PAVEMENT FAILURE INVESTIGATION ALONG ZARIA - FUNTUA ROAD

BY

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D E C L A R A T I O N

This is to certify that this dissertation has been composed and compiled by myself and that it is a record of my own research work.

It has not been accepted in any previous application for a higher degree or diploma.

Hassan Umar Danjuma

1992.

D E D I C A T I O N

I wish to dedicate this thesis work to my late mother, Malama Maryam Umar Danjuma whose death occured on 27th March 1992, may her soul rest in perfect peace.

C E R T I F I C A T I O N

This thesis entitled "Pavement Failure Investigation along Zaria-Funtua Road" by Danjuma, Hassan Umar meets the regulations governing the award of the Degree of Masters of Science (Civil Engineering Highway Materials) of Ahmadu Bello University and is approved for its contribution to knowledge and literary presentation.

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A C K N O W L E D G E M E N T

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A B B R E V I A T I O N

AASHO	-	American Association of State Highway Officials
AADT	-	Annual Average Daily Traffic
ADT	-	Average Daily Traffic
ADT_p	-	Present Average Daily Traffic
C.B.R.	-	California Bearing Ratio
PSI	-	Present Serviceability Index
OMC	-	Optimum Moisture Content
MDD	-	Maximum Dry Density
ESAL	-	Equivalent Standard Axle Load
FTG	-	Future Total Growth factor
DDF	-	Direction Distribution Factor
LDF	-	Lane Distribution Factor
TRDF	-	Texas Road Development Fund
FMWH	-	Federal Ministry of Works and Housing
PEU	-	Pavement Evaluation Unit.

N O T A T I O N S

LL	-	Liquid Limit
PL	-	Plastic Limit
PI	-	Plastic Index
P_t	-	Percent Trucks
N_D	-	Design Traffic
F_{avg}	-	Average 8,200kg single axle load equivalence factor
S_v	-	Mean Slope Variance
R	-	Roughness value
GI	-	Group Index.

A B S T R A C T

This study was aimed at investigating the causes of premature failures on Zaria - funtua Road linking Zaria in Kaduna state and Funtua in Katsina State. The road was asphalted in 1979. The riding quality on the road had deteriorated in the past couple of years with sections of the road developing various types of distress. Both field and laboratory tests were carried out. Results were utilized to draw conclusions on the type of axle loading, quality of construction materials and efficiency of the construction.

On the whole, it was found that the failures were mainly caused by heavy axle load, use of poor strength materials and low degree of compaction. The degree of compaction was generally less than the desired 100%, whereas 42% of axles sampled exceeded the ten tons legal limit specified. The values of Atterberg limits did not meet the Federal specifications of minimum of 30% for the liquid limit and 12% for plasticity index. Also, the C.B.R. values were found to be below the minimum of 00% for base, 30% for sub-base and 5% for the subgrade set-out by the Federal specifications.

Finally, both short and long term measures have been recommended to remedy the situation.

1.0 INTRODUCTION

1.1 Preamble

The current global recession calls for judicious use of available resources especially in the developing countries which are badly hit by economic constraints. Road construction projects, apart from the major role they play in economic development, are costly ventures. They should therefore be properly undertaken with a view to reducing total life-cycle costs.

However, a number of roads in Nigeria start to deteriorate soon after they have been constructed. This situation makes it necessary to investigate causes of road failures and determine the best measures of repairs. Such investigations would also provide valuable guidelines for future construction as well as design and hopefully open new areas of interest to both the Research and Maintenance Engineers.

1.2 Study Target

Pavement structures involve massive quantities of various materials with inherent variations in their physical as well as other engineering properties. The variations, apart from being complicated, differ in both horizontal and vertical directions. This implies that in investigating pavement failures, substantial data is required for analysis before logical conclusions can be drawn on causes of the failures. Thus, it will be diffi-

cult and time consuming to simultaneously study all roads in order to come out with the analysis of their modes of failure. Rather, a case by case approach is more realistic and this approach has been adopted here.

This study is therefore aimed at carrying out failure investigation of Zaria - Funtua road as a case study. It is hoped that results obtained, when reviewed with similar research works on other roads, will shed more light on the subject and make understanding of failure characteristics of flexible pavement structures easier.

1.3 Objectives

A main thrust of this work is to propose rehabilitation measures for the studied road. However, it is obvious that the type of road rehabilitation measures suitable in any instance depends upon the causal factors of the failed sections.

The objectives of this research work will, therefore, include:-

(i) Assessing the adequacy of the pavement structure through evaluation of the strength properties of the various materials making up the pavement at the failed sections.

(ii) Assessing the physical condition of the road.

(iii) Surveying traffic volume and axle loads to which the pavement has been subjected.

(iv) Determining likely causes ofthe failure.

(v) Proposing the appropriate measures of rehabilitation to be adopted for the pavement.

In achieving the listed study objectives, the major factors causing pavement failure were identified through the following processes:

- Condition survey
- Materials characterisation
- Evaluation of Design
- Evaluation of Construction methods.

1.4 Scope of Work

The research work was specifically aimed at a case study of pavement failure characteristics by taking Zaria - Funtua road that links Kaduna and Katsina States of Nigeria.

Previous case studies on road deterioration undertaken in Nigeria and abroad were reviewed. Material characterization, traffic volumes, axle loads and condition surveys were then undertaken along some selected locations along the road. Further investigations involving laboratory tests were carried out on samples obtained from the field. Results were analysed and compared with appropriate standards or specifications.

It was on this basis that conclusions were drawn on probable causes of the observed failures. Rehabilitation strategies had also been formulated in accordance with factors deemed to have caused the failures.

1.5 Description of Site

Zaria - Funtua road is a single carriageway with an average width of 7.5m and shoulder width of 2.0m on either sides. It was built in the seventies with surface dressing. It was rehabilitaed with an asphalt overlay in 1979. The road links Zaria, in Kaduna State and Funtua in Katsina State of Nigeria. It has a length of about sixty six kilometers and lies in the guinea savannah vegetation regio. with a mean annual rainfall of 1150mm and weighted mean annual air temperature of 26^{o}C. Also, it has a gently rolling terrain.

For the most part of the early eighties, the riding quality on the road was satisfactory to good. However, in past couple of years, the quality had deteriorated. At the period when the study was initiated, sections of the road had developed various types of distress ranging from different classes of cracking to rutting, pot-holing and shear deformations. Typical failure modes as observed along the sections of the road are shown in Plate I - X.

ILLUSTRATION OF FAILED SECTIONS ALONG

ZARIA - FUNTUA ROAD

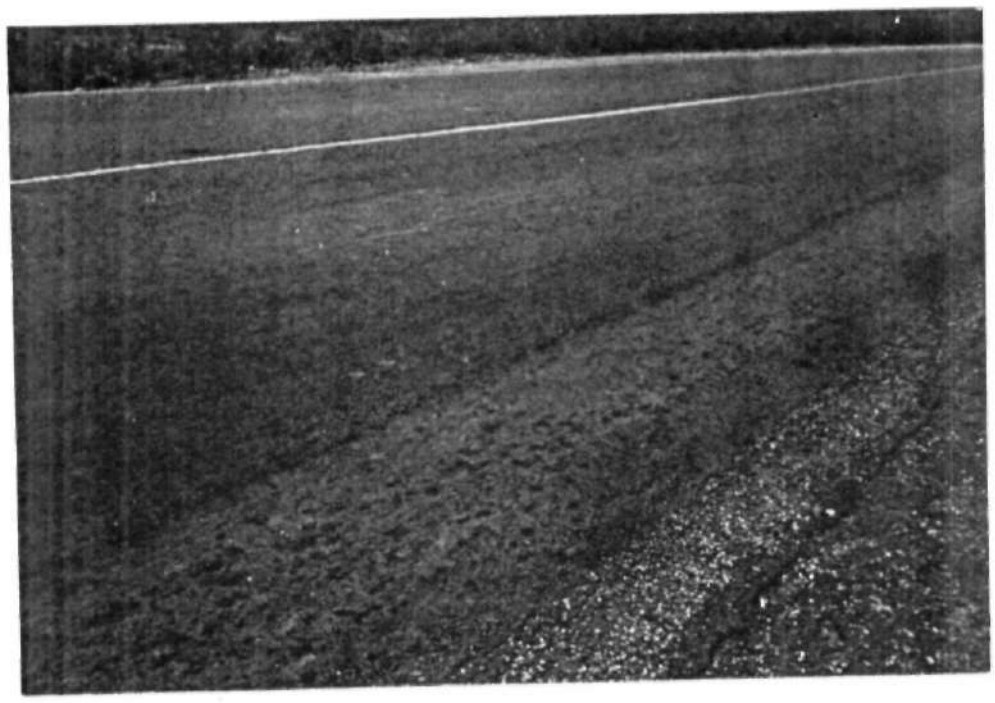


PLATE I: Primary Cracking (Class I cracks)



PLATE II: Secondary Cracking (Class II cracks)



PLATE III: Tertiary Cracking found along a section of the road (Class III cracks)



PLATE IV: Shoulder Failure threatening the carriageway

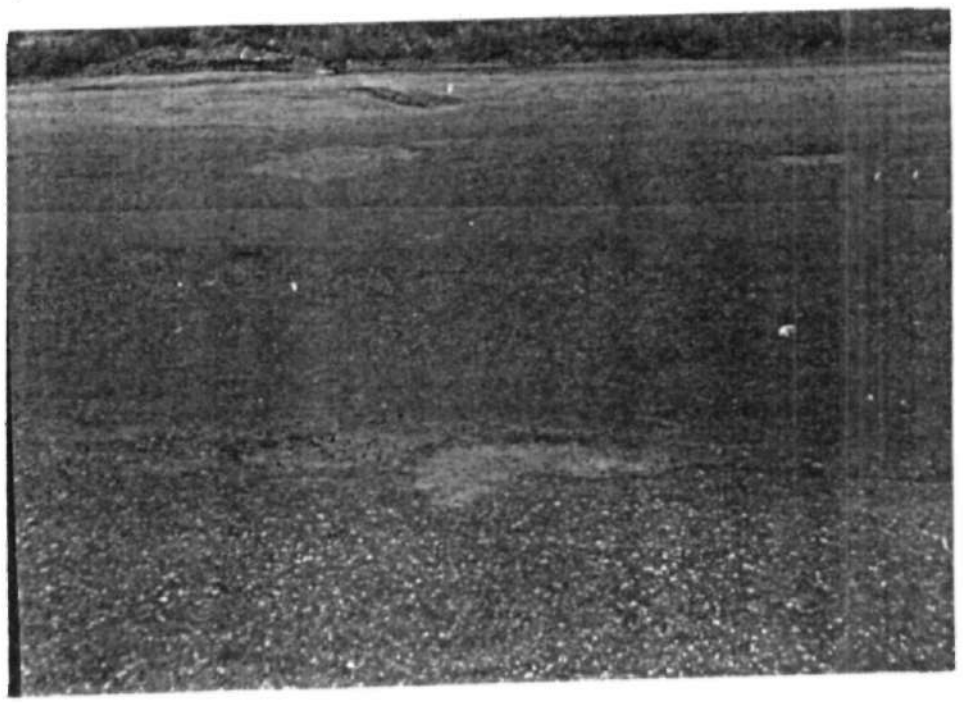


PLATE V: potholing of section of the carriageway

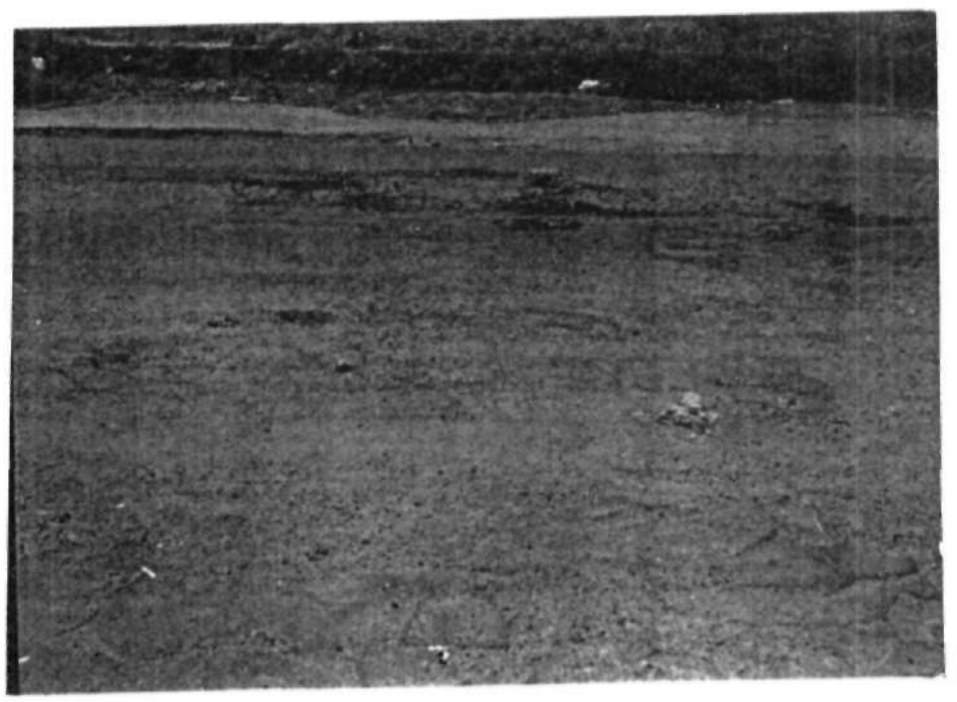


PLATE VI: Total Failure of a section along the road



PLATE VII: Another form of shoulder failure attributed to poor drainage.

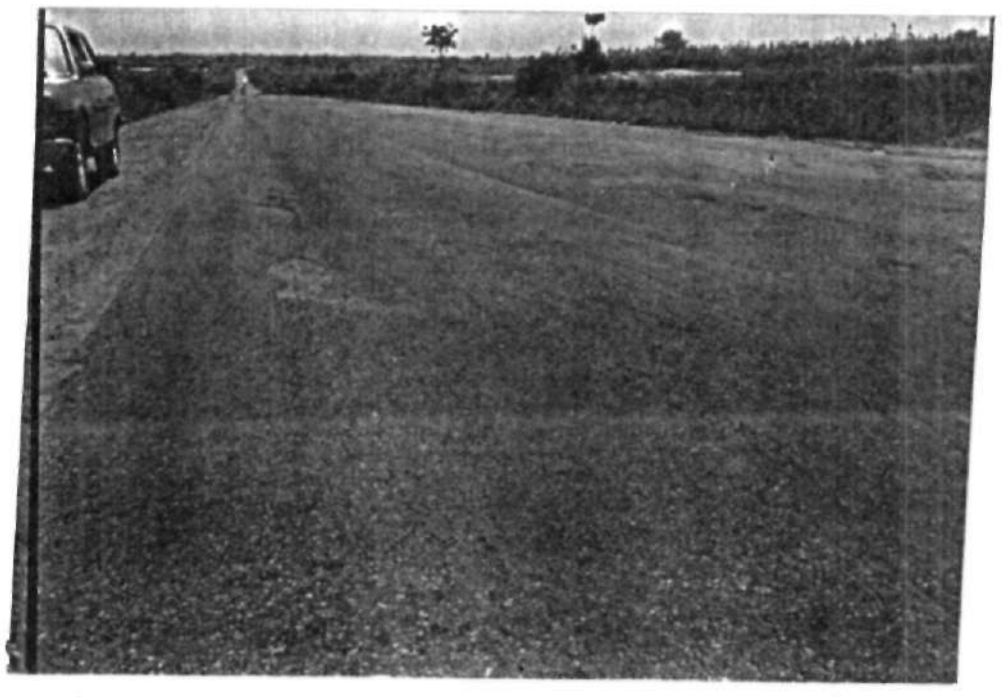


PLATE VIII: Rutting of a section of the road.



PLATE IX: Shoving as noted around KM 7+00.

