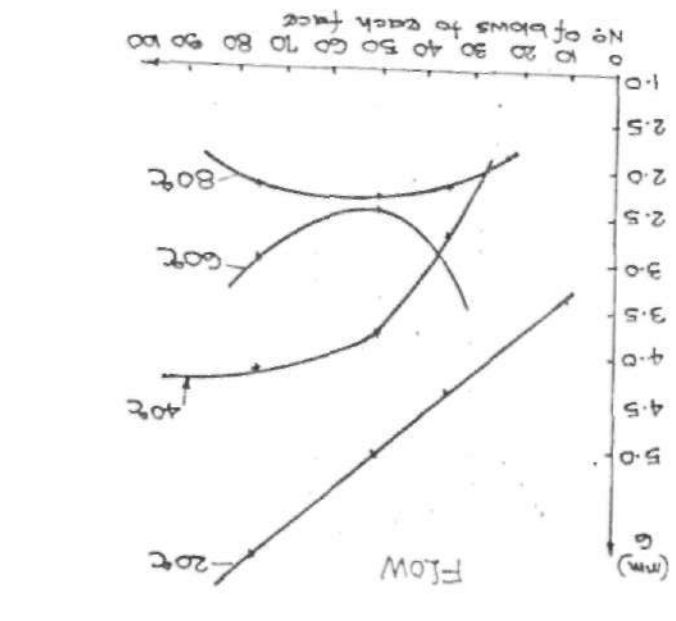


Fig. 4.4.2.- Relationship between flow values, testing temperature and compactive effort.

c) On Marshall Stiffness:- From Fig. 4.4.3 and Table 4.4.3, it can be observed that temperature and compactive effort affects marshall stiffness of asphaltic concrete in a similar manner as it did to it's stability, however in the case of the stiffness there seems to be an optimum value of the compactive effort. At low temperatures of 20°C to 60°C a compactive effort of 50 blows in each face seems to be the optimum value. It should be noted that the same optimum value for the compactive effort was obtained for flow at a higher temperature of 80°C.

d) On CDM:- From the results obtained in table 4.4.1, and shown on fig. 4.4.4, it can be noted that CDM of the Asphaltic concrete is directly proportional to the amount of compactive effort given to the mix. A straight line equation of the following order is obtained from the result so obtained:-

$$CDM = \frac{n}{800} + 2.25 \quad \ldots \quad \ldots \quad (1)$$

where CDM = Compacted density of the mix in g/cm^3.

n = number of hammer blows on each face.

Although, the result obtained may be perculiar to the particular mix of 50% Stone Content and binder content of 7%, it is indicative to the fact that compacted density of the mix is improved by increasing

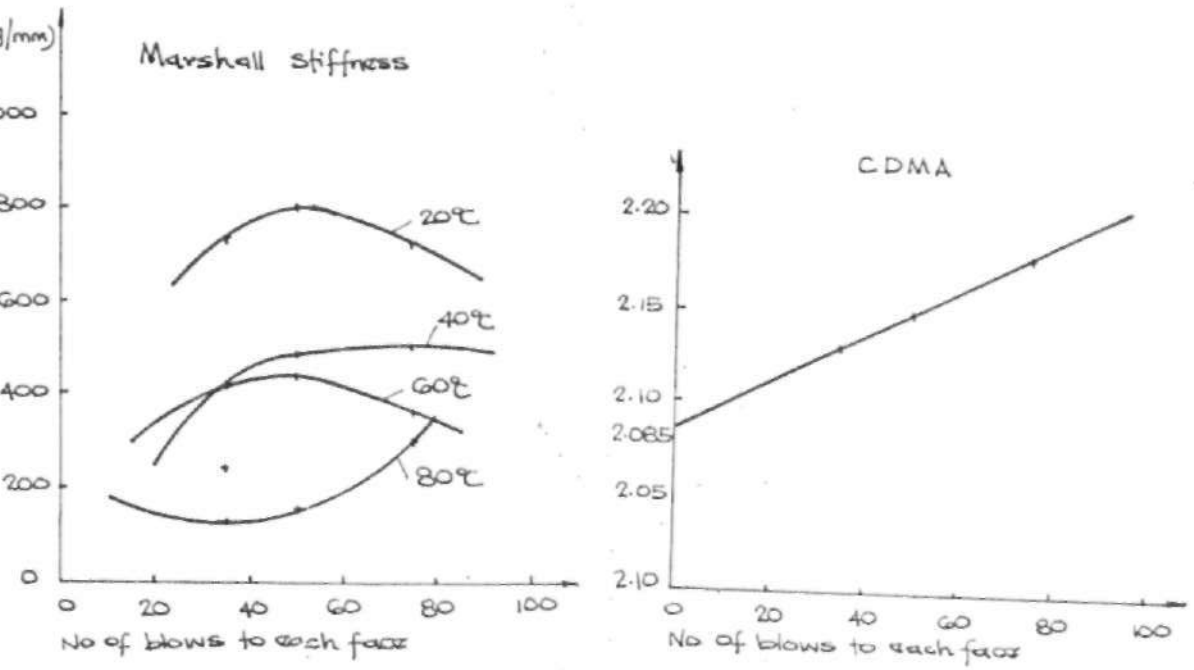


Fig. 4.4.3 - Relationship between Marshall stiffness, Testing temperature and compactive effort.

