EVALUATION OF ROAD PERFORMANCE CONDITION IN FCT ABUJA

BY

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CERTIFICATION

This dissertation "An Evaluation of Road Performance Condition In Keffi, Nasarawa State" meets the regulations governing the award of Master of Science, Faulty of Environmental Science of Nasarawa State University, Keffi.

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DECLARATION

I hereby declare that this thesis has been composed by me and that it is the outcome of my research in "Road Performance" and that it has not been presented for the award of any degree in any higher institution. All the sources of information are specially acknowledged by means of references.

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DEDICATION

To Almighty Allah my creator and to the entire family especially my brother Alh. Isyaka Ibrahim and my Mother Alh. Rabiat Asande. For their love and support towards my education.

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ABSTRACT

This study is an investigation of the effect of rainwater and its permeation (geotechnical engineering properties) through the laterite soil of subgrade and- base---soils used in the construction of road in Keffi Local Government Area. Soil sample were collected with the aid of the tools; digger, tape, soil color chart, sieve and trowel from the borrow pit in Panda Development Area in Nasarawa state; the samples were subjected to the following laboratory tests; Particle (grain) size analysis, Atterberg limit test, Compaction test, California bearing ratio (CBR) test and Specific gravity test. Analysis of the grain size curve shows that soil sample A is well graded with coarse grained. The plasticity and shrincages indicate low plasticity of 19. 14%. As the water content increases, its effect becomes progressively easier that disturbed the soil structure and dried density achieved with a working comparative effort increases. The soil exhibits moderate swelling potentials with plasticity index of 12.60 with 90% consolidation in 50.5 minutes. The use of soil as road construction materials should however, be used with caution in the study area. The summary of the pavement condition result shows that; roads were generally moderate to poor and showed sign of fatigue cracking and patched areas and numerous. The study recommended that appropriate methods of improving the engineering properties of non-suitable laterite should be adopted.

CHAPTER ONE

INTRODUCTION

1.1 BACKGROUND OF THE STUDY

The increase in economic and commercial activities and the production of goods and services have resulted in the deterioration of roads in various parts of FCT-Abuja All the roads were constructed with soil materials that were locally available. The linking network must be of high quality with adequate maintenance all year through the linking of various land uses in different parts of FCT-Abuja is so important that one cannot talk of any success without calling to mind the performance condition of roads in FCT-Abuja.

As a result of the importance and frequent usage of the roads, the rate of deterioration is on the increase thereby leading to unprecedented delays and discomfort, safety risk of goods, lives and services are endangered, at the same time regularity is adversely affected. If the right skills and materials are put into the construction of the, roads, the financial involvement in terms of maintenance will be reduced drastically and the life span extended. Due to the inexhaustible importance of roads in FCT-Abuja it is very important to take a look into the various performance conditions and maintenance of the roads and the standards at which the work is done.

1.1.1 Pavement Condition Survey

This involves visual inspection on identify pavement distress features (such as cracks, and potholes), pavement, distortions (such as rutting, and corrugation) and edge failure, This must be supplemented by quantitative techniques, such straight' edge, crack and roughness measurements.

1. Straight edge: This technique is used for measuring rutting and other pavement distortions. The straight edge is laid across the top of the distortion, and the distance to the lowest level is measured from the bottom of the straight edge. The value of the measurement is affected by the length of the straight edge and is normally referred to a standard length say, 2.0m.

2. Cracking measurement: This technique measures the extent of- pavement surface cracking. It may be as simple as placing a shallow box that is open-ended top and bottom and that has inside dimensions of 1m2 on the pavement surface, and measuring the total length of cracks within that area (Smith, 1998).

3. Roughness Measurement: Both stationary and rolling methods are in use, although rolling methods are preferable because of speed of execution. The bump integrator, which measures the vertical movement of a single wheel relative to its mounting frame as it travels over the road at a uniform speed, has become a standard measurement method. Other methods such as Maysmeter, and Profilometer may be used. Other techniques and methods used for collecting pavement condition data are

field California Bearing Ration (CBR) testing, the dynamic cone. The highway Maintenance management cycle.

To execute maintenance operations are to supervise and control them to ensure that the programmed activities are carried out within the budgeted limits are at the required standard of performance. The person carrying out these works must thoroughly familiar with the various maintenance techniques available and must have an appreciation of cost and time.

4. Maintenance of Shoulders, Slopes and Drainage
   Surface water or groundwater is a major obstacle to road, because it erodes slopes, weakens pavements, destroys shoulders and slopes, and washes out culverts, embankments and even bridges. The focus of highway drainage maintenance, therefore, is to safeguard that components of the drainage system (ditches, drainage culverts, causeways, manholes under drains, and even the roadway surface to the roadside ditches and drains) remain free of obstructions and retain their intended cross sections and slopes. In addition, the objectives of shoulder maintenance activities are directed at retaining the cross-section and grade by grass cutting, shoulder grading and dressing. The maintenance engineer must be particularly on the lookout for should bulking which would prevent water from drainage from the roadway surface towards the side drains and ditches. The objective of slope maintenance is to protect the slopes against the damaging effects of water so that they retain their shape and stability (Agbede & Coker, 1997).

5. Maintenance of Unpaved Roads
   Unpaved roads include earth roads (those constructed with the natural soil occurring along the alignment, and gravel roads (those surfaced with a layer of material, which is mechanically better than the natural soil). The overriding objective of the maintenance of unpaved road is to permit the immediate disposal of surface water which, if allowed to remain, quickly causes the formation of ruts and potholes. Maintenance activities comprising grading, dragging, patching and re-gravelling are required to restore the camber (4-6 percent) to reestablish gravel from the road side and to remove deep potholes in addition corrugations by removing loose material from the road surface; to fill and rectify soft sports as well as to effect general repairs to the surface; and to add or replace gravel material in order to correct other severe defects, or upgrade an earth road to gravel road.

6. Maintenance of Pave Roads

   Pave roads maintenance techniques under the routine and preliminary periodic activities include sanding (in the case of bleeding); sealing of cracks and small areas showing loss of aggregate; reshaping (in the case of subsidence); patching of potholes and removal of surface irregularities such as ruts and penetrometer, field and lab skid testing, and precise leveling to record longitudinal and automatic traffic counts are carried out to record vehicle movements and chart growth, and fixed and portable ·weighing devices are used to record axle loads.

1.1.2 Pavement Surface Deflection Measurements

Highway engineering inputs revolve around pavement conditiG11analysis the pavement being the single most apparent measure of effectiveness because it affects the user, by way of side comfort and operating costs. Road roughness is relatively easy to measure but, as an indication of the structural problems associated with the surface distortions, it is not enough. Measures of the residual strength of road pavements are therefore necessary. A number of measurements techniques available to the engineer are:

(1) Rolling Wheel: This technique measures pavement surface deflection when a standard moving wheel load is applied. The deflection beam method, notably the Benkelman beam, deflectograph are the most well-known techniques.

(2) Stationary loading: This technique measures pavement surface deflection when a standard weight is allowed to fall on to a circular plate of

Standard dimensions. Such methods are the falling weight deflectometer plate bearing tests are included in this class. Techniques for assessing the condition of a pavement range from visual to sophisticated electromechanical methods as follows:

1.2 Statement of the Research Problem

'Sustainable Development Goals (SDGs) 9 and 11: Goal 9: Build resilient infrastructure, promote inclusive and sustainable industrialization and foster innovation (United Nations, 2015)' and

Goal 11: Make cities and human settlements inclusive, safe, resilient and sustainable (United Nations, 2015)'

In line with SDGs 9 and 11 of the UN stated above that seek to develop quality, reliable. sustainable and resilient infrastructure to support economic development and human wellbeing with a focus on affordable and equitable access for all, Facilitate sustainable and resilient infrastructure development in developing countries, and By 2030, provide access to safe, affordable, accessible and sustainable transport systems for all, improving road safety, notably by expanding public transport, with special attention to the needs of those in vulnerable situations, women, children, persons with disabilities and older persons is the major focus of this study.

Evidence from reconnaissance survey indicate wide spread pavement failures have occurred in most of the completed roads and many road sections have been rendered Untrafficable barely two years after completion. The Central Market-Kofar Goriya Road; the Central Market-Ungwar Mada Road; the Tudun Kofa- Yelwa Road has more than 80% of their entire length broken up (failed).During the rainy season, problems encountered include the complete failure of sections of roads under the combine influence of rain and traffic. Many of the feeder roads are waterlogged and marshy after rainfalls that rendered such roads unfit for use; these deficiencies are not normally remedial. The problems are further aggravated by inadequate waste disposal systems; hence, one can see collection of refuse disposal in the middle of roads in FCT-Abuja. This no doubt constitutes an obstacle to movement as sight distance and traveled way is greatly reduced. This study, therefore, intends to report the visual condition survey of some roads in FCT-Abuja in order to evaluate their performance. It is hoped that the study will help in both the maintenance strategies of existing roads as well in planning new roads in FCT-Abuja.

1.3 Aim and Objectives of the Study

Primarily, the study aimed at evaluation of visual condition of FCT-Abuja roads in order to evaluate their performance conditions. Various pavement distress defects and conditions are rated, for its density and severity, with a view to fulfill the following objectives:

a) Investigate the effect of rainwater and its permeation (geotechnical engineering properties) through the laterite soil of subgrade and base soils used in the construction of roads in FCT-Abuja Local Government Area;

b) Examine the causes of pavement failures; and

c) Determine the maintenance requirements of the roads in FCT-Abuja.

1.4 Justification and Relevance of the Study

Availability of good performing roads in FCT-Abuja is needed for efficient transportation of goods and services. The use of good and well-graded laterite materials will effectively reduce the cost of maintaining roads, minimize risk of accidents on FCT-Abuja roads and cost of repairs on vehicles will be minimal.

CHAPTER TWO

CONCEPTUAL FRAMEWORK AND LITERATURE REVIEW

2.1 MAINTENANCE MODELS

2.1.1a RTIM2 Model: This model was developed at the Transport and Road Research Laboratory (TRRL) Road Investment Model for Developing Countries. It is designed to assist the engineer and planner to study various aspects of a road investment project such as the optimum maintenance standards for the road, the effects of providing an earth, gravel or bituminous pavement, or the differing benefits that can be obtained by adopting various staged construction options. The detailed flow diagram of the model is shown in Figure 2.1 (Aderibighe, Akeju & Orangun, 1983).

Following specification of the investigation period and definition of the alignment of a road, either construction standards, cost or road surface condition are input (if the road exists), or construct costs are computed (if the road is new), although estimated already construction cost may also be input for a new road. Cumulative traffic to date, plus traffic forecasts, are then input along with the proposed maintenance standards, and for every simulated year hereafter, the worsening of the road surface is estimated and the cost of maintenance and vehicle operation computed, in terms of present value. Three levels of maintenance (namely, routine, recurrent and periodic) are defined in the model.

A maintenance policy input to the model for paved roads might consist of specified recurrent maintenance based on threshold values of surface distress (e.g. patch all areas with cracks in excess of 5m1m2) and periodic maintenance based on time or accumulated axle-load intervals. For unpaved roads, periodic re-gravelling might also be based on residual thickness of gravel after gravel loss. Any combination of strategies might be tested. The material quantities are computed and multiplied by the input costs to get maintenance costs.

2.1.1 b HDM-IIII: This model is designed with the same basic objectives as RTIM2, The major differences are in the number of road links it can evaluate simultaneously, the facility for endogenous comparison of alternatives and the inclusion of an appraisals many different road linics with sections and each has different design standards and environmental conditions.

2.1.2a MAINTENANCE MANAGEMENT

Maintenance management comprises of: planning, budgeting, programming and execution.

1) Planning means

a) Determining the maintenance activities to be carried out not only on both a long term and near term basis, but also on a fiscal year basis, and

b) Estimating the resources necessary to carry out the activities (i.e. labour, equipment, and materials).

2) Budgeting

Proper planning establishes budgetary requirements. Budgeting itself is a political process, which is initiated by the budgetary requirements. The budgetary allocations are very often less than the requirements and it is. the job of management -to—determine which of .the planned projects should be executed to maximize the social benefits. Detailed programming for maintenance operations should not take place until the budget is determined as illustrated in the diagram. below..

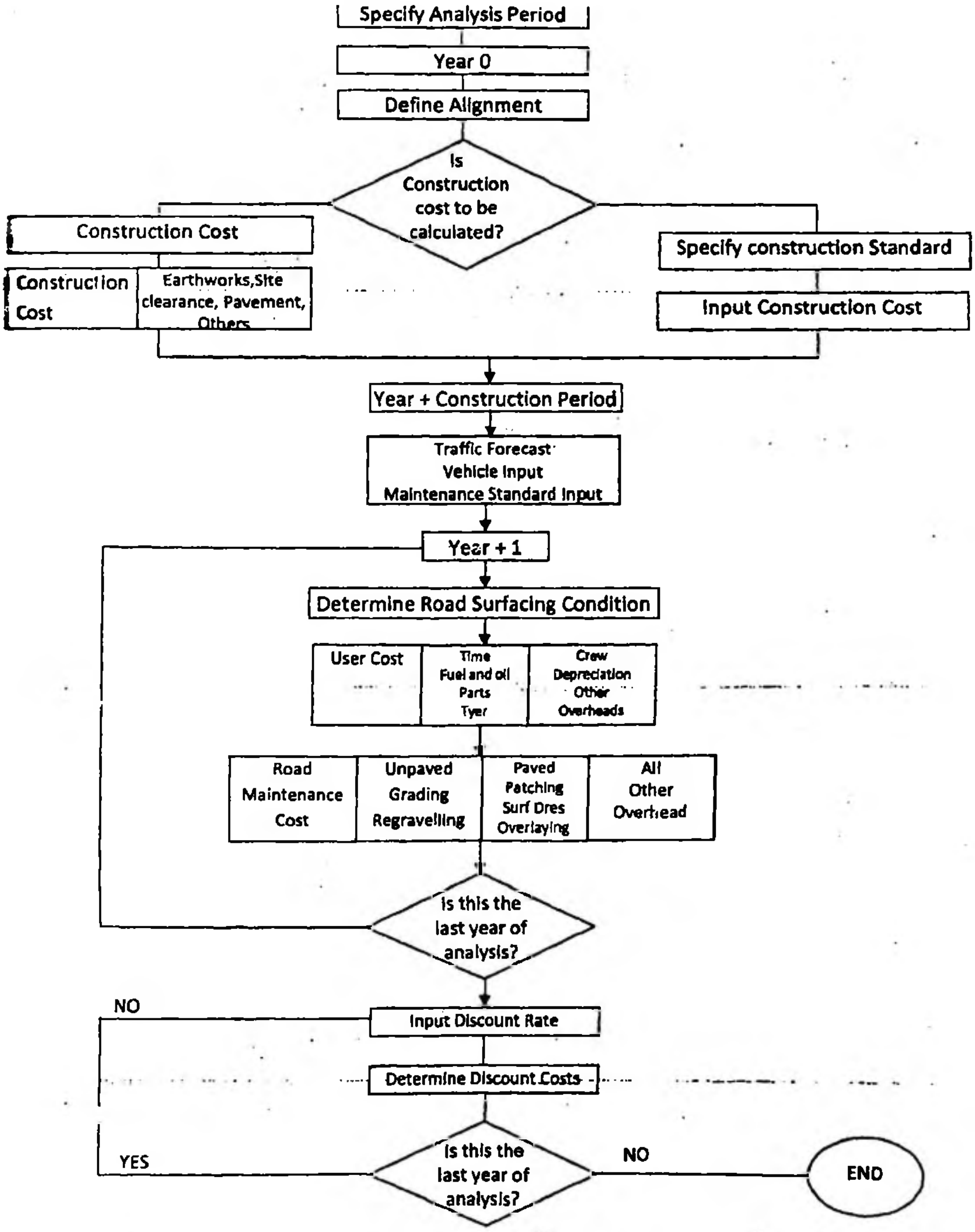


Figure2.1: An analysis table

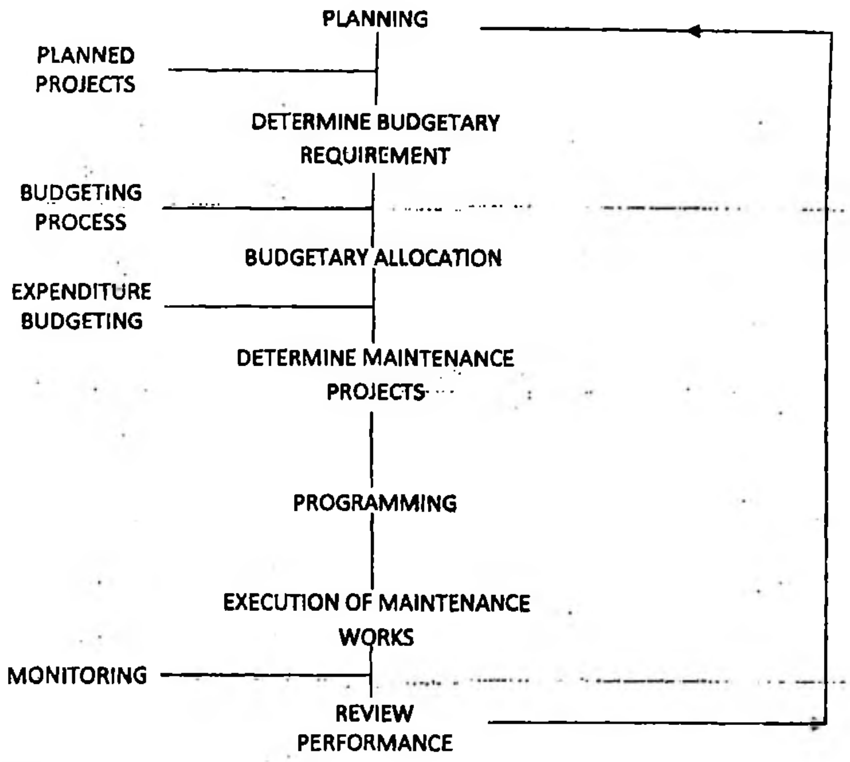


Figure 2.2: Planning established budget

For instance surface repairs, mixture of sands, aggregates and bitumen are used, while for repairs to the pavement structure itself natural and crushed stone materials, as well as bituminous mixtures, are used in a layered operation. Surface dressing consists of the application of a thin layer of binder material followed by the spreading of gravel or stone Chippings which are then rolled. The functions of the dressing are to arrest surface deterioration, provide an impermeable surface and to improve skid resistance. Surface dressing is usually done after the necessary general repairs have been made to restore cross-section geometry and is generally applied over the complete width of the pavement as a single surface dressing, double surface dressing, surfaces suffering from bleeding, cracking, glazing (i.e. embedment of chippings below the binder), stripping/fretting/raveling, streaking (i.e. embedment of chippings below aggregate parallel to the centerline) and local deformations, are candidates for surface dressing treatment.

— Surface dressings should- be rolled with a rubber-tyred roller- Over4ays are used where there is structural failure or where it may be deemed necessary for the upgrading of an important and heavily trafficked road, or to arrest progressive deformations on a surface. Overlays always consist of hot plants mixes of bitumen and aggregate that meet the specifications for a new construction and are spread in single lifts with a maximum thickness of about 5cm. it is necessary to apply a tack coal to the prepared and / or repaired existing surface before application of the overlay. Overlays are also rolled with a rubber tyred roller.

2.1.2b DESIGN OF PAVEMENTS FOR HIGHWAYS

A roadway cross section consists of two parts, the pavement structure and the subgrade with the pavement structure always supported on the subgrade. A pavement is therefore that portion of the roadway on which bears directly the wheel load. A pavement may be flexible, rigid or composite, depending on the type of materials making it up. Whatever type however, its main functions are:

(1) To distribute wheel loads over an area so that the bearing capacity of the subgrade is not exceeded.

(2) To provide smooth, skid resistant, non-dusty surface over which the wheel moves.

(3) To provide a protective coating against adverse weather conditions (e.g. rainfall) for the underlying subgrade.