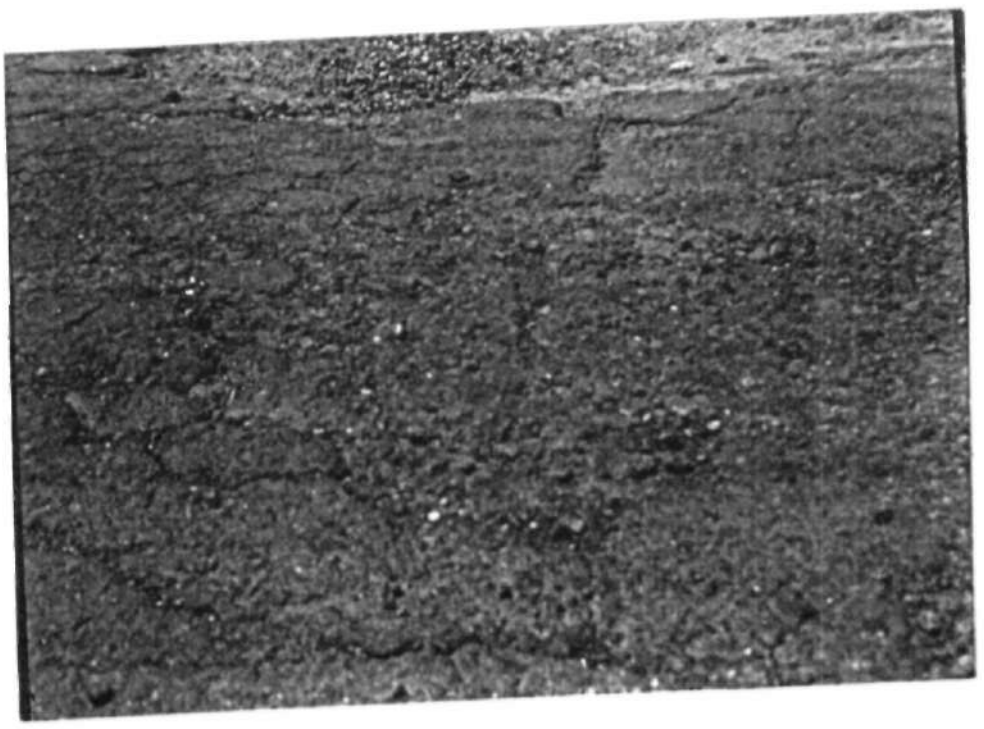


PLATE X: Ravelling as observed at a location.

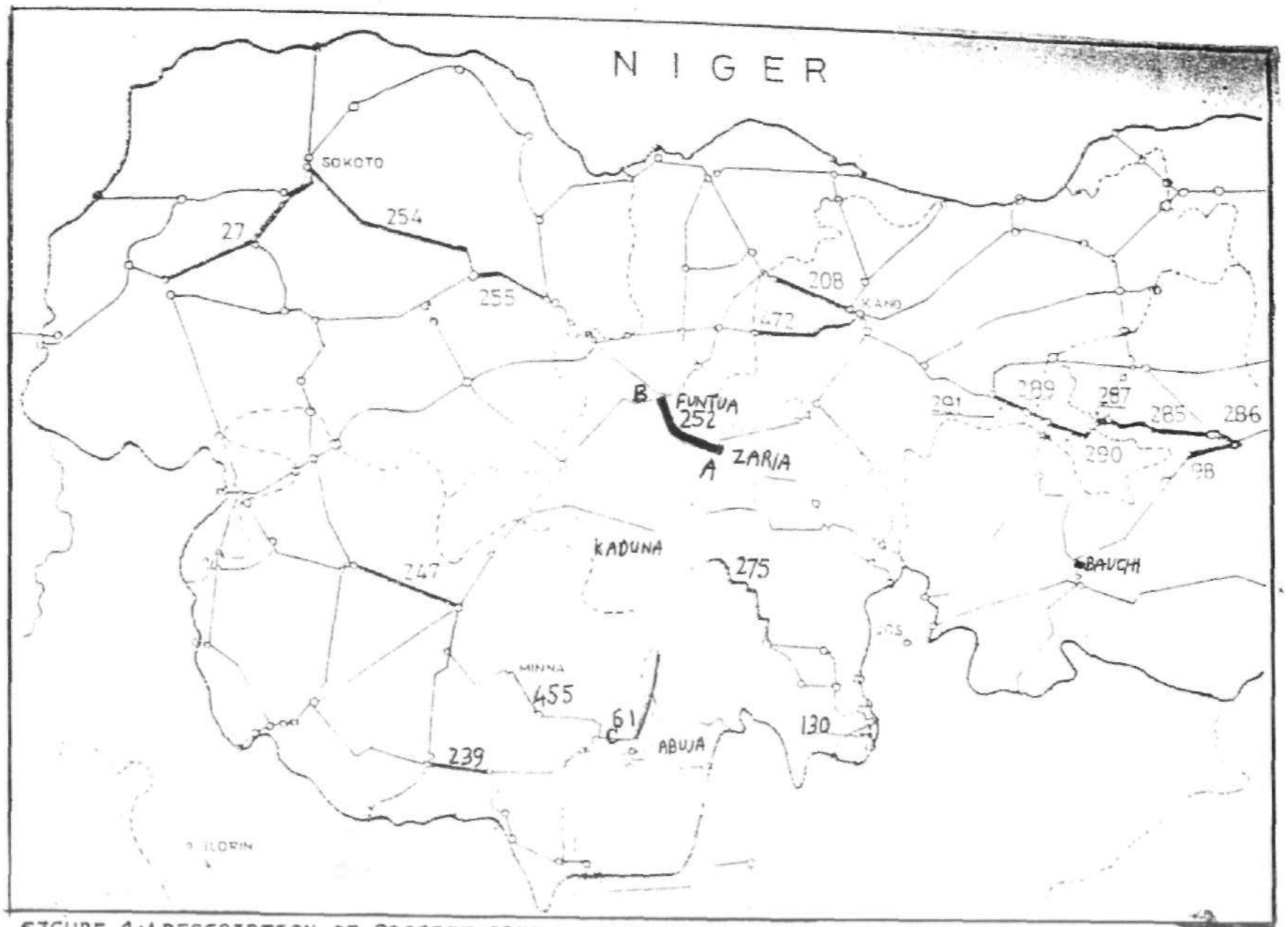


FIGURE 1:1 DESCRIPTION OF PROJECT AREA. KEY: A–B TOTAL PROJECT AREA

1.6 Methodology

In order to realize the objectives earmarked for this research, Zaria - Funtua road was taken as a case study. The investigation consisted of field studies and laboratory tests.

1.6.1 Field Studies

The studies included:-

(i) Assessment of the physical conditions of the road during which observed defects were noted. This involved closely examining the carriage-way, shoulders and drainage facilities. In as much as possible, detailed characteristics of each type of defect was noted and documented with photographs.

(ii) Traffic surveys were carried out to determine the Average Daily Traffic (ADT) at some suitable locations along the road.

(iii) Samples of materials comprising the pavement structure were collected at the failed sections for laboratory tests.

(iv) Truck weighing data was collected and used to determine axle loading for comparison with allowable legal limit.

1.6.2 Laboratory Tests

Laboratory tests were carried out on recovered materials in layers making up the pavement structure. The specific tests undertaken are outlined as follows:

(a) Surfacing

(i) Bitumen Content.

(ii) Specific gravity.

(iii) Grading test on left-over aggregates.

(b) Base Layer

(i) California Bearing Ratio (CBR) test.

(ii) Compaction test.

(iii) Sieve Analysis.

(iv) Atterberg limits.

(c) Sub-base Layer

(i) California Bearing Ratio (CBR) test.

(ii) Compaction Tests.

(iii) Sieve Analysis.

(iv) Atterberg limits.

(d) Sub-grade Layer

(i) California Bearing Ratio (CBR) test.

(ii) Compaction Test.

(iii) Sieve Analysis.

(iv) Atterberg limits.

CHAPTER TWO

2.0 LITERATURE REVIEW

2.1 Review of Causes and Modes of Pavement Failure

A road is required to provide a smooth and skid-resistant surface with long life and little maintenance (2). It is also primarily required to ensure adequate stability under traffic loads and environmental impacts (5). A road can be said to have failed if it cannot meet these basic requirements, though, defining "failure criterion" constitutes the biggest question and affects variation in design thicknesses obtained by various methods (1).

Two forms of failure have been defined; structural and functional failures (1). Structural failure is said to occur when either the entire pavement structure collapses or there is a breakdown of one or more of the pavement components of such a magnitude that it is incapable of sustaining the imposed loads. On the other hand, functional failure may or may not be accompanied by structural failure but is such that the pavement will not be able to carry out its intended functions without causing discomfort to motorists or high stresses on the vehicles that pass over it due to surface irregularities. A knowledge of the type of failure as well as the causal factors is important so as to know the kind of remedial measures to adopt. For example, a pavement that has developed a functional failure may only require some maintenance measures

to restore the riding quality of the wearing surface. The same pavement may break and crack due to overload which is structural failure (1).

It is particularly important to know whether the pavement distress is progressive which will lead to eventual failure or non-progressive.

The causes of pavement failure are varied and include a number of factors ranging from high load repetitions, quality of materials, environmental factors to design and construction methods. An investigation on Lagos - Ibadan expressway (9) was carried out when it was noted that a section of road had started to deteriorate barely one year after official opening. The causes of failure were found to include (i) use of poor quality materials (ii) lack of subdrains and (iii) heavy axle - loads. Axle loading on road pavement in Nigeria is generally believed to constitute a major contributory factor to premature road failures (8). A survey conducted on Kachia and Pambegua roads (8) showed the probable causes of the failures to be the combined effects of heavily loaded vehicles and high traffic volume, poor drainage, thermal and other environmental factors.

The overload, like excessive gross loads and high load repetitions and high tyre pressure, can cause surface irregularities to develop. Others are:

(a) Volume change resulting from wetting and drying or improper drainage (see Plate IV) can also cause disintegration of pavement materials.

(b) Breakdown of the surface course materials generating fines to unstable mix to develop (see Plate IX).

(c) Susceptibility of the subgrade materials to climatic conditions.

(d) Construction practices may also have some effect.

(e) Lack of maintenance like sealing of cracks and joints at proper intervals to ensure tight wearing surface.

Also structural failures in flexible pavements may also result from surface fatigue, consolidation or shear failure developing in the subgrade, sub-base, base or surfacing (1). The basement soil under the load is compressed and partially displaced laterally. This causes the pavement to bend and develop tensile and cleavage stresses. When the critical level of stress is reached, failure of the pavement with the collapse of its part results (2).

Displacement of material from beneath the area of loading (plastic flow) which leads to loss of strength of the pavement may occur in non-cohesive and poorly cohesive materials in the underlying soil. Also, repeated applications of load may cause roughening and cracking that ultimately load to complete failure (4). The role which each type of deformation plays in the destruction of the pavement varies for different types of pavement structures. Rate of deterioration of a road largely depends on such factors as original design stand-

ard, traffic load and prevailing climate (4). This shows that roads with different structures will experience different rates of deterioration.

Causes of structural failure have been summarized (5) to consist of:

(a) excessive stresses due to traffic causing increased deformation with each application of load with consequent failure of layers above. This has been categorized as the most important cause of failure and one considered in most design methods.

(b) Excessive stresses due to secondary causes such as moisture movement and temperature fluctuations.

(c) Stripping of the surface which occurs due to insufficient strength of adhesive bond between aggregates in surface and base layers.

Movements of vehicles generate vertical and horizontal stresses and consequently bring about deformations in various forms (5). Each time a vehicle passes, some deflection of the surface and underlying layers occurs on account of elastic deformation, consolidation of the base and sub-soils or a combination of elastic and plastic deformations (4). In elastic deformation, some recovery of the deformation is achieved so that pronounced permanent unevenness does not occur. However, in highly resilient soils, deflections under repeated heavy loads can cause fatigue failures in the surfacing (4). Consolidation deformation is permanent and is sometimes accompanied

by rutting while plastic deformation is progressive under load repetition. The latter results in shear failure and is accompanied by movement of the affected layers. This is believed to constitute one of the major causes of failure of pavement surfaces because the base, sub-base and subgrade layers are all susceptible to it (4).

The effects of the factors listed are in the form of instability, disintegration or fracture (2). Since it manifests itself, instability can be visibly identified in the form of shoving, distortion, corrugation and rutting. Disintegration of the paving materials can also occur in the form of ravelling, potholing and stripping. Fracture occurs in the form of cracking of various forms and can be as splitting into sections (2). The two most important types of cracking observed in Nigeria are the Alligator cracking and edge cracking (7).

In general, the mode of failure is strongly linked to its causes (5). The following is a brief description of the common modes of failures in road pavements.

(a) Cracks: Cracks occur in many forms, i.e.

(i) Transverse cracking - which occurs at right angles to the centre line of the road and is mainly caused by movement in the underlying layers and low temperature stresses.

(ii) Longitudinal cracks - which occur in the longitudinal direction and are attributed to weak joints between layers due to insufficient internal friction. They may also be caused by improper compaction of fill materials and sliding of side slopes.

(iii) Alligator cracks - which occur in the form of series of inter-connected small polygons. Their occurence is a primary indication of surface distress due to excessive deflection of the sub-grade and lower layers. They are normally traced back to improper compaction of the underlying layers.

(iv) Shrinkage cracks - which take the form of larger inter-connected polygons normally with sharp angles at the corner and are mainly due to moisture movement in lower layers.

According to AASHTO Road classification fatigue cracks belong to three classes, namely:-

Class I (Fatigue Cracking):- is longitudinal cracking in the wheel path inter-connected by short transverse cracks. They are generally wavy and may criss-cross in some instances.

Class II (Fatigue Carcking):- are cracks that have progressed into an interconnected form in a grid type pattern having the appearance of chicken-wire or Alligator skin. They are basically Alligator cracks except that in this

instance, the pieces are still intact as if disintegration has not begin.

Class 3 (Fatigue Cracking) - is the progression of Class 2 cracking with spalling occuring at the edges, loss of integrity between pieces and segments of the pavement loosening and moving under traffic.

(b) Rutting

Rutting occurs in form of longitudinal depressions formed under the wheel paths and arises primarily through differential compaction ofthe road structure as a result of consolidation or lateral movement in the underneath layers or displacement of the asphaltic surface.

(c) Ravelling

Ravelling involves the disintegration of aggregate from the surface or edges of pavement. Its major causes are improper compaction, construction during wet weather, use of dirty or disintegrating aggregate, deficient asphalt mix or overheating of the mix.

(d) Shoving

Shoving is the lateral displacement of paving materials due to the action of traffic resulting in bulging of the surface.

(e) Corrugations

These are transverse undulations at regular intervals in the surface forming closely spaced valleys and crests. They are caused by instability in the surfacing materials.

(f) Potholes

Potholes are holes in the surface caused by use of poor materials or other defects and can occur in conjunction with Alligator cracking (Class 2 cracks). Their presence in the pavement indicates structural failure.

(g) Bleeding

Bleeding is caused by excessive use of bitumen in the surfacing and its occurence leads to loss of skidding resistance.

2.2 Review of Past Case Studies

Over the years various studies have been carried out on road deterioration both in the country and abroad. The approaches differed in scope depending on resources and the level at which the research was being carried out. For example, while in some cases the test sections were limited to only small locations of the road (9), others cover larger portions or quite a number of lengths (18, 14). However, the procedure generally involved field and laboratory tests on sections of the road from which conclusions were drawn regarding the causes of the failure. A number

of factors have been found to contribute to road deterioration and the findings have been quite useful in shedding light on the subject.

One of the early related works in this country was an investigation on the first expressway in the country, the Lagos-Ibadan expressway (9). Premature failures were observed between KM 92 + 00 and KM 98 + 00 barely one year after officially opened. The Civil Engineering Services Division of FMWH was directed to carry out an investigation. Visual inspection of the road was first undertaken to detail extent and severity of failures. Test locations were then selected and drainage characteristics of each area noted. Tests were then carried on failed and unfailed sections to determine whether the unfailed sections were potential areas of failures. An effort was also made to investigate the effect of the high water table found in some locations after the road had been constructed. Boreholes were sunk and soil samples were collected for laboratory tests. The data was used to produce boring log. Pipes were installed in locations where water was met in order to facilitate close study of water table movement. Traffic count and axle load survey was also conducted to determine traffic count and axle weighings. Analysis of field and laboratory tests results showed the causes of failure to include use of poor materials especially subgrade fill and sub-base materials, lack of sub-drains in the perched water table as a result of presence of rock close to the surface and heavy axle loads of traffic in the east carriageway. Remedial measures recommended were provision of subdrains

using graded-filter materials, open side drains to carry run-off, removal of existing sub-base and reconstruction. Others were placing of standard crushed stone base and concrete surfacings, an overlay and enforcement of the legal axle load limits.

A similar research was carried out along Kaduna - Abuja Highway in 1989 (14). The main link to the new Federal Capital from Kaduna, the road had been experiencing repeated road failures and investigation concerned assessment of the structural and functional adequacy of the pavement along the section experiencing repeated failure. The test section consisted of the first fifty - five kilometers from Kaduna found to be experiencing repeated failures. Surface deflections tests combined with visual inspection were carried out to locate weak spots and identify types of pavement distresses. The results of the deflection tests and visual inspection were used to locate test pits for field tests and collection of samples for laboratory tests. Ahmad (14) concluded that the failures were due to poor quality control during construction resulting in the use of sub-standard materials, poor drainage condition, high stability asphaltic concrete surface over a weak base leading to fatigue cracks. This led to high degree of surface roughness and reduced stability. Some sections were found to have completely failed due to failure of the subgrade to support the upper layers of the pavement. The poor condition of some sections led vehicles to divert to other portions which accentuated deformations in some sections. Ahmad finally concluded that the pavement was structurally and functionally inade-

quate and therefore recommended reconstruction and stregthening measures to be adopted. However, he did not take into consideration the axle overloading factor which is a major factor causing road failures (9).