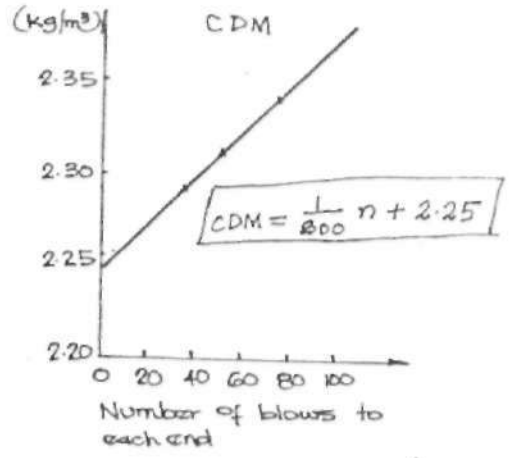


Fig. 4.4.4. Relationship between CDM and compaction

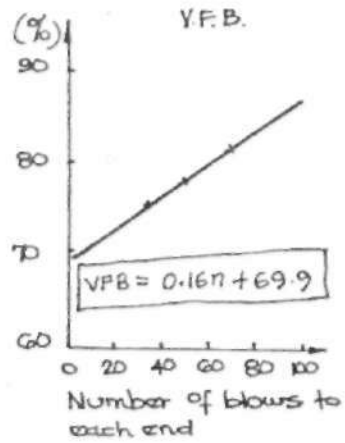


Fig. 4.4.5 - relationship between VFB and compaction

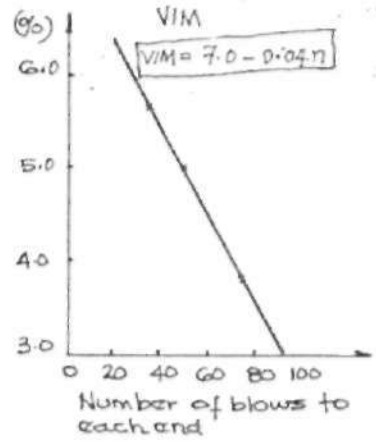


Fig. 4.4.6 - Relationship between VIM and compaction.

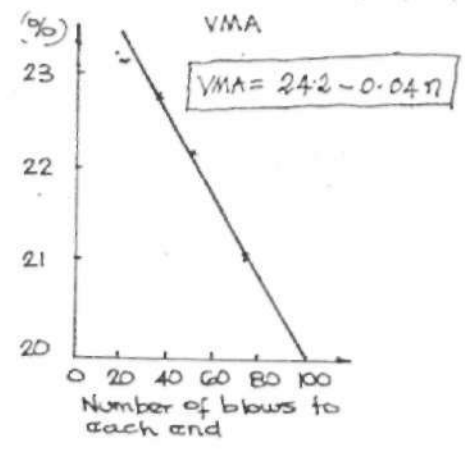


Fig. 4.4.7- Relationship between VMA and compaction

the compactive effort.

e) On VFB:- From the results contained in Table 4.4.2 and shown on fig 4.4.5, it can be seen that the amount of voids filled with bitumen is also dependent upon the compactive effort put. A linear relationship of the following ofder was obtained:-

$$VFB = 0.16n + 69.9 \quad \ldots \quad \ldots \qquad (2)$$

where VFB = Voids filled with bitumen (m%)

n = no of hammer blows to each face.

f) On VIM and VMA:- Both the voids in the mix and mixed aggregates were found to be inversely proportional to the compactive effort. Figs 4.4.6 and 4.4.7 give the straight line equation describing the relationship of the two parameters with the amount of compactive effort put. From the figures, the following equation were obtained:-

$$VIM = 7.0 - 0.04n \quad \ldots \quad \ldots \qquad (3)$$

$$VMA = 24.2 - 0.04n \quad \ldots \quad \ldots \qquad (4)$$

where VIM = Voids in the mix (n%)

VMA = Voids in the mix Aggregates (m%)

n = number of hammer blows to each face.

4.5. DETERMINATION OF DESIGN BINDER CONTENT

As reviewed in Chapter 2, the designed binder content is determined as the average of the optimum binder contents corresponding to maximum marshall stiffness, compacted density of the mix, tensile strength, compressive modulus and 4% air voids of the mix. From the result contained in table 4.5.1 and shown on figs 4.3.3 to 4.3.7, the design binder content is obtained as 7% as can be seen in table 4.5.2. This value of the design binder content was checked for minimum/maximum design criteria as contained in the Marshall design procedure and is presented in Table 4.5.3. From the result of the table 4.5.3 it can be said that the design binder content of 7% by weight of the mix is generally satisfactory with respect to the marshall criteria.

TABLE 4.5.1 :- RESULTS OF OPTIMISATION OF THE BINDER CONTENT.

BINDER CONTENT	No. of SPECIMENS	MARSHALL STABILITY		FLOW VALVE		MARSHALL STIFFNESS		CDM		TENSILE STRENGTH		COMPRESSIVE MODULUS		VIM	
	Mean	Mean	CV	Mean	CV	Mean	CV	Mean	CV	Mean	CV	Mean	CV	Mean	CV
		Kg	%	mm	%	Kg/mm	%	Kg/m^3	%	KN/m^2	%	KN/m^2	%	%	%
4	4	550	18.1	2.38	3.3	231	13.1	2.18	1.4	155	17.7	38.270	17.6	15.0	8.1
5	4	813	26.9	2.30	3.6	353	24.6	2.24	1.2	181	10.1	40.666	9.0	10.9	9.7
6	4	981	18.1	2.23	4.3	439	15.1	2.28	0.6	218	19.6	42.906	9.7	7.6	6.7
7	4	917	19.8	2.28	4.2	403	20.8	2.30	1.1	242	9.5	44.948	21.4	5.4	19.8
8	4	933	16.2	2.63	12.2	279	8.5	2.28	0.9	214	3.1	27.055	12.5	4.7	18.3

TABLE 4.5.2: DESIGN BINDER CONTENT

Property	Optimum binder content (%w)
Maximum marshall stiffness, b_1	6.2
Maximum compacted density, b_2	7.0
Maximum tensile strength, b_3	7.0
Maximum compressive modulus, b_4	7.0
4% air voids b_5	7.8%

AVERAGE (DESIGNED BINDER CONTENT 7.0%

TABLE 4.5.3 CHECK FOR DESIGN BINDER CONTENT

PROPERTY	Marshall Design criteria	Design value	Remarks
Marshall stability	$\not<$ 340kg	917 Kg	Okay
Marshall flow	$\leqslant$ 4.1mm	2.28mm	okay
Air void Content (VIM)	$\leqslant$ 5% for surface course	5.4%	Marginally okay
Voids in the mix aggregates	$>$ 15%	22.5%	Okay

NOTE: The above result is for Aspahltic concrete made of grading with 50% stone content and given 50 number of hammer blows on each face. The testing temperature was at 60°C.