Pavement design aims at providing a pavement structure that will serve traffic safely, conveniently and economically during the design life of the pavement when distress features appeal on the road surface they lead to discomfort as well as higher vehicle operating costs The designer therefore needs to understand the functions of the various layers of the road pavements so that his complete design will minimize the likelihood of this happening. Flexible pavements are nearly always multilayered structure, usually made up of an asphalt surfacing (or bituminous surface treatment), a base and a sub-base. The strength of a flexible pavement is derived from the composite effect of the various layers of the pavement. These layers are thus arranged in such a way that layer strength increases from the subgrade upwards, with the strongest material being placed on the surface. A typical flexible pavement structure is shown below. Below are the functions. of the various layers of the pavement:

(1) **Surfacing**: Generally provide a running surface capable of carrying wheel loads without undue discomforts to motorists.

a. Basecourse:

(i) It supports the wearing course and also assists in protecting the road

b. Wearing course:

(i) Provides a skid resistance surface

(ii) Waterproofs the pavement

(iii) Withstands the direct loading of the traffic

2. Road Base: The main load spreading layer of the structure

3. Sub-base:

(i) Assists in the load spreading

(ii) Assists sub-soil drainage (if a free drainage material is used)

(iii) Acts as a temporary road for construction traffic

(iv) Provide some protection to the subgrade as soon as it is exposed.

The Design of a flexible pavement is based on the fact that the loads of any magnitude are dissipated by being. carried deep into the subgrades trough successive layers of granular materials. The intensity of the loads diminishes in geometrical proportion from the top layer down to the subgrade by virtue of being spread over an increasingly larger area. Because of this reduction in Icadintensiry with increasing depth of pavement as shown below, materials of progressively lower bearing values are employed-in the lower-layers. -Only very small tensile stresses can be resisted in a flexible pavement, and any deformation of the underlying subgrade is always inflected on the surfacing. A rigid pavement is that which has concrete as its riding surface and as a rule the material (if any) between it and the sub grade is usually called a sub-base arid not a base. This is usually a two-layered structure.

The design of this type of pavement aims at providing sufficient strength in a structural slab made of Portland cement concrete to resist the destructive action of traffic. The two layered system is made up of the concrete surfacing of finite thickness and the sub grade of infinite thickness. IT has high load distribution properties by virtue of its beam and slab action over small weaknesses and depressions in the subgrade. It can withstand tensile stresses as high as $4.85N/mm^2$.

Composite pavements employ a combination of both flexible and rigid construction, i.e. about halfway flexible and halfway rigid. An example of this is a bituminous surfacing "laid" on a lean concrete base as part of initial construction or a bituminous surfacing "over laid" on an existing concrete surfacing as major maintenance operation.

The need for composite pavement may be dictated by material and equipment sources nearby. A composite pavement seldom meets first considerations buniiteiializes when the need arises to rehabilitate an existing rigid pavement.

It is thus, clear that the basic difference between a flexible and rigid pavement is the manner of transfer of load to the subgrade. Whereas wheel loads are spread through progressively weak materials in a flexible pavement, loads are spread through one material of high structural strength (high rigidity and modules of elasticity) in a rigid pavement. This reason alone makes the subgrade no paramount importance the design of a rigid pavement as against that of a flexible pavement, in which the depth above the subgrade depends on a consideration of both wheel and load intensity and subgrade supporting capacity. It should therefore be obvious that the terms "flexible and rigid" are arbitrary and serve mainly to distinguish between pavements made of asphalt and Portland cement concrete surfacing.

Rigid pavements are usually reserved for use at road junctions with high intensity of turning traffics, as well as on airport aprons, holding bays, hanger and runways. They are also used at parking areas where wheels stand for long periods. This is because of their initial high costs. Flexible pavements are used on longer stretches of roadways and on not too heavily loaded airport runways.

2.1.2c FACTORS AFFECTING THE CHOICE OF PAVEMENT .TYPE

The main factors that affect the choice of which pavement type to use (apart from those bordering on location as mentioned above) are:

1) Volume and character of traffic - both flexible and rigid pavements can be designed to meet most traffic requirements under given condition.
2) Cost comparison: This should include cost of maintenance In addition to the initial construction costs.
3) Subgrade soil characteristics
4) Weather conditions: Seasonal variations in rainfall will influence the bearing capacity of the subgrade soil as well as have a direct effect in the form of moisture, on pavement wearing surfaces by way of cracks, heaving and depressions reflected on the surface. These will push up the maintenance costs.
5) Performance record judging from the designers rating of the performances of pavements in the immediate area of his jurisdiction as past performances is always a useful guide in planning the future. Other factors which may influence the choice of the pavement type to use are:

a. Traffic safety

b. Availability of local materials

c. Adjacent existing pavements

d. Stage construction

e. Conservation of aggregate

f. Other construction considerations e.g. direct labour, available construction equipments.

2.1.2d ANALYSIS OF FLEXIBLE PAVEMENT STRUCTURE

The behavior of pavements subject to wheel load as applied has been examined using full-scale experiments as well as theoretically. Examples of full-scale experimental test roads are the WASHO and AASHOT roads in the US and the Borough Bridge and Alcanbury Hill test roads in Britain. Although very promising, these approaches have not yet reached the stage where a fundamental method of flexible pavement design can be evolved, and reliance is still placed on empirical design procedures and on existing pavement performance results. The following include some of the more common mathematical models.

1. Boussinesa Elastic Theory

Given a uniformly-distributed circular load on a similar layer of unlimited depth, the Boussinesq theory defines the stress at a give depth as follows:

$$\sigma_z = P\left(1 - \frac{z^3}{(a+z)}1.5\right)$$

Where

P applied surface pressure or intensity of loading at the surface

σ_z = Vertical stress along the vertical axis of loading

$\sigma_x = \sigma_y$ horizontal stress on the vertical axis of loading

a = radius of applied circle of loading

z = distance of the point from the surface; and

μ = Poisson's ratio

This theory works on the assumption that the pavement and the subgrade form an idealized elastic homogenous and isotropic mass with semi-infinite dimensions both laterally and vertically from the point of application of the load in the surface which is level. In other words, the theory considers the pavement and underlying subgrade as one layer i.e. single layer structure. In the above equation, the vertical stress (σ_z) is the major principal stress while ($\sigma_x = \sigma_y$) is the minor principal stress.

The maximum shear stress at any point is given as:

$$\tau_{max} = \frac{\sigma_z \ \sigma_x}{2} \quad \text{or} \quad \frac{\sigma_z - \sigma_y}{2}$$

The equations have also been adapted to given the vertical elastic displacement at the surface under the center of the applied loading as

$$\Delta = \frac{2pa\,(1-\mu^2)}{E}$$

E = Modulus of elasticity of the soil

A = Displacement

The businessq equations are not directly applicable to pavement design because of the many discrepancies between the assumptions and real life. They are only useful in elastic theory in estimating the stresses and deformation or deflection which a pavement may be subjected to under traffic loading.

2. BURMISTER'S TWO LAYER THEORY

For this theory, it is assumed that the top layers (consisting of the pavement structure) is an infinite elastic horizontal slab overlying a semi-infinite solid of lower elastic modulus, with the interface assumed to be either perfectly smooth or perfectly rough. The surface loading is assumed to be uniformly-distributed over a circular area. In simplified form burmister's two-layer system can be used to compute the elastic deflection at the pavement surface using the following expression:

$$\Delta = \frac{1.5PaF}{E2} \quad F \ldots\ldots 1$$

Where:

P = Intensity of applied load

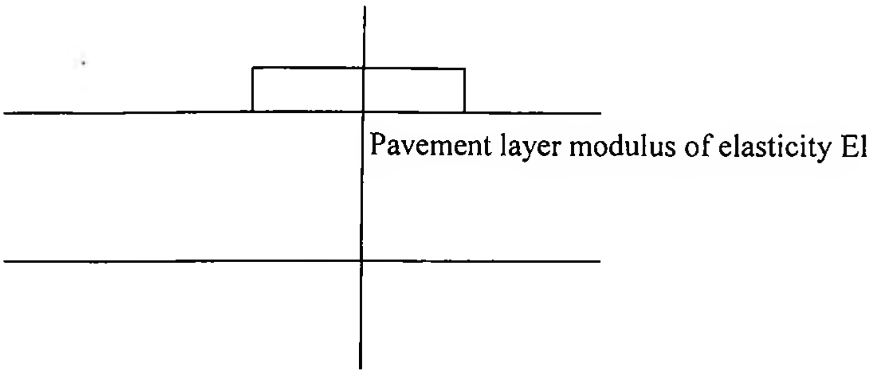
a = radius of circular area of loading

F = a displacement factor depending on the thickness of the top layer and the ratio E1/E2 (ranging from 0.02 to 1.0 for rating of E1/E2 between about 10000 and 2, respectively.

El Elastic modulus of the top layer (pavement structure)

E2 = Elastic modulus of the bottom layer.

The system is as shown below:



Subgrade: semi-infinite layer modulus of elasticity, E2 $\mu = 0.5$ for both layers

Other Models:

There are a number of other models which have been used to analyze pavement structures for stresses or displacements. Meyerhof's method of analysis is based on the Burmister's two layers system. and the guiding expression gives the transient deflections in the following:

$$\Delta = \frac{0.52W}{E2 \text{ cub.rt } (nt)}$$

Where:

Δ= Transient deflection

W = Applied load

t = pavement

n = El/E2

El = Elastic modulus of the pavement

E2 = Elastic modulus of the subgrade

Three layer and multilayer pavement systems have been analysed using the power of the modern computer. In this case the pavement system or structure is represented as a multilayered system but it does require that strength properties (the elastic or stiffness modulus and poisons ratio) for each layer are known. Pavement loading is then introduced and analysis carried out to determine the stresses or strains at critical points in the structure.

The stresses and strains obtained are compared with allowable values for the various materials used in the pavement structure. If the calculated stresses or strains are greater than these allowable values, the design is repeated using thicker layers or alternative materials. Finite element technique was used to carry out structural analysis of a multi layer system. The technique involves dividing a structure into finite elements, each of which is a simple unit whose structural behavior can be readily analyzed. The solution to the complete system is obtained by assembling the elements.

The limitations of mathematical models in the design of pavement structures are:

a) Mathematical models developed on the basis of assumptions that may not apply to the problem being. considered. For example, stress/strain relationships for road pavement materials are generally non-linear and are dependent on loading time as well as On temperature.
b) b) It is also difficult to model the fatigue characteristics of road pavement material
DESIGN OF PAVE ROADS
c) Many design methods have been developed to suit different climatic—and traffic loading conditions. For historical reasons most of the design methods for tropical countries were adopted from those technological for the European temperature climate.
d) Flexible pavement design methods can be divided broadly into empirical and analytical methods. Empirical and semi-empirical design techniques developed on the basis of long term pavement performance for specific traffic loading and environmental conditions was used. This means as long as conditions for which these techniques developed continues, the performance of the pavement is satisfactory. The following techniques shall be considered:

1) GROUP INDEX METHOD

e) This empirical design method, developed in the USA, is based on the particle size distribution and plasticity of the subgrade materials. The following is the design formula.
f) GI O.2a + O.OO5ac + O.Olbd
g) Where:
h) a = That portion of the % passing the No. 200 sieve which is greater than 35 and which does not exceed 75, expressed as a positive whole number (0 — 40).
i) .b = That portion of the % passing the No. 200 sieve which is greater than 15 and which does not exceed 55, expressed as a whole number(0 — 40).
j) c. = The portion of the numerical liquid limit which is greater than 40 and which does not exceed 60, expressed as a positive whole number (0 — 20); and
k) d = That portion of the numerical plasticity index which is greater than 10 and which does not exceed 30, expressed as a positive whole number (0 to 20).
l) GI ranges between 0 and 20, GI = 0 implies very good material (high bearing capacity), and GI = 20 implies very poor material (low bearing capacity).
m) The design chart shown in figure. 6.25 provides the pavement layer thickness for value of GI corresponding to different traffic loading levels.

ADVANTAGES

n) 1. It is a very simple design procedure. -

o) 2. No extra tests required data collected forms parts of standard soil classification procedure.

p) 3. It is scientific — caters for both traffic conditions and characteristics of subgrade soil.

q) 4. Pavement thickness are on the safety side.

DISADVANTAGES

r) 1. Uneconomical pavement thickness are often constructed.

s) 2. Usage depends on particular conditions of subgrade moisture and compaction

t) Example 3

u) A soil sample from a proposed highway site was tested in a laboratory and the following

v) parameters were obtained.

w) • Soil passing sieve no. 200 (Bs) 55%

x) • Liquid limit = 40%

y) • Plasticity index = 20%

z) It was further estimated that a traffic volume comprising 200% will be the sub-grade when it

aa) is opened. Determine the pavement thickness using the group index method of design.

Solution: The indexes expression is as follows:

GI	=	0.4a	+	0.004ac	+	0.02bd
a	=	55	-	35	=	20
b	=	55	-	15	=	40
c	=	40	-	40	=	0
d	=	20	-	10	=	10
GI	=	(0.4 x 20) + (0.004 x 20) + (0.01 x 39 x 10) = 8				

Using the group index design chart with Gi = 8 and 200 daily trucks and buses. From curve B, it is seen that approximately 350mm of pavement is required above the subgrade. While Curve A indicates that only 200mm of this can be of subbase material. However, figure 6,25 requires that no less than 4inches granular base should be used this is equivalent to 100mm. the top bituminous surfacing will carry the remaining 50mm.

Surfacing

Base

Subbase

... Total thickness = 35.0cm

CALIFORNIA BEARING RATIO (CBR) DESIGN METHOD

The CBR design method, as developed by the California state highway department, involves the determination of the CBR value.

Pavement layer thicknesses are then selected from the chart shown, on the basis of the relevant design wheel load. This method has undergone considerable modification over the 'ears to accommodate varying traffic loading patterns, as well as different environmental conditions.

This method has been adopted and incorporated into the FMW x H highway manual design method,

Design Steps = Clause 1 — 306.04

1. **Traffic**: The anticipated traffic for the design life of the pavement must be determined. This figure will be expressed as the number of vehicles per day exceeding 3 tons loaded weight. Lane distribution of these vehicles must be taken into consideration for multilane roadways as show in the table below:

REDUCTION VALUES FOR MULTILANE ROADWAYS

Number of Lane Both Direction	**Factors to be Applied Traffic**			
	Lane No. 1	**Lane No. 2**	**Lane No. 3**	**Lane No.4**
2	100	-	-	-
4	100	100	-	-
6	20	80	80	-
8	20	20	801	80

Lane No 1 is next to the center line or median on the drivers left.

2. Sub-base and base materials

Constituents designated for use in the erection of a flexible pavement must be evaluated to provide information for an adequate and economic design. The materials must also be checked to determine quality and to establish compaction requirements. The mechanical strength test to be used will be the California Bearing Ratio (CBR).

3. After the CBR value for the subgrade and the estimate of traffic have been determined, the thickness of the pavement structure can be determined from figure 1-306.1. The values from the chart should be rounded upward to the nearest centimeter.

4. Minimum surfacing thickness: The recommended minimum asphalt surfacing thickness is as follows:
   Light traffic 2inches (50mm) curves A, B and C
   Medium traffics 3 inches (75mm) curves D and E
   Heavy traffic 4inches (100mm) curves F.

Sub-base Thickness

Cement stabilized soil (sub-base) with CBR of 15% plotting this value in the curve shows a pavement depth of 250 centimeters.

Total thickness of cement stabilized sub-base

= 400cm — 25 Oem

= 150cm

Base Thickness

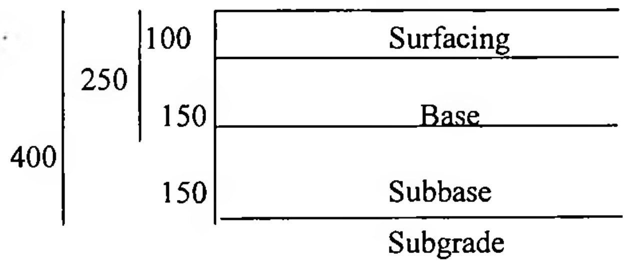
With a crush stone CBR of 75% and plotting on the curve, shows a depth of 4inches remaining.

Total thickness of crushed stone base = 250cm — 100cm

= 150cm

Surface Thickness

4 inches of asphalt is required to complete the pavement structure



TRANSPORT AND ROAD RESEARCH LABORATORY ROAD NOTE 29 DESIGN METHOD

Road Note 29 (RN29) presents a guide to the structural design of pavements for new roads for UK conditions. RN29 has however, been used in some tropical countries. Sub-base thickness is selected on the basis of subgrade California Bearing Ratio (CBR), and the expected cumulative standard axles during the pavement design life. Base and surfacing thicknesses are determined from charts, on the basis of the type of construction material and the design life of the pavement.

Design Steps

1) Find number of commercial vehicles per day at time of construction and traffic growth rate.
2) Decide on design life –
3) From chart for particular growth rate obtain cumulative number of commercial vehicles for the design life.
4) Multiply this number by the appropriate factor from table 2 to obtain cumulative number of axle loads (factor is based on type of road).
5) 5) Reduce number of cumulative axle loads to equivalent standard axles.
6) From charts 6 (six) obtain the sub-base thickness for the appropriate cumulative number of standard axles.
7) From chart 7 to 10 obtain the thickness for the base and surfacing for the appropriate cumulative number of standard axle loads for different types of roadbase i.e. rolled asphalt roadbase figure 7, dense macadam roadbase figure 8, lead concrete, soil cement and cement bound granular roadbases. Figure 9, and we mix and dry bound macadam roadbases figure10.

Note:

To allow for comparison of the relative damaging effect of various axle loads, a standard axle of 8.2t was adopted following the AASHO Road Test conducted between 1959 and 1960 in the USA. Other axle loads can be converted to equivalent standard value using the following approximate equation.

$$\left[\frac{D_L}{D_{8.2}}\right] \quad \left[\frac{L}{8.2}\right]^n$$

Where DL = Measure of damage caused by an axle load of Lt

D8.2 measure of damage caused by an axle load of 8.2t. n=a factor of about 4.5, which depends on the thickness of the pavement. Thus, one passage of axle load L is equivalent to [L/8.2]n passages of the standard axle.

TRANSPORT AND ROAD RESEARCH LABORATORY LR 113 DESIGN METHOD

This method has been developed from the large databank of experimental road test results, research in material characteristics used in pavement construction, methods of construction and mathematical models of pavement behavior. The performance of the experimental roads has been interpreted in the light of structural theory. Thus the pavement is designed for the ultimate condition of structural failure (collapse and deformation of the pavement surface). This has necessitated the redefinition of design life of a pavement as the cumulative traffic that can be carried before strengthening of the pavement by the application of a bituminous overlay become necessary. The criteria used in the development of this design method are:

1) Ability of subgrade to sustain traffic loading without excessive dilapidation by controlling the vertical compressive stress or strain at the starting level.
2) The use of bituminous materials and cement-bound materials in roadbase plans and not crack under the influence of traffic. Controlled by the horizontal tensile stress or strain at the bottom of the roadbase.
3) In pavements comprising a considerable depth of bituminous material. The internal distortion of these materials must be limited; their deformation isa function of their creep characteristics.
4) The load distribution ability of granular sub-base and capping layers must be satisfactory to provide adequate construction platform.

The thickness of the pavement layers are determined from design charts for each material type used in the roadbase. Figure 3,5 and 6 of TRRL 1132 refers to bituminous, wet mix and lean concrete roadbase respectively. The subgrade CBR must not be less than 5 percent otherwise a capping layer must be provided. The capping layer provides a working platform on which sub-base construction can proceed with minimum interruption from wet weather and to minimize the effect of a weak subgrade on road performance. The design assumed granular subbase of standard thickness of 225mm in all cases.

The probability of survival of the pavement to its design life has also been incorporated into the design, 85 percent probability is used. Thus for a chosen roadbase material, the thickness of the roadbase and surfacing can be determined given the cumulative traffic in million standard axles.

Design Steps

1) Estimate the annual average daily flow for the design year

2 Convert to total number of commercial vehicles by use of the formula

$$Tn = \frac{365F_0[(1+r)(a-1)]}{r} P$$

Where

r = growth rate

p = proportion of commercial traffic in design year

Fo = initial daily flow for design year

n design life.

3) Convert the equivalent standard axles by multiplying Tn by the vehicle damaging factor given below:

$$\Delta = \frac{0.35}{0.93 + 0.082}\left[0.26/0.92 + 0.82\left[1.9/3.9(f/1550)\right]\right]$$

DESIGN OF UNPAVED ROAD

The development of rural areas in many developing countries will continue to depend on road transportation for the foreseeable future. Because limited funds are available and because of low traffic volumes, rural roads are generally constructed to gravel standard only.