A review study of pavement materials was undertaken from data collected by PEU at various locations around the country by John Harvey under the TRDR programme in 1986 (18). The objective was to examine the data from past Highway construction projects to determine the level of compliance with FMWH specifications and identify possible deficiencies in construction and quality control methods. The field data for the study was collected from similar TRDF projects which include

(a) The master Test section testing program for development of the PEU overlay design procedure in 1986.

(b) Evaluation for overlay design of 3,349KM of trunk road network in 1987

(c) The evaluation of three sections of trunk road near Numan Gongola in 1987; and

(d) The evaluation of a section of highway A2 between Kaduna and Abuja in 1987.

The laboratory data was taken from tests on disturbed samples collected during the above studies. A computer format was used for the study. The procedure was based on fatigue cracking and permanent deformation (rutting) criteria. The method incorporates a mechanistic prediction of permanent strains using elastic layer theory, field deflection measurements, condition survey results, material strength tests, environmental factors such as

temperature and rainfall and estimates of past and future traffic in terms of equivalent axle loads (10). The results showed that pavements in Nigeria were underdesigned in terms of quality of materials, workmanship and axle loading. The combined effects of these factors contributed to the premature failures in the road network. The recommendations given included axle load regulation, standard design based on Nigerian conditions, maintenance and quality control. The research could be described as one of good efforts made in the country on this subject. It was spectacular considering the volume of data used and the methodology.

Axle loading factor was found to be one of the major factors contributing to premature failure of roads in this country (9). Typical examples were the Lagos - Ibadan expressway (9) and Entrance road to Kaduna Refinery off Kaduna-Kachia road (8). A study on this factor would therefore beuseful. An attempt was made by Mohammed Yakubu (8) on two federal Highways in Kaduna State i.e. Kaduna - Kachia road and Zaria - Pambeğua road. A condition survey was undertaken in which an attempt was made to relate the existing failure conditions to such factors as types of vehicles, parking habits, environmental conditions etc. The survey showed that a large proportion of the vehicles were overloaded with number of axles greater than the legal maximum high. The failure was however found to be functional caused by the combined effect of heavy trucks, poor drainage and other environmental factors as well as bad parking habit of trucks

along the road. The study showed that axle loading as major factor contributing to road failures does not exist alone. The failures were mostly as a result of combined effects of many factors.

Jones in 1984 (12) carried out a research on the deterioration of unpaved roads under TRRL. The research was aimed at providing new and improved deterioration relationships for unpaved roads for use in computer model for estimating the construction costs, maintenance and vehicle operating costs for roads in developing countries. Test sections were located on public road network in Kenya and their rates of deterioration were measured and quantified in such terms as gravel loss, surface roughness, depth of loose surface material and rut depths. The test sections were monitored for a period of two years. The deterioration was related to cumulative traffic, design standards and construction, maintenance strategy and climate. The findings of the research showed that the unit rate of gravel loss was higher than that predicted by the road investment model. It was also found that the total loss of gravel from unpaved roads in developing countries was increasing with additions to the road network. The study was also able to quantify separate relationships for different types of gravel materials which was an important improvement on previous studies of gravel roads. The effect of rainfall was found to increase surface roughness and effective maintenance was found to

minimize amount of loose materials and reduce vehicle operating costs. Overloading was also found to be minimal.

The study helped in the development of a computer model to aid investment decision within the road sector. A model has also been developed on the basis of this work by the World Bank. The results of the study, although of direct benefit to Kenya could also be applicable to other developing countries.

A similar research was undertaken by Roberts in 1984(13) under TRRL and Ghana Highway Research Programme. The study was on the performance of selected lengths of surface dressed roads in Ghana monitored over a period of two and half years. Road surface condition measurements were related to pavement deterioration factors. Criteria for maintenance interventions and user cost savings resulting from road maintenance were given. The study determined that the rate deterioration was slower in the drier parts that those areas where the annual rainfall exceeded 150mm. It was also noted that lack of maintenance led to serious deterioration of the test sections. Breaking of the edge of the wearing course accounted for a significant portion of the damage to the surface. The quality of construction was also found to have great influence on pavement deterioration. For example, some surfaces continued to carry heavy traffic without appreciable deterioration after many years whereas poorly surfaced sections carrying light traffic deteriorated extensively within three years.

The results of the study also showed that effective patching and resealing could bring about significant reductions in unit vehicle operating costs. It went further to recommend a wheel track intensity of $0.5m/m^2$ as a threshold value indicating probable need for resealing. Hand sealing of individual cracks threshold value was given as $0.1m/m^2$. Standards of pothole patching were found to be poor and improvement was recommended. Finally emphasis was recommended for rational system for establishing priorities among maintenance projects and roads carrying more than 200veh/day with a roughness greater than 400mm/km were recommended for patching and sealing. However, roads carrying more than 400veh/day and roughness greater than 4000mm/km needed resurfacing.

CHAPTER THREE

3.0 DATA COLLECTION

3.1 Condition Survey

Pavement condition survey is important in determining the relative need for rehabilitation based on practical engineering considerations. This is through evaluation of such factors as user-related factors, maintenance need and overall protection of the investment (18). Thus, it serves as a means of judging the adequacy of the existing pavement under the current traffic (14).

The condition survey was carried out with the aims of:-

(i) Identifying the various modes of failures along sections of the road through observation of the specific distress types and their severities.

(ii) Assessing the surface condition of the carriageway through rating of the riding quality as well as evaluating such factors as skid resistance, roughness and pothole potential.

(iii) Assessing the shoulder and drainage conditions through evaluation of the distress types occuring on the shoulder surface and drainage facilities.

(iv) Assessing through closer examination, the extent and type of failure. This would be through further evaluation of the distress types, severities and quantities.

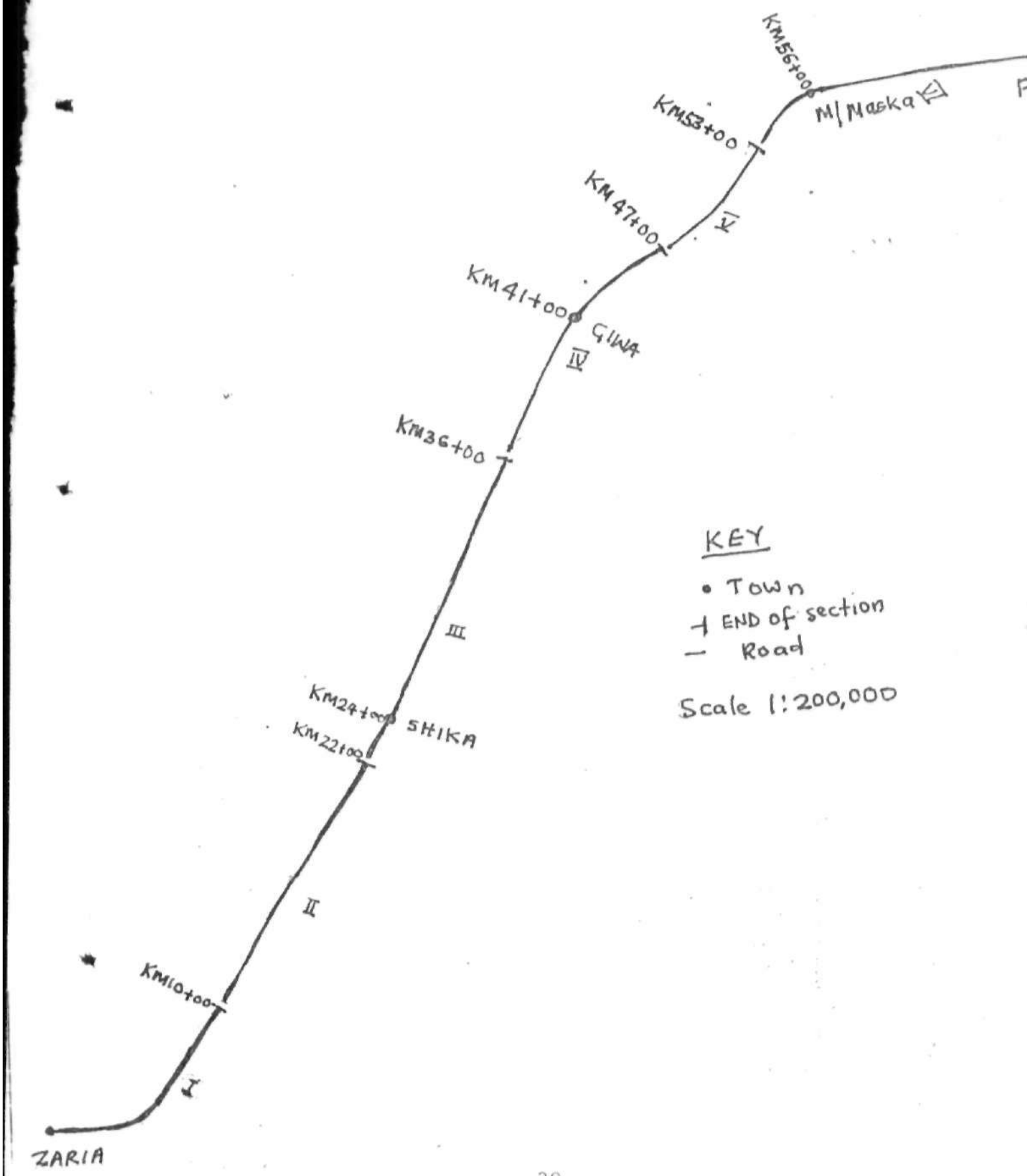
The total length of the road was about sixty six kilometers. KM 0 + 00 had been arbitrarily fixed at the overhead bridge at Hanwa and KM 66 + 00 at Brinin Gwari junction at Funtua. The road had also been divided into six sections for the purpose of the study. This was arbitrary and strictly for convenience. A map showing the approximate length and location of each section is presented in Fig. 3.1.

Major elements considered were carriageway, shoulders, verges and drainage facilities.

3.2.1 Test Pits

Four locations, found during the condition survey to be the worst distressed and representing the weakest portions, were selected for detailed investigations. Four pits were dug from each location bringing the total to sixteen pits. For each selected pit location, the asphalt concrete over the pit was removed carefully after cutting. In-situ tests were carried out on each layer. After conducting the tests, the pit was excavated until the next layer was encountered. The same series of tests were then carried out on each succeeding layer.

FIG 3.1 A map showing the sections along the road



The in-situ tests carried out included:-

(a) Dry density test

(b) Moisture content

(c) Measurement of layer thickness.

Sufficient quantities of the materials were packed and transported to the laboratory of Department of Civil Engineering, Ahmadu Bello University, Zaria for testing. The in-situ dry density tests were carried out using the Sand Replacement Method as contained in B.S 1377 test No. 10A on base, sub-base and sub-grade layers. The average relative degree of compaction from each section was computed by comparison with the maximum dry density later obtained from laboratory investigations.

3.2.2 Laboratory Tests

A summary of tests undertaken on the different layers of the pavement is as follows:-

(i) Surfacing

(a) Determining the bitumen content

(b) Grading tests on left-over aggregates

(c) Specific gravity.

(ii) Base

(a) Atterberg limits tests

(b) Grading tests

(c) Compaction tests

(d) Laboratory CBR tests (soaked and unsoaked)

(iii) Sub-base

(a) Atterberg limits tests

(b) Grading tests

(c) Compaction tests

(d) Laboratory CBR tests (soaked and unsoaked).

(iv) Subgrade

(a) Atterberg limits tests

(b) Grading tests

(c) Compaction tests

(d) Laboratory CBR tests (soaked and unsoaked).

3.2.3 Compaction/CBR Tests

(a) Base and Sub-base Materials

The base and sub-base materials were compacted using the West African Standards (WAS) in accordance with the Federal Ministry of Works and Housing specifications. The samples were compacted in a Standard mould (15.24cm dia x 12.70cm height) in five layers with 25 blows using 4.53kg hammer and 45.72cm drop.

The compacted specimen was then tested first without soaking, in the C.B.R. apparatus and later immersed in water for a period of 24 hrs before it was tested again. A dial gauge was also attached to measure swelling during the period of soaking.

(b) Subgrade Materials

The subgrade material was compacted using the British Standard Method of compaction as required by the FMW & H General specifications. The specimen was compacted in a standard mould (10.16cm dia x 11.64cm height) in three layers each layer approximately 1.53cm thick and given 25 blows each with 2.5kg hammer and 30.48cm drop.

The specimen was tested first in the C.B.R. apparatus before being immersed in water for a period of 48 hrs and tested again. Similarly a dial gauge was attached to measure swelling during the period of immersion.

3.2.4 Traffic Survey

Manual traffic count was used to record the traffic volume and vehicle classification (AASHO classification). Three points were selected for the exercise:-

i) Samaru located around KM 07 + 00

ii) Gangara located around KM 38 + 00; and

iii) Bakin Dutse located around KM 63 + 00 at the Funtua end.

The exercise was carried out for a period of three days (twelve hours everyday) at the different locations mentioned above. Wednesday was chosen to avoid market days so that local traffic increase would not affect the results obtained.

The present Average Daily Traffic (ADT_p) and percent trucks were determined. Similar records were obtained from Pavement Evaluation Unit (PEU) of FMW & H on the same exercise carried out in 1987. The average growth rate was estimated and analysed by comparing with an estimate based on econometric data (11). The results were also used to estimate the Equivalent Standard Axle loads (ESALs) likely to have been carried by the pavement since it was overlaid in 1979 (i.e. past and future traffic).

3.2.5 Axle Load Survey

Axle load data is required to accurately quantify the loads applied to highways. Average axle loads combined with AADT and percent trucks will allow the determination of the type of loads applied to a roadway. The non-availability of weigh bridge and other constraints made it impossible to carry out a current axle load survey. However, secondary data was obtained from a similar exercise conducted by the Pavement Evaluation Unit (PEU) of the Federal Ministry of Works and Housing (FMW & H).

The Axle load data which consists of the mix of axle weights of trucks using a highway is converted to an equivalent number of applications of 8,200kg (18,000lb) single axle load, by means of equivalence factors.

An equivalence factor relates the damage done by one pass of a given axle load to an equivalent number of standard axle load applications to cause the same damage. Truck axle weight data was collected on representative samples of trucks using the road. Each axle of each truck was weighed separately.

It should also be noted that single and tandem axles are treated separately by the use of a different standard reference load. The reference load for single axles is equal to the standard 8,200kg ESAL. Due to the better spreading abilities of tandem axles and considering that there are two axles, a reference load of 14,500kg is used. Thus, a tandem axle load of 14,500kg has an 8,200kg single axle load equivalence factor of 1.0 and this implies that one application of a 14,500kg tandem axle load causes the same damage as one application of an 8,200kg single axle load.

The equivalence factors are computed from wheel load data taken from one side of the vehicle by first multiplying the measured load by two to get the total load on the axle. Also, wheel weights for tandem axles are added to get the total load on the pair of axles. In the computations, equation 3.1 was used (11), i.e.

$$F_j = 1.02 \left[\frac{L_j}{L_s}\right]^{3.81} \qquad 3.1$$

where F_j = 8200kg single axle load factor for axle load j.

L_j = axle load j in kg

L_s = standard axle load, 8200kg for single axle and 14500kg for tandem axle.

To determine the average 8,200kg single load equivalence factor, the equivalence factor of all axles are summed and divided by the total number of trucks. This is shown in equation 3.2

$$F_{avg} = \left[\sum_{j=1}^{NA} F_j \right] / NT \qquad 3.2$$

where F_{avg} = average 8,200kg single axle load equivalence factor per truck.

NA = number of axles

NT = number of trucks; and

F_j = equivalence factor for axle load j.

To determine an average equivalence factor, all single axle weights are tabulated together and all tandem are put together. Equivalence factors are computed for each axle individually. The overall average factor is then calculated as the sum of tandem factors plus the sum of single factors divided by the total number of trucks weighed.

CHAPTER FOUR

4.0 RESULTS PRESENTATION, DISCUSSION AND ANALYSIS

4.1 Condition Survey

(a) Section 1 (KM 0 + 00 - KM 10 + 00)

The length of this section was estimated to be ten kilometers and from the survey, six locations had been identified to have completely failed with various forms of failure manifested. It was also found that more than forty percent of the length of the section exhibited at least one form of failure or the other. This ranged from primary cracks (Class I) to secondary and tertiary cracks (Class II & III) (see plate III - X). Also, the area of pot-holing had been estimated to be about $20m^2/km$.

The condition of the shoulders was also generally poor as they were characterized by shoulder drop-offs which in some instances, reached the edge of the carriage-way. The shoulder level was also high in some locations leading to overflooding of the carriageway during rain-storms. This probably lead to increase in the rate of deterioration of the pavement as the surface was exposed to greater water infiltration (see plate IV).

The drainage condition was similarly very poor as drainage facilities were either lacking or where provision was made, had been blocked or overgrown by weeds (Plate VII).

On the whole, the section was found to be characterized by various types of distress with over 40% of the length exhibiting one form of failure or the other. Also, more than 20% of the length showed signs of potential failure mostly in the form of primary cracks.

Most of the locations associated with complete failure were in the fill sections. However, primary cracking which shows signs of imminent failure was observed in both cut and fill sections.