CHAPTER 5

FINDINGS, CONCLUSIONS, AND RECOMMENDATION

In line with the objective of the study as introduced in chapter one together with the literature review of the existing knowledge in bituminous pavement engineering and the conduct of the experimental programme as described in chapter three, the following findings, conclusions and recommendations were made on the basis of the result discussions in chapter four.

5.1 Findings of the Study

Within limits of technicalities of the experimental conduct, weather influence and other variables the findings of the study are:

1) On the effects of Aggregate gradation and Stone Contents

The result of the study showed that the properties of the asphalt concrete mixes are influenced by the type of aggregate gradation and stone content. The aggregate grading that give stone content between 50% to 65% are observed to give the highest maximum values of the Marshall Stability, Flow, Stiffness, Compacted density of the mix and mixed aggregates

at the binder content of 7%. This result is quite comparable to the previous findings by Ngboer (1948) and Brien (1978) who found the optimum stone content to be 52% and 55% respectively.

2) On the effects of Aggregate shape and surface texture.

It was observed that:-

i) Crushed angular aggregate which have rougher surface texture produced mixes with higher maximum stability and stiffness than the smooth textured round aggregates even though their crushing value of the aggregates and the compacted density of the mix may be the same. This finding agrees with that made earlier by Jimenez (1962) and Brien (1978) that as long as there is sufficient binder, the rough textured aggregate will perform better than the smooth textured aggregate.

ii) the smooth textured round aggregate gave too much air voids in the mix (11.3%) than the crushed singular rough textured aggregates (5.4%) at optimum binder content.

iii) The rough surface textured crushed aggregates gave the highest optimum binder content of 7% to that given by smooth surface textured aggregates optimum value of 5%. This also agrees with the findings of Jumenez (1962) that optimum binder (asphalt) content for fatigue resistance is of the order 6% binder

content for smooth aggregate and 7.5% for rough textured aggregate.

3) On the effect of Binder Content:- The results of the study show that all the properties of the asphalt concrete mix are dependent upon the amount of the binder in the mix. The values of the optimum binder content obtained for the aspahlt concrete performance properties are as follows:-

a) For Maximum Marshall Stability = 6.4%
b) " " " Stiffness = 6.2%
c) " " Compacted density of the mix = 7.0%
" aggregates
d) " " tensile strength = 7.0%
e) " " compressive strength = 7.0%
f) " 4% voids in the mix = 7.8%
g) " minimum voids in the mixed aggregates = 6.2%

The above values gives the design optimum binder content at 7% by weight of the mix in accordance with the reviewed mix designed method suggested in chapter two.

Also it was found that more binder content is required for maximum tensile properties (7%) than required for stiffness (Marshall) properties (6.2%) which agrees with the findings of Brown (1978) that deformation resistance properties (Marshall properties) require low binder content and fatigue resistance properties (tensile properties) require high binder content.

4) On the effect of Testing temperature:

It was found that increasing the testing temperature from $40^{o}C$ to $80^{o}C$ reduces the Marshall stability and stiffiness of the mix by 79% and 68% respectively for a 50 blows compactive effort as contained in Table 4.4.2. This agrees with the findings of Bissada (1972) that the resistance of the aspahlt mix to fatigue life decreases with increase in temperature.

5) On the effect of Compaction

The result of the study indicated that increasing compaction is beneficial to the Marshall stability, stiffness compacted density of the mix and mixed aggregates and voids filled with the bitumen of the asphaltic mix concrete mixes. Also increase in compaction reduces the amount of voids in the mix and mix aggregates. The relationships between Compactive effort and CDM, VFB, VIM and VMA has found to be as follows:-

$$CDM = \frac{n}{800} + 2.25 \quad (10^3\ kg/m^3) \quad \cdots \quad (1)$$

$$VFB = 0.16n + 69.9 \quad (\%) \quad \cdots\cdots \quad (2)$$

$$VIM = 7.0 - 0.04n \quad (\%) \quad \cdots\cdots \quad (3)$$

$$VMA = 24.2 - 0.04n \quad (\%) \quad \cdots\cdots \quad (4)$$

where n = no. of blows to each face.

Increase in compaction is also found to improve the performance of the asphalt mixes against the adverse effect of temperature. It was found that when the compactive effort was increasedfrom 50 to 75 blows on each face, the Marshall stability and stiffness of the asphalt mix is improve by 11% and 31% respectively when the testing temperature is increased from 40°C to 80°C.

5.2 CONCLUSIONS

Based on the findings of this study, the following conclusion are made:-

1) That the high temperature prevailing in the country atimes as high as 60°C during the month of March in some areas (See Appendix J), could be a major contributing factor to the pavement failure in Nigeria. This is because with such high temperature, the effective stiffness of the modulus of Asphaltic corponent of the pavement is lowered, which in turn causes a decrease in it's load spreading properties and in the stability of the pavement as a whole.

2) That insufficiency of Stone Content below the Optimum Value (50%-65%) could lead to production of weak aspahltic concrete thereby excessive transfer of load to weaker layers below.

3) That the use of low binder content (as the author suspect in reference to present practice in Nigeria) than is required for both fatigue resistance and deformation

resistance properties could contribute to bad performance of the mixture. Therefore the design asphalt (binder) content should include the tensile properties of the mixture as well. It had been observed by Maupin Jr (1972) that stiff, mixes yield shorter fatigue lives that did the flexible mixes of the same aggregate gradation.

4) That Rough angular crushed aggregate perform better as aggregate for the production of good performance asphaltic concrete than the smooth round aggregates.

5) That compaction is very influential to the performance of Asphaltic concrete. High performance can be achieved by increasing the compactive effort.

5.3 RECOMMENDATIONS

1. That the authorities concerned should look into the possibility of incorporating the Tensile test procidures in the present design policy for paving mixtures in Nigeria.
2. That adequate construction control should be intensified to control the air voids in the Asphaltic concrete by applying sufficient and adequate copaction.
3. That further research should be cerried out to correlate the existing compaction procedures on the site with the result of the laboratory studies on compaction.

4. That adquate supervision should be given both during production and laying of asphalt concrete in accordance with the design specifications.