(i) The design of a gravel wearing course is generally based on the bearing capacity of the subgrade and the expected traffic volume. The following equation has been used to determine the total thickness of the wearing course for new gravel roads in Kenya.

DDI +NxGL

Where D Total thickness required

DI minimum thickness given in table below

N Period between regravelling operations (in years)

GL Annual gavel loss (in mm)

$$= \frac{fT^2}{T^2+50} (42+0092+3.50R2 + 1.88v)$$

Where

T =Total traffic volume in the first year in both direction (in vehicles x 103)

R = Average annual rainfall in (m)

V = Total (rise + fall) as a percentage of the road

And F* = 0.9 for lateritic gravels

= 1.1 for quartzitic gravels

= 0.7 for volcanic gravels

= 1.5 for coral gravels.

Subgrade strength CBR (%)	Initial daily commercial vehicles (both direction)			
	<15	15—50	50-150	150-500
2—5	350	425	500	575
5-10	225	275	325	375
7-13	175	225	250	275
10-18	150	175	200	225
15-30	125	150	175	200
>30	-	-	-	-

In general, shoulders should preferably be made up of the same material as the gravel wearing source and a cross fall of a percent should normally be provided. The design should consider the possibility of upgrading the gravel road to a paved road.

DESIGN OF RIGID PAVEMENTS

THEORETICAL AND PRACTICAL DESIGN CONSIDERATIONS FOR RIGID PAVEMENTS

The deformation of concrete stabs may be due to tensile, compressive and flexural stresses of varying magnitudes. These stresses are caused by applied wheel loads, changes in temperature, changes in moisture content in the concrete, as well as volumetric changes in the foundation soil. The theoretical values of these stresses as well as their practical implications in rigid pavement design will now be discussed.

Wheel Load Stresses

Failure of concrete slab due to wheel load (just like any other load) is by bending, where the flexural stresses developed by excessive bending moments exceed the modulus of rupture of the concrete. The modulus of rupture is taken as the extreme fibre stress under the action of bending loads. These stresses are of varying magnitude depending on the position of the wheel on the slab area.

The first known attempt at the theoretical analysis of stresses in rigid pavements was that by Dr. N.H Westergard of the University of Illinois in 1925. He examines three critical conditions of loading as illustrated below. These are at the corners, edges, and interior of the slab for which ~~he developed the three equations below:~~

$$\sigma_0 = \frac{0.31625\ P}{h^2}\left[4\log_{10}(1/b)+1.0693\right] \text{..........}(1)$$

$$\sigma_0 = \frac{0.31625\ P}{h^2}\left[4\log_{10}(1/b)+0.3593\right] \text{..........}(2)$$

$$\sigma_c = \frac{3P}{h^2}\left[1-\frac{(ad2)^{0.6}}{1}\right] \text{..........}(3)$$

Where:

P = point load, ibf

a = max, tensile stress, IBF/1n^2 at the bottom of the slab directly under the load, when the load is applied at a point in the interior of the slab at a considerable distance from the edges directly under the load at the edge and in a direction parallel to the edge.

Ac=Man, tensile stress, ibf7in2 at the top of the slab in a direction parallel to the bisector of the corner angle, due to load applied at the corner

h = slab thickness, in

μ = Poisson's ratio for concrete (=0.15in these equations)

E = Modulus of elasticity of the concrete, IblYIn2

K= subgrade modulus Ibf/In^2

a = Radius of area of load contact, in (the area is circular for corner and interior loads, and

semi-circular for edge loads),

b = Radius of equivalent distribution of pressure at the bottom (1 .2a2 +b2)5 — O.675h,

and I = Radius of relative stiffness, in

$$\left[\frac{Eh^3}{12(1-\mu^2)k}\right]^{1/4}$$

in developing these equations, Westerguard assumed that:

1) The concrete slab acts as a homogenous isotropic elastic solid in equilibrium.

2) The reactions of the subgrade are vertical only and they are proportional to the deflections of the slab

3) The reaction of the subgrade per unit area at any given point is equal to a constant k multiplied by the deflection at the point. The constant k is termed the modulus of subgrade reaction and is assumed to be constant at each point, independent of the deflection and to be the same at the points within the area of consideration.

(4) The thickness of the slab is uniform

(5) The load at the interior and at the corner of a slab is distributed uniformly over a circular area of contact, for the corner loading the circumference of this circular area is tangent to the edge of the slab.

(6) The load at the edge of a slab is distributed uniformly over a semi-circle being at the edge of slab:

The most circular situation illustrated by the above equation which were later modified refers to the corner loading were due to local depressions of the subgrade or warping of the slab, the corner portion may become unsupported, in extreme circumstances behave as a contilever. An important variable in .the corner formula is the modulus of subgrade reaction, k which is a measure of the stiffliess of the supporting soil subbase or subgrade, values of k vary very widely, depending upon the soil type, the unit weight and its moisture condition. This variation notwithstanding, the influence of the modulus of sugrade reaction is such that a large change in its value has but a relatively small influence on the calculated stress in a slab. This is one of the reasons why it is usually considered unnecessary to build up the supporting capacity of the subgrade beneath a rigid pavement with sub-base material so long as uniformity of subgrade material is guaranteed.

The third situation considered by Westerguard was when the wheel load is at the edge of the slab but a considerable distance from any corner when the load is applied, the edge deflects downward immediately under the load and upwards at a distance away. The critical tensile stress is therefore immediately beneath the centre of the area on the underside of the slab the tensile stress at the upper surface of the edge at a distance with a considerably smaller than that at the bottom of the slab beneath the centre of the semi-circle. Westerguards original stress formular for edge loading was later modified to take into account the situation where the slab was in a warping condition.

The initial formular for the second condition was also modified to allow the reactions of the wpporting soil closely concentrated about the load being greater than if the reaction is assumed tɔ be proportional to the deflection.

There are however, other considerations which affect the magnitude of these stress. These are the effect of multiple wheel loads (since the stress relationships were developed for single wheel load at rest), impact from moving wheel load, and frequency of loading or intensity of traffic. These effects are taken into consideration in practice by increasing the static wheel (design load by some factor of safety for impact and multiple wheel load effect (USA practice), by estimating and comparing the number of expected and allowed repetition of wheel loads (USA practice) for the frequency of loading, and by estimating the total number of cumulative equivalent standard axle load (ESAL) over the design life of the pavement (British practice) for the intensity of traffic.

WARPING STRESSES

A) Temperature Warping Stresses

These are due to temperature differentials throughout the thickness of the slab. They are induced by the restraints caused by the weight of the slab itself and by load transfer devices or friction at the pavement joints. The absence of the restraints will produce negligible stresses and no problem with warping of the slab (curling — in of the slab edges) will arise. The problem of temperature warping was also considered by Westerguard using the assumption that the temperature gradient from the top to bottom of a concrete road slab is the form of a straight line, he developed equations for the three cases. In the simplest one of these, he assumed that the slab was, infinitely large and derived the expression.

$$\sigma_0 = \frac{E\varepsilon t1}{2(1-\mu)} \quad \text{(A)}$$

Where:

σ_0 = Tensile stress developed, IB/in2

E = Modulus of elasticity of concrete, lb/in

ε= Coefficient of linear thermal expansion of concrete per degree FO

M = Poisson's ratio for concrete.

T1 = Temperature difference between the top and the bottom of the slab Investigations have shown that the temperature gradient is a curve and that in practice the stresses induced by temperature warping are not as detrimental as might otherwise be expected for the reason below:

(1) At slab corners, where the load stresses are actually the greatest, the warping stresses are negligible since the tendency of a slab to curl at these locations is restricted by. only a very small amount of concrete.

(2) At the interior of a slab and along its edges, significant curling stresses may be developed which under certain circumstances, are additive to load stresses. However, since concrete slabs are normally designed to have a uniform thickness based on the corner load needs, the margin of strength present in the interior and edge of a slab is usually sufficient to offset the warping stresses which are produced at these locations.

(B) Moisture Warping Stresses

Differences in moisture content between the top and bottom of a slab also cause warping stresses. This is due to the ability of concrete to shrink when its moisture content is decreased and to swell when the moisture content is increased. Very little is know about the extent to which this type of warping occurs, and as yet it has not been possible to develop a method of analysis which enables the stress produced by this phenomenon to be calculated. Warping stresses caused by moisture differences are not considered by most practical design methods utilizing calculations to obtain a thickness since, infact, the omission of this calculation generally results in a situation which error on the side of safety and recues the possibility of failure of the pavement. The reason of this is that the slab normally lies on a moist subgrade and therefore the bottom may have a moisture content which average, is significantly higher than that of the top. Furthermore, because of its higher water content, the bottom of the slab is in a more expanded condition and so there is a tendency for the slab to curl upwards. The net result is that the stresses produced by the moisture differential act to resist the stresses caused by the wheel loads, and very often the warping stresses caused by the temperature differential within the slab.

FRICTIONAL STRESSES

(A) Temperature Frictional Stress

These stresses are produced in the slab as a result of the changes in the average temperature in the slab, the net result of which is expand or contraction of the slab. As the pavement slab temperature increases or decreases, each end of the slab tries to move away from or towards the slab centre. A crack occurs about the centre of the slab if cooling takes place uniformly. If expansion is excessive and as might be on a very hot afternoon after cooling morning and night and adequate joints are not provided, then blow ups can result in adjacent slabs being jock knifed into the air.

Friction between the slab and the supporting soil results in compressive stresses being produced at the underside of the slab as it attempts to expand. As the slab contracts due to decrease in average temp, the same type of restraint is exerted but this time it results in. tensile stresses in the bottom of the slab. Stresses resulting from restraint of this type are only important when the slabs are quite long (e.g. .30m). Hence shorter lengths of slabs are used in practice. They are critical only when considerations allow them to applied when combined loading and warping stresses from other sources are at their maximum. Since the max tensile stress due to frictional restraint only occurs when a slab is contracting and since the warping stresses resulting from temp gradients are not at their max at this time, the net result is that in practice these restraint stresses are usually be neglected when calculating the maximum tensile stresses in a concrete road slab for thickness design purposes.

The magnitude of the restraint tensile stresses which are developed is heavily depended on the temperature conditions prevailing at the time the slab is laid. If a pavement is laid,, in countries where there are marked seasonal changes, at temperature of about 320 or normal air temp at afternoon, it can be expected that the pavement will spend a greater part of its life in a state of permanent contraction, with consequent development of permanent tensile stresses. If, on the other hand, it is laid at much lower temperature then much of its life will be spent in an expanded state and the stresses developed will be compressive ones which the concrete is usually capable of withstanding.

In Nigeria there is however no marked seasonal changes except in the north where half of the year is very hot (between March and October) and the other half cold (harmattan, October and March). In this case concrete pavements are better laid in the second half. In the south concrete pavements can be laid anytime but better still in the mornings.

(B) MOISTURE FRICTION STRESSES '

A concrete slab will also expand or contract as its average moisture content changes, throughout its life. Little is known about the stresses developed as' a result of these actions but it can be assumed that they are developed in direct opposition to the temperature stresses, when the temperature is raised (and the slab tends to expand) the moisture content is lowered. and this result in a tendency for the slab to contract. Investigations have suggested that the frictional stresses resulting from moisture movements are always less than those resulting from temperature changes. It is usually considered that they can be omitted from concrete road stress calculations since the moisture stresses are opposition and hence subtractive to the temperature.

THICKNESS DESIGN

ROAD NOTE 29 METHOD

Steps:

1. Determine the equivalent number of standard ales for the design life of the concrete pavement.

2. From table 5 (TRRL RN 29), determine the minimum thickness of subbase required for the type of subgrade (classified on the basis of the subgrade CBR).

3. For the value obtained from step 1, determine the minimum thickness of the concrete pavement for reinforced or unreinforced design using figure 11. The thickness obtained should be rounded upward to the next 10mm intercept.

4. For reinforced design, determine the minimum weight of reinforcement-for -the concrete slab. for the cumulative number of standard axles from figure 12.

5. Determine maximum spacing of joints reinforced concrete slab from figure 1 for the actual weight of reinforcement obtained in step 5. Every third joint should be an expansion joint, the remainder being contraction joints, with the provision that expansion joint may, at the discretion of the engineer, be replaced by contraction joints.

For unreinforced concrete slabs, the maximum spacing of expansion joints recommended in the RN is 60m for slabs of 200mm or greater thickness and 40m for slabs of lesser thickness, with intermediate contraction joints at 5m interval where aggregate other than limestones are used, where limestone is used throughout the depth of the slab, the maximum expansion joint spacing is increased to 722m and 48m respectively with intermediate contraction joints at 6m intervals.

For both cases, (reinforced and unreinforced) longitudinal joints should be provided so that slabs are not more than 45m wide.

PORTLAND CEMENT ASSOCIATED METHOD

The PCA method is based on the philosophy that pavement life is limited by concrete fatigue from repeated loads. The method based on this phytosophy utilizes the controlled parameters of traffic, concrete quality in terms of modulus of rupture, foundation support in terms of modulus of subgrade reaction and a factor of safety. This procedure is based on the analysis of westerguard (19)v and Pickett, (1951). It is the procedure adopted by the Portland cement Association (196) for rigid pavement design. The steps in the PCA-or design by fatigue accumulation method are as follows:

1. Determine traffic demand over pavement design life — classify total axle loads in increments of 20001b (2klb), separating single and tandem axles and calculate the repetition of the different axle loads for the design life using an approximate traffic growth rate. The projection factor, p, for a growth rate of r% over a design life of n years is given by:
   $P = (1 + r)^n$

2. Find the modulus of rupture, MR, of the concrete, and the modulus of subgrade reaction k, for the subgrade

3. Using a thickness estimate, scale off the concrete stress o, for each axle and category from figure 6 (Design chart for single axle) and figure 7 (Design chart for tandem axle)

4. Calculate the stress ratio σ_0Mr, for each axle load category and find, from table, the number of allowable repetitions of each load that will produce failure. Stress ratio less than or equal to 50 means the concrete will withstand virtually unlimited stress repetitions without loss in load bearing capacity.

TABLE 3:

Stress ratios and allowable load repetitions (PCA)

Stress Ratio	Allowable Repetition	Stress Ratio	Allowable Repetition	Stress Ratio	Allowable Repetition
0.51	400,000	0.64	11,000	0.77	270
0.52	300,000	0.65	8,000	0.78	210
0.53	240,000	0.66	6,000	0.79	160
0.54	180,000	0.67	4,500	0.80	120
0.55	130,000	0.68	3,500	0.81	90
0.56	100,0000	0.69	2,500	0.82	70
0.57	75,000	0.70	2,000	0.83	50
0.58	57,000	0.71	1,500	0.84	40
0.59	42,000	0.72	1,100	0.85	30
0.60	32,000	0.73	850		
0.61	24,000	0.74	650		
0.62	18,000	0.75	400		
0.63	14,000	0.76	360		

(4) From the ratio of expected axle loads to repetition to failure or allowable repetitions, find the fatigue resistance used in each category of axle load.

(5) Sum up the figures obtained in steps to get the total fatigue resistance used. Theoretically, the total fatigue resistance used should not exceed 100%. In practice about 100% is suitable for designs based on the 90-days modulus of rupture. For designs based on the 28-day

modulus of rupture, fatigue consumption can be increased to about 125% especially if the load is to be opened to traffic after a much longer period, say 90 days and beyond.

In the event that the total fatigue resistance consumed exceeds the allowable maximum value (100% - 15%). A thicken pavement is tried and the procedure repeated.

LOAD SAFETY FACTORS (LSF)

The following load safety factors are recommended

(1) For inters state, multilane and single lane projects there will be an uninterrupted traffic flow and heavy volumes of truck traffic = 12

(2) In the case of highways, arterial and streets where there were moderate sizes of truck traffic, LSF=l.l

(3) On the highways, residential streets and others that carry small volume of truck traffics LSF = 1.0

2.1 An Overview of Road Condition

Inability to maintain roads is identical to an act of disinvestment; it implies the loss of the past investments in roads. Inadequate maintenance of roads is responsible for the loss of an estimated $45 billion in developing countries. This loss could have been averted, with preventive maintenance costing of less than $12 billion (World Bank, 1988).

A large network of roads, have not been adequately maintained and utilized, resulting to deterioration of roads in many Third World countries. A significant number of roads are in a state of disrepair to the extent that maintenance is not adequate and-effective. These roads. now require rehabilitation or reconstruction at three to five times the cost of timely preventive maintenance and strengthening. And many more roads, whose, deterioration is not yet visible, will soon reach that point if they are not better maintained. Much of the problems of road maintenance are rooted in its economic and institutional aspects. Inadequate incentives and weak accountability derived from the characteristic separation of responsibility and control between the providers and users of roads. The prospects are not encouraging.

This has led to 'the creation of the Petroleum (Special) Trust Fund (PTF) by the "Aba" administration that engaged in road construction, re-construction and maintenance of rural and urban roads in the county. Inconsistencies in government policies and lack of political will have led to the scrap of PTF by the Obasanjo's administration and yet created another agency, The Federal Road Maintenance Agency (FERMA) for the maintenance of roads in Nigeria. Even with the current road maintenance project supported by the Federal.

Government, unless more resources can be put into road maintenance and applied effectively deterioration will continue. If so, it will take far more costly rehabilitation to forestall the almost total collapse of road transport in the county.

2.2 Laterite of Road Soil

Apart from "Lateritic soil" or "Laterite" the word Maloma (1977) has suggested "Red soil". He said the word "Red soil" is used to cover all accumulation of oxides iron and alumna. In other words, red soil is highly weathered materials rich in secondary of iron and aluminum. According to Alexander and Candy (1962), it is nearly devoid of bases; and primary silicates, but it may contain larger amount of quartz and kaolinite. It is either hard or capable of hardener on exposure to wetting and drying. Two group of red soil has been identified for geotechnical evaluation on the basis of the type of parent rock weathering condition made of genesis, and degree of dehydration in situ, Gidigasu (1971). These are "stable and "Sensitive" red soil stable red soils are form generally in areas of moderate climate. Low natural moisture contents and low paucit due to high degree of desiccation generally characterize them.

Sensitive red soils by virtue of other genesis structure, mineralogy and or degree of dehydration are very sensitive to drying and or remolding and evaluation of either property is unreliable when used upon standard procedures. These have been nerally identified as weathering in regions of recent volcanic activity and or of continuous wet climate, (Agbede, 1992);

2.3 Physical Properties of Laterite

Townsend et al (1967), Gidigasu (1974), Malomo (1977) and Grant and (1970), showed that two properties peculiar to red soils and not found in the soil are susceptible to change as a result of the addition of small levels of Thermal or Mechanical energy.

2.4 Thermal Instability

When red soils are subject to drying, the soil materials undergo chemical and mineralogical changes. These changes are mainly due to changes in temperature, hence thermal instability can be considered as energy precipitated reaction. An addition of energy to the system leads to higher Energy State. The soil can be stable or unstable at this state depending on the initial condition (Agbede, 1992; Gidigasu, 1977).

2.5 Mechanical Instability

This is made manifest in changes that occur on remolding and manipulation of the soil from the in-situ state. This causes the breakdown of segmentation and the structure of the soil, and affects such parameters as grain size. Atterberg limits, particles size distribution etc. (Townsend et al, 1969, New, 1961). Mechanical instability is due to dislocation of one particle with respect to another leading to the failure of the bounds or actual removal of the cementing agents (Agbede 1992). Terzaghi (1958) observed that the high bearing strength, low plasticity and high permeability association with undisturbed red soil appear to be lost upon working on the soil that is mixing, compacting or employing any type of extensive mechanical manipulation in the presence of water.

2.6 Pedogenic Process of Tropical Weathering and Laterization

Tropical weathering and laterization essentially involve chemical and physicochemical alteration and for transformation of primary needs forming minerals into materials rich mainly in 1.1 Lattice change minerals and laterite constituents (Fe, Ai, Ti, & Mn). Three major stages of the process have been identified as follows. The first stage (decomposition) is characterized by physic-chemical break down of primary minerals and the release of constituent elements (Sio_2, $A1_20_3$, Fe_20_2.CaO, MgO, NaO). (Longhman, 1969). The second stage (Laterization) involves the leaching, under appropriate drainage condition of combined silica and bases and the relative accumulation of enrichment from outside source of oxides and hydroxide of sesquioxides (mainly $A1_20_3$, Fe_20_3 and $Ti0_2$). The soils and condition under which the various elements are rendered soluble and removed through leaching or combinations with other substance appear to drainage-conditions (Pickering, 1962 and Longhman, 1969).