(b) Section 2 (KM 10 + 00 - KM 22 + 00)

The length of the section was estimated to be twelve kilometers with only one location identified as showing complete failure around KM 15 + 00 (see Plate 3). However, about 30% of the surface area was observed to display signs of imminent failure which was mostly in the form of primary cracking (Class I). Also, the section was identified to be characterized by potholing where the area of potholing was estimated to be $80m^2/km$ (see Plate VI).

As in the previous section, provision for drainage was also found to be generally poor as facilities were either lacking or blocked and overgrown by weeds where provided. The shoulder condition was similarly poor and characterized by cracking, washouts and potholing. However, the surface riding quality could be described as fair. This was because though there were a lot of potholes, other types of major distress which cause

riding discomfort like secondary and tertiary cracks, rutting, corrugations and ravelling were not severe.

The failures found in the section were not restricted to either the cut or fill sections.

(c) Section 3 (KM 22 + 00 - KM 36 + 00)

The approximate length of the section was estimated at about fourteen kilometers. The peculiarity of the condition of the section was that none of its portions was found to have completely failed. However, over 25% of the total surface area was observed to display signs of imminent failures mainly in form of primary cracks (Class I) (see plate 5). These were in the longitudinal direction along the wheel path interconnected by short transverse cracks indicative of surface distress (section 2.1). Also, instances of potholing, estimated at $10m^2/km$, were not severe and were located mainly along the centre-line of the road in many locations. The pattern was indicative of improper joint during construction. Patching work, which is mainly a form of routine maintenance, had only been carried out in some localized sections and not satisfactorily as driving on them was bumpy.

The drainage condition was generally very poor as in other sections while the shoulder condition could be described as fair because instances of drop offs and potholing were not severe.

The surface riding quality could be rated as good with occurance of major types of distress not common. The failures were found in both the fill and cut sections and, on the whole, the road condition could be rated as fair with none of its portions completely failing, pot-holing not severe and fair shoulder condition.

(d) Section 4 (KM 36 + 00 - KM 47 + 00)

The length of the section was approximately eleven kilometers and situated between KM 36 + 00 and KM 47 + 00. Two locations had been identified to have completely failed. They were located around KM 44 + 00 and KM 46 + 00 respectively. It had also beenobserved that signs of imminent failure were not severe with less than 20% of the total carriageway surface showing primary cracking (Class I). Also, shoulder condition was good as cases of washouts, potholing and cracking less severe. However, the drainage condition was generally very poor as in other sections.

Cases of potholing was also less severe and was estimated to be $15m^2/km$ which was relatively small compared to other sections. The surface riding quality could be rated as fair with instances of potholing, rutting, corrugations and other types of major distresses not rampant.

Generally, the road condition could be described as fair.

(e) Section 5 (KM 47 + 00 - KM 53 + 00)

The length of the section was estimated to be six kilometers located between KM 47 + 00 and KM 53 + 00. As in the case of section 3, none of the portions was identified as completely failing. The section could be rated as good and obviously the best compared to other sections.

Instances of potholing, severe cracking, rutting, shoving and ravelling were minimal. The area of potholing was estimated at about only $5m^2$/km of the total carriageway surface area. Few locations also showed signs of potential failure estimated at less than 20% of the total carriageway surface area.

However, both the drainage and shoulder conditions were found to be generally very poor. The shoulders were characterized by drop-offs, potholing and cracking whereas drainage facilities were mostly not provided.

On the whole, the road condition could be described as good but the shoulder and drainage were in a very poor condition. This could lead to fast deterioration of the carriageway itself unless necessary remedial measures were taken.

(f) Section 6 (KM 53 + 00 - KM 66 + 00)

The length of the section was about thirteen kilometers and was situated between KM 53 + 00 and KM 66 + 00. Three of its portions were found to have shown complete

failures displaying many forms of failure like secondary and tertiary cracks (See Plates 6 & 7), shoving and potholing, etc.

A number of points were also observed to display signs of potential failure mainly in form of primary cracks estimated at greater than 20% of the total carriageway surface area. Cases of potholing were also found estimated at about $20m^2/km$. However, the surface condition could be rated as fair in terms of riding quality. Though potholing was relatively significant, other forms of failure like rutting, corrugations, ravelling etc. which accentuate riding discomfort were less severe.

The shoulder condition was fair unlike in the previous sections as instances of drop-offs, potholing and cracking were not common. The drainage condition was very poor as in the case of other sections.

The condition survey data was summarized in Table 4.1.

The condition survey data showed that the road was generally characterized by various forms of failure, ranging from different classes of cracks to potholing, rutting, stripping, ravelling and corrugations. Ten locations were identified as showing total failure in most cases accompanied by Class III cracks, stripping to potholing and total disintegration (Plates II - X). It was also observed that the failures were not restricted to either cut or fill sections but larger proportions

TABLE 4.1: SUMMARY FINDINGS FROM THE CONDITION SURVEY

SECTION	LENGTH (km)	NO. OF COMPLE-TELY FAILED SEC-TIONS	RIDING QUALITY	20% OF TOTAL SURFACE SHOWING SIGNS OF FAILURE	POTHOLING m^2/km	SHOULDER CONDI-TION	DRAINAGE CONDI-TION	REMARKS
1	10	6	POOR	YES	20	POOR	V. POOR	Over 40% of area showing at least one form of failure
2	12	1	FAIR	YES	80	POOR	V. POOR	Characterized by a lot of pot-holing
3	14	NONE	GOOD	YES	10	FAIR	V. POOR	Road condition fair.
4	11	2	FAIR	NO	15	GOOD	V. POOR	Road condition fair.
5	6	NONE	GOOD	NO	5	POOR	V. POOR	Road condition good.
6	13	3	FAIR	YES	20	FAIR	V. POOR	Over 20% of area at least one form of failure.

occured in the cut sections. This could be attributed to the fact that cutting tends to bring the formation level closer to the water table which increases the susceptibility of the pavement structure to changes in moisture. It is most probable that the high moisture content contributed to the premature failures found in these sections.

The fill sections where failure occurred also were mostly locations at stream crossings or waterlogged areas. The failures found in the fill sections could likely be due to improper compaction of the fill materials coupled with high moisture condition of the locations. The highest number of failed sections were found in section 1 (KM 0 + 00 - KM 10 + 00) while none could be found in section 4 (KM 36 + 00 - KM 47 + 00) and section 5 (KM 47 + 00 - KM 53 + 00).

The surface riding quality ratings ranged from poor to good. Section 1 was rated as poor which indicates its poor state as shown above, Sections 2, 4 and 6 were rated as fair while Sections 3 and 5 were rated as good. On the average, the road could be rated as fair in terms of its riding quality. This shows that despite the poor quality materials used and improper compaction, the road was still capable of maintaining a fair riding quality.

Apart from the sections identified to have completely failed, a large proportion of the carriageway surface was observed at close examination to display signs of imminent failure. This was mostly in form of

small and interconnected longitudinal cracks (AASHTO Class I) (Plate I). It was estimated that over 70% of the sections had more than 30% of their lengths exhibiting signs of potential failures. This is indicative of surface distress due to movements in the underlying layers and shows the likelihood of improper compaction of the pavement materials. It could also be related to the use of poor quality materials with high plasticity as determined from the laboratory tests (Section 4.2.3), leading to excessive deformation under traffic loads. It could also be concluded that since Class I cracks are primary indication of surface distress as a result of deflection in the underlying layers under traffic loads, more portions would be expected to fail as the progression develops unless necessary remedial measures were adopted.

Instances of rutting were not severe and only occured in few locations (Plate VIII) while corrugations were mostly observed in Sections 1 and 6. Section 2 was found to be characterized by a lot of potholing occuring mainly along the centre-line of the road (Plate XI) estimated at $80m^2/km$ followed by Sections 1 and 6 ($20m^2/km$) while Section 5 had the least area of potholing.

The shoulders were surface dressed with average width of 2.0m on both sides. The condition of the shoulders could be generally described as poor, characterized by drop offs and potholes (Plates IV & VII). However, their condition in Section 4 (KM 36 + 00 - KM 47 + 00) was observed to be still good while Sections 3 (KM 22 + 00 - KM 36 + 00) and 6 (KM 53 + 00 - KM 66 + 00) was fair. Those of Sections 1, 2 and 5 were poor.

The drainage facilities were also generally poor in most locations. They were either lacking or, where provided, had been blocked or overgrown by weeds (Plate VII). Runoff water along the roadway, could not be properly carried to the discharge point and this could lead the saturation and subsequent weakening of the pavement layer. The damaging effects of this phenomena might have contributed to the premature failures thereby shortening the service life of the road.

4.2 Test Results on Soil Samples

4.2.1 Dry Density and Moisture Contents

The in-situ dry density were carried out using the sand-replacement method as contained in BS1377 (test No. 10A) on base, sub-base and subgrade layers. The average relative degree of compaction from each section was computed by relating the results to the maximum dry density later obtained from the laboratory. The results are given in Tables 4.2 and 4.3.

. The relative compaction results showed that none of the tested samples met the requirement set by FMW & H General Specifications which is 100% for base and sub-base materials and 95% for subgrade materials. However, 42% of the samples tested were found to be within 90% of the specified values for all the pavement layers while none was below 75% of the specified values.

The low degree of compaction could be attributed to changes in climatic conditions particularly moisture changes which could occur due to infiltration from surface cracks and rise in water table. The majority of failed sections were found in cut-sections, cutting tends to raise the water table and this shows the likelihood of the rise in water table as discussed in Section 4.1. Also, failed sections in fill areas were found at river-crossings and water-logged area which could have been caused by rise in water table and improper compaction. Although, variation in density should be expected after construction, it is significant and could not be due purely to climatic changes since the density is expected to increase with surcharge weight and traffic loads. Therefore, a likely reason may be weakening due to increased moisture content and improper initial compaction during construction. The variation between field and maximum dry densities are shown in Figs 4.1 - 4.3. The variations of degree of compaction with the specification are shown in Figs 4.4 - 4.6.

The distributions of the in-situ moisture contents compared with the optimum moisture contents (OMC) are shown in Figs 4.7 - 4.10. For the base and subgrade layers, 50% of the materials tested had in-situ moisture contents (IMC) greater than the OMC, 25% within four percent of the OMC while 25% had more than four percent less than the optimum values. However, for the sub-base layer, only 38% of the samples tested had IMC greater than the OMC, 13% within four percent while 49% more than four percent less than the OMC indicating a more stable condition for the sub-base layer.

The conclusion that can be drawn from the foregoing analysis is that the moisture condition of the pavement materials was tending to saturation particularly for the base and subgrade materials, indicating weak state. This could be attributed to the fact that the tests were carried out during the rainy season when the moisture content is expected to be high. Another reason could be the fact that the pits were located in failed sections of the road where infiltration through surface cracks or potholes might have occured. The lower moisture content of the sub-base layer might be due to the protection of the base and subgrade layer from above and beneath respectively. On the whole, it can be concluded that there was high moisture condition that might have accentuated the weakening of the materials and increased deformation.

TABLE 4.2: SUMMARY OF FIELD SOIL TEST RESULTS

LOCATION	TEST PIT	PAVEMENT LAYER	LAYER THICKNESS (mm)	MOISTURE CONTENT (%)	DRY DENSITY UNIT
1	A	BASE	180	11.2	1.82
		SUB-BASE	230	15.9	1.81
		SUBGRADE	-	9.4	1.79
	B	BASE	235	13.0	1.85
		SUB-BASE	300	15.15	1.83
		SUBGRADE	-	17.1	1.81
	C	BASE	230	13.4	2.04
		SUB-BASE	290	8.8	1.90
		SUBGRADE	-	7.0	1.76
	D	BASE	290	16.5	1.75
		SUB-BASE	265	3.5	1.74
		SUBGRADE	-	6.4	1.80
2	A	BASE	162	10.1	1.68
		SUB-BASE	196	10.9	1.95
		SUBGRADE	-	11.5	1.90
	B	BASE	172	11.1	1.94
		SUB-BASE	190	13.0	1.90
		SUBGRADE	-	12.30	1.79

TABLE 4.2 (CONT'D)

LOCATION	TEST PIT	PAVEMENT LAYER	LAYER THICKNESS (mm)	MOISTURE CONTENT (%)	DRY DENSITY UNIT
2	C	BASE	193	14.4	2.14
		SUB-BASE	183	11.4	1.89
		SUBGRADE	-	6.0	1.92
	D	BASE	182	8.0	1.49
		SUB-BASE	187	13.9	1.91
		SUBGRADE	-	16.1	2.14
3	A	BASE	200	7.5	1.49
		SUB-BASE	215	15.5	1.80
		SUBGRADE	-	17.7	1.69
	B	BASE	215	9.3	1.75
		SUB-BASE	218	15.3	1.50
		SUBGRADE	-	15.0	1.73
	C	BASE	190	9.0	1.92
		SUB-BASE	189	5.1	1.74
		SUBGRADE	-	13.4	1.68
	D	BASE	195	9.7	1.80
		SUB-BASE	193	15.7	1.67
		SUBGRADE	-	21.0	1.70

TABLE 4.2 (CONT'D)

LOCATION	TEST PIT	PAVEMENT LAYER	LAYER THICKNESS (mm)	MOISTURE CONTENT (%)	DRY DENSITY UNIT
4	A	BASE	186	7.7	1.85
		SUB-BASE	194	18.5	1.71
		SUBGRADE	-	12.9	1.70
	B	BASE	192	6.9	1.69
		SUB-BASE	174	12.6	1.73
		SUBGRADE	-	10.5	1.75
	C	BASE	149	18.5	1.89
		SUB-BASE	193	9.0	1.76
		SUBGRADE	-	9.0	1.80
	D	BASE	170	5.6	1.69
		SUB-BASE	182	7.1	1.77
		SUBGRADE	-	10.5	1.75

TABLE 4.3: RELATIVE DEGREE OF COMPACTION OF LAYERS

LOCA-TION	PAVEMENT LAYER	FIELD DRY DENSITY (g/cm^3)	LAB MAXIMUM DRY (g/cm^3)	DEGREE OF COMPACTION (g/cm^3) %
1	BASE	1.87	2.09	90
	SUB-BASE	1.82	2.15	85
	SUBGRADE	1.79	2.11	85
2	BASE	1.93	2.10	92
	SUB-BASE	1.92	2.11	91
	SUBGRADE	1.92	2.23	86
3	BASE	1.74	2.23	78
	SUB-BASE	1.68	2.14	79
	SUBGRADE	1.70	2.23	82
4	BASE	1.75	2.14	82
	SUB-BASE	1.74	2.17	80
	SUBGRADE	1.75	2.29	76