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a) INDIVIDUAL MARSHALL TEST SPECIMEN CHARACTERISTICS AND RESULTS FOR MIX OF 3% BITUMEN CONTENT
(ANGULAR CRUSHED AGGREGATE TYPE)

SPECI-MEN	Binder content	Volume content cm^3	Stability Dial Reading	Factor	Corr. Dial Read.	Kg	Flow mm	Marshall Stiffness	Weight in air	in H_2O	CDM	CDMA	SGMA	SGM	VIM	VMA	VFB	VIMO
4W1		556.2	175	0.89	155.8	701	2.70	260	1171	625	2.15	2.05		2.53	15.02	25.8	39	23
4W2	4	567.5	200	0.86	172.0	774	2.50	310	1170	612	2.10	2.02	2.73		17.00	26.01	25	26
4W3		561.8	210	0.86	180.6	813	2.60	313	1165	610	2.10	2.02			17.00	26.01	35	26
4W4		567.5	210	0.56	180.6	813	2.90	280	1169	606	2.09	2.00			17.79	26.74	35	26
5W1		533.5	200	0.96	192.0	864	2.90	298	1148	619	2.17	2.05			12.85	24.54	48	20
5W2	5	532.7	260	0.96	249.6	1122	2.20	510	1128	600	2.14	2.03	2.73	2.49	14.06	25.64	45	22
5W3		535.1	225	0.96	216.0	972	2.30	235	1143	608	2.14	2.03			14.06	25.64	45	22
5W4		541.6	235	0.96	225.6	1015	2.40	243	1142	608	2.14	2.03			14.06	25.64	45	22
6W1		530.2	220	0.96	211.2	950	2.30	413	1187	657	2.24	2.11			8.57	22.71	62	13
6W2	6	541.6	205	0.93	190.7	858	2.20	390	1184	648	2.21	2.08	2.73	2.45	9.80	23.81	59	15
6W3		540.8	275	0.93	255.8	1150	2.10	548	1185	645	2.19	2.07			10.61	24.54	75	57
6W4		536.7	220	0.93	111.5	503	2.30	219	1154	617	2.15	2.05			12.25	26.01	53	19
7W1		527.0	170	0.96	163.20	735	2.10	350	1153	630	2.21	2.05			8.30	24.54	66	13
7W2	7	544.0	230	0.93	213.9	964	2.20	438	1172	633	2.17	2.00	2.73	2.41	9.54	25.64	63	15
7W3		521.3	215	1.00	215.0	968	2.00	484	1129	610	2.18	2.03			9.54	25.64	63	15
7W4		513.2	150	1.00	150.0	675	2.30	294	1118	614	2.22	2.07			7.88	24.18	67	12
8W1		518.9	122	1.00	122.0	550	2.10	262	1134	620	2.12	1.95			9.40	28.57	67	15
8W2	8	516.4	175	1.00	175.0	787	2.10	275	1130	617	2.20	2.02	2.73	2.34	5.98	26.00	77	9.0
8W3		527.0	173	0.96	166.1	747	2.30	325	1170	650	2.21	2.03			5.56	27.11	78	9
8W4		527.0	143	0.96	137.3	618	2.20	281	1172	640	2.20	2.04			5.98	28.00	77	9

b) INDIVIDUAL MARSHALL TEST SPECIMEN CHARACTERISTICS FOR BITUMEN MIX OF 5% BITUMEN CONTENT
ANGULAR CRUSHED AGGREGATE TYPE

SERIES-NO.	Binder content	Volume content cm^3	Stability Dial Reading	Stability Correction factor	Stability Corr. Dial Reading	Stability KG	Flow mm	Marshall Stiffness	Weight in air	Weight in H_2O	CDM	CDMA	SDM	SDMA	VIM	VMA	VFB	VIMO
4A1		552.92	110	0.89	97.9	441	2.20	201	1160	627	2.18	2.07				14.84	24.28	38.88	29.68
4A2	4	561.84	130	0.86	111.8	504	2.40	210	1154	628	2.13	2.02	2.70	2.56	16.80	26.81	37.34	33.60
4A3		532.65	135	0.96	129.6	584	2.40	243	1156	632	2.20	2.11				14.06	23.55	40.30	28.12
4A4		545.63	160	0.93	148.8	670	2.50	268	1167	646	2.19	2.10				14.45	23.91	39.57	28.90
5A1		526.98	135	0.96	129.60	554	2.30	241	1162	633	2.20	2.13				12.35	22.9	49.14	24.7
5A2	5	529.41	200	0.96	192.0	864	2.30	376	1178	652	2.24	2.12	2.76	2.51	10.76	22.83	52.57	21.52
5A3		510.76	240	1.00	240.0	1079	2.40	450	1160	646	2.26	2.15				9.96	22.10	54.93	19.92
5A4		534.27	175	0.96	168.0	756	2.20	344	1168	662	2.25	2.11				10.36	22.46	53.87	20.72
6A1		522.11	215	1.00	215.0	967	2.30	420	1150	651	2.28	2.14				7.69	22.46	65.76	15.38
6A2	6	509.95	275	1.00	275.0	1236	2.30	537	1174	664	2.30	2.17	2.76	2.47	6.88	21.74	68.35	13.76
6A3		518.87	190	1.00	190.0	855	2.20	389	1175	660	2.28	2.14				7.69	22.46	65.76	15.38
6A4		523.73	200	0.96	192.0	864	2.10	411	1167	653	2.27	2.13				8.10	22.83	64.52	16.20
7A1		502.65	205	1.04	213.20	959	2.40	400	1143	649	2.31	2.15				4,94	22.10	77.65	9.88
7A2	7	494.55	150	1.09	163.50	736	2.20	335	1100	615	2.27	2.11	2.76	2.43	6.58	23.55	72.06	13.16
7A3		488.87	235	1.09	256.15	1151	2.20	523	1125	642	2.33	2.17				4.12	21,37	80.73	8.24
7A4		535.09	190	0.96	182.4	821	2.30	357	1207	679	2.29	2.13				5.76	22.83	74,77	11,52
8A1		495.38	130	1.09	141.7	638	2.40	255	1127	633	2.28	2.10				4.60	23.91	80.76	9.20
8A2	8	490.49	130	1.09	141.7	638	2.40	266	1117	628	2.28	2.10	2.76	2.39	4.60	23.91	80.76	9.20
8A3		494.55	180	1.09	196.2	883	3.10	285	1149	650	2.30	2.12				3.77	23.19	83.74	7.54
8A4		499.41	165	1.04	171.6	772	2.50	309	1159	644	2.25	2.07				5,86	25.00	76.56	11.72

c) INDIVIDUAL MARGVALL TEST SPECIMEN CHARACTERISTICS AND RESULTS FOR MIX OF 50% ..... CONTENT

(AIRING SWOLLEN AGGREGATE TYPE)