Chemical weathering of the primary minerals is the second stage under which the condition of low chemical and soil forming activity, the- physico-chemical weathering doesn't occur beyond the forming stage and this intend to produce end products consisting of clay minerals, predominantly requested represented by Kaolinite and occasionally by hydrated or oxides of iron and aluminum (Mohr, 1984) under intense and prolonged physicochemical weathering, however change minerals are destroyed and silica is leached; the remainder will merely consist of aluminum oxide such as gibbsite or hydrous iron oxide such as limonite or goethite, and with hematite (Hamilton, 1964).

2.7 Morphological Characteristics of Laterite Soil

Genetically, two major groups of laterite soils have been identified. They are two alumni (bauxite) and ferruginous laterite soils the significant genetic characteristics of the group of laterite soils are summarized in table 2.1 below.

Table 2.1; morphological characteristics of laterite soils

Characteristics	Aluminous Laterite soil	Ferroginous Laterite soil
Site	Old forms	Principally deep form
Induration	Slight to moderate	Moderate heavy and even very heavy
Density	Low	High
Structure	Basically Siorraceous	Extremely carved; Pisolithic alveolar, Lameliar etc.
Chemical	Strongly hydrated (20%)	Slightly hydrated (10%) plenty of insoluble material, Kaolinite & goethite
Composition	Little insoluble material Mineralogical composition Principally gibbsitc, goethite, quarts, absent or present non abundant	

Source: After Maignen, 1966

2.8 Composition of Laterite

The proportion of sesquioxides of iron (Fe203) and aluminum (A1203) relative to other chemical components is a feature characteristic of the grades of laterite soils at higher level. (Maignen, 1966) have shown that two groups in which the iron oxide predominate (ferruginous laterite soils) and those in which alumna predominate (alumnus laterite soils). The chemical composition of typical laterite soils from different tropical area is given in table 2.2 below. Note that generally bases are almost completely absent. Combine silica is generally believed to be low, but some soils may have significant amount, probably in the form of kaoline, which appears to be the commonest silicate clay mineral in laterite soils.. Other common chemical constituents of laterite soils are oxides of manganese (Mn), titanium (Ti), chromium (Cr) and vanadium (V). Titanium oxide doesn't commonly occur in significant amount in most varieties but it may sometimes be a major constituent in some laterite soils (Maignen, 1962).

Quartz may be absent or present in only limited amounts but in rocks rich in quartz, it is a significant or major component, as in granites. It is considered as important factor in differentiating aluminous laterites from ferruginous ones. The bases (Mno, Mgo, 1(20, Na20) are almost absent in most laterite soils except. in some ferruginous crust development is alluvium and concretionary horizons in some ferrignous types of tropical soils. Buchana (1807) attached special importance to the role iron oxides play in laterite rocks seem to consist mainly of the crystallization of the amorphous iron oxides and dehydration (Alexander and Cady, 1962). The presence of iron in laterite soils is also considered to be the most important factor, which influences their engineering behavior. Maignen (1966) proposed the use of molecular silica sesquioxides ratio instead of the silica- alumna ratio.

Table 2.2: Chemical Composition of Typical Laterite Soils Mohr, 1962)

Type	Feroginous bauxite Laterite	Laterite	Concretionary iron-stone	Aluminous Laterite
Locality	Satara India	Nigeria	British Gurana	Goaso Ghana
$Si0_2$	0.9	26.5	5.7	21.91
Al_20_3	26.3	19.9	7.1	15.75.
Fe_20_3	36.0	30.7	76.3	45.13
$Ti0_2$	3.6	1.1	0.3	2.17
H_20	14.4	-	8.02	13.60

2.9 Chemical and Mineralogical Characteristic of Laterite

Chemical analyses do not usually reveals the origin,, nature or even the composition of laterite (Maignen, 1966). He further explains that physically similar laterite may have different chemical composition and chemically similar laterite the clay minerals most common is the kaolinite; others are the Halloysite, illite and Moutmorrilonite. Similarly, there are variations in colour of laterite ranging from very dark red, and reddish brown to shade yellow or pink depending on the quantity of iron oxide present and its state of hydration. The chemical and mineralogical characteristics of all laterite soils exhibit some similarities as observed by Clare and O'Kelly (1960).

2.10 Permeability Characteristics of Laterite

Permeability is said to be a measure of the rate at which fluid passes through a porous medium. Soils are said to be permeable if they permit the passage of fluid through the voids between the soil grains (Osinubi, 1998). Taylor (1948), stated that the coefficient of permeability for a given soil is used to indicate the permeability level of the soil. Structure has generally been observed as the most important variables influencing-the permeability of compacted latente, investigation by Olsen (1962) has shown that the permeability of a cluster structure is controlled largely by flow through intercellular pores, rather than through the cluster themselves. At low water- contents the clusters have high strength and should be able to resist the static compaction pressure without appreciable distortion. On the other hand, Mitchell et a! (1976) observed that clay particles were randomly flocculated with aggregates or cluster that formed even during carefully controlled hand mixing of soil and water prior to compaction(Ola, 1983). According to the study on the permeability of compacted clay by Mitchell (1976) he found that the permeability of compaction curve was many times higher than on wet side. Lambe and Whitman (1969) explained that reason for this phenomenon in terms of particle diameter, properties of the porefluid, the void ratio, the shape and arrangements of the pores and of the soil particles. Many soil compacted dry of OMC had more random particle orientations and larger average pore size than when compacted wet of optimum water content, where the particles had more parallel arrangement. The larger the individual pores for any given total pore area, the greater the flow would be, since permeability varies as a power function of pore size.

2.11 Compressibility of Laterite Soils.

Evaluation of the consolidation characteristics of residual laterite and other tropically weather days on the basis of Terzaghi's theory of consolidation has been found to be useful in predicting the settlement of structure (Lumb, 1962). Experience has shown that the decrease of the void ratio with applied Nomian pressure in residual clays follows the same law, which governs the consolidation phenomenon for sedimentary clays. Vargas (1953) has also reported that casagrande's reconsolidating presume somewhat similar; that is up to a certain load, the decrease in the void ratio by consolidation is very small, after the applied load exceeds that limit, the relation between the void ratio and the applied presume follows the consolidation law.

2.12 Isotropic Compression and Swelling

The general behavior of soil during isotropic compression and swelling is illustrated in figure 2.1 below. This shows soil, which the grains are loosely packed as described by Atkinson (1995). Initially at Po: at 0 compressed to A, unloaded to B and Ke loaded through C to D when the grains are more densely packed. Compression is primarily caused by rearrangement of the grains and so the stiffness will increases from loose state (where there are plenty of voids for grains to move into) to dense state (where there is much less opportunity for grains to rearrange). This is the mechanism of volume change in soil due to rearrangement of the grain accounts for the non-linear bulk stiffness behaviors. For the unloading reloading loop ABC the soil is very much stiffer (i.e. the volume change are less) than for the first loading because the grains will obviously not re-arrange them on unloading behaviour.

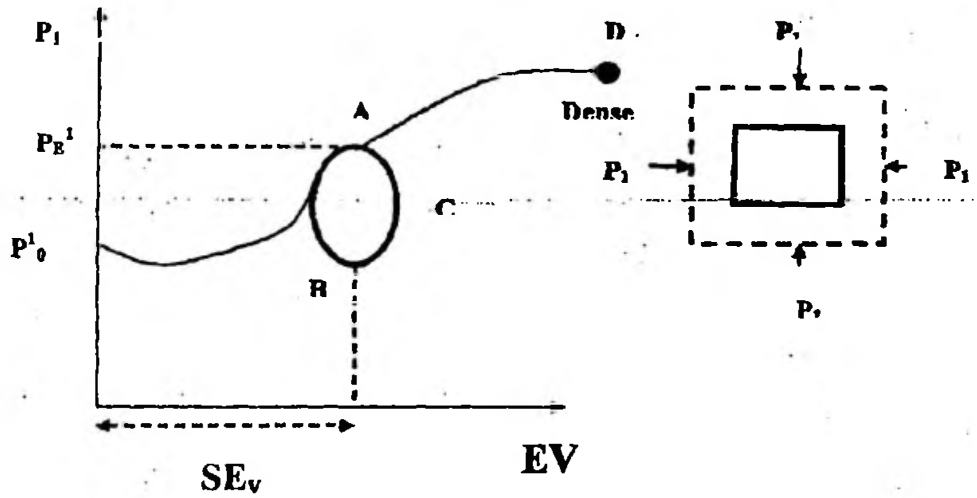


Figure 2.1 Isotropic Compressions and Swelling of Laterite Soil

2.13 Volume Changes Characteristics

As soil is loaded or unloaded to change of effective stress, It will gradually charge in volume. However, because the soil grains themselves are very stiff the volume change of the grains is negligible so that volume change of the soil must be due to re-arrangement of the grains and change in volume of voids at small effective stress. The spacing of the grains may be base ard act high stresses it will be decreasing. Atkinson (1993).

2.14 Shrinkage and Swelling

In considering the clay materials structures and interlayer bonding. It would be expected montmorillonites and vermiculite undergoes greater volume clac wetting and drying than do Kaolinites and hydrous micas. Because of the problem encountered in the performance of structures founded on high volume change soils, several attempts have been made to develop reliable methods for their identification. The most successful of these are based on the determination of some factor that is directly related to clay mineral composition such as shrinkage limit, plasticit index, and percentage less than 1μm and activity (Holt and Gibbs, 1956; and Lungren, 1962). Swell test on artificial sand-clay mineral mixture were used as the basis for development of one method according to the seed Laungren, 1962). The standard for comparison was the expansion of laterally confined specimen, at optimum water content using standard AASFIO compactive effort, and allowed swelling under a surcharge of IPS 1.

An excellent correlation was found expressed both the and compositional factor of clay, according to

$S = 3.6 \times 10^{-5} A^{244} C^{3.44}$

Where S Percent swell of sample prepared and test under the specific conditions.

A = Percent activity

C = Percent clay size <2Nm).

Further analysis of the relationship indicated that for natural soil the swelling potential could be related to the plasticity include with an accuracy of 135 percent (Seed, Wood Ward, & Lungreen, 1962)

S = 2. 16 X HY(P1j2

A Slightly different relationship was found by Rang Nathan and Satyanarayana, 1965) to better classify the swell potential of some soils, especially the black cotton compositional factors relating to swelling and is defined by

SA=SI V C V

Where SI is the shrinkage index,

The resulting relationship is $S\ 41.13 \times 10^{-5}\ SI^{2.67}$

2.15 Physical Interaction in Volume Change

Physical interactions between particles that are important during soil compression include particle bending, particle sliding, particle rolling and particle crushing. In general the coarser the gradation, the more important physical interaction relative to physico-chemical interaction are particle bending is important in the case of platy particles. The presence of even small amounts of mica in coarse-grained soil can greatly increase compressibility. Mixtures of a dense sand having rounded grains with mica flakes can duplicate the form of the compression and swelling curves of day. A sample of Chattahoochie River sand with mica content of 5% is twice as compressible as one with on mica (Moore, 1971).

2.16 Factor Controlling Resistance to Volume Change

The amount of compression or swell in any case depends on both compositional and environment factors and meaningful quantitative prediction and only if undisturbed samples or in- situ tests are used for evaluation of need parameter. The following factors are important in determining the resistance to volume change.

2.16.1 Physical Interaction

Physical interaction includes bending, sliding, rolling and crushing of soil particles. Physical interactions are more important than physico-chemical interaction at high presume and low void ratios.

2.16.2 Physico-Chemical Interactions

These interactions depend on particle surface forces and are responsible for double layer interactions, surface and ion hydration, and inter-particle attractive forces. Physicochemical interactions may be more important than physical interaction at low presume and high void ratios, (Seed, Mitchell and Chan, 1962).

2.17 Utilization of Lateritic Soil:

In the past, researchers have discovered the use or laterite soil in many ways, Laterite soils have been one of the major building materials In Nigeria for a long time. Wails of the large percentages of residential homes in rural areas have been building and continue to be built with lateritic soils in different forms. Mesida (i978) has established that lateritic soils in Okitipupa area of Ondo state need only 12% cement for stabilize laterite soilto produce blocks of the same order ?f compressive strength as for cement sand mix otherwise called concrete blocks (Lasisi. 1977).

Aderibigbe et al, (1983) while working on some local laterite soils in Lagos area reported that, stabilized laterite can be use in many areas of building construction i.e. as local bearing walls, partition walls etc. Agbede and Coker (1997) concluded that although laterite soil is largely, indispensable as a building material, yet some stabilizing measures aimed at making up for any deficiency in the soil must be in the place before its application especially in compress earth brick technology. Laterite can as well be selected and will perform very well as. a road building material; In case of gap graded laterite an almost ideal base course material can be obtained, provided the proper mixing ration of gravel to clay sized particle present in the material. After compaction, the pavement layer will then consist of a skeleton of gravel sized particles the voids of which are filled with clay and silt sized material. This material will behave properly in the dry condition but may readily loose its strength on wetting (Sweere and Galjaard, 1984). All these studies have been generally geared towards the attainment of results have that will enable popper utilization of the abundantly available laterite.

2.18 Laboratory Test

Purposes of laboratory test include among other things (Atkinson 1993.)

2.1 For description and Classification of a particular soil.

2.2 To investigate the mechanical behaviour of soil and to develop theories of soil behaviour.

2.3 To determine design parameters.

Soil can be classified in the field or in the Laboratory, field techniques are usually based upon visual recognition. Laboratory techniques include several specialized tests.

2.18.1 In the Field

Visual examination: All disturbed soil samples, both jar and bulk, should be examined individually, described and a permanent record made either on site or shortly after they arrive in the laboratory. It is customary during samples or to examine the sample obtained from the cutting which were used, or both. However, all undisturbed samples should be re-examined, each time a specimen is taken testing. When it is know that no further soil testing is likely to be required the remaining undisturbed sample should be extruded and split down the middle for examination and description, with special attention being given to noting the fabric and structure of the soil.

2.18.2 In The Laboratory

The common laboratory test on soil described in BS (5930: 1981) table 4, together given reference given detail methods.

Soil Classification test

Soil Chemical test

Soil Compaction test

Pavement design test Soil Strength test

Soil Deformation test

Soil Permeability test

Soil corrosives test

2.18.3 Sample Quality

Handling and quality of samples are defined according to BS (5930: 1981). It is essential that sample used is of sufficiently high quality for the test in question. When sample arrive in the laboratory, all necessary steps should be taken to ensure that they are preserved and store at their natural moisture contents and suffer the minimum amount of shocks and disturbance. Where the test is carried out on an "undisturbed" sample, the sample may, in truth, be far from undisturbed.

2.18.4 Sample Size

For disturbed sample the amount of soil required for any particular test is given in test I' or 13S (5930.198 1). As the behaviour of the ground is greatly affected by discontinuities, undisturbed sample should ideally be sufficiently' large to include a representative pattern of their discontinuities. This can often be achieved by the use of large undisturbed sample.

2.8.5 Condition of Test

Where, as in case of soil strength, the test can be the one in the condition approximate to those that will exist in the field at the being considered in the design.

CHAPTER THREE

STUDY AREA AND RESEARCH METHODOLOGY

3.1 The Study Area

FCT-Abuja Local Government Area is largely an urban Local Government Area and FCT-Abuja town is one of the oldest towns in Nasarawa state that enjoy the eminent position of being a foremost centre of history. Metropolitan FCT-Abuja currently has ten political wards namely — Tudun Kofa, Gangaren Tudu, Yara, Goriya, Limabaji, Ungwar Rimi, Iya I, Iya II, Sabon Gari and Jigwada, these serve as spatial reference points for analysis. In the high density areas, there are usually large cluster of old houses with high concentration of households and indecent surroundings. The situation becomes even worse with the ever-increasing migration of low or no income earners from the surrounding rural neighborhoods into the town. This has resulted among other things, in the increase of petty businesses, which competes for accommodation with households in the same buildings and therefore, has aggravated the problems of shorter dwelling units.

Similarly, houses within the high residential areas reflect the images of their inhabitants, particularly in the complete separation of women, and in directing day time activities toward an inner court. The basic emotional needs of utmost to the residents, the need for privacy, and family entertainments. These are emotionally connected, and were taken into consideration to a great extent in terms of the physical layout of the houses. The physical and mental health as well as the social well-being of the inhabitants of low and medium density areas are therefore, by virtue of their relative cleaner, more refined and quieter living environment, generally more conducive to a better quality of life, than those of their fellow citizens residing in the high density areas. Abubakar (2000) observed that many potentially harmful materials such as solvent and pesticide containers, medical waste and asbestos debris, even though prohibited are already present in the collected waste in FCT-Abuja. FCT-Abuja is located between Latitude 8°50'55" N a.nd7°52'25' E or & 84861 and 7.8736.1 (in decimal degrees).

Transportation network in urban FCT-Abuja is limited to motor vehicles and motor cycles. Roads in FCT-Abuja can be categorized into the following hierarchy: Arterial and collector roads that pass through the town from Akwanga (with only one lane) to Abuja (with two lanes each) and also foot/cycle paths, they are the intercity roadways linking the city to neighboring states, and they also form the boundaries of the phases of the city. They also receive traffic from local streets and deliver to the access roads which are meant to serve the residential areas; Local streets have single two lane carriageways with foot/cycle-paths,; and Access roads with single two lane carriageways and cycle paths. The mass-Transit System for moving large numbers of people into FCT-Abuja center and out again is mainly the buss (vehicle) system with the main transportation terminal being motor parks and the FCT-Abuja central market, these are the locations where buses (vehicles) originate and terminate. The main water supply to FCT-Abuja is the River Macla Water Works at Gudi. It has a maximum capacity of supplying water to a population equivalent of 350,000 inhabitants which means that it is not enough to meet the ultimate city requirements. The transportation of water to FCT-Abuja is by gravity through a series of water tanks around the periphery of the city. This is a unique system because the clear water reservoir of the treatment plant is elevated enough to facilitate the gravitational flow right to the city. However, there are areas in FCT-Abuja where the water pressure is too low due to head losses and for those areas booster stations would be necessary. Municipal solid waste in FCT-Abuja is composed of paper, plastics, food, yard wastes, glass, metals, and wood among others.

3.1.2 Physical Characteristics

3.1.2.1 Geology and Relief

The landscape of FCT-Abuja forms part of the low plains of the Benue trough. This plain is believed to be tectonic in origin and is lying in depression. The Maloney Hill in FCT-Abuja is of historical significance. FCT-Abuja is drained by numerous fast-flowing streams that take their sources from Jama'a catchment in Kacluna State and flow into the River Antau (Hamillton, 1960).

FCT-Abuja is well-endowed with both renewable and non-renewable natural resources. The major non-renewable resources include the constructional materials such as gravel, sand, clay and earth. Sand is obtained both on land and from river beds. The major renewable natural resources include water resources, a wide variety of economically important timber species, pole-wood, fuel-wood; edible vegetables, fruits, nuts and seeds; medicinal plants, palm wine and other paln products; fibers; and tannin.

3.1.2.2 Climate

a) Pattern of Mean Rainfall in FCT-Abuja

FCT-Abuja is characterized by a dry season of seven months or more, usually between October/November and April/May. Data from the National Meteorological Department, Lafia, 2015 suggest that the mean annual rainfall distribution in FCT-Abuja ranges from 1000mm to 1200mm. The minimum rainfall received during the dry season ranges from less than 50mm to 100mm.

Similarly, the data further show that, the mean onset date of rains in the area is currently before 10th of April (for early onset), the late rains generally come before 20th April. On the average, rains terminate in October, but sometimes may extend beyond November However, the, mean cessation dates is generally between 17th and 27th October. The mean length of rainy season (LRS) ranges from more than 150 days to within 180 days.

b) Sunshine

The duration of sunshine per day, combined with solar radiation intensity are two important parameters that determine the drying power of the ambient air. FCT-Abuja enjoys high sunshine hours of.8-9 hours per day. Hence daytime lengths are on the average about 8.5 hours for most of the dry season; this may mean increase in evaporation. The high evaporative power of between 16 and 20 millimeter of water in November also suggest a very high drying power. Extreme dryness is usually associate with values higher than 20 ml when relative humidity (RH) is 40% or below. The mean monthly temperature ranges from 25.7°C in August to 30.9°C in March.

c) Effective Temperature Factor

The rate of all developmental processes is directly regulated by temperature, being accelerated when temperatures are raised and retarded when they are lowered.