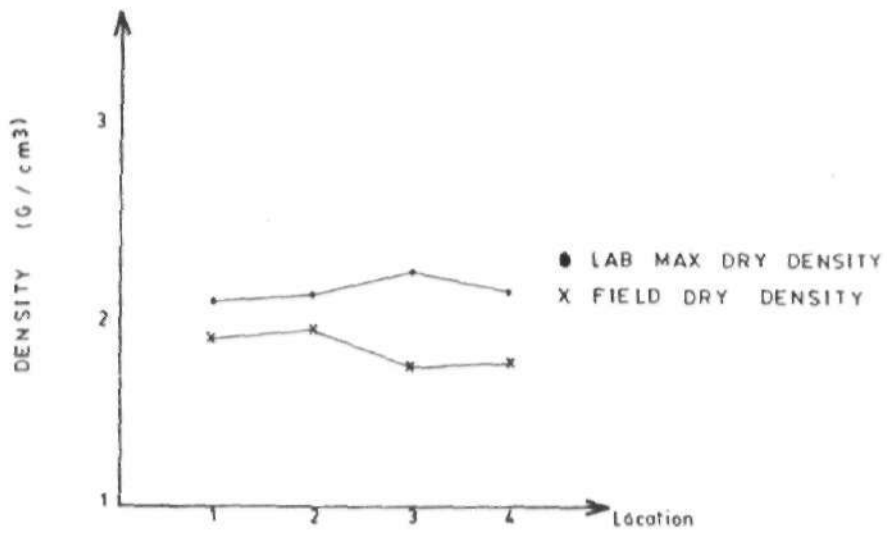


FIG. 4.1 LOCATION VS FIELD/LAB MAX DRY DENSITY VARIATION FOR BASE LAYER

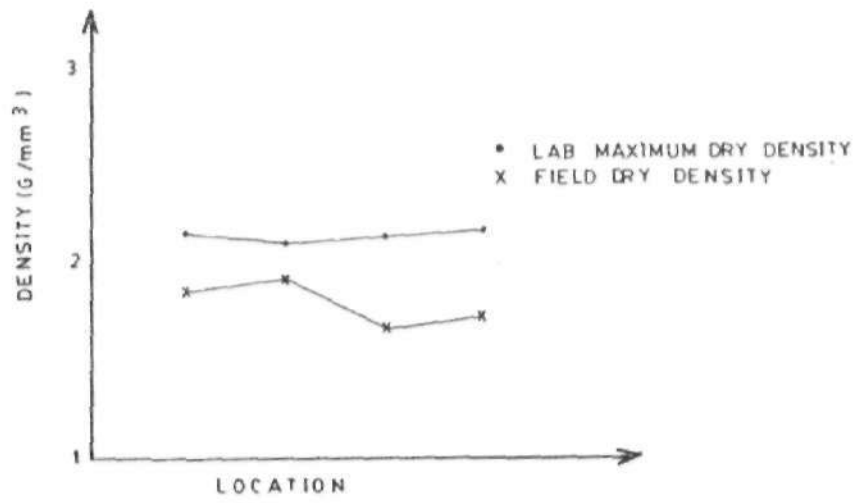


FIG. 4.2 LOCATION VS FIELD/LAB MAX DRY DENSITY FOR SUB BASE LAYER

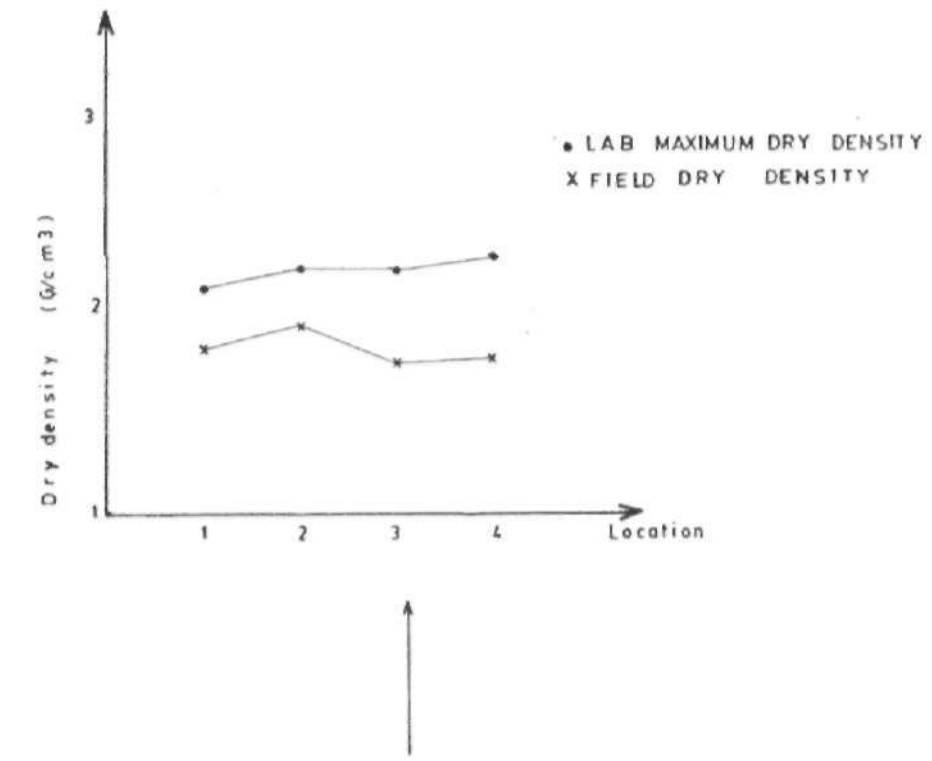


FIG. 4.3 LOCATION VS FIELD / LAB MAX DRY DENSITY FOR SUB GRADE LAYER

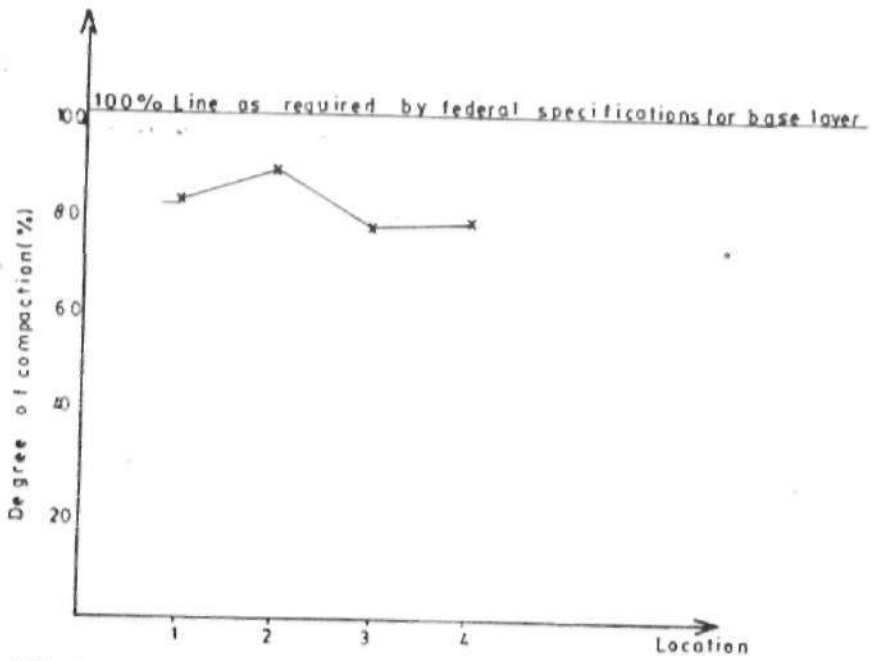


FIG. 4.4 RELATIVE COMPATION VARIATION WITH LOCATION FOR BASE LAYER

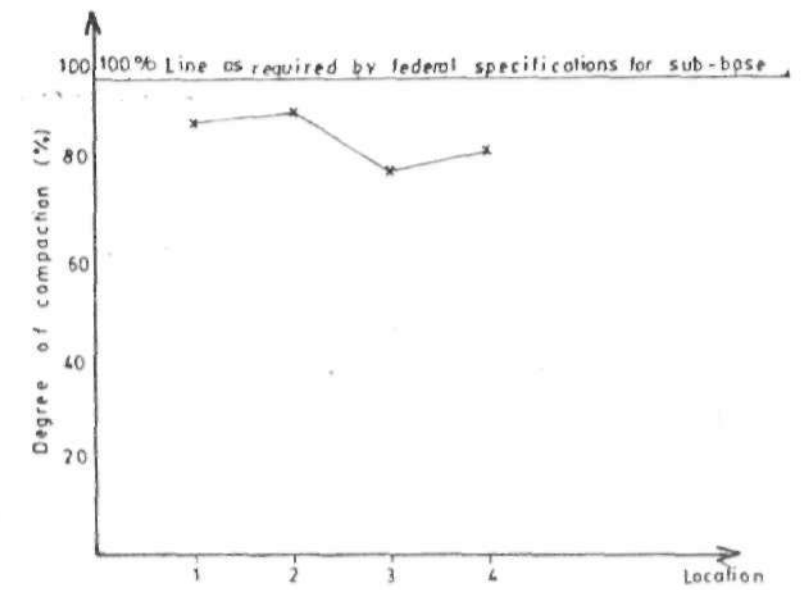


FIG. 4.5 RELATIVE COMPACTION VARIATION WITH LOCATION FOR SUB BASE

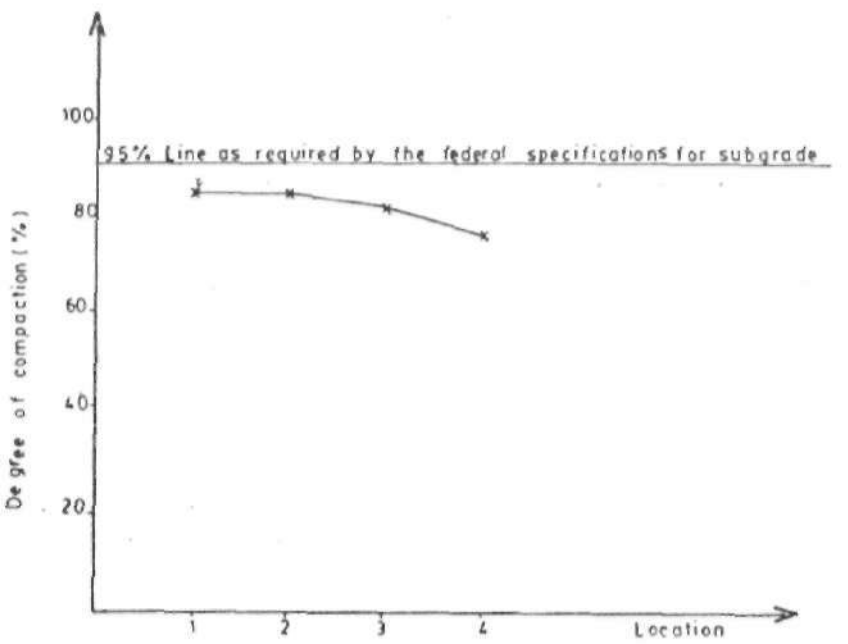


FIG. 4.6 RELATIVE COMPACTION VARIATION WITH LOCATION FOR SUB GRADE LAYER

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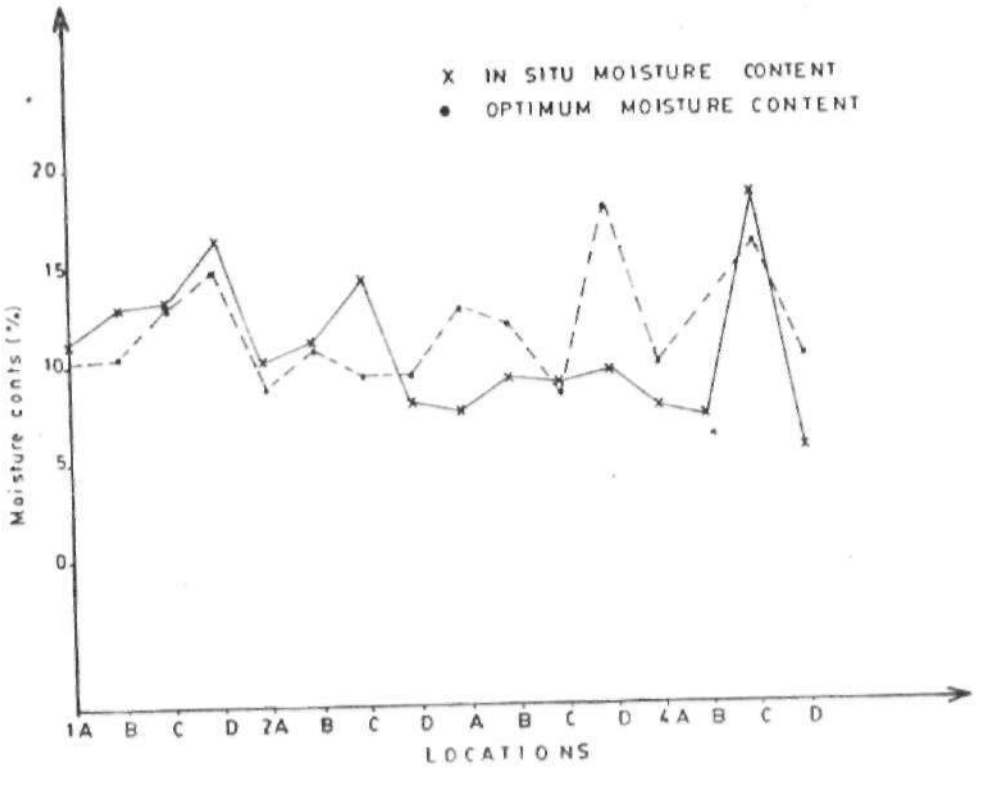


FIG. 4.7 PLOT OF IN SITU / OPTIMUM MOISTURE VS LOCATION FOR BASE LAYER

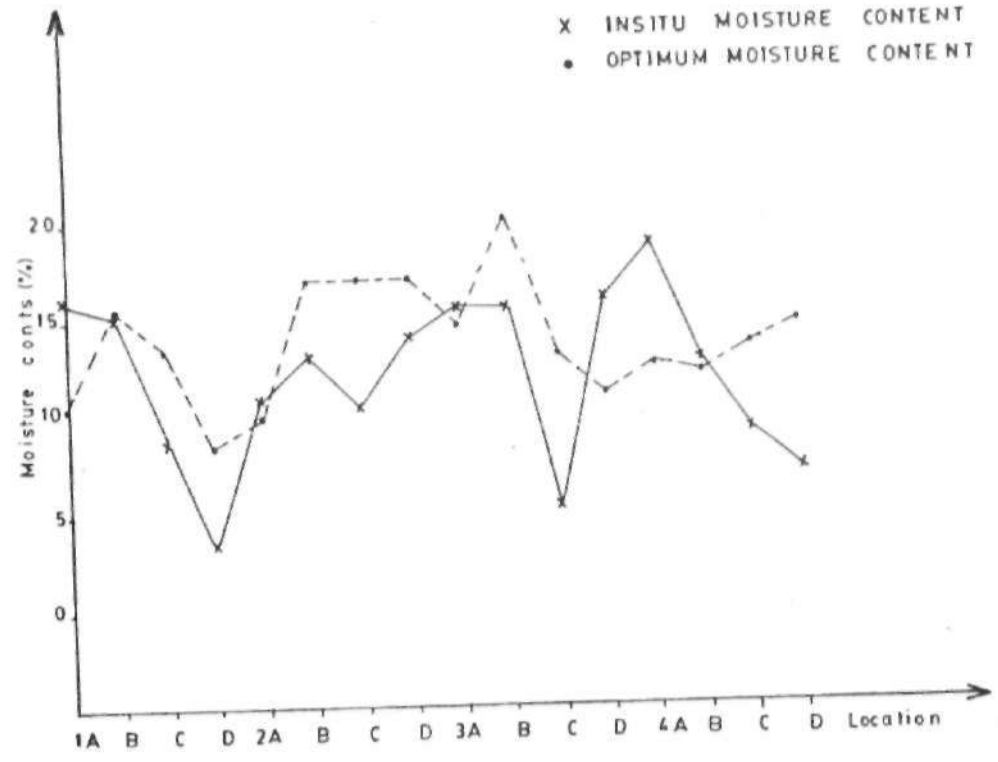


FIG. 4.8 PLOT OF INSITU/OPTIMUM MOISTURE CONTENTS VS LOCATION FOR SUB BASE

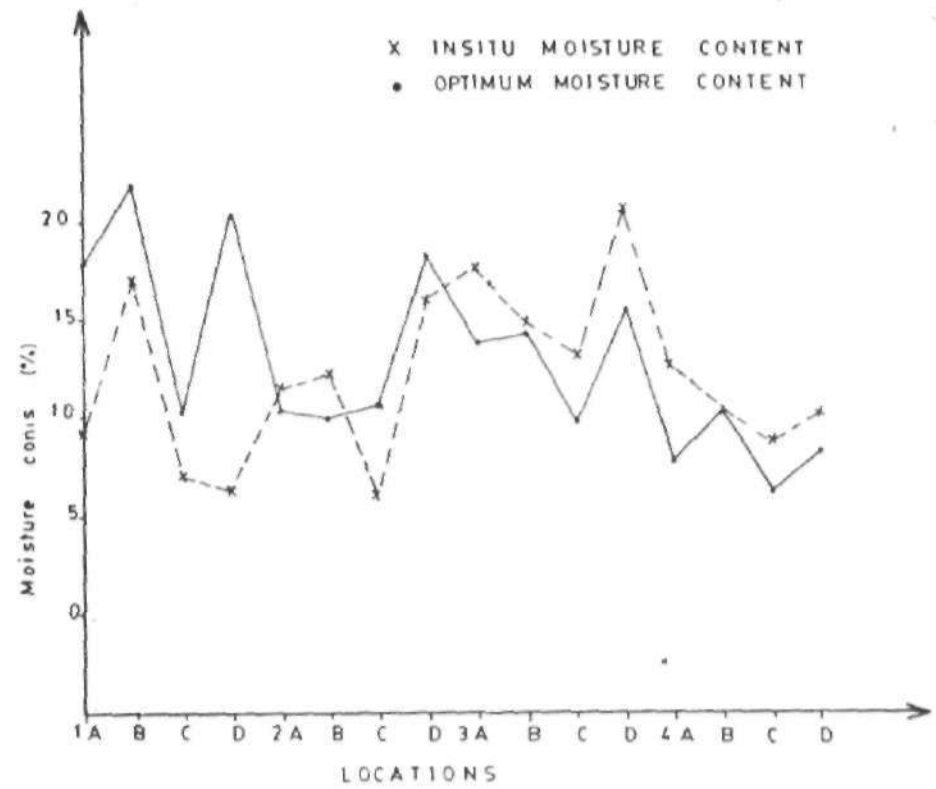


FIG. 4.9 PLOT OF INSTU/OPTIMUM VS LOCATION FOR SUBGRADE LAYER

4.2.2 Measurements of Thicknesses of Layers

The thickness of the surfacing, base and sub-base layers at each test pit was measured and the results are presented on Tables 4.2 and 4.4.

The results showed that the thickness of the surfacing varied from 19mm to 46mm with a mean value of 36.0mm. For the base layer, it varied from 135mm to 290mm with a mean value of 190mm. That of sub-base layer varied from 162mm to 300mm with a mean value of 210mm. It can be seen that the mean total depth of construction from the above data is 436mm (say 440mm). Also, the thicknesses of the base and sub-base is on the average above the minimum specified by FMW & H General Specifications of 150mm.