SPECIMEN IDENT.	Elapsed Volume cont. cm³	Volume content cm³	Slump Test Data Time in Secs.	Vebe Test Data Time in Secs.	Air Content %	Flow %	Workability Stiffness mm	Weight in air	Weight in H_2O	CDM	CDMA	SDM	SDMA	VIM	VMA	VFB	VIMO
6R1		522.92	195	0.96	187.20	842	2.2	383	1128	642	2.29	2.15		10.55	20.96	49.67	21.10
6R2	6	528.60	150	0.96	144.00	648	2.1	309	1163	647	2.25	2.12	0.72	2.56	12.11	22.06	45.10	24.22
6R3		532.65	170	0.93	158.10	712	2.3	310	1168	640	2.26	2.12			11.72	22.06	46.87	23.44
6R4		518.87	230	1.00	230.00	1034	2.4	431	1184	664	2.28	2.14			10.94	21.32	48.69	21.88
7R1		524.54	170	0.96	158.10	712	2.1	339	1172	648	2.24	2.08			11.46	23.53	51.30	22.92
7R2	7	568.33	210	1.04	218.40	982	2.2	446	1159	648	2.27	2.11	2.72	2.53	10.28	22.43	54.17	20.56
7R3		505.90	150	1.04	186.00	702	2.0	351	1140	633	2.25	2.09			11.07	23.16	52.70	22.14
7R4		508.33	150	1.04	156.00	702	2.4	293	1136	625	2.22	2.07			12.25	23.90	48.75	24.50
8R1		497.80	135	1.04	140.40	632	2.2	287	1141	628	2.22	2.05			11.20	24.63	54.53	22.40
8R2	8	514.82	125	1.00	125.00	563	2.4	235	1155	632	2.21	2.03	2.72	2.50	11.60	25.37	54.28	23.20
8R3		508.33	125	1.04	130.00	586	2.0	293	1154	633	2.22	2.04			11.20	25.00	55.20	22.40
8R4		500.22	160	1.04	166.40	749	2.4	312	1146	633	2.23	2.06			10.80	24.26	55.48	21.60
5R3		478.33	140	1.14	182.40	821	2.3	357	1060	606	2.30	2.10			10.85	19.49	44.33	21.70
5R4		476.71	150	1.14	171.20	770	2.4	321	1045	587	2.28	2.17			11.63	20.22	42.48	23.26
6R1		522.92	195	0.96	187.20	842	2.2	383	1128	642	2.29	2.15			10.55	20.96	49.67	21.10
6R2	6	528.60	150	0.96	144.00	648	2.1	309	1163	647	2.25	2.12	0.72	2.56	12.11	22.06	45.10	24.22
5R2	5	470.23	215	1.19	255.85	1150	2.2	523	1057	595	2.29	2.17	0.72	2.58	11.24	20.22	44.41	22.48
5R1		476.71	160	1.14	182.4	820	2.1	391	1063	597	2.28	0.17			11.63	20.22	42.48	23.26
4R4		486.44	210	1.09	228.90	1029	2.20	468	1072	600	2.27	2.16			13.03	19.85	34.36	26.06
4R3		488.87	185	1.09	201.65	907	2.10	432	1077	602	2.27	2.12			13.03	19.85	34.36	26.06
4R2	4	489.68	185	1.09	201.65	907	2.20	412	1077	598	2.25	2.17	0.72	2.61	13.29	20.59	33.03	27.58
4R1		480.44	130	1.09	141.7	638	2.60	245	1062	584	2.22	2.13			14.94	21.69	31.12	29.88

d) INDIVIDUAL MARSHALL TEST SPECIMEN CHARACTERISTICS AND RESULTS FOR MIX OF 60% STONE CONTENT
(ANGULAR CRUSHED AGGREGATE TYPE)

SPECI-MEN	Binder content	Volume cm^3	Stability Dial Reading	Factor	Corr. Dial Read.	Kg	Flow mm	Marshall Stiffness	Weight in air	in H_2O	CDM	CDMA	SGM	SGI	VIM	VMA	VFB	VIMO
3B1		534.3	200	0.96	192.00	864	2.7	320	1160	640	2.22	2.15			14.9	22.1	32	37
3B2	3	539.1	150	0.93	139.00	628	2.1	299	1152	692	2.22	2.15	2.76	2.61	14.9	22.1	32	37
3B3		555.4	160	0.89	142.4	641	2.4	267	1170	639	2.20	2.13			15.7	22.8	31	39
3B4		548.1	175	0.89	155.8	701	2.0	351	1173	646	2.21	2.14			15.3	22.5	32	38
4B1		533.5	175	0.96	168.0	756	2.6	291	1184	658	2.25	2.16			12.1	21.7	44	30
4B2	4	537.5	235	0.93	218.5	985	2.9	339	1180	653	2.24	2.15	2.76	2.56	12.5	22.1	43	31
4B3		526.2	190	0.96	182.4	821	2.1	391	1169	649	2.25	2.16						
4B4		532.7	195	0.96	187.1	855	2.4	356	1164	643	2.26	2.15			12.2	22.1	43	31
5B1		541.7	215	0.93	200.0	902	3.2	282	1151	647	2.28	2.17	2.76	2.51	9.2	21.4	57	23
5B2	5	522.9	215	0.96	206.4	928	2.9	320	1154	645	2.27	2.16			9.4	21.7	56	23
5B3		544.8	235	0.93	218.6	983	2.1	468	1148	644	2.28	2.17			9.2	21.4	57	23
5B4		518.9	290	1.00	290.0	1303	2.2	592										
6B1		561.0	285	1.04	296.4	1332	2.9	839	1150	662	2.37	2.23			4.1	19.2	79	10
6B2	6	514.8	225	1.0	225.0	1012	2.9	349	1164	668	2.35	2.21	2.76	2.47	4.9	19.9	76	12
6B3		494.6	270	1.09	294.3	1322	2.5	529	1126	644	2.34	2.20			5.3	20.3	74	13
6B4		502.7	325	1.04	338.0	1518	2.7	562	1139	653	2.34	2.20			5.3	20.3	74	13
7B1		522.1	245	1.00	245.0	1101	2.5	440	1152	659	2.34	2.18			3.7	21.0	82	9
7B2	7	488.9	285	1.09	310.65	1396	3.0	465	1139	656	2.37	2.20	2.76	2.43	2.5	20.3	88	6
7B3		515.6	215	1.00	215.0	967	3.2	306	1124	640	2.32	2.16			4.5	21.7	79	11
7B4		517.3	235	1.00	235.0	1057	2.9	365	1146	657	2.34	2.18			3.7	21.0	82	9