3.1.3 Socioeconomic Characteristics

3.1.3.1. Socio-economic Activities

Human activities in FCT-Abuja includes: primary - fishing and farming, quarrying, and river sand mining have become major production activities in FCT-Abuja secondary manufacturing, and various traditional industrial activities (weaving, carving, dyeing, smiting and so on) found in the informal sector of the economy; and tertiary — commerce, administration, banking and finance, information, transportation and local traditional marketing through the traditional rural periodic markets and the urban markets. There have been some modest improvement in the construction of new roads within FCT-Abuja by the present administration under Governor Umaru Tanko Almakura. Generally, inadequate transport system has been a major constraint on social and economic development in FCT-Abuja.

3.1.3.2 Demographic Characteristics

According to National Population Commission (NPC, 2006), it is observed that population growth in FCT-Abuja is influenced by declining mortality and stable high fertility level as well as influx of people because of its proximity to FCT Abuja. This increase had been further influenced by factors such as improved environmental sanitation, raising income level, peace and political order. Similarly, decline in mortality rate, increase in fertility had (still is) consistent in FCT-Abuja.

Early marriage has been important factors in high fertility in the FCT-Abuja. Apart from natural population increase, FCT-Abuja also experiences a relatively high level of immigration as a result of movement of the seat of Federal Government from Lagos to Abuja. Until recently migration in FCT-Abuja followed the common developing country pattern of young people leaving their rural villages to seek work in larger urban centers. Thus FCT-Abuja has an average population growth rate of about 3% which is higher than the national average of about 2.5 per cent (NPC, 2015). FCT-Abuja is home to Nasarawa State University FCT-Abuja, School of Health Technology, and a large number of primary and secondary schools among which is Government College FCT-Abuja.

3.2 RESEARCH TOOLS AND METHODS

3.2.1 Materials and Methods

3.2.1.1 Sampling

Samples of subgrade were collected in-situ from the construction works of Sabon Layi, and Ungwar Tiv roads in FCT-Abuja Local Government Area, Nasarawa State, at chainages 0 + 00, 1.5 + 00, 2.5 + 00, 3.5 + 00, 4.5 + 00 at depths ranging between 1.00m — 2.00m. Sample of base was collected from the borrow pit (Table 3.1).The soil samples collected were sieve in 2mm sieve to determine the Particle Size Distribution (PSD) i.e. determine the different size of grains of particle that make up the soil and the percentage of clay, silt and sand present in the soil.

Data were collected with the aid of the tools; digger, tape, soil color chart, sieve and trowel. Others are weighing pan, weighing scale, plastic bottle, measuring cylinder, mechanical shaker, hydrometer, thermometer, sodium hexameter, phosphate and distill water.

Table 3.1: Identification of soil layer/profile and colors (collected on August 21, 2015).

Sample pit	Soil layer profile	Depth of layer/profile	Soil colour as identified
A	Layer I	0.00m-.-0.7m	7.5YR(5/4) brown
	Layer II	0.7 - 1.2m	5 YR (7/3) Pink
	Layer III	1 .2m - 1 .5m	5YR (5/8) Yellowish Red
	Layer IV	1.5 - 3.0m	5YR (4/3) Reddish Brown

Similarly, the following observations were made

- Recent weather - Rainy season
- Vegetation cover - Grass and shrubs
- Erosion - Slight erosion
- Slope/Topography/Landform- Relatively flat
- Rock outcrop - Nil
- Parental material - "MARMARA"
- Drainage - Moderately drained
-Surface soil - Fine/Loamy
- Land use - Agriculture/Grazing
- Age of the soil - Oil soi!
- All the layers are relatively dry -

- Presence of insect at the third layer was noticed

- Elevation is 678m above sea level

- There are roots within the first to third layers

- 1 and 2nd layers are fine 3rd layer is rough

- 4th layer is coarse

3.2.2LABORATORY TESTS

3.2.2a Preliminary Tests on Sample:

Preliminary tests carried out on the soil sample were:

1. Classification test:-

• Particle size distribution

• Moisture Content

• Atterberg limit (LL & PL)

• Linear Shrinkage

• Specific gravity.

2. Soil Composition Test: Standard proctor method was used. This establishes the degree of compaction that can be achieved at different moisture content 3. Soil Permeability Test: Falling head method was used. This determines the drainage characteristics of sample under study.

4. Potential Volume Change: Free swell percent method used. This determines the swelling characteristics of the samples.

5. Consolidation test: The above tests were performed in accordance with B S 1377; 1990 Specifications (method of test for soils for evil engineering purposes).

3.2.2.1 Particle Size Analysis

Soil particles analysis/test was done to determine grain size distribution of the soil samples. Wet/dry sieving method was used. Grain size is determined by sieving a quantity of soil through a stack of sieves, of progressively smaller mesh opening from up to bottom of stack by pouring the soil sample into the Reflie box. The quantity of the soil retained on a given sieve in a stack is termed one of the grain sizes of the soil sample (all particles present in the soil containing different sizes of particles present in the soil). The tests were performed in accordance with B.S. 1377, 1990. The result of the grain size analysis is shown in Appendix A. in order to remove moisture content in the soil sample, it was kept in the oven at a temperature of 106°C for 24 hours with the aid of crucible, and soil sample was a weighed with the aid of weighing balance.

Wet sieving was also carried out on the sample in order to remove clay/silt, particles finer than sieve No.200. The sample was poured/soaked in a tray filled with water and was stirred, washed, sieved with sieve No.200 (75μm) under tap until water became clean. The retained particles in the sieve were collected into the crucible and oven dried for 24 hours to expel moisture content in preparatory for dry sieving. Dry sieving was accomplished bypassing/pouring the particles through assemblage of sieves of various sizes. These sieves were shaken for some time so that each sieve could retain particles not finer than the sieve and weight of particles retained in each determined, from where percentage retained and percentage passing were deduced.

3.2.3Atterberg Limit Test

The liquid limit and plastic limit test were performed in accordance with B.S 1377, 1990.

a. Liquid Limit

The water content above which the soil behaves as a viscous liquid or at which 25 blows of the liquid limit (EL) machine close a standard groove. cut in the soil pat for a distance of 12.7cm. Casagrande (1958) have modified the test as initially proposed by Atterberg so that it is fewer operators-subjective and more reproductive.

b. Plastic Limit

The water content above which the soil is no longer behaves as plastic materials. It is in range of water contents between WL and the Plastic Limit (PL)W that the soil behaves as a plastic material. This range is termed the plasticity index and is computed.

IP=WL-WP

c. Shrinkage Limit

The water content, defined at degree of saturation = 100 percent, below which no further soil volume change occurs with further drying. The test has a wide application in the identification and classification of soil. The results are given in Table 3.2 below.

Table 3.2:Atterberg Limit Test Results

Sample	LL%	PL%	P1%	SL%
A	31.73	19.14	12.60	5.9

3.2.4 Compaction Test

Compaction tests were carried out using the - standard proctor test method in order to determine Optimum Moisture Content (OMC) and Maximum Dry Density (MDD). The tests were performed in accordance with B S 1377 (1991). This involving the moisture content relationship, dry-density was carried out using soil sample air-dried for one day, and was determine by the following relation:

$$\% \text{ Moisture} = \frac{\text{Weight of Moisture}}{\text{Weight of dry Sample}} \times 100$$

$$\text{Dry Density} = \frac{\text{Wet density} \times 100}{\% \text{ Moisture} + 100}$$

The compactive efforts utilized throughout the test were the standard proctor. The compaction hammer of 25kg mass falling through a height of 30cm was used to prepare specimen 10. 1cm in diameter in a 1000cm3 mold in 3 layers with 25 blows per layer. The results are as shown in Table 3.3 below.

Table 3.3 Compaction Characteristics Results Showing Maximum Dry density (MDD)

And Optimum Moisture contents

Sample	OMC%	MDD (Mg/M)
A	9.8	1.95

OMC = Optimal Moisture Content, MDD = Maximum Dry Density

3.2.5 Permeability Test

The method adopted for the investigation is the falling head. The permeability test were performed in accordance with BS 1377, the first step in carrying out the permeability test was to saturate the specimen in the mould. The saturation process involves the placing of the permeameter in a small water container. A transparent plastic tube was connected between the opening the top of the cell cap. The permeability tests were performed in accordance with BS 1377 (1990), and the coefficient of were reported in Table 3.4 below.

Table 3.4 Results of Permeability Characteristic

Sample	Sample Permeability Sample Coefficient (K) (cmlsec)
A	2.62×10^{-4}

3.2.5.1 Limitations and other Factors Affecting coefficient of Permeability (K)

The Laboratory test for determination of K is very unreliable, with considerable attention to test procedure and equipment design necessary to obtain even the correct order of magnitude of the permeability coefficient. Some of the factors effecting the results are:

1. Soil in-situ if generally stratified, and it is hard to duplicate in situ conditions in the laboratory test.

2. The field soil structure is invariably lost in the laboratory because an undisturbed sample cannot be fully certain, since it would have to be transferred from the recovery device to the Permeameter.

3. The small size of laboratory samples leads to effects of boundary conditions such as smooth sides of the test chamber affecting flow and air bubbles either in the water or trapped in the test sample affecting the test results.

3.2.6 Volume Change Test

Free swell test method: (Holt et al, 1980) Known quantities of the samples were taken and carefully grinded with pestles and the sample was allowed to pass through 200mm sieve. The sieved sample was poured into a beaker contain water to make solution. The initial water level of the solution is noted and recorded. The solution was allowed to settle down for 24 hours. At the end of this period, final level of the sample is also observed and recorded. The percent swell was calculated from the difference in observed level. The result is as shown in Table 3.5.

Sample	Initial height of specimen (ML)	Final height of specimen (ML)	Percent swell (%)
A	45	32.4	6.6

(Final — Initial) Level x 100 = percent swell
Initial Level Oedometer

3.2.7 Consolidation Test

The rate and extent of the settlement arising from volume change in the soil are generally predicted from the result of test in an oedometer. The tests were performed in accordance to B S 1337 (1990). A section of soil is cut from an undisturbed sample of the soil, and is filled into a steel ring. Porous plate is placed at the top and button of the sample, which is therefore free to drain from both faces. The sample I loaded vertically, and a record is made of settlement against time. The result presented using 5OKN load. The result is shown in Table 3.6a below.

Table 3.6 (a) consolidation parameters (50 KN LOAD)

Sample	Time @ 90% consolidati on (Mim)	Initial Void Ration (e_o)	Final Void Ratio Change e_1	Void Ratio Change □e	Co-efficient of consolidation on Cv (mm^2/S)	Co-efficient of Compressible on Cv (mm^2/K)
A	82.8	0.786	0.762	0.024	0.139	0.000011

Source: Authors' field work, 2015

Table 3.6 (b) consolidation parameters (880 KN LOAD)

Sample	Time @ 90% consolidati on (Mim)	Initial Void Ration (e_o)	Final Void Ratio Change e_1	Void Ratio Change □e	Co-efficient of consolidation on Cv (mm^2/S)	Co-efficient of Compressibili ty on Cv (mm^2/K)
A	72.3.38	0.544	0.534	0.010	0.139	0.000011

[illegible]

Source: Author's field work, 2015

3.2.8 Specific Gravity Test

Specific gravity is defined as the ration of unit weight of materials, to the unit weight of water. The test generally involves the use of displacement of water to measure the total volume of the irregular shape making up the test sample. The specific gravity test is usually performed using a closed container to which a vacuum can be applied to remove any entrapped air, according to B.S 1337 (1990).

3.2.9 Moisture Contents

Moisture or water content of a soil is defined as the ratio of the weight of water in the pores of a soil, to the weight of soil solids.

$$Mc = \frac{\text{Weight of water}}{\text{Weight of soil}}$$

The tests were performed in accordance with B.S 1377 (1991).

3.2.10 California Bearing Ratio (CBR)

California Bearing Ratio (CBR) Machine was carried out in order to estimate bearing capacity of the soil used in sub-grade and base course. The wet soil was compacted and placed on the CBR machine. Two gauges; proofing ring, and plunger penetration were set at zero. The gauges started working simultaneously soon as the plunger penetration made a contact with the soil, and readings taken on the proofing ring gauge at every 25 division on the plunger penetration gauge. The first 10 readings were referred to as first pointer and the 10th reading being the correct reading was adopted and multiplied with a multiplication factor of 0.2 while the last 10 readings were referred to as second pointer, and so also, the 20th reading was adopted and multiplied with a multiplication factor of 0.11. The test was done on both top and bottom of the compacted wet soil. The higher of the two values was chosen as actual CBR. The average of the top and bottom was however the final actual CBR. The same was done for the remaining three compacted wet samples.

3.2.10 PAVEMENT CONDITION SURVEY

3.2.10.1 Field Technique

A visual condition survey was carried out on all the roads in the study area to evaluate their physical condition. The Federal Ministry of Works and [lousing pavement evaluation methodology currently in operation was adopted. It involves a typical recording on the pavement evaluation chart during the field study. In assessing the pavement conditions of the roads, the following distresses/defects were considered; (a) cracking, (b) surface deformation, (c) surface defects, (d) failed areas (e) shoulder conditions aid (f) drainage conditions. The description and major causes of these distresses/defects are given in Table 4.3. Each distress/defects was quantitatively rated, on the basis of its extent (density) and severity whereas drainage conditions were qualitatively assessed (see Table 4.4). A typical recording on the pavement evaluation chart used during the field study is shown in Table 4.5

The deflexion survey took place during the month of August and September, 2015. The procedure followed was recommended in Federal Ministry of Works pavement condition standard. Measurements were taken in each wheel track in both lanes of the carriage way, at l00m spacing. Extra measurements were made to define locations where high deflexions were recorded, and at other locations where deterioration of the pavement was severe.

3.2.10.2 Surface Condition Rating

At each measurement point, the surface condition was assessed for degree of deformation and amount of crack. Where the deformation of cracking is of value 5, it can be assumed that the pavement has failed. Generally, the worst surface rating was measured in the near side wheel track, Due to the amount of patching that has been done, these condition ratings are taking only as guide to the nature of the surface. On the other roads where a full depth bitumen running has been provided, the ratings give a good impression of the roads condition. However, where the road has only been surface dressed, and has been extensively patched, as in this case, it is difficult to define the reference for measurement.

3.2.10.2 Traffic Census

The traffic count provided essential information on the amount of usage of roads and junctions as factor that determined the overall performance of-roads in FCT-Abuja. The aim of the studies is to determine the volume of traffic flow at the designated areas as detailed in the following sections below:

3.2.10.3Apparatus

Manual traffic count and a stop watch

3.2.10.4 Procedure

a) Count sites were suitably located on either side of the carriage way (data on the traffic census and flow along the corridors (carriageway) at the Antau Bridge - FCT-Abuja Round About-Emir's Palace to Pyanku Road Corridor).
b) Traffic flows were then recorded in the categories of "light", "medium", and "heavy" vehicles respectively during the enumeration exercise.
c) In conducting (b) above care of the two-way traffic movement system in each case was observed.
d) Two hours of traffic enumeration (1000am to 100pm) was carried out on the assigned date.
e) The maximum traffic counts were then summed-up from each group that carried out the studies. – V
f) Produce traffic signal design using Webster methods from the results of(a) above.

3.2.11 Interview

An interview is defined as a process in which a researcher and participant engaged in a conversation focused on questions related to the research study (DeMarrais, 2004). Face- to-face interview which is the most common form of interview was used to collect data from key stakeholders as far as road maintenance is concerned from the following road maintenance agencies:

a. Federal Roads Maintenance Agency, and

b. State Ministry of Works, Transport and Housing

The interview's main purpose was to obtain a special kind of information. It was open ended conversation format.

A list of questions (interview guide) was asked. Gentle probing was employed as suggested by Merriam (2009) and that the fewer the questions the better. The type of interview guide used in this study was a semi-structured type which according to Merriam includes a mix of more and less structured interview questions with all questions used flexibly.

3.2.12 Secondary Sourced Data

Data was obtained from library books and diagrams from various sources, articles, maps arid internet sources to review literature. The study included review of different maintenance requirement to give a view of the maintenance requirement status.

CHAPTER FOUR

RESULTS AND DISCUSSIONS

4.1.1 GRAIN SIZE ANALYSIS

The shape of the grain size curve somewhat indicative of the grain distribution. Grading curve of sample obtained from the borrow pit as shown in Appendix A. It is observed from the curve that the soil sample A is well graded, with coarse grained. The small particles of the well graded soil will pack the spaces giving a dense mass of interlocking particles with low compressibility, this soil are ideal fill materials, they are closely compact and the proportions of clay that exist will acts as a binder, giving tough dense materials, see Table 4.1 below. Unified Soil Classification System (USCS) classified the soil as CL. American Association of State Highway and Transport Officials (AASHTO) classification system classified the soil as A-6, that is, the index and geotechnical properties of the soil use for the construction of roads within FCT-Abuja Metropolis. -

4.1.2 Atterberg Limit

Table (3.2) shows the plasticity results and Appendix B shows the liquid limit curves. The sample has liquid limit range of 3 1.73%. The, plasticity (11.6%) and shrinkage (19.4%) result indicates high plasticity, see Table 4.1. in general, the larger the plasticity index, the greater will be the engineering problem associated with using the soil as an engineering material, such as road sub-grades.

The shrinkage limit of value of the sample is 5.9%, then when the in-situ water enter exceeds this value, the soil will begin to expand. The smaller the shrinkage ii it the more susceptible a soil to change in volume.

4.1.3 Effect of Water Contents on Compaction of Laterite

The objective of the compaction is the improvement of the engineering properties of the soil mass. The main advantages, which occur through compaction, are:

a. Reduction in settlement due to reduced void ratio;

b. Increases in. soil strength

c Reduction in shrinkage

The results of the compaction test performed for the sample are shown in the Table (3.3). and the corresponding curve is shown in Appendix C. The Maximum Dry Density (MDD) for the samples is 2 01mg/rn2, while its corresponding Optimum Moisture Content (OMC) is 8.2%. At low water contents, the shearing resistance to relative movement of the soil particle is large. As .the water content increases, it become progressively easier to 'disturb the soil structure, and the dry density achieved with a working comparative effort increase. The highest dry densities are produced in well graded coarse-grained soil as empirically shown with smooth rounded particles.

4.1.4 Suitability of 'Compacted Laterite As A Fill Materials

The tested soil is suitable as a refill in engineering, m an excavation works or void, and also as a sub-base for a road and also to providing made-up ground to support a structure, for greater stability, less settlement, deformation, less water absorption and less risk of heave.

4.1.5 Effects of Water and Its Permeation on Laterite

Permeability has a wide range of application from evaluating the amount of seepage to rate of settlement studies where soil volume change, occurs as water is expelled from the soil void. The results of permeability test at optimum moisture contents are shown in Table 3.4.

The coefficient of permeability is 1.04 x 10^{-4} cm/s. This permeability value is adequate for granular materials, being typical of values- measured in clays—generally. Lambe and Whiteman (1969) found that the highest permeability will occur in samples which are in most flocculated state and samples in the most dispersed state will have the minimum permeability. The distribution of the clay through the soil, forming a matrix, which surrounds the coarser fraction, means the permeability is controlled by the clay fraction. Flow of water through laterite soil depends on the void ratio and fabric orientation. The soil therefore, will be more permeable.

4.1.6 Volume Change Characteristics

It is known that swelling of soil is affected by the orientation of clay particles and its flocculated structure. From the free swell test, it is observed that the sample has significant changes in volume potentials. The soil have 4.5% swell which is about an average.

In considering the relationship between Atterberg limit results in Table 3.2 and free swell test results in Table 3.5. It is observed that the soil exhibit moderate swelling potential with plasticity index of 19.42%. According to Malomo (1977), the greater the plasticity index the more the engineering problems associated with using the soil as base, sub-grade material in foundation material, because such soil will have greater shrinkage-swell potential.

4.1.7 Effects of Loading Conditions on the Volume Change of Laterite

The consolidation analysis performed resulted introduction in void ratio (or voids volume) as the pore fluid is displaced. The method of analysis predicts the magnitude of the time settlement and the length of time for the major portion of the soil to reach 90 percent consolidation. The result shows that, the initial load of 501CN applied on the soil specimens takes the highest time of 82.8mm to attain 90% consolidation and its co-efficient of consolidation was 0.139rnm2lsec, which is about the average. See Table 3.6a-b for details. As the load increases over time to 800KN-settlement and void volume reduction continues through the entire soil and attain there 90 consolidate at a period of 72.3 minutes respectively. Its corresponding coefficient of consolidation was 0.139mm^2/sec. The volume change associated to the settlement consolidation analysis were deduce from plotting the void ration change and the pressure load change. It was observed that the soil sample has the coefficient of volume change of 1 .04x 10 mm^2/KN.