4.2.3 Laboratory Tests

Standard laboratory tests were performed on recovered samples from each layer and the results are tabulated in Table 4.5.

4.2.3a Base Layer

The results of laboratory tests undertaken on the base layer are shown in Table 4.6. The mean values, standard deviation and coefficients of variation are computed and shown in the Table.

TABLE 4.4: ASPHALT THICKNESS

LOCATION	PIT NO.	ASPHALT THICKNESS (mm)	ASPHALT THICKNESS (IN)	AVERAGE THICKNESS
1	A	41	1.62	
	B	34	1.34	
	C	46	1.81	
	D	33	1.30	39
2	A	46	1.81	
	B	35	1.38	
	C	36	1.42	
	D	35	1.38	38
3	A	29	1.14	
	B	19	0.75	
	C	28	1.10	
	D	38	1.50	29
4	A	43	1.70	
	B	38	1.50	
	C	31	1.22	
	D	42	1.65	39

TABLE 4.5: SUMMARY OF LABORATORY SOIL TEST RESULTS

LOCATION	PIT NO.	PAVEMENT LAYER	LAYER THICK-NESS (mm)	LL (%)	PL (%)	PI	AASHTO CLASSI-FICA-TION.	GI	OMC (%)	MDD (g/cm^3)	UNSOAKED C.B.R. %	SOAKED C.B.R. %	SPECIFIC GRAVITY (g/cm^3)
1	A	BASE	180	27.0	19.3	7.7	A-2-4	0	10.2	2.12	31.1	6.1	2.86
		SUB-BASE	230	34.0	23.2	10.8	A-2-4	0	10.5	2.10	7.7	4.2	2.67
		SUBGRADE	-	27.0	11.5	15.5	A-2-6	0	18.0	2.10	17.0	4.8	3.02
	B	BASE	134	29.0	15.3	13.7	A-2-6	0	10.5	2.16	11.6	6.0	2.51
		SUB-BASE	300	34.0	21.0	13.0	A-2-6	0	15.5	2.18	6.0	3.7	2.81
		SUBGRADE	-	34.0	26.7	11.0	A-2-5	1.0	22.0	2.08	8.1	6.3	2.58
	C	BASE	230	49.0	32.0	19.9	A-2-7	0	13.8	2.12	6.4	4.8	2.92
		SUB-BASE	290	34.0	19.2	26.0	A-2-6	0	13.5	2.29	20.3	11.0	3.41
		SUBGRADE	-	45.0	21.2	16.6	A-2-7	0	10.5	2.13	29.5	8.0	3.09
	D	BASE	290	34.0	15.2	16.4	A-2-6	0	14.6	1.98	22.3	4.7	2.63
		SUB-BASE	265	20.0	18.7	15.3	A-2-7	0	8.5	2.05	8.4	3.7	2.36
		SUBGRADE	-	32.0	20.0	17.0	A-2-6	0	18.0	2.13	8.2	6.4	2.75

TABLE 4.5 (CONT'D)

LOCATION	PIT NO.	PAVEMENT LAYER	LAYER THICK-NESS (mm)	LL (%)	PL (%)	PI	AASHTO CLASSI-FICA-TION	GI	OMC (%)	MDD (g/cm^3)	UNSOAKED C.B.R. %	SOAKED C.B.R. %	SPECIFIC GRAVITY (g/cm^3)
2	A	BASE	162	35.0	15.1	12.2	A-2-6	0	9.0	2.19	23.7	6.1	2.56
		SUB-BASE	196	48.0	22.1	15.7	A-2-6	0	10.0	2.05	8.7	5.1	2.17
		SUBGRADE	-	41.0	8.0	11.0	A-2-7	1.0	10.5	2.23	4.0	2.3	2.61
	B	BASE	172	34.0	16.4	19.0	A-2-6	0	10.9	2.23	14.3	7.9	2.61
		SUB-BASE	190	33.5	24.1	18.0	A-2-6	0	17.0	2.12	6.0	3.7	2.51
		SUBGRADE	-	41.0	27.0	14.8	A-2-7	1.0	10.0	2.26	10.7	6.4	2.33
	C	BASE	193	37.5	12.0	5.5	A-2-6	0	9.5	2.16	9.7	7.3	2.45
		SUB-BASE	183	39.0	18.5	16.8	A-2-6	0	17.0	2.10	20.5	16.0	2.18
		SUBGRADE	-	39.0	19.2	17.8	A-2-6	0	10.7	2.24	8.0	4.0	2.43
	D	BASE	182	27.0	22.1	11.8	A-2-6	0	9.5	1.99	20.0	3.3	2.43
		SUB-BASE	187	36.0	16.3	17.3	A-2-6	0	17.0	2.16	19.8	11.3	2.69
		SUBGRADE	-	39.0	19.5	9.0	A-2-6	0	18.5	2.21	8.7	6.0	2.56

TABLE 4.5 (CONT'D)

LOCATION	PIT NO.	PAVEMENT LAYER	LAYER THICK-NESS (mm)	LL (%)	PL (%)	PI	AASHTO CLASSI-FICA-TION	GI	OMC (%)	MDD (g/cm^3)	UNSOAKED C.B.R. %	SOAKED C.B.R. %	SPECIFIC GRAVITY
3	A	BASE	200	27.0	15.1	11.9	A-2-6	0	13.0	2.05	23.3	4.2	2.42
		SUB-BASE	215	36.0	22.1	13.9	A-2-6	0	14.5	2.19	9.3	5.4	2.46
		SUBGRADE	-	23.0	8.0	15.0	A-2-6	0	14.0	2.13	17.6	11.6	2.61
	B	BASE	215	33.0	16.4	16.6	A-2-6	0	12.0	2.19	9.6	8.2	2.76
		SUB-BASE	218	29.0	24.1	4.9	A-2-6	0	20.0	1.98	13.3	11.3	2.68
		SUBGRADE	-	36.0	27.0	9.0	A-2-6	0	14.5	2.16	11.9	2.3	2.61
	C	BASE	190	27.0	12.0	15.0	A-2-6	0	8.5	1.97	10.7	7.4	2.43
		SUB-BASE	189	36.0	18.5	17.5	A-2-6	0	13.0	2.09	5.1	3.2	2.86
		SUBGRADE	-	30.0	19.2	10.8	A-2-6	0	10.0	2.35	24.0	5.4	2.74
	D	BASE	195	26.5	22.1	4.4	A-2-6	0	18.0	2.20	6.3	9.5	2.76
		SUB-BASE	193	37.5	16.3	21.2	A-2-6	0	11.0	2.32	6.0	4.0	2.24
		SUBGRADE	-	35.0	19.5	15.5	A-2-6	0	15.7	2.23	5.5	3.0	2.65

TABLE 4.5 (CONT'D)

LOCATION	PIT NO.	PAVEMENT LAYER	LAYER THICK-NESS (mm)	LL (%)	PL (%)	PI	AASHTO CLASSI-FICA-TION	GI	OMC (%)	MDD (g/cm^3)	UNSOAKED C.B.R. %	SOAKED C.B.R. %	SPECIFIC GRAVITY
4	A	BASE	186	38.0	17.9	20.1	A-2-6	0	10.0	2.16	19.3	3.6	
		SUB-BASE	194	12.5	9.0	3.5	A-2-6	0	12.5	2.05	16.0	7.0	2.86
		SUBGRADE	-	24.5	18.2	6.3	A-2-6	0	8.0	2.29	12.4	9.1	2.83
	B	BASE	192	20.0	16.0	4.0	A-2-6	0	13.0	2.15	8.7	5.1	2.50
		SUB-BASE	174	31.0	26.3	4.7	A-2-6	0	12.0	2.19	8.1	5.6	2.95
		SUBGRADE	-	43.0	27.8	15.2	A-2-7	0	10.5	2.29	17.3	5.3	2.71
	C	BASE	149	51.0	36.7	14.3	A-2-7	0	16.0	2.32	8.9	4.7	3.06
		SUB-BASE	193	51.0	12.0	14.3	A-2-6	0	13.4	2.15	7.7	4.2	2.90
		SUBGRADE	-	51.0	10.0	14.3	A-2-6	0	6.5	2.28	22.3	5.3	2.64
	D	BASE	170	27.0	15.2	12.5	A-2-6	0	10.0	2.28	3.7	3.0	2.65
		SUB-BASE	182	23.0	9.5	13.5	A-2-6	0	14.5	2.28	16.8	4.4	2.59
		SUBGRADE	-	41.0	16.2	24.8	A-2-7	0	11.0	2.36	17.7	5.4	2.89

The liquid limit and plasticity index for the base materials tested ranged from 26.5% to 51.0% and 20.1% to 4.4% respectively. The corresponding mean values were 32.68% and 12.8% respectively. From the data, 50% of the tests results had liquid limits greater than 30% and the Federal specifications allows the Engineer to disallow the use of materials with liquid limit greater than 30%. Also, 56% had plasticity index greater than 12%, the maximum specified. The mean values as indicated above did not meet the Federal Specifications. It can thus be said that base materials used were significantly of low quality.

The distribution of the AASHTO classification showed the base material to belong to the low quality A-2-6 to A-2-7 groups. Their rating even for use as subgrade ranges from fair to good. However, the percentage of fines (% passing B.S. 200 sieve) and the group index results showed the percentage of plastic fines to be low. While too much fines will reduce maximum dry density and strength attainable, some fines is required to allow for compaction to the appropriate density.

The C.B.R. test results ranged from a minimum of 3.7% to a maximum of 30.0% with a mean value of 14.3% whereas the Federal specifications gives 80% to be the minimum. Also, the relative compaction results showed the degree of compaction to be below the specified values. The Federal specifications required is 100% whereas the data showed a range of 78% to 92%.

TABLE 4.6: LABORATORY TESTS RESULTS FOR BASE LAYER

PIT NO.	LL(%)	PI(%)	GI	AASHTO CLASSI-FICA-TION	CBR %	% PASSING BS 200 SIEVE
1 A	27.0	7.7	0	A-2-4	31.0	5.0
B	29.0	13.7	0	A-2-6	11.6	3.0
C	49.0	19.9	0	A-2-7	6.4	5.0
D	34.0	16.4	0	A-2-6	22.3	4.0
2 A	35.0	12.2	0	A-2-6	9.6	6.0
B	34.0	19.0	0	A-2-6	23.7	6.0
C	37.5	5.5	0	A-2-6	14.3	3.0
D	27.0	11.8	0	A-2-6	9.7	3.0
3 A	27.0	11.9	0	A-2-6	20.0	6.0
B	33.0	16.6	0	A-2-6	23.3	6.0
C	27.0	15.0	0	A-2-6	10.7	4.0
D	26.5	4.4	0	A-2-6	6.3	3.0
4 A	38.0	20.1	0	A-2-6	19.3	4.0
B	20.0	4.0	0	A-2-6	8.7	5.0
C	51.0	14.3	0	A-2-7	8.9	7.0
D	27.8	12.5	0	A-2-6	3.7	3.0
Mean value	32-68	12.8	-	-	14.3	4.6
Stand-ard devia-tion	8.28	5.2	-	-	7.9	1.36
Coef-fici-ent devia-tion	0.25	0.41	-	-	0.55	3.4

Thus, improper compaction coupled with low strength base material combined to give poor performance and led to failure under increased traffic loads.

4.2.3b Sub-base Layer

The laboratory test results are given in Table 4.7. The liquid limits and plasticity indices of the sub-base materials tested ranged from 12.5% to 51% and 4.7% to 26% respectively. The corresponding mean values were 33.3% and 14.1% respectively. 75% of the samples tested had liquid limits and plasticity indices above the maximum set by the Federal specifications (liquid limit not greater than 30% and plasticity index not greater than 12%). Even the mean values of the liquid limit and plasticity index did not meet the Federal specifications. AASHTO classification for the materials showed the sub-base materials tested to be within the A-2-6 group which is a low quality material. The group index was found to be zero for all the samples tested indicating high quality and low percentage of fines with the percentage passing B.S. 200 mean value of 6% (less than 35%), though the liquid limits and plasticity indices showed the material to be a little plastic (LL and PI greater than 30% and 12% respectively) as indicated above.

The soaked (24hrs) CBR values ranged from 3.2% to 16.0% with a mean value of 6.5% as against the minimum of 30% required by the Federal specifications after 24hrs

TABLE 4.7: LABORATORY TESTS RESULTS FOR SUB-BASE LAYER

PIT NO.	LL(%)	PI(%)	GI	AASHTO CLASSI-FICA-TION	CBR		% PASS-ING NO. 200 SIEVE
					UN SOAK-ED	SOA-KED	
1 A	34.0	10.5	0	A-2-4	7.7	4.2	3.0
B	34.0	13.0	0	A-2-6	6.0	3.7	12.0
C	20.0	26.0	0	A-2-6	20.3	11.0	8.0
D	48.0	15.3	0	A-2-7	8.4	3.7	15.0
2 A	33.5	15.7	0	A-2-6	8.7	5.1	5.0
B	39.0	18.0	0	A-2-6	6.0	3.7	6.0
C	36.0	16.8	0	A-2-6	20.5	16.0	2.0
D	33.0	17.3	0	A-2-6	19.8	11.3	4.0
3 A	36.0	13.9	0	A-2-6	9.3	5.4	5.0
B	29.0	4.9	0	A-2-6	13.3	11.3	6.2
C	36.0	17.5	0	A-2-6	5.1	3.2	3.0
D	37.5	21.2	0	A-2-6	6.0	4.0	6.0
4 A	12.5	3.5	0	A-2-6	16.0	7.0	5.0
B	31.0	4.7	0	A-2-6	8.1	5.6	4.0
C	51.0	14.3	0	A-2-6	7.7	4.2	7.0
D	23.0	13.5	0	A-2-6	16.8	4.4	6.0
MEAN	33.3	14.1	-	-	11.23	6.49	6.08
Stand-ard Devia-tion	9.46	6.0	-	-	5.6	3.8	3.34
Coeff-icient of Varia-tion	0.28	0.43	-	-	0.5	0.58	0.55

soaking. Also, relative compaction ranged from 79% to 89% which showed that none met the requirement of 100% specified.

It can therefore be concluded from the foregoing discussion that the sub-base is also characterized by low strength and poor quality which might have led to the unsatisfactory field performance and premature failures.

4.2.3c Subgrade Layer

The results of laboratory tests on the subgrade layer are presented in Table 4.8.

The distribution of the AASHTO classification group showed that 75% of the samples tested were in the A-2-6 group while 19% were in the A-2-7 group. These groups give fair to good performance as subgrade materials. The percentage of fines (% passing BS 200 Sieve) which indicates the amount of fine materials was low with a mean value of 8.63% as compared with the maximum specified of 35%. The group indices of the materials also gave a range of between 0 and 1.0 indicating high quality with low percentage of expansive clays.

The C.B.R. tests results for the unsoaked and soaked condition gave mean values of 13.73% and 5.9% respectively, showing a loss of strength of over 50% as a result of soaking. This clearly indicates the susceptibility of the subgrade to weakening due to moisture increase. About 63% of the tested samples had C.B.R.

TABLE 4.8: LABORATORY TESTS RESULTS FOR SUBGRADE LAYER

PIT NO.	LL(%)	PI(%)	GI	AASHTO CLASSI-FICA-TION	CBR		% PASSING BS 200 SIEVE
					UN SOAKED	SOA-KED	
1 A	27.0	15.5	0	A-2-6	17.0	4.8	4.0
B	34.0	11.0	1.0	A-2-5	8.0	6.3	16.0
C	45.0	16.6	0	A-2-7	29.5	8.0	4.0
D	32.0	17.0	1.0	A-2-6	8.2	6.4	19.0
2 A	41.0	11.0	0	A-2-7	4.0	2.3	20.0
B	41.0	14.8	1.0	A-2-7	10.7	6.4	18.0
C	39.0	17.8	0	A-2-6	8.0	4.0	4.0
D	39.0	9.0	0	A-2-6	8.7	6.0	4.0
3 A	23.0	15.0	0	A-2-6	17.6	11.6	6.0
B	36.0	9.0	0	A-2-6	11.9	2.3	7.0
C	30.0	10.8	0	A-2-6	24.0	5.4	4.0
D	35.0	15.5	0	A-2-6	5.5	3.0	13.0
4 A	24.5	6.3	0	A-2-6	12.4	9.1	3.0
B	43.0	15.2	0	A-2-6	17.3	5.3	5.0
C	51.0	14.3	0	A-2-6	22.3	5.3	7.0
D	41.0	24.8	0	A-2-6	17.7	5.4	4.0
MEAN	36.3	14.0	-	-	13.73	5.9	8.63
Stand-ard Devia-tion	7.68	4.4	-	-	7.2	2.46	6.42
Coeff-icient of Varia-tion	0.21	0.32	-	-	0.52	0.42	0.72

values greater than 5% after soaking while the corresponding value for the unsoaked condition is 94%. Similarly, 6.3% had C.B.R. value greater than 10% after soaking while 63% is the corresponding value before soaking.