INDIVIDUAL INDIRECT TENSILE TEST RESULT FOR MIX OF 50% STONE CONTENT

(ANGULAR CRUSHED AGGREGATE TYPE)

SPECIMEN NUMBER	BINDER CONTENT	APPLIED LOAD (P_f) TO FAILURE: DIAL READ.	CONVERSION kN	CONVERSION kg	VERTICAL DEFORMATION LDV: DIAL READ/CONVERSION 0.001	mm	P/Y kN/mm	t thickness mm	TENSILE STRENGTH ST. (KN/m^2)	COMPRESSIVE MODULUS E_Y (kN/m^2)
		0.01286kN/ Division								
4T1		140	1.80	180	85	2.2	81.8	667	162.9	42,884
4T2	4	150	1.93	193	90	2.3	83.9	680	171.3	43,144
4T3		100	1.29	129	90	2.3	56.1	683	114.0	28,722
4T4		145	1.87	187	105	2.7	73.0	666	171.0	38,328
5T1		132	1.70	170	90	2.3	73.9	641	160.1	40,314
5T2		160	2.06	206	110	2.8	73.6	633	196.4	40,658
5T3	5	140	1.80	180	120	3.1	66.5	638	170.3	36,448
5T4		160	2.06	206	100	2.5	82.4	635	195.9	45,376
6T1		140	1.50	180	100	2.5	72.0	620	175.2	40,608
6T2	6	170	2.19	219	120	3.1	707	630	209.8	39,242
6T3		170	2.19	219	110	2.8	78.2	634	208.5	43,131
6T4		200	2.57	257	130	3.3	77.9	560	277.0	48,643
7T1		190	2.44	244	115	2.9	84.1	620	237.5	47,432
7T2	7	190	2.44	244	120	3.1	78.7	624	246.0	44,102
7T3		170	2.19	219	150	3.8	57.6	619	213.5	32,539
7T4		210	2.70	270	110	2.8	96.4	605	269.3	55,717
8T1		160	2.06	206	200	5.1	40.4	605	205.5	23,350
8T2	8	170	2.19	219	160	4.1	53.4	596	221.8	31,330
8T3		170	2.19	219	175	4.5	48.7	613	215.6	27,780
8T4		165	2.12	212	190	4.8	44.2	600	213.2	25,760

INDIVIDUAL TEMPERATURE VARIATION TEST SPECIMEN CHARACTERISTICS AND RESULTS

a) (75 HAMMER BLOWS TO EACH FACE)

SPECIMEN	TEST-ING Temp. (^oC)	VOL.	STABILITY Dial Read	Factor	Corr. Dial Read	Conversion	FLOW	MARSHALL Stiffness	WEIGHT in air	WEIGHT in H_2O	CDM	CDMA	VIM	VMA	VFB
75T20-1		501	958	1.04	996	4491	6.1	736	1167	668	2.34	2.18	3.7	21	82
75T20-2	20	500	952	1.04	990	4464	6 .2	720	1177	670	2.32	2.16	4.7	.2	79
75T20-3		486	910	1.09	992	4473	6.1	733	1139	654	2.35	2.19	3.3	21	84
75T20-4		486	905	1.09	987	4450	6.3	706	1136	652	2.35	2.19	3.3	21	84
75T40-1		506	445	1.04	463	2088	4.3	486	1178	674	2.34	2.18	3.7	21	82
75T40-2	40	502	400	1.04	416	1876	4.1	458	1142	652	2.33	2.17	4.1	21	81
75T40-3		486	446	1.09	486	2191	4.3	510	1129	647	2.34	2.18	3.7	21	82
75T40-4		503	445	1.04	463	2088	4.0	522	1164	667	2.34	2.18	3.7	21	82
75T60-4		511	240	1.00	240	1082	3.5	309	1184	677	2.34	2.18	3.7	21	82
75T60-2		508	232	1.04	241	1086	2.6	388	1178	673	2.33	2.17	4.1	21	81
75T60-3	60	486	210	1.09	229	1032	2.7	382	1137	650	2.33	2.17	4.1	21	81
75T60-4		488	200	1.09	218	983	2.8	351	1114	635	2.33	2.17	4.1	21	81
75T80-1		486	138	1.09	150	676	2.20	307	1175	668	2.32	2.16	4.7	22	79
75T80-2	80	490	135	1.09	147	663	2.3	288	1132	649	2.34	2.18	3.7	21	82
75T80-3		478	110	1.14	125	564	2.0	282	1113	637	2.34	2.18	3.7	21	82
75T80-4		478	140	1.14	160	720	2.2	327	1123	648	2.36	2.19	2.9	21	86

INDIVIDUAL TEMPERATURE VARIATION TEST SPECIMEN CHARACTERISTICS AND RESULTS

b) (50 HAMMER BLOWS TO EACH FACE)