Table 4.1: Geotechnical Properties of Laterite Soils used for Construction of Roads in FCT-Abuja

Properties	Values
Liquid Limit (LL)%	31.02
Plastic Limit (PL)%	11.6
Plastic Index (PI)%	19.42
Shrinkage Limit (SL)%	6.0
Maximum Dry density MDD (mg)mg/m	2.01
Optimum Moisture Content (OMC) %	8.2
Specific Gravity (sq)	2.6
Natural Moisture Content %	29.3
Permeability (CMJS)	1 .04x104
Volume Change % at (OMC)	4.5
USCS	CL
AASHTO	A-6
% Passing 200	36.9

4.1.8 California Bearing Ratio (CBR)

The results of California bearing ratio test revealed that subgrade samples at chainages 0+000, 1.5+000, 2.5+000, 3.5+000, 4.5+000have CBR values of 99%, 100%, 79%, 132%, 69% while base sample has a CBR value of 69%.According to clause 6201 of Federal Ministry of Works and Housing (F.M.W & H) Specification Requirement, the minimum strength of base course material shall not be less than 80% C.B.R (unsoaked) while minimum strength for subgrade/fill shall not be less than 10% after at least 48 hours soaking. consequently, base sample is not fit to be used since it exhibits CBR of 69% (unsoaked) which is less than the stipulated 80%. The subgrade samples are good because they exhibit CI3R (unsoaked) values (99%, 100%, 79%, 132%, and 69%) that are even higher than what is stipulated in the specification. Therefore, subgrade samples are better than the base sample used for the construction of the road which is evident in their CBR values.

4.2.1 PAVEMENT CONDITIONS

The summary of the pavement conditions is given in Table 4.4 their conditions were generally moderate to poor, although there are some few good ones. All the roads showed signs of fatigue cracking and patched areas are numerous, indicating previous widespread pavement failures. The worst road in terms of surface deformations and pavement failures are; Sabon Layi-Stadium junction and Alh. Babai-Modibbo's road. The whole pavement has completely broken down just barely one year after the completion. At present, the whole section (1.2km) and (1.1Km) are been rehabilitated and reco nstructed. Similarly, the roads has no organized shoulders and the low level of the finished roadway has resulted in poor drainage and frequent flooding/water logging.

Where organized shoulders exist in some of the roads, they are poorly compacted and therefore cannot withstand the weight of vehicles during the wet season. The shoulder drop-off on most roads is severe, especially where they are unpaved and have been seriously affected by erosion. The drainage conditions of the roads were also poor. It is a common occurrence to find portions of the pavement and shoulder water logged for days. It should also be mentioned that excessive traffic loading especially during market days appears to play a significant role in the pavement failures of some of the roads. The outgoing lanes of these roads have more extensive severe pavement failures because of the heavily loaded vehicles evacuating importing and exporting. goods. The relatively good pavement conditions of some of the roads in Keffi are associated with factors such as low/light traffic, low water table and stable shoulder.

Table 4.3: Description and Causes of the Pavement Distresses/Defects/Conditions Studied

General Condition	Specific Type of Distress	Description	Causes
Cracking	Fatigue or alligator Longitudinal Transverse	Longitudinal cracking in the wheel path. Longitudinal cracking between wheel paths, along centerline, and between outside wheel path and shoulder. Cracking at right angles to the pavement axis.	Traffic loading , Poor construction, and Cyclic environmental condition
Surface deformation	Wheel path rutting	Pavement failures along wheel path	Mix stability, excessive traffic loading, poor base and sub-base compaction.
Surfacing defect	Raveling/stripping, bleeding	Many aggregates (large and small) come off the pavement, excess of asphalt on pavement surface.	Lack of asphaltic bitumen and compaction between aggregates too much asphalt or insufficient aggregates.
Pavement failure	Patched area, failed area	Patched area of pavement, potholes, base, sub-base/sub-grade failures, and pavement disintegration.	Poor compacted or incompetent base, sub- base or sub-grade.
Shoulder conditions	Unpaved with vegetation, un-paved with soil, unpaved with gravel, paved shoulder drop-off.	Height difference between pavement and shoulder	Poor construction, lack of shoulder, and erosion.

Source: Author's field work, 2015

Table 4.4:Rating of Pavement Condition

Road Conditions	None	Slight	Moderate	Severe
CRACKING (Severity) (% area/length coverage)		Numerous wavy cracks of 2-3m long, criss-crossing in places, less than 5mm wide. 1-10%	Numerous interconnected cracks with paragon and octagon shapes, 5-12mm wide, 11-50%	Severe cracking with small pieces loosened and some missing; width of crack is greater than 12.5mm. Greater than 50%
SURFACE DEFORMATION (Severity) (% area/length wheel path coverage cutting)		Less than 2cm 1-10%	2-5cm deep 11-50%	Depth greater than 5cm Greater than 50%
SURFACING DEFECTS (% length coverage)		Some noticeable aggregate loss, faint colouring on wheel - path. 1 -10%	Opened textural surface with streaks of lost aggregates; free asphalt material in wheel path. 11 — 50%	Disintegration of surface and formation of potholes wet look on pavement, noise like water beneath tyre. Greater than 50%
FAILED AREAS (size) % area coverage		$1m^2$ 1- 5%	$1 - 2m^2$ 10 - 20%	$2m^2$ 20%
SHOULDER (drop-oft) (% length coverage)		5cm 1- 10%	5 - 12.5cm 11 - 50%	12.5cm 50%
Rating Condition	**Excellent (4 Points)**	**Good (3 Points)**	**Moderate (2 Points)**	**Poor/very poor (1. Point)**
DRAINAGE	Well drainage side slopes, high embankment and good run-off ditches	Adequate side slopes and ditches, water most probably run-off reasonably well to surrounding streams and rivers	Moderate drainage with some side slopes and ditches, of moderate sizes. Run-off is less adequate, some water logging of roadway; pavement level is close to or same as the s4rrounding terrain,	Little or no side slopes and ditches, sections may be slightly lower than the surrounding terrain, roadway probably water logged frequently after heavy rain; slow run-off, constant water into base, sub-base and subgrade

Source: Author's field work, 2015

Table 4.5: Pavement Condition Survey

Table 4.5:Pavement Condition Survey

Road	Year of Construction	Distance (Km)	Cracking	Surface Deformation	Surfacing Defects	Failed Areas	Shoulder Conditioas	Drainage Condition
From FCT-Abuja main market Stadium Junction (Sabonl.ayi Road)	2015	3.52	Slight	Slight	Slight	Slight	4	4
From Alh. Babai's house to Modibbo's house	2000	1.85	Slight	Slight	Slight	Slight	4	4
From Emir's palace Masallacin Idi	.1996	2.1	Slight	Slight		Moderate	3	
From Emit's palace to Kofar Hausa	1996	1.8	Severe	Severe		Severe	4..	
From main Market Agyadi's house down Amosun Hospital	2000	1.2	Severe	Severe		Sever	4:	

Source: Author's field work, 2015

4.2.2 Lane Analysis of Traffic Count along Antau Bridge — FCT-Abuja Round About — Emir's Palace to Pyanku Road Corridor

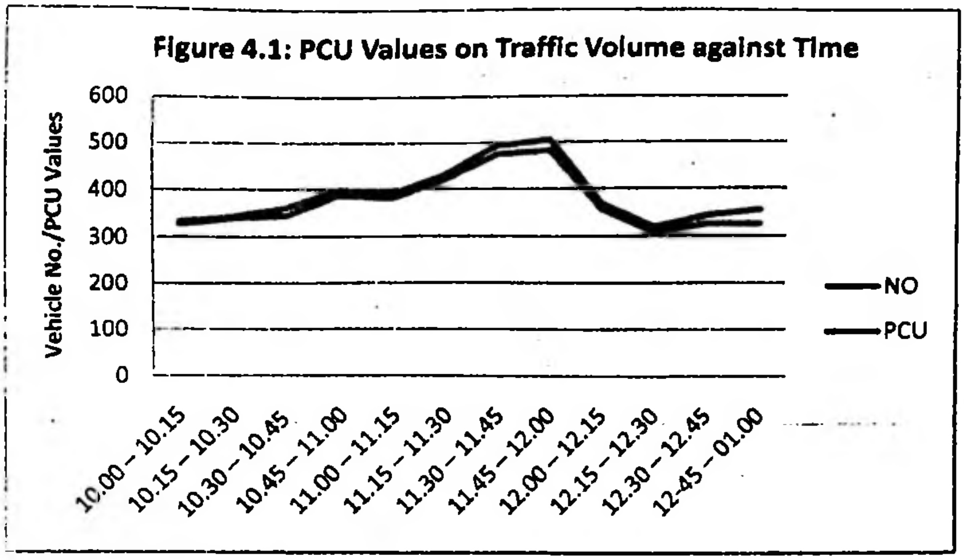
The volume of traffic and their corresponding P.C.U. values were obtained as shown on Table 4.5 below, the subsequent pages. It is observed that traffic composition of the areas under study was characterized by trading activities; thus traffic concentration are heavy and light commercial vehicles due to high conduct of commercial activities consequently, the environmental impact on the traffic flow situation.

Analysis of the data revealed that the "peak period" of the traffic flow was characterized by massive movements of school pupils and students returning to their respective homes after the close of schools during the same period (11.30am to 12.30pm). Furthermore, commercial motorcyclists, popularly known as "Van Achaba" was observed and significantly contributed to the traffic logjam during the peak period.

Table 4.5:Lane Analysis of Traffic Count along Antau Bridge — FCT-Abuja Round About

— Emir's Palace to Pyanku Road Corridor

	LV		MV		HV		TOTAL		% VOLUME	
TIME	NO	PCU	NO	PCU	NO	PCU	NO	PCU	NO	PCU
10.00 -10.15	323	323	4	8	1	3	328	334	7.33	7.2
10.15- 10.30	337	337	3	6	-	-	340	343	7.6	7.39
10.30-10.45	335	335	4	8	6	18	345	361	7.71	7.78
11.00-11.00	380	380	3	6	4	12	387	398	8.65	8.56
1.00 - 11.15	374	374	5	10	2	6	381	390	8.51	8.4
1.15 -11.30	413	413	6	12	2	6	421	431	9.41	9.29
1.30 - 11.45	460	460	9	18	5	15	474	493	'10.59	10.62
1.45 - 12.00	467	467	7	14	8	24	482	505	10.77	10.88
12.00-12.15	349	349	5	10	3	9	357	368	7.98	7.93
12.15-12.30	300	300	5	10	3	9	308	319	'6.88	6.87
12.30-12.45	315	315	4	8	7	21	326	344	7.28	7.41
12.45 - 01.00	320.	320	3	6	3	·9	326	355	7.28	7.65
							4475	4641		



4.2.3 Assessment of FCT-Abuja Roundabout Junction Flow

Saturation flow (s) pcu/hr = 525W, for W> 5.0m

Y — Value = Flow (pcu/hr)

Sat. Flow (pcu/hr)

Summaticn of Design Y — Value (Y) = $Y_N + Y_S + Y_E + Y_W$

Y 0.106 + 0.058 + 0.159-+0.125 = 0.448

Lost time: Total lost time (L), Sec PL + P (I - a)

P = No. of signal phases 4

L = Starting delay assumed to 2 sec

I = Inter-green period, assumed to be 7 sec

a = Amber period; assumed to be 3 sec.

Table 4.6

Approach width (m)	Saturation flow	Y-values	Design Y values
7.3	3,833	0.106	0.106
		0.053	
		0.018	
3.65	3,150	0.058	0.058
		0.047	
		0.012	
7.3	3,833	0.159	0.159
		0.04	
		0.092	
7.3	3,833	0.125	0.125
		0.107	
		0.012	

∴ Total Lost Time (L) = 4 x 2 + 4 (7.3)

= 8 + 16 = 24 seconds

Optimum Cycle Time (Co)

$$Co = \frac{1.5L+5}{1-Y} = \frac{1.5(24)+5}{1-0.448} = 74.285 sec.$$

Total effective green time (G) = Co – L

G = 74.28 – 24 = 50.28 second

Effective green time $\delta_{1} = \frac{y_n}{Y} \times G$

$$\delta_1^0 = \frac{y_1}{Y} \times G = \frac{0.159}{0.448} \times 50.28 = 17.84 seconds$$

$$\delta_2^0 = \frac{y_2}{Y} \times G = \frac{0.125}{0.448} \times 50.28 = 14.03\ seconds$$

$$\delta_3^0 = \frac{y_3}{Y} \times G = \frac{0.106}{0.448} \times 50.28 = 11.90\ seconds$$

$\delta_4^0 = \frac{y_4}{Y} \times G = \frac{0.058}{0.448} \times 50.28 = 6.51\ seconds$

Degree of Saturation $(x) = \frac{q_c}{\delta_s^0}$

$Phase\ I\ Flow = \frac{1100}{3600} = 0.306 veh/sec$

$Saturation\ Flow = \frac{3833}{3600} = 1.065\ veh/sec$

$$\delta_1^0 = 17.84\ sec, C = 50.28 sec$$

$$\therefore x_1 = \frac{0.306 \times 50.28}{17.84 \times 1.065} = 0.810$$

$Phase\ II\ Flow = \frac{937}{3600} = 0.261\ veh/sec$

$Saturation\ Flow = \frac{3833}{3600} = 1.065\ vehs/sec$

$$\delta_2^0 = 14.03\ sec, C = 50.28 sec$$

$$\therefore x_2 = \frac{0.261 \times 50.28}{14.03 \times 1.065} = 0.878$$

$Phase\ III\ Flow = \frac{678}{3600} = 0.188\ veh/sec$

$Saturation\ Flow = \frac{3833}{3600} = 1.065\ vehs/sec$

$$\delta_3^0 = 11.90\ sec, C = 50.28 sec$$

$$\therefore x_3 = \frac{0.188 \times 50.28}{11.90 \times 1.065} = 0,746$$

$$Phase\ IV\ Flow = \frac{368}{3600} = 0.102\ veh/sec$$

$$Saturation\ Flow = \frac{3150}{3600} = 0.875\ vehs/sec$$

$$\delta_4^0 = 6.51\ sec, C = 50.28sec$$

$$\therefore x_4 = \frac{0.102 \times 50.28}{6.51 \times 1.065} = 0.900$$

4.2.3 Data Analysis from Interview with Maintenance Agencies through the use of Interview Guide

Maintenance Agency: Ministry of Works, Transport and Housing (MWT&H), FCT-Abuja Zonal Office

Respondent: Director Engineering Services

Date of Interview: 29th March 2017

The findings showed that the State Ministry for Works, Housing and Transport claimed to conduct yearly assessment of road maintenance at zonal scale. This task normally involves five main responsibilities:

1. Planning the annual programme of maintenance work for his area, assessing the resources needed and preparing an appropriate budget estimate

2. Ensuring that funds are fairly distributed to the various parts of the road network, and undertake the full programme.

3. Authorizing and scheduling work

4. Making sure that his staff knows how to carry out the work methodically and efficiently.

5. Monitor and ensure compliance to all the specification and effectiveness of maintenance programmes. The District Engineer also claimed that he is actively involved in site visit to assess and identify trouble spots and places that require immediate attention for maintenance.

Consequently, this knowledge of road conditions made him to decide on the required operations needed, On lengths of road where maintenance is, straightforward and easy to specify in advance, it should be possible to leave day-today work in the hands of suitably trained foremen or contractor's staff. Road situation requiring highly technical treatment and professional judgment, the maintenance team in the Ministry normally invite contractors in determining what needs to be done and supervising the work, the key point is that he should not let his time be taken up by simple operations which less qualified staff are able to manage. In addition, knowledge and competence to fulfill the duties they are given, the Maintenance Engineer has to make sure that supervisor, foremen and other personnel receive the necessary training (and include practical on the job experience as well as more formal courses), and, that there are enough trained staff to carry out his instructions (so that competent staffs are available to take over when more experienced personnel arc promoted or transferred to other duties). This means that training is an important part of his responsibilities.

MWT&H further claimed that it focused more on maintenance activities in terms of their frequency as follows:

a. Routinely maintained and repair pot holes; and ruts, dragging, grading, grass cutting: drain clearing; re-cutting ditches; culvert maintenance; road signs maintenance
b. Recurrent maintenance, required at intervals during the year with a frequency that depends on the volume of traffic using the road, for example, re-gravelling.
c. Periodic maintenance, required only at intervals of several years, for example, resealing (surface dressing, slurry sealing, fog spray,) re-graveling shoulders; road surface marking
d. Urgent maintenance, needed to deal with emergence and problems calling its immediate action when a road is blocked, for example, removal of debris and other obstacles; placement of warning signs and diversion works.

Maintenance Agency: Works Department, FCT-Abuja Local Government Area
Respondent: Director Works and Housing (DW&H)
Date of Interview: 15thApril 2017

The findings showed that although their responsibilities as stated in their mandate requires that they carryout maintenance of roads at regular and systematic fashion ensuring compliance to specification, Works and Housing Department (W & HD) does not and has never carried out any form of maintenance on 'any road in FCT-Abuja. The finding further revealed that W & H1) had no funds to carry our maintenance of roads within FCT-Abuja LGA because of the dwindling governmental works expenditure. There is an urgent need of W & HD to step up to its duties of frequent monitoring and maintenance of roads so that: good grading of traffic service; removal of loose material; and required specification is ensured. Neglect of these responsibly is accounting for those problems enunciated at the section 1.2 of this study.

Other findings revealed that W & HD lacked technical manpower (skill labour). Virtually almost all the staff in the Department, none has a Diploma in Civil Engineering, this constitute the most significant important problem iii the maintenance of roads in FCT-Abuja. Consequently, determination of work schedule for construction — type and size of road materials; the amount of traffic usage; the nature of local topography; weather and climatic regime; and practical test will be done not to specification. This problem is aggravated by lack of basic maintenance tools and equipment, such as graders; stone rollers, lorry driver among others.

In conclusion, the findings from this study is in line with answers to the research questions revealed that there is high percentage of lack of resources in terms of personnel and equipment which in turn determined their lack of inability to carry out maintenance work of roads within FCT-Abuja metropolis. The enunciated problems in section 1.2 of this study depicts a low level of road maintenance and that the dwindling government expenditure to works is responsible.

CHAPTER FIVE

CONCLUSIONS AND RECOMMENDATION

5.1 Summary, Conclusions and Recommendation

5.1.1 Conclusion

The visual condition survey carried out on some major roads in FCT-Abuja shows that the performance of pavements is generally poor. This is manifested in such distresses and defects as surface deformations, complete failures, fatigue cracks and pot holes. Poor drainage and lack of organized shoulders as observed have contributed to the poor condition of these roads.

This study evaluates the engineering properties Of FCT-Abuja laterite as well as performance conditions with focus on permeability and potential volume change characteristics. Within the scope of this study, coupled with the available laboratory test and results, the following can be deducing about the laterite soil samples under study.

When this soil are well compacted, especially at water contents less than its optimum value, the soil will have less swelling potential less influence by rain water. This attributes will makes the soil perform very well under the influence of heavy rainfall and traffic.

The use of soil for road and building materials should however be used with caution. Though the volume change value associated this soil are moderate, but the soil exhibit slight potentials for volume change. The soil will not withstand the influence of traffic during heavy rainfall. Sections of the roads constructed with this soil will always have problem during rainy season. This result has confirmed the statement of the problem identified.

5.1.2 Recommendations

The review of this project has allowed the following recommendations:

1. The reactions encountered with this soil type enabled us to conduct preliminary test on soil samples.

2. We further recommend that appropriate method of improving the engineering properties of the non-suitable investigated laterite.

3. Further research into chemical composition and erosive potential of the investigated borrow pit laterite is highly recommended. These will further reveal the depth analysis not cover by scope of this study.

4. The contractors working on some of this road need to improve their working performance, and maintain prescribed standard for roads construction in FCT-Abuja.

5. To enhance the performance of the roads in FCT-Abuja, pavements must be built on suitable embankments with well-defined paved shoulders and gutters.

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SIEVE ANALYSIS OF SOIL (WET/DRY SIEVING)

Job:.. Operator: ....