On the whole, the subgrade material cannot be said to be absolutely of low strength from the above findings though the Federal specifications required a minimum of 10% after 48hrs soaking.

The relative compaction results data showed the likelihood of improper compaction while the high moisture content showed the weak state of the subgrade layer which could have possibly led to failure of the layers above as a results of increased deformation under surcharge and traffic loads.

4.3 Traffic and Axle Load Survey

The results of the manual traffic count carried out at the different locations are presented in Table 4.9 and the truck weighing data in Table 4.13.

From the results, the growth rate was estimated at 7.5% which was considered unrealistic when compared with the econometric data. The ADT data from the Funtua end and growth rate from the econometric data were then used. The initial ADT_i when the road was overlaid in 1979 was determined as follows:-

TABLE 4.9: TRAFFIC COUNT AND VEHICLE CLASSIFICATION

LOCATION	AVERAGE DAILY TRAFFIC ADT (VEHS/DAY)	NUMBER OF TRUCKS	PERCENT TRUCKS (%)
SAMARU (1) (KM 7 + 00)	18,864	1547	8.2
GANGARA (2) (KM 48 + 00)	5,304	679	12.8
BAKIN DUTSE (3) (KM 63 + 00)	6,672	840	12.6
PEU	6,240	481	7.7

$$ADT_p = ADT_i (1+r)^n \quad - \quad \text{eqn 4.3.1}$$

where ADT_p = Present Daily Traffic and

ADT_i = Initial Average Daily Traffic when the road was overlaid.

r = growth rate (Table 4.12).

(The growth rate was estimated based on econometric data analysis (11) to be 10% between 1978 - 1981 and 4% from 1981).

n = number of years

From eqn 4.3.1

$$ADT_i = \frac{ADT_p}{(1+r)^n}$$

$$ADT_{1981} = \frac{ADT_{1991}}{(1+0.04)^{10}}$$

$$= \frac{6672}{(1.04)^{10}}$$

$$= 4507 \text{ vehs/day}$$

$$ADT_{1979} = \frac{ADT_{1981}}{(1+0.10)^2}$$

$$= \frac{4507}{1.10^2}$$

$$= 3725 \text{ vehs/day.}$$

The Initial ADT was therefore 3725 Vehs/day.

The total number of vehicles over the design period Q_f is given by the following equation;

$$Q_f = ADT \times FTG \quad \text{.....} \quad \text{eqn 4.3.2}$$

where FTG = Future Total Growth Factor

For a design period n = 10 yrs and from table 4.12, FTG = 4460.

$$Q_f = 3725 \times 4460$$

$$= 16.6 \times 10^6 \text{ vehs.}$$

Similarly, for a design period n = 20 yrs and from table 4.12, FTG = 11,000 and therefore,

$$Q_f = 3725 \times 11000$$

$$= 41 \times 10^6 \text{ vehs.}$$

The Equivalent Standard Axle Loads (ESALS) over the design period is calculated using the following equation;

$$N_D = Q_f \text{ (DDF)(LDF)}(P_t)(F_{avg}) \quad \text{... eqn 4.3.3}$$

where N_D = Design ESALS for the design lane

Q_f = Total number of estimated future vehicles during the design period in both directions.

DDF = Direction distribution factor (0.5 for a 2 lane highway).

LDF = Lane distribution factor (1.0 for two lane highway)

P_t = Percent trucks

F_{avg} = Average 8200kg single axle load equivalence.

For a typical Nigerian situation and for a two lane divided highway eqn 4.3.3 can be simplified as follows:

$$N_D = 0.5 \cdot Q_f \cdot P_t \cdot F_{avg} \qquad 4.3.4$$

F_{avg} = 7.18 (Table 4.9).

For a design period n = 10 yrs (1989),

$$N_D = 0.5 \times 16.6 \times 10^6 \times \frac{12.6}{100} \times 7.18$$

$$= 7.5 \times 10^6 \text{ ESALS.}$$

$$\underline{N_D = 7.5 \times 10^6 \text{ ESALS for 10 yrs.}}$$

For a design period of n = 20 yrs

$$N_D = 0.5 \times 41.0 \times 10^6 \times \frac{12.6}{100} \times 7.18$$

$$= 18.5 \times 10^6 \text{ ESALS}$$

$$\underline{N_D = 18.5 \times 10^6 \text{ ESALS for 20 yrs.}}$$

The design ESALS for subsequent years are tabulated in Table 4.10.

The results of the traffic counts carried out at the three locations indicated significant variations particularly point 1 located in Samaru. The computed ADT_s showed that of point 1 to be more than three times greater than the other two points. Though, the point could be regarded as too close to the town and therefore subject to the influence of local traffic, the value should not totally be disregarded. As indicated in section 4.1, the point was within section 1 which was identified to be worst characterized by many forms of failure. Also, over forty percent of the total length of the section exhibited one form of failure or the other. Thus, this could be attributed to the high traffic volume carried by the section, although the percent truck

TABLE 4.10: FUTURE DESIGN EASLS

END OF YEAR	DESIGN PERIOD (YEARS)	N_D (EASLS)
1991	12	9.46×10^6
1992	13	10.47×10^6
1993	14	11.48×10^6
1994	15	12.58×10^6
1995	16	13.68×10^6
1996	17	14.87×10^6
1999	20	18.55×10^6

TABLE 4.11: LANE DISTRIBUTION FACTORS

TOTAL NUMBER OF LANES BOTH DIRECTIONS	LANE DISTRIBUTION FACTOR
2	1.0
4	0.9
6	0.7

Source:- TRDF "Overlay Design Manual" FMW & H Final Project.

TABLE 4.12: ESTIMATED PAST GROWTH FACTORS

(Source reference 11).

Region	States	Period (Years)		
		1965-1978 Period 1	1978-1981 Period 2	1982-1986 Period 3
1	Kaduna, Kano	20	10	2 - 4
2	Bauchi, Borno	16	8	2 - 4
3	Bendel, Eastern Kwara, Lagos, Ogun, Ondo, Oyo	16	8	2 - 4
4	Fed. Capital Territory, Western Kwara, Niger, Sokoto	20	10	2 - 4
5	Benue, Plateau, Gongola	20	10	2 - 4
6	Anambra, Cross Rivers, Imo Rivers	10	5	2 - 4

TABLE 4.13: FUTURE TOTAL GROWTH FACTOR

Time Period, Years - (n)

	1	2	3	4	5	6	7	8	10	15	20
1	367	738	1120	1490	1870	2257	2646	3039	3840	5910	8080
2	369	738	1130	1520	1920	2325	2741	3164	4040	6360	8940
3	371	751	1150	1550	1970	2396	2839	3294	4260	6900	9950
4	372	760	1160	1580	2020	2469	2940	3430	4460	7450	11000
5	373	767	1180	1610	2070	2544	3046	3572	4710	8080	12400
6	376	774	1200	1640	2130	2622	3155	3720	5000	8750	13800
7	378	7825	1210	1680	2170	2701	3268	3874	5220	9490	15500
8	379	789	1230	1710	2230	2783	3385	4036	5500	10300	17400
9	382	797	1250	1740	2290	2868	3507	4204	5800	11200	19500
10	383	805	1260	1760	2340	2955	3633	4379	6090	12200	22000
12	386	819	1304	1847	2455	3136	3899	4754	6782	14408	27847
14	390	835	1341	1919	2578	3329	4185	5161	7541	17098	35499
16	393	850	1379	1994	2706	3532	4491	5603	8390	20327	45399
18	397	865	1418	2070	2840	3748	4820	6084	9337	24200	58203
20	400	881	1457	2149	2980	3976	5171	6606	10394	28842	74748

Table presents solution to: $365 \quad (1 + i)^n - 1) \ / \ \ln(1 + i)$

Source (reference 11).

which contribute largely to the damaging effect of the pavement was lower than other sections. It was found to be 8.2% as against 12.8% and 12.6% for point 2 (Gangara) and point 3 (Bakin Dutse) respectively.

Also, the percent trucks for the current ADT was found higher than the 1987 value. An average value of 11.2% for the current ADT was estimated as against 7.7% estimated in 1987. This shows a highly significant increase of about 45%.

An attempt to estimate the traffic growth rate from the 1987 and the 1991 data gave unrealistic results. Actually, such estimates require sufficient historical data accummulated over a number of years before reasonable values could be obtained. Such data is very scarce in Nigeria and growth rates have been estimated based on econometric data (11). These were the figures used and the recommended values are shown in Table 4.12.

The axle load survey data is shown in Table 4.14 - 4.15. A total of forty two trucks (24 in Class 3 and 18 in Class 6) were sampled-out for weighing. 57% of the trucks weighed were in Class 3 while 43% in class 6 (AASHO classification and adopted by FMW & H). The average equivalency factor for Class 3 was 3.00 while that of Class 6 was 11.35 giving an overall average equivalency factor of 7.18. The standard deviations for Class 3 and Class 6 were 2.03 and 7.8 respectively. Similarly, the single maximum factors were 8.4 and 22.93

TABLE 4.14: AXLE LOAD SURVEY DATA FOR VEHICLE CLASS 6

1st AXLE			2ND AXLE			3RD AXLE (TANDEM AXLE)				
HALF AXLE	AXLE LOAD	EQUIVALENCY FACTOR	HALF AXLE	AXLE LOAD	EQUIVALENCY FACTOR	1ST WHEEL	2ND WHEEL	AXLE LOAD	EQUIVALENCY FACTOR.	OVERALL EQUIVALENCY FACTOR
FRONT WHEEL	2 x HALF AXLE	e_1	WHEEL LOAD	2 x HALF AXLE	e_2	LOAD (L_1)	LOAD (L_2)	$2(L_1+L_2)$	e_3	$e=e_1+e_2+e_3$
3150	6300	0.38	7550	15,100	10.38	6750	6650	26,800	1.78	12.54
2750	5500	0.22	6250	12,500	5.06	6100	6150	24,500	7.49	12.77
2250	4500	0.10	2000	4,000	0.07	1540	1500	6,080	0.04	0.21
2500	5000	0.15	2500	5,000	0.15	1750	1850	7,200	0.07	0.37
2200	4400	0.09	2000	4,000	0.07	1700	1750	6,900	0.06	0.22
2850	5700	0.25	6500	13,000	6.05	6200	6250	24,900	7.96	14.26
3250	6500	0.41	7950	15,900	12.63	6750	6800	27,100	10.98	17.44
2250	4500	0.10	6350	12,700	5.38	6200	6250	24,900	7.96	13.44
3120	6240	0.35	7550	15,100	10.38	6930	7000	27,860	12.20	22.93
2750	5500	0.22	6500	13,000	5.88	6250	6000	24,500	7.49	13.59
3050	6100	0.33	7250	14,500	8.90	6950	6530	26,960	10.77	20.00
2750	5500	0.22	6800	13,600	6.98	6500	6000	25,000	8.08	15.28
2750	5500	0.22	5830	11,660	3.89	5250	4500	19,500	3.14	7.25
2600	5200	0.18	5800	11,600	3.81	4850	4700	19,100	2.91	6.90
3150	6300	0.37	7500	15,000	10.12	7000	6550	27,100	10.98	21.47
2250	4500	0.10	5200	10,400	2.52	4450	4400	17,700	2.18	4.80
2500	5000	0.15	2000	4,000	0.07	1850	1750	7,200	0.07	0.29
3250	6500	0.41	7850	15,700	12.04	6500	6000	25,000	8.08	20.53
									TOTAL	204.3
									MEAN	11.35

TABLE 4.15: AXLE LOAD SURVEY FOR CLASS 3

1ST AXLE			2ND AXLE			EQUIVALENCY FACTOR
1ST HALF AXLE WHEEL LOAD	AXLE LOAD 2 x HALF AXLE LOAD	EQUIVALENCY FACTOR (e_1)	2ND HALF AXLE WHEEL LOAD	AXLE LOAD 2 x HALF AXLE LOAD	EQUIVALENCY FACTOR e_2	$e = e_1 + e_2$
2050	4100	0.07	5150	10300	2.43	2.53
1950	3900	0.06	4850	9700	1.93	1.99
2550	5100	0.17	6750	13500	6.75	6.92
2750	5500	0.22	5850	11700	3.93	4.15
2050	4100	0.07	5050	10100	2.25	2.32
1750	3500	0.04	3850	7700	0.80	0.84
1950	3900	0.06	4750	9500	1.79	1.85
2250	4500	0.10	5750	11500	3.69	3.79
2150	4100	0.07	5500	11100	3.11	3.18
2250	4500	0.10	5950	11900	4.2	4.3
1900	3800	0.05	4750	9500	1.79	1.84
1750	3500	0.04	1650	3300	0.03	0.07
2150	4300	0.09	5750	11500	3.67	3.76
2050	4100	0.07	5250	10500	2.61	2.68
1800	3600	0.04	4250	8500	1.17	1.21
2050	4100	0.07	4750	9500	1.79	1.86
3100	6200	0.36	7050	14100	8.00	8.36
2150	4300	0.09	6230	12460	5.00	5.09
2100	4200	0.08	4150	8300	1.07	1.15
2050	4100	0.07	5700	11400	3.57	3.64
2830	5660	0.25	6300	12700	5.38	5.63
2500	5000	0.15	5050	10100	2.25	2.4

TABLE 4.15 (CONT'D)

1ST AXLE			2ND AXLE			EQUIVALENCY FACTOR
1ST HALF AXLE WHEEL LOAD	AXLE LOAD 2 x HALF AXLE LOAD	EQUIVALENCY FACTOR (e_1)	2ND HALF AXLE WHEEL LOAD	AXLE LOAD 2 x HALF AXLE LOAD	EQUIVALENCY FACTOR e_2	$e = e_1 + e_2$
2150	1300	0.09	5000	10000	2.16	2.25
1750	3500	0.04	1700	3400	0.04	0.08
					TOTAL	71.88
					MEAN	3.00

TABLE 4.16: AVERAGE EQUIVALENCY FACTOR

VEHICLE CATEGORY (AASHO CLASSIFI-CATION)	AVERAGE EQUIVA-LENCY	NUMBER OF TRUCKS	STANDARD DEVIA-TION	MAXIMUM INDIVI-DUAL FACTOR
Class 2	0.0	0	0.0	0.0
Class 3	3.00	24	2.03	8.40
Class 4	0.0	0	0.0	0.0
Class 5	0.0	0	0.0	0.0
Class 6	11.35	18	7.80	22.93
Class 7	0.0	0	0.0	0.0
Class 8	0.0	0	0.0	0.0
Others	0.0	0	0.0	0.0
OVERALL	7.18	42	6.72	22.93

% of axles exceeding the legal load limit = 42%

(SOURCE:- PEU, FMW & H).

for Class 3 and Class 6 respectively. Also, the percentage of axles exceeding the 10,000kg legal axle load limit specified was found to be 42%.

From the above analysis and discussions, it can be concluded that heavy trucks number and overloading coupled with adverse climatic and environmental factors and poor quality materials might have likely caused the premature failure.

4.4 Adequacy of the Pavement Design

The average depth of construction of the pavement was estimated at 440mm from the measurement of the layer thicknesses (Section 4.2.2).

Using the Group Index (GI) Method

GI = 1.0 (for the subgrade) and,

for commercial vehicles > 300 per day, total thickness required H = 356mm and thickness of sub-base required = 150mm.

Using the C.B.R. Chart

Subgrade CBR (soaked) mean value = 5.9%,

Required depth of construction H = 400mm.

Sub-base CBR (soaked) mean value = 6.5%,

Minimum H above the base = 370mm.

Base CBR mean value = 14.3%,

Minimum H above the base = 200mm.

From the above design, the required thicknesses can be distributed as follows:

Sub-base = 150mm

Base = 150mm

Surfacing = 100mm.

Asphalt Institute Method

The subgrade CBR values are 2.7, 3.3, 3.9, 5.9, 6.2, 6.4 and 6.4%. The resilient modulus (Mr) can be approximated from the C.B.R. values using equations 4.4.1 and 4.4.2.

M_r (MP_a) = 10.3 CBR equation 4.4.1

M_r (PS_i) = 1500 CBR equation 4.4.2

TABLE 4.17: SUBGRADE RESILIENT MODULII

OCBR (MPa)	CBR (PSi)	NO. EQUAL TO OR GREATER	PERCENTAGE EQUAL TO OR GREATER
74.2	10,800	1	12.5
65.9	9,600	3	37.5
65.9	9,600	-	37.5
63.9	9,300	4	50.0
55.6	8,100	5	62.25
40.2	5,850	6	75.0
34.0	4,950	7	87.5
27.8	4,050	8	100.0

From Fig. 4.7 Design M_r = 33.0 MP_a

For a design period of 10 yrs

Design EASLS = 7.5×10^6

Asphalt concrete = 125mm

Untreated Base = 300mm

Total Construction thickness = 425mm.

For a design period of 14 yrs (1993),

Design EASLS = 11.48×10^6

Asphalt Concrete = 130mm

Untreated Base = 300mm

Total Construction thickness = 430mm

For a design period of 15 yrs (1994),

Design EASLS = 12.58×10^6

Asphalt Concrete = 135mm

Untreated Base = 300mm

Total depth of construction = 435mm.