SPECIMEN	TEST-ING TEMP. (^oC)	VOL.	STABILITY Dial Read	Factor	Corr. Dial Read	Conversion	FLOW	MARSHALL Stiffness	WEIGHT in air	WEIGHT in H_2O	CDM	CDMA	VTM	VMA	VFB
50T20-1		503	845	1.04	879	3962	5.5	720	1147	651	2.31	2.15	4.94	22	82
50T20-2	20	499	860	1.04	894	4032	5.1	791	1124	638	2.31	2.15	4.9	22	81
50T20-3		482	820	1.14	935	4215	4.9	860	1104	628	2.32	2.16	4.5	22	80
50T20-4		501	855	1.04	889	4009	4.8	835	1118	634	2.31	2.15	4.9	22	78
50T40-1		511	380	1.00	380	1713	3.5	489	1158	656	2.30	2.14	5.4	25	77
50T40-1		476	400	1.14	456	2056	3.8	541	1100	628	2.33	2.17	4.1	21	81
50T40-3	40	507	365	1.04	380	1712	3.9	439	1143	652	2.33	2.17	4.12	21	81
50T40-4		501	370	1.04	385	1735	3.8	456	1148	657	2.34	2.18	3.70	21	82
50T60-1		503	205	1.04	213	959	2.4	400	1143	649	2.31	2.15	4.9	22	78
50T60-2	60	495	200	1.09	218	983	2.2	447	1100	625	2.27	2.11	6.58	24	72
50T60-3		489	235	1.09	256	1151	2.2	523	1125	642	2.33	2.17	4.10	21	81
50T60-4		503	190	0.96	182	821	2.3	357	1207	670	2.29	2.13	5.8	23	75
50T80-1		517	92	1.00	92	413	2.2	189	1158	654	2.30	2.14	5.4	22	77
50T80-2	80	519	78	1.00	78	352	2.3	153	1175	668	2.32	2.16	4.5	4.1	21
50T80-3		507	95	1/-4	99	445	2.7	165	1140	646	2.31	2.15	4.9	22	78
50T80-4		511	65	1.00	65	293	2.5	117	1132	630	2.26	2.10	4.5	222	71

INDIVIDUAL TEMPERATURE VARIATION TEST SPECIMEN CHARACTERISTICS AND RESULTS

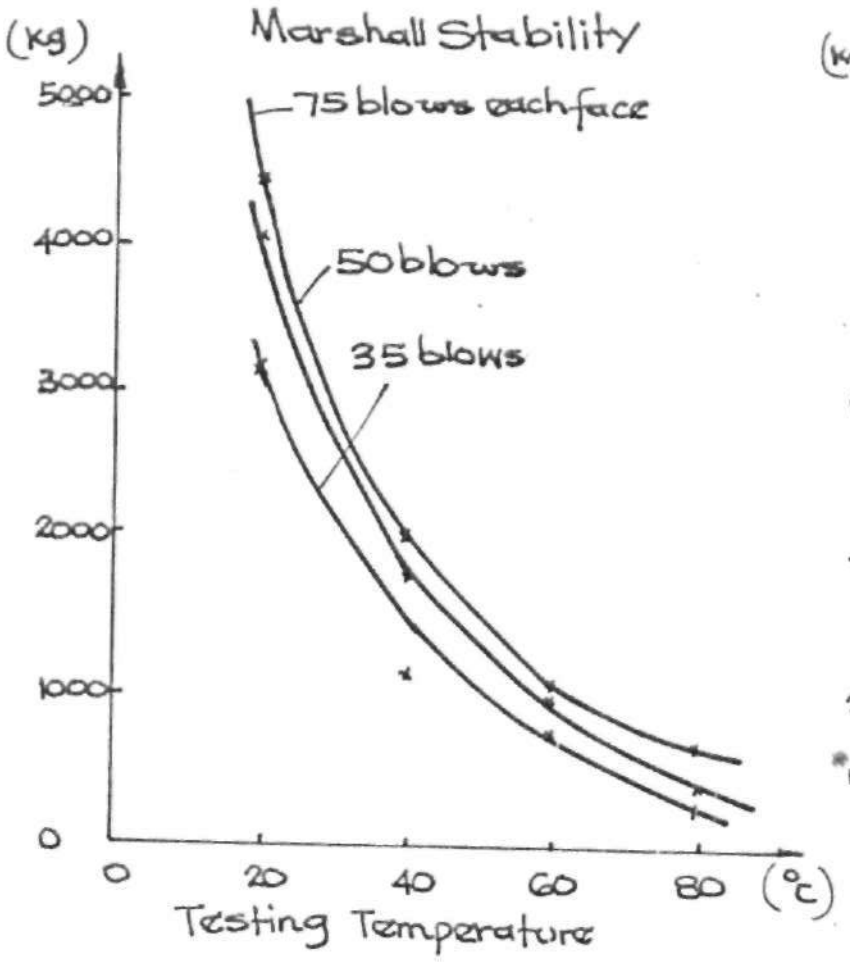
(35 HAMMER BLOWS TO EACH FACE)