Site: ....Panda Pit hole................ Date:..........................10/8/2015..........................

Sample No: ....A.................... Description: ....Bark - brown..........................

Total Weight of dry sample:.........................300.........................gr ....

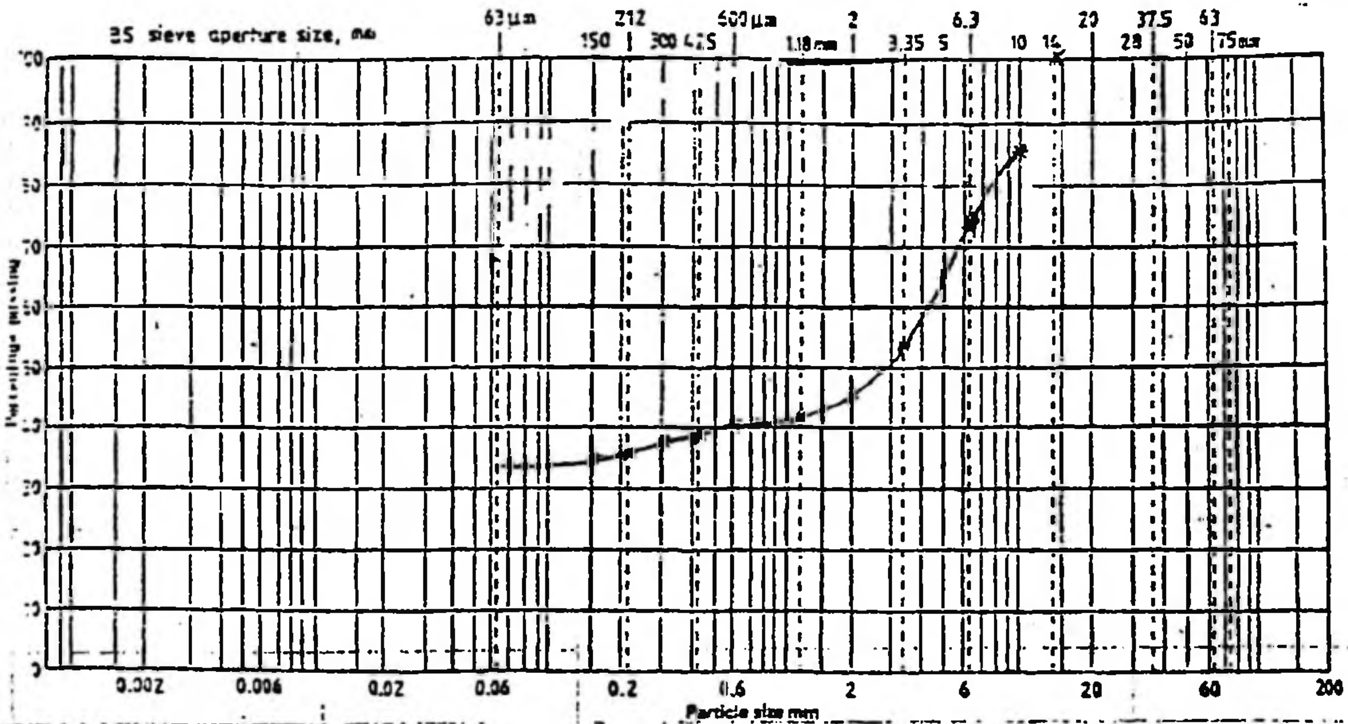
B. S. Sieve size	Weight retained	Weight Passing	Percent retained	Total Passing	Remarks	Max Sieve Load
	gr.	gr	%	%		gr.
3 in.						
2 ½ in.						
2 in.						4500
1 ½ in.						5300
1 in.						2500
¾ in.						2000
½ in. 14	-	-	-	100		1500
⅜ in. 10	38.21		12.74	87.26		1000
¼ in. 6.3	31.80		10.16	76.66		750
3/16 in. 5	33.16		11.05	65.61		500
Passing 3/16 in.	9.20		3.04	62.54		
Riffiled sample passing 3/16 in.						
⅛ in. 3.35	23.30		7.77	54.77		300
No. 7 2	24.4		8.13	46.64		200
No. 14 1.18	13.26		4.42	42.22		100
No. 25 600	1.46		0.49	41.73		75
No 36 425	6.70		2.23	39.50		75
No 52 300	1.90		0.63	38.87		50
No. 72 212	1.60		0.53	38.34		50
No. 100 150	2.70		0.90	37.44		40
No. 200 75	0.90		0.30	37.14		25
Passing 200 75	0.71		0.24	36.90		
Total						

BS 1377 : Part 2 : 1990

Particle size distribution chart

Form 2.N

Location	Soil description	Job ref.	Sample no.
Panda	Dark brown very gravelly sand	Borehole/Pit no.	E
			Depth 2 m
Test method	BS 1377 : Part 2 : 1990 : 9.2/9.3/9.4/9.6/9.7 *		Date 10/3/02



CLAY	SILT			SAND			GRAVEL			COBBLES	BOULDERS
	Fine	Medium	Coarse *	Fine	Medium	Coarse	Fine	Medium	Coarse		

* Coarse as appropriate

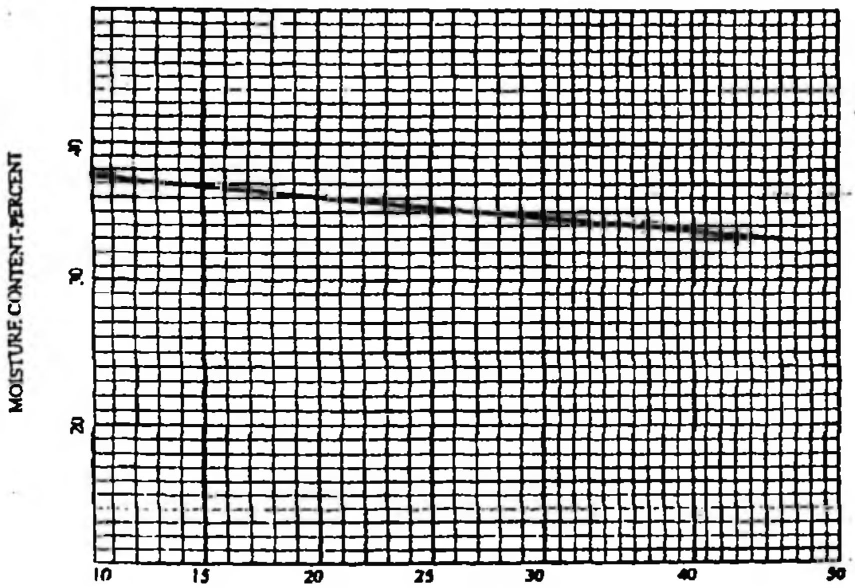
Operator	Checked	Approved

FIG 8.0

LIQUID AND PLASTIC LIMITS, LINEAR SHRINKAGE

Sample No. E .... Liquid Limit 31.02 .... Plastic Limit 11.6
Date .... Plasticity Index 19.4% .... Linear Shrinkage 8.0
Job .... Description of Soil ....
Site ....
L.L. Machine No .... Operator ....
Proportion of sample retained on No 36 B. S. Sieve. = ... Percent

Type of test	LL		LL		LL		LL		LL	
No of blows/shrinkage %	43		36		24		18		11	
Container No	89		28		X69		24		74	
Wt. of wet soil and container gr	24.76		24.76		26.60		27.84		29.26	
Wt. of dried soil and container gr	21.40		21.38		24.46		24.7		24.24	
Wt. of container gr	11.35		11.45		14.2		14.46		11.0	
Wt. of dry soil (Wd) gr	10.05		9.90		10.26		10.25		13.24	
Wt. of moisture (Ww) gr	3.35		3.41		2.15		2.94		5.01	
Moisture content 100(Ww/Wd) %	33.3		34.4		20.9		28.7		37.8	
Type of test		PL		PL		PL		PL		
No of blows/shrinkage %		—		—		—		—		
Container No		16		10.5		120		87		
Wt. of wet soil and container gr		10.6		11.7		11.8		11.64		
Wt. of dried soil and container gr		12.26		10.85		11.41		11.26		
Wt. of container gr		8.38		8.24		8.25		8.4		
Wt. of dry soil (Wd) gr		3.88		2.31		3.16		2.86		
Wt. of moisture (Ww) gr		0.34		1.16		0.39		0.39		
Moisture content 100(Ww/Wd) %		8.8				12.34		13.7		



NUMBER OF BLOWS

Fig 16

APPENDIX B

COMPACTION

Sample Calculation using D/Tofa (A) sample.

Bulk density r = $\frac{w2 - w1\ (mg/m^3)}{1000}$

Wt mould and wet soil (w2) gr = 6270

Wt of mould (w1) gr = 4540

Wt of wet soil (w2- wi) gr = 1.73

Wt of wet soil + container = 53.30g

Wt of dry soil + container = 48.80g

Wt of container 14.3g

Wt of dry soil (wd) gr = 34.5 gr

Wt of Moisture (wm) 4.5

Moisture content 100 (wm/wd) 13.4

DETERMINATION OF THE MOISTUREIDENSITY RELATION OF SOILUSING STANDARD PROCTOR COMPACTION

Location........Panda.......... Jample No.......E.......................Operator.............................
Sub-Grande/Sub-Base/Base...Date....................
Amount retained on 3/4" b.S. Sieve..gr. Total weight of sample.....3 000...........gr.
Mould type..

Wt. of mould and wet soil ....(W2) ......gr	6400	6770	6740	6690			
Wt. of mould ....(W1)..................gr	4540	4540	4540	4540			
Wt. of wet soil ....(W2-W1) ....gr	1860	2230	2200	2150			
Bulk density Y= W2-W1 / X ....lb/cu ft	1.86	2.23	2.20	2.15			

For B. S. Mould X =1000m' For C.B.R Mould X =37.11

MOISTURE CONTENT DETERMINATION

Container No...	67	62	41	165	81	68	25	69				
Wt. of wet soil and container .....gr	50.2	57.8	62.6	65.9	79.0	78.6	50.8	85.2				
Wt. of dry soil and container .....gr	48.5	57.4	58.7	60.7	70.8	69.3	71.45	73.60				
Wt. of container .....gr	12.65	11.37	14.76	10.5	19.0	11.6	14.55	11.4				
Wt. of dry soil (Wd) .....gr	36.15	46.03	46.9	50.3	70.6	57.7	57.0	62.7				
Wt. of moisture (Wm) .....gr	1.40	1.85	3.9	4.6	5.8	9.27	4.35	10.15				
Moisture content (100 Wm/Wd) .....%	1.71	4.06	7.66	8.12	5.82	13.64	14.07	16.74				
Average moisture content (m) .....%	3.29 / 1.89		7.89 / 2.87		13.43 / 1.93		15.70 / 1.67					
Dry density (yd=100y/100+m) .....lb/cuft												
C.B.R. (Mean of the arat bottom) .....%												

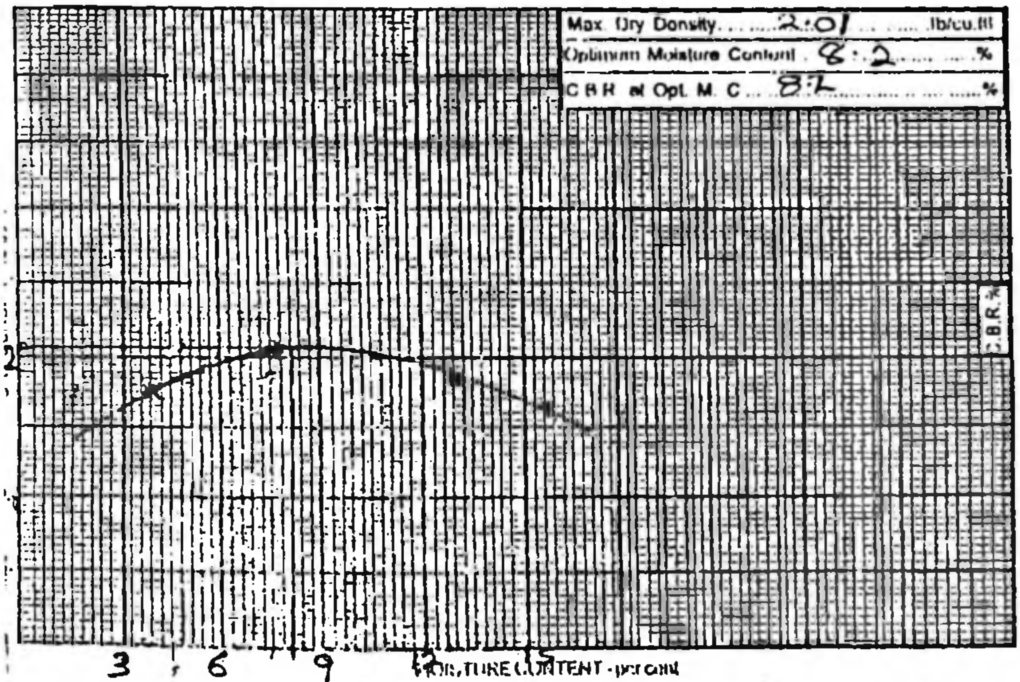


FIG 24.0

APPENDIX C

PERMEABILITY SAMPLE CALCULATION

Using falling head method

The Datum Level-146cm

Co-efficient of permeability $^{(k)}$ is calculated thus

$$K = \frac{ax\ 1 \times 2.303}{A \times 60 \times 1 \times 100} \quad \frac{\log 10h1}{h2}$$

Parameters

K — Co-efficient of permeability (failing —head

A Cross-sectional Area of the soil sample

a = Cross sectional area of the standing pipe

L = Length of specimen

t= Time in seconds

hi /h2 = heights of stand pipe permanent level.

2.3 03 = constant

Using sample —E as illustration

A = 68.7cm

A = 60.4mm2

T=6Omm

Hi = 76.8cm

h2 = 52.6cm

Constant =2.303

Using above formula:

$$K = \frac{60.4mm^2 \times 11.20cm \times 2.3\ 03\ \log_{10} 76.9}{68.7cm2 \times 60 \times 60 \times 100 \quad 52.6} \qquad = 10.04 \times 10^{-4}$$

APPENDIX D

CONSOLIDATION SAMPLE CALCULATION

Using soil sample A (Panda)

Thickness of the sample (H) 19.00mm (DRAINAGE PATH)

Mean thickness D 18.505mm

One-half height / thickness d/2 = 9.252

Moisture contents (after the test) = 17.5%

Specific gravity (Gs) = 2.51

From the graph t90 = I 56.2mm

Coefficient of consolidation CV

$$K = \frac{d^2}{4\ xt90x60} = \frac{(9.252)^2}{4x156.2x60}$$

= 0.007mm+/sec

Final void ratio e1 = Mc x G3

= 0.175x2.5

el = 0.439

Initial void ratio e0 = e1+ e

1H(1-t-e1)

$$\text{Void ratio change } \Delta e = \frac{\Delta H\ (1 + e1)}{H - \Delta H}$$

ΔH, Change in thickness

$$\Delta e = \frac{0.495(1.439)}{18.505}$$

= 0.0385

Void ratio change e0 = 0.439 + 0.03 85

= 0.4775

Co-efficient of compressibility Mv

$$Mv = \frac{1}{1 + e_0} \quad x \quad \frac{e_0 - e_1}{r^1_1 - r^1_0}$$

$$= \frac{0.0385}{1.477(5)}$$

$= 0.0052m^2KN$

COEFFICIENT OF COMPRESSIBILITY MV CALCULATIONS

From the void ratio e versus pressure P graph ... see graph for details

The slope $a_v = \frac{\Delta P}{\Delta e}$

The coefficient of compressibility Mv is given as:

$$Mv = \frac{av}{1 + e} = \frac{\Delta e}{\Delta P \text{ x } (1+e)}$$

Using Dawakin Tofa (A) for illustration

$$Mv = \frac{\Delta e}{\Delta P \text{ x } (1+e)} = \frac{0.0114}{1000 \text{ x } (1.0114)} = 1.0 \text{ x } 10^{-5}m^2/KN$$

Using Kiru for Illustration

$$Mv = \frac{\Delta e}{P \text{ x } (1+e)} = \frac{0.0114}{650 \text{ x } (1.0114)} = 0.00002 \text{ x } 10^{-5}m^2/KN$$