For a design period of 16 yrs (1995),

Design EASLS = 13.68×10^6

Asphalt Concrete = 140mm

Untreated Base = 300mm.

Total depth of construction = 440mm.

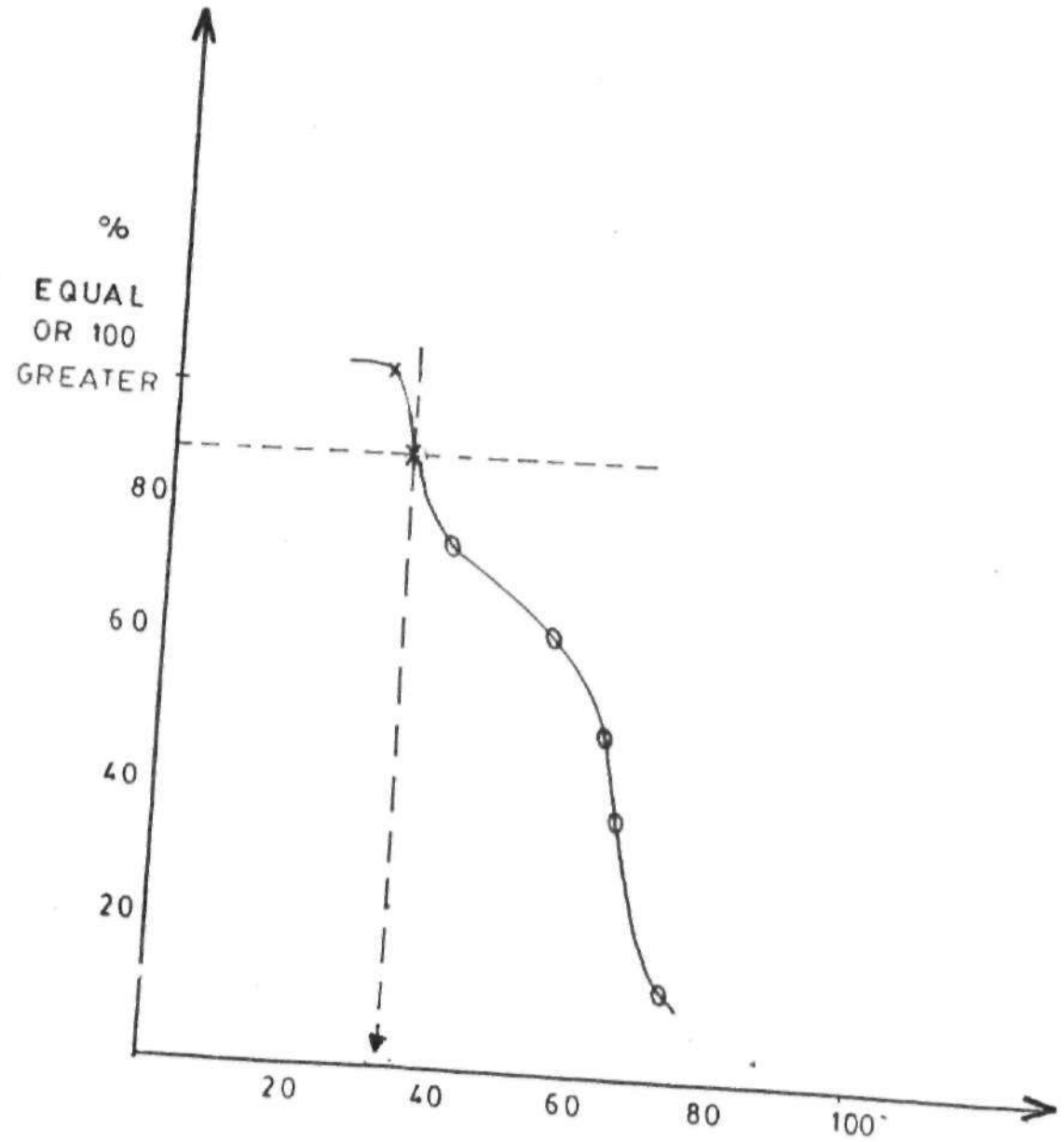


FIG. 4.10 RESILIENT MODULUS (MPA)

From the foregoing analysis, it can be concluded that the constructed total depth of the pavement was adequately designed (for at least design period of 16 yrs). However, the thickness of the surfacing was found to be inadequate. In most cases, it was less than 50mm whereas the minimum specified for the estimated axle load (over 10^6 ESALS) was 125mm (Appendix II). The surface course as the uppermost layer is in addition to providing safe and comfortable riding, expected to protect the roadway from disintegrating effects caused by vehicles. Provision of inadequate thickness could therefore lead to the pavement failures.

4.5 Discussion of Findings/Causes of Failures

From the analysis of results of field and laboratory tests, the likely causes of failure include the following:-

(a) The use of poor quality materials for construction, particularly, the base and sub-base layers. The materials have been found to fall within the A-2-4 to A-2-7 class which can give satisfactory performance only as subgrade materials. The materials used have low strength, as found from the C.B.R. test results and the layers could not therefore support adequately the wheel loads under the prevailing traffic condition.

(b) The high number of trucks coupled with axle over load also contributed to the failure. The truck load data analysis has shown that about half of the axles of trucks using the road are overloaded with an average equivalence factor of 7.18.

(c) The sample designs showed that the pavement was adequately designed as far as total thickness of pavement was concerned. However, the surface course which plays an important role of protecting the pavement from the damaging effects of the traffic loads did not meet the minimum thickness specified for the imposed traffic loading. This shortcoming could have also contributed to the observed premature failures.

(d) The drainage condition was found to be generally poor as facilities were either lacking or not adequately maintained where provided. Water could not therefore be drained to the appropriate discharge points which subsequently led to the weakening of the pavement and its eventual failure.

(e) Lack of proper supervision to maintain quality control during construction resulting in inadequate compaction of materials to the required densities. This might have accounted for the observed differential settlement and development of cracks on the surface particularly along the wheel paths.

(f) Lack of proper maintenance both routine and preventive such as sealing of cracks and patching of potholes might have probably led to the progression of the failure. Further infiltration of water from surface cracks and potholes caused progression of cracks and other modes of failure leading to total breakdown of the pavement structure.

CHAPTER FIVE

5.0 CONCLUSIONS AND RECOMMENDATIONS

5.1 Conclusion

It can be concluded that the causes of failures in the studied pavement are several and vary in the degree of their contribution to the observed defects. The failures were likely due to the combined effects of use of substandard materials, heavy axle loads, inadequate supervision, inadequate drainage facilities, lack of proper maintenance culture and other environmental factors. This led to the development of various forms of distress and allowed for infiltration of water which accentuated the progression of the failures and eventual total breakdown of the pavement structure.

5.2 Recommendations

In order to remedy the current conditions of the road, the following are recommended:-

(a) Reconstruction of the locations that have completely failed by scarifying the existing surface, reconstructing or stabilizing the base and sub-base layers. A prime coat using MC1 bitumen and constructing asphaltic concrete surfacing.

(b) An asphaltic overlay is similarly recommended for the other sections that have only severely cracked. This could be through firstly patching the weak areas and then applying a single coat overlay of 75mm.

(c) Provision of side drains to carry run-off from the road surface to the discharge pointsand the existing drainage facilities including bridges and culverts should be adequately maintained through inspection and clearance of silt, debris and weeds.

(d) The government should through its law enforcement agencies like the Federal Road Safety Corps (FRSC), regulate the axle load on the highways. This will reduce the rate of damage currently caused by overloaded axles as evidenced by the calculated equivalency factor. This could be through the introduction of weigh bridges and enforcement of strict compliance. The current practice is that the driver is penalised either through fines or reducing the excess load at the spot but combining the two, in the view of the author, will ensure compliance of the regulation.

(e) Special attention should also be paid to quality control during construction to ensure provision of strong pavement that will reduce future huge maintenance costs as a result of premature failures due to poor workmanship like improper compaction and useof poor quality materials. The practice

where supervision is provided by consultants may lead to connivance with contractor which can result to poor quality work. However, supervision by Government Agencies is not similarly be fool-proof and a better solution may only be a legislation to control construction quality.

(f) The shoulders should also be adequately maintained through patching and sealing of potholes to protect the entire pavement.

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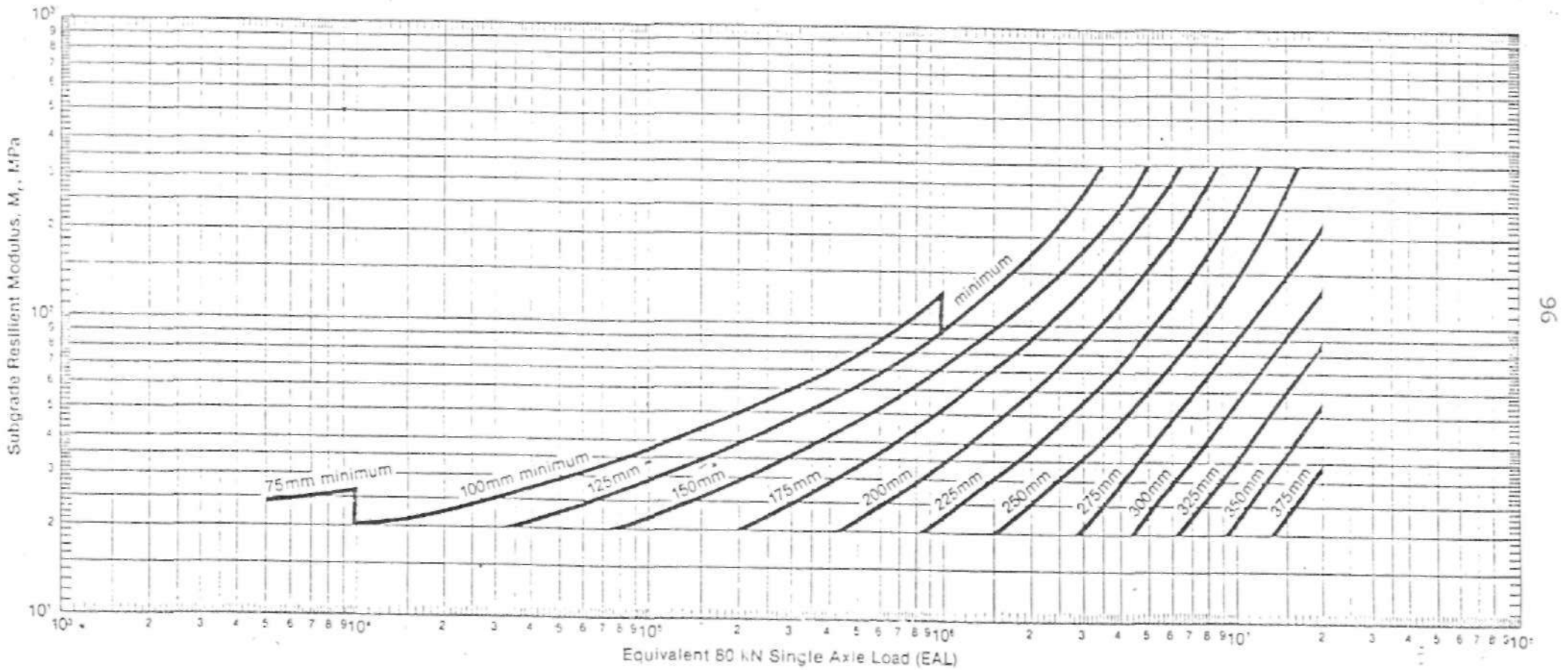
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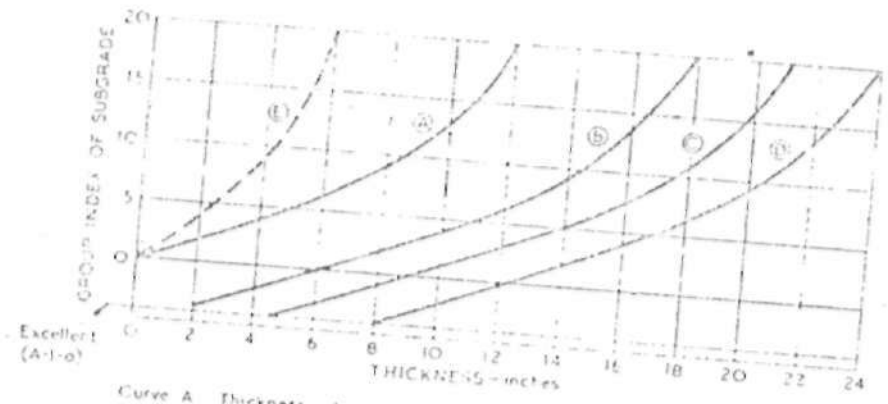
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Untreated Aggregate Base 300mm Thickness



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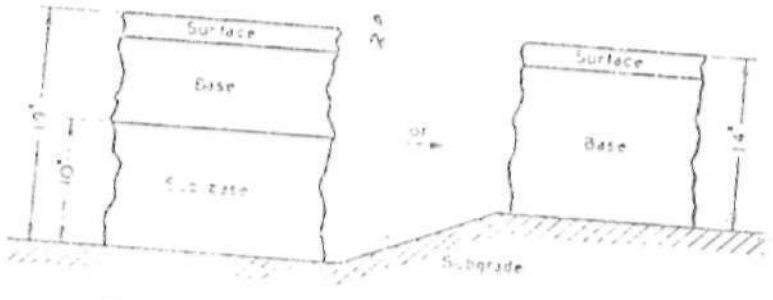
FIG 9



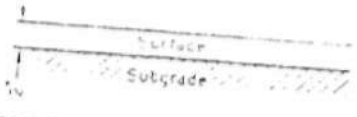
Curve A Thickness of sub-base required
Curve B Total thickness of surface, base and sub-base – Light traffic
Curve C Total thickness of surface, base and sub-base – Medium traffic
Curve D Total thickness of surface, base and sub-base – Heavy traffic
Curve E Thickness of additional base which may be substituted for sub-base of Curve A

Light traffic – [illegible] commercial vehicles/day
Medium traffic – 50-300 commercial vehicles/day
Heavy traffic – > 300 commercial vehicles/day

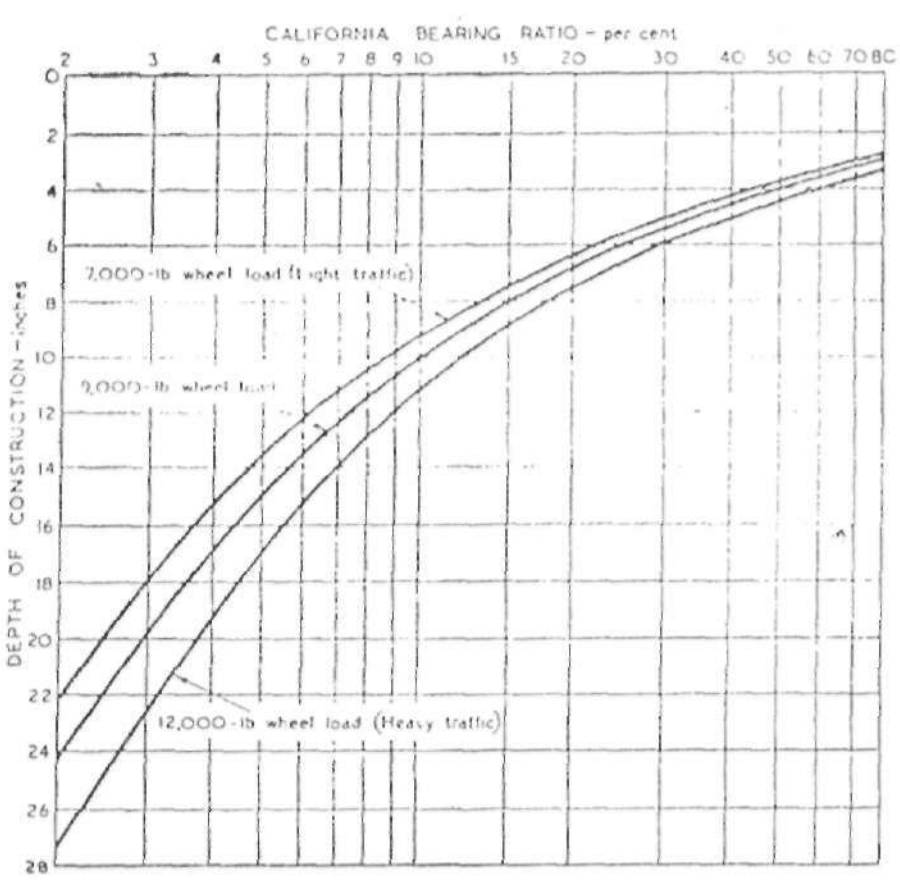
Probably the base an A-1 material and the sub-base an A-2 material



EXAMPLE 1 Clay subgrade – A-6 Group index=12
Medium traffic



EXAMPLE 2 Excellent gravel subgrade – A-1-a
Group index=0 Light traffic



DESIGN CURVES FOR ROADS, CALIFORNIA BEARING RATIO METHOD