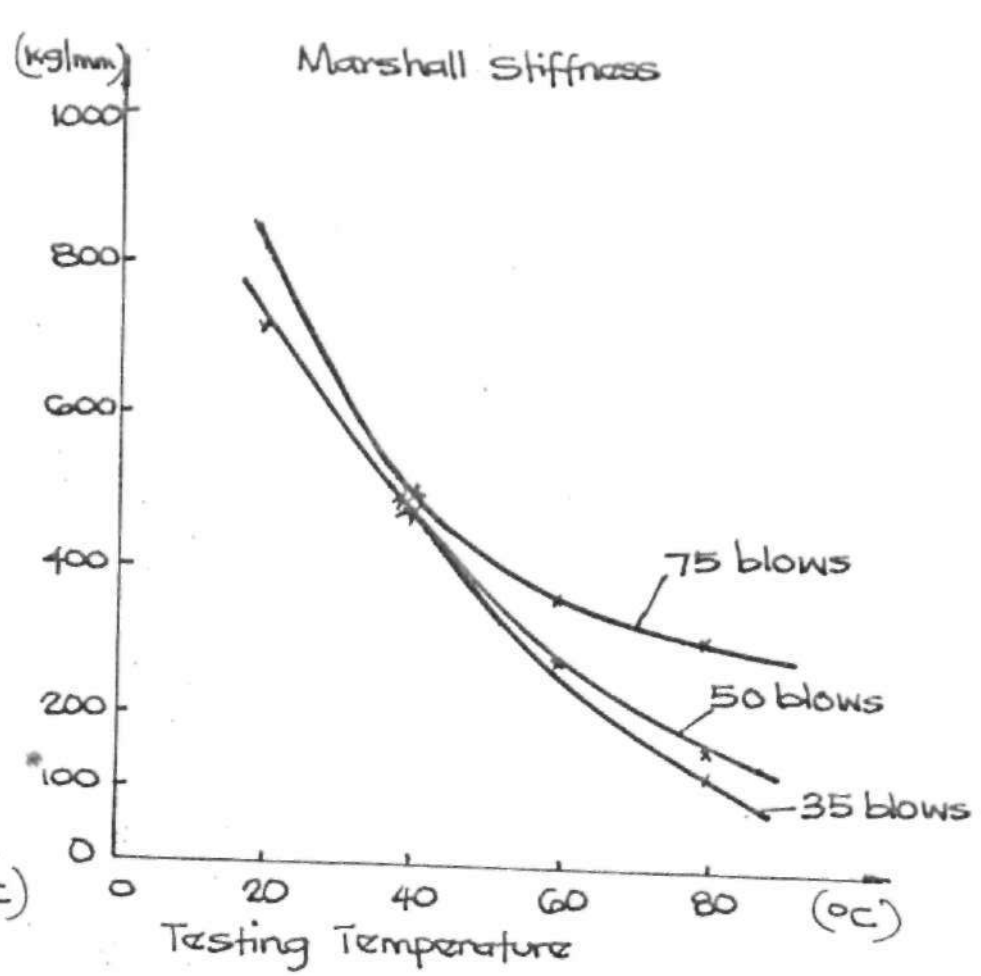
SPECIMEN	TEST-ING Temp. (^{o}C)	VOL.	STABILITY Dial Read	Factor	Corr. Dial Read	Conversion	FLOW	MARSHALL Stiffness	WEIGHT in air	WEIGHT in H_2O	CDM	CDMA	VCB	VMA	VFB
35T20-1		501	630	1.04	655	2954	4.2	703	1134	642	2.30	2.14	5.4	32	77
35T20-2	20	486	630	1.09	687	3096	4.4	704	1111	637	2.34	2.18	3.7	31	82
35T20-3		477	700	1.14	798	3593	4.7	766	1090	622	2.33	2.17	4.1	21	80
35T20-4		492	640	1.09	698	3145	4.3	732	1092	616	2.29	2.13	5.2	23	75
35T40-1		507	235	1.04	244	1104	2.5	441	1110	622	2.28	2.12	6.2	23	73
35T40-2	40	503	245	1.04	255	1149	2.7	426	111106	622	2.29	2.13	5.8	23	75
35T40-3		499	265	1.04	276	1243	2.8	444	1132	642	2.31	2.15	4.9	22	78
35T40-4		514	245	1.00	245	1105	2.6	425	1170	664	2.31	2.15	4.9	22	78
35T60-1		511	175	1.00	175	789	3.0	263	11144	640	2.27	2.11	6.6	24	73
35T60-2	60	511	130	1.00	130	586	2.9	202	1138	634	2.26	2.10	7.0	24	71
35T60-3		507	160	1.04	167	844	3.2	264	1139	642	2.29	2.13	5.8	23	75
35T60-4		507	145	1.04	151	680	3.0	227	1112	6243	2.28	2.12	6.2	23	73
35T80-1		512	60	1.00	60	271	2.3	118	1147	644	2.28	2.12	6.2	23	73
35T80-2	80	504	50	1.04	52	235	2.0	118	1130	634	2.28	2.12	6.2	23	73
35T80-3		507	70	1.04	73	328	2.1	156	1160	657	2.31	2.15	4.9	22	78
35T80-4		515	50	1.00	50	225	2.2	102	1168	642	2.27	2.11	6.6	24	73

SOUTHERN REGION DAILY MAXIMUM TEMPERATURES (A.S.R) IN 1981
(AS) DAYS ENDED IN PERIOD

MONTH DAYS	JAN	FEB	MARCH	APR	MAY	JUN	JULY	AUG	SEPT	OCT	NOV	DEC.
A)												
1 - 10	88.0	90.1	97.1	105.0	104.1	102.6	98.0	88.7	89.4	92.0	97.3	91.0
11 - 20	91.0	89.1	99.0	105.0	103.0	98.3	90.0	88.7	94.1	95.6	93.1	94.0
21 - 31	91.1	90.0	100.0	103.0	102.0	96.1	89.0	87.0	93.4	93.0	90.9	90.7
B.SOUTH												
1 - 10	89.0	90.7	100.1	103.4	103.1	98.6	91.0	87.3	88.1	93.0	97.3	93.7
11 - 20	90.3	89.2	100.7	104.6	101.3	93.9	90.2	86.2	90.7	96.7	94.2	95.0
21 - 31	92.7	100.1	102.2	100.0	99.4	92.4	88.3	86.8	91.4	97.7	92.3	91.0
C) JOS												
1 - 10	56.7	60.2	70.1	66.1	65.1	63.0	60.4	62.4	61.0	62.2	62.1	59.0
11 - 20	56.7	61.7	73.1	63.1	64.3	63.6	60.0	62.7	62.0	63.7	60.7	59.1
21 - 31	58.5	60.7	72.7	65.6	64.4	63.7	60.1	62.1	62.0	62.7	59.1	57.3

- CALCULATED FROM 1 JANUARY 1981 FIGURES AT STATE OF NORTHERN NIGERIA.


(kg)
Marshall Stability
75 blows each face
50 blows
35 blows
5000
4000
3000
2000
1000
0
0
20
40
60
80
(°C)
Testing Temperature


(kg/mm)
Marshall Stiffness
1000
800
600
400
200
100
0
75 blows
50 blows
35 blows
0
20
40
60
80
(°C)
Testing Temperature

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ZARIA, NIGERIA.