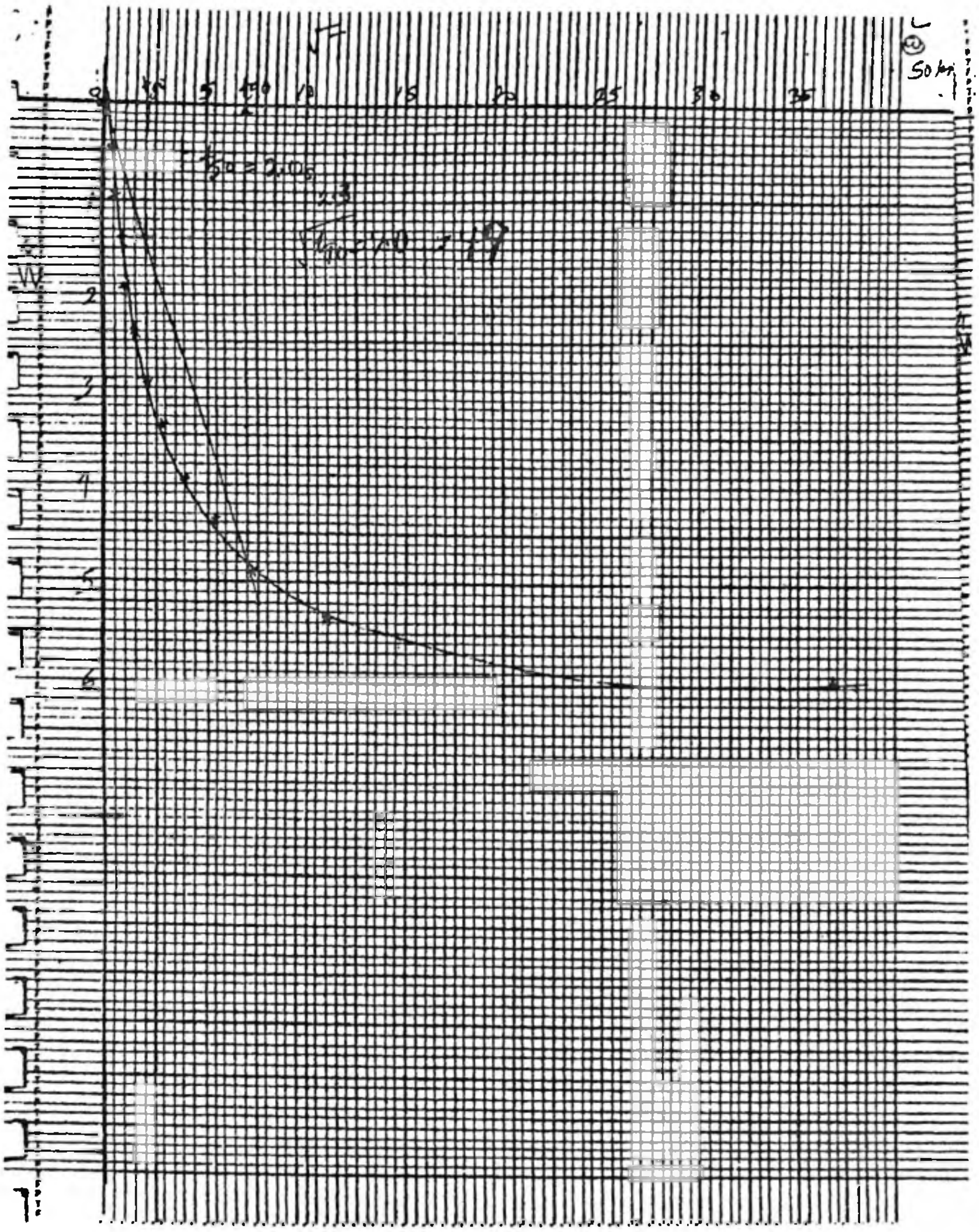
Same for others

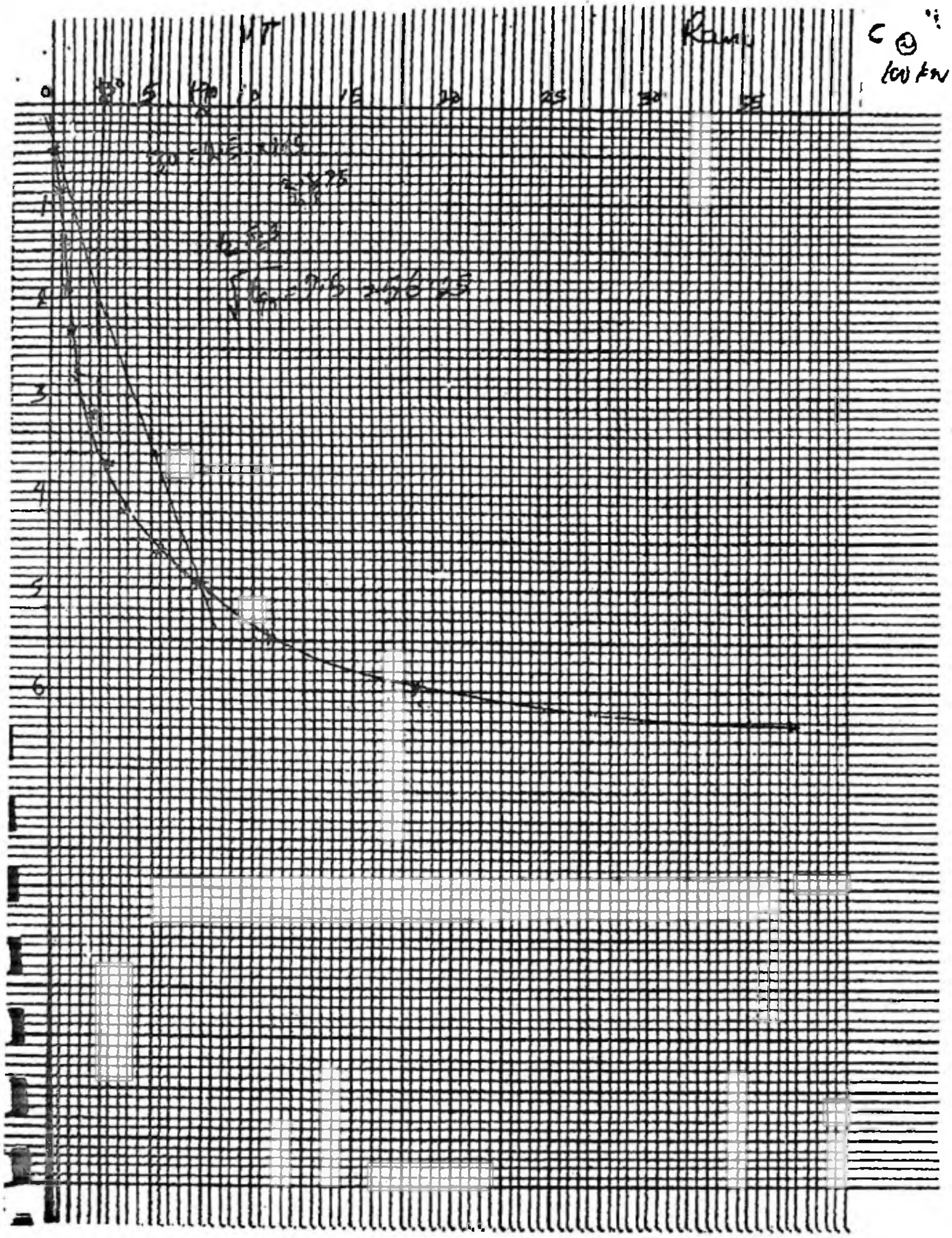
PANDA PIT HOLE

Pressure KN/M	Time @ 90% consolidation (Min)	Initial Void Ratio (e_0)	Final Void Ratio change e_f	Void Ratio change □e	Co-efficient of consolidation C_V (mm^2/S)
50	82.8	0.642	0.618	0.024	0.013
100	121.0	0.722	0.700	0.022	0.0056
200	81.0	0.533	0.514	0.019	0.0084
400	73.3	0.422	0.407	0.015	0.0093
800	72.3	0.544	0.534	0.010	0.095

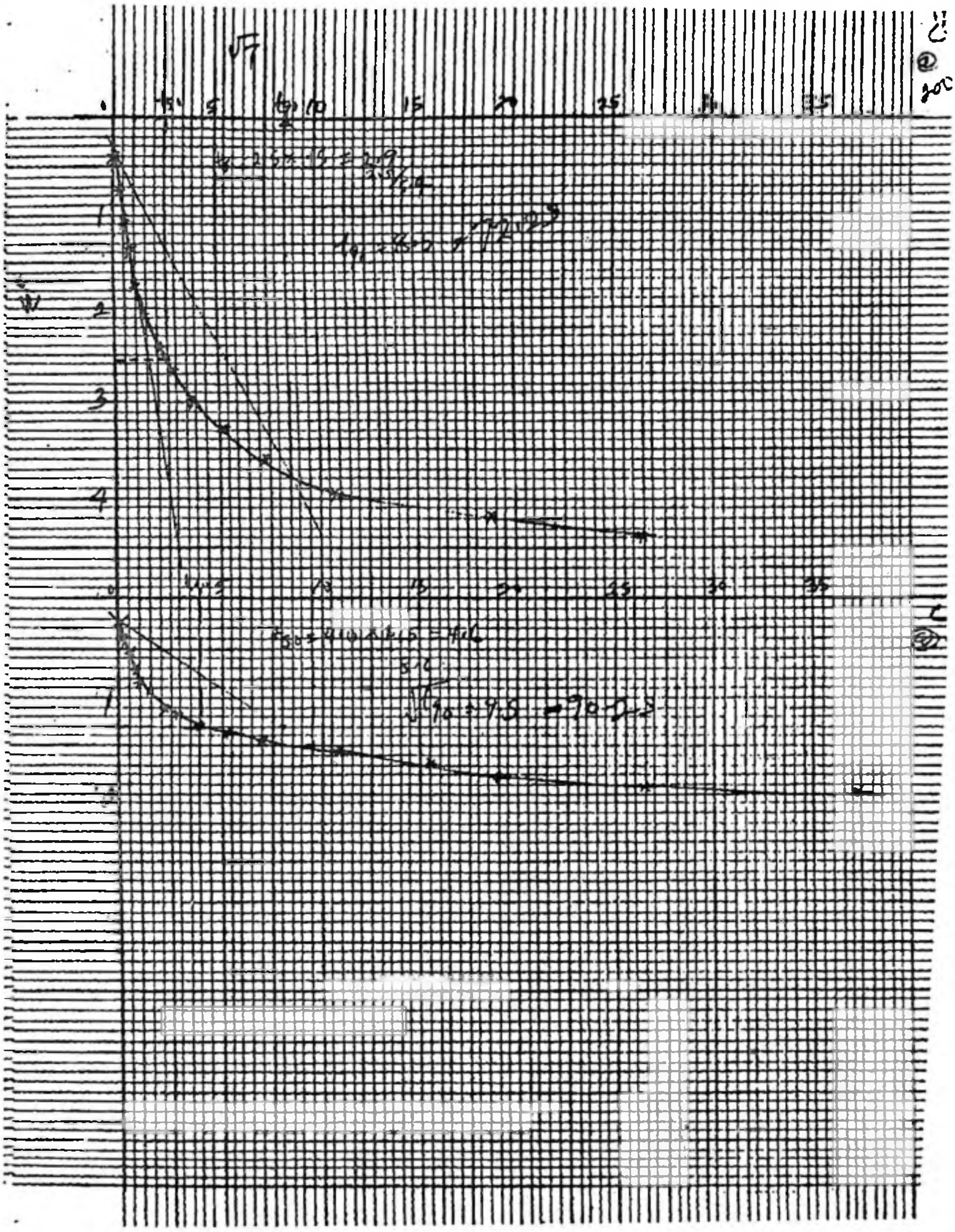
PANDA PIT HOLE

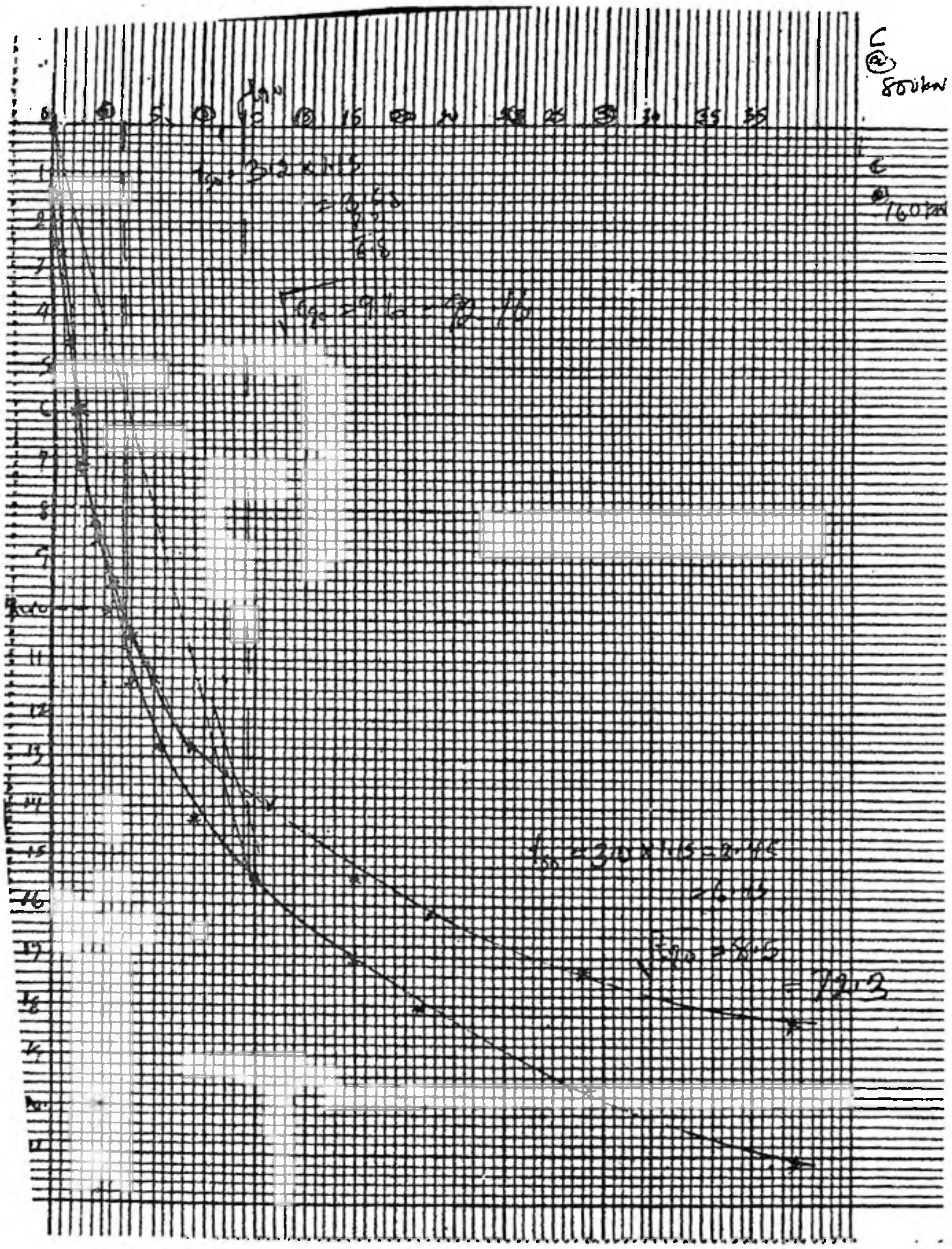
Pressure KN/M	Time @ 90% consolidation (Min)	Initial Void Ratio (e_0)	Final Void Ratio change e_f	Void Ratio change □e	Co-efficient of consolidation C_V (mm^2/S)
50	54.0	0.421	0.386	0.035	0.016
100	80.13	0.642	0.611	0.031	0.091
200	121.52	0.533	0.510	0.023	0.059
400	215.70	0.590	0.578	0.012	0.003
800	124.45	0.674	0.667	0.007	0.006


50 km
$\frac{1}{50} = 2.06$



C
100 kW





DEPARTMENT OF CIVIL ENGINEERING

Sample Location:......Panda Pit Hole....... Press No : ..CN4.......... Date: ....10/8/2015......

Sample Depth:..2m............................ Loading :

Sample No: ....B.................... Load: ....50KN...................

Consolidation test (lab data) $H = 19.00\,mm$

Calculation	Date	Clock Time	Time	Wt √T	Dial reading ×10.000	Change in thicknes δh (mm)	Change in thickness of specimen h (mm)
Weight of ring		8·30	0	0			
Weight of wet sample			10 sec.	0·41	0,489	0489	
			15 sec.	0·50	0, 484	0978	
			30 sec.	0·71	0, 481	1459	
			1 min.	1·00	0, 476	1935	
			2 min.	1·41	0.475	2410	
			4 min.	2·00	0.474	2884	
			8 min.	2·83	0.472	3356	
			15 min.	3·87	0,287	3643	
			30 min.	5·48	0·124	3762	
		9·30	1 hr.	7·75	0·084	3725	
		10·30	2 hr.	10·96	0,020	3795	
		12·30	4 hr.	15·50	0·019	3814	
		2·30	6 hr.	18·97	0·012	3826	
		8·30	12 hr.	26·83	0·011	3837	
		8·30	24 hr.	37·95	0·010	3847	

INITIAL VOIDS RATIO = FINAL VOIDS RATIO =

DEPARTMENT OF CIVIL ENGINEERING

Sample Location:....Panda Pit Hole........ Press No : .... Date: ....10/8/2015......

Sample Depth:...2m............................. Loading :

Sample No: ....8.................... Load: ......100KN............

Consolidation test (lab data) H=19.00mm

Calculation	Date	Clock Time	Time	Wt √T	Dial reading 10.000	Change in thickness δh (mm)	Change in thickness of specimen h (mm)
Weight of ring		8.10	0		0,10	0010	
Weight of wet sample			10 sec.	0.41	1.410	1420	
			15 sec.	0.50	1.334	2754	
			30 sec.	0.71	1.272	4026	
			1 min.	1.00	1.250	5276	
			2 min.	1.41	1.238	6514	
			4 min.	2.00	1.236	7740	
			8 min.	2.83	1.234	8974	
			15 min.	3.87	1.227	10207	
			30 min.	5.48	1.221	1422	
		9.10	1 hr.	7.75	1.200	12622	
		10.10	2 hr.	10.96	1.187	13809	
		12.10	4 hr.	15.50	1.175	14,984	
		2.10	6 hr.	18.97	1.173	16157	
		6.10	12 hr.	26.83	1.171	17,328	
		8.10	24 hr.	37.95	1.163	18,491	

INITIAL VOIDS RATIO =

FINAL VOIDS RATIO =

DEPARTMENT OF CIVIL ENGINEERING

Sample Location:.....Panda Pit Hole....... Press No : ..CN4.......... Date: ....10/8/2015.....

Sample Depth:..2m.............................. Loading :

Sample No: ....B.................... Load: ....200KN............

Consolidation test (lab data) $H = 19.00 mm$

Calculation	Date	Clock Time	Time	Wt √T	Dial reading 10.000	Change in thickness ∫h (mm)	Change in thickness of specimen h (mm)
Weight of ring		8.15	0		1,163	1163	
Weight of wet sample			10 sec.	0.41	2,452	3615	
			15 sec.	0.50	2.436	6051	
			30 sec.	0.71	2.364	8415	
			1 min.	1.00	2.338	10753	
			2 min.	1.41	2.332	13085	
			4 min.	2.00	2.315	15400	
			8 min.	2.83	2.314	17714	
			15 min.	3.87	2,300	22311	
			30 min.	5.48	2.297	24592	
		9.15	1 hr.	7.75	2.281	26869	
		10.15	2 hr.	10.96	2,277	29159	
		12.15	4 hr.	15.5	2.220	31408	
		2.15	6 hr.	18.97	2.269	31409	
		8.15	12 hr.	26.83	2.260	33668	
		8.15	24 hr.	37.95	2.255	35923	

INITIAL VOIDS RATIO = FINAL VOIDS RATIO =

DEPARTMENT OF CIVIL ENGINEERING

Sample Location:......Panda Pit Hole....... Press No : .... Date: ....10/8/2015......

Sample Depth:...2m............................. Loading :

Sample No: ....B.................... Load: ....400KN...............

Consolidation test (lab data) $H = 19.0$mm

Calculation	Date	Clock Time	Time	Wt √T	Dial reading 10.000	Change in thickness δh (mm)	Change in thickness of specimen h (mm)
Weight of ring		8·10	0		2,255	2255	
Weight of wet sample			10 sec.	0·41	2035	4290	
			15 sec.	0·50	2,010	6300	
			30 sec.	0·77	2,002	8302	
			1 min.	1·00	3,499	11801	
			2 min.	1·41	3,495	15296	
			4 min.	2·00	3,491	18787	
			8 min.	2·83	3,490	22277	
			15 min.	3·87	3,478	25755	
			30 min.	5·48	3,475	29230	
		9·10	1 hr.	7·75	3,457	32687	
		10·10	2 hr.	10·96	3,450	36137	
		12·10	4 hr.	15·50	3,446	39583	
		2·10	6 hr.	18·97	3,444	43027	
		6·10	12 hr.	26·83	3,435	46462	
		8·10	24 hr.	37·95	3,424	49886	

INITIAL VOIDS RATIO =

FINAL VOIDS RATIO =

DEPARTMENT OF CIVIL ENGINEERING

Sample Location:......Panda Pit Hole....... Press No : .... Date: ....10/8/2015......

Sample Depth:...2m............................... Loading :

Sample No: ....B..................... Load: ....800KN..............

Consolidation test (lab data)

H = 19.0mm

Calculation	Date	Clock Time	Time	Wt √T	Dial reading 10.000	Change in thickness δh (mm)	Change in thickness of specimen h (mm)
Weight of ring		8-15	0		3,424	3424	
Weight of wet sample			10 sec.	0.41	3,214	6658	
			15 sec.	0.50	3,200	9838	
			30 sec.	0.77	3,186	13024	
			1 min.	1.00	3,170	16194	
			2 min.	1.41	3,145	19339	
			4 min.	2.00	3,188	22427	
			8 min.	2-83	3,127	28604	
			15 min.	3-87	3,124	28728	
			30 min.	5-48	3,122	31850	
		9-15	1 hr.	7.75	3,121	34971	
		10-15	2 hr.	10.96	3,106	38077	
		12-15	4 hr.	15.5	3,100	41177	
		2-15	6 hr.	18.97	3,078	44255	
		8-15	12 hr.	26.83	3,064	47319	
		8-15	24 hr.	37-95	3,044	50363	

INITIAL VOIDS RATIO =

FINAL VOIDS RATIO =

DEPARTMENT OF CIVIL ENGINEERING

Sample Location:......Panda Pit Hole....... Press No : .... Date: ....10/8/2015......

Sample Depth:...2m............................. Loading :

Sample No: ....B.................... Load: ....400KN..........

Consolidation test (lab data)

$H = 19.0 mm$

Calculation	Date	Clock Time	Time	Wt √T	Dial reading 10.000	Change in thickness δh (mm)	Change in thickness of specimen h (mm)
Weight of ring		8·10	0		2,255	2255	
Weight of wet sample			10 sec.	0·41	2,635	4290	
			15 sec.	0·50	2,010	6300	
			30 sec.	0·77	2,002	8302	
			1 min.	1·00	3,499	11801	
			2 min.	1·41	3,495	15296	
			4 min.	2·00	3,491	18787	
			8 min.	2·83	3,490	22277	
			15 min.	3·87	3,478	25755	
			30 min.	5·48	3,475	29230	
		9·10	1 hr.	7·75	3,457	32687	
		10·10	2 hr.	10·96	3,450	36137	
		12·10	4 hr.	15·50	3,446	39583	
		2·10	6 hr.	18·97	3,444	43027	
		6·10	12 hr.	26·83	3,435	46462	
		8·10	24 hr.	37·95	3,424	49886	

INITIAL VOIDS RATIO = FINAL VOIDS RATIO =

DEPARTMENT OF CIVIL ENGINEERING

Sample Location: Panda Pit Hole Press No : .... Date: 10/8/2015

Sample Depth: 2m Loading :

Sample No: B Load: 800KN

Consolidation test (lab data) $H = 19.0mm$

Calculation	Date	Clock Time	Time	Wt √T	Dial reading 10.000	Change in thickness δh (mm)	Change in thickness of specimen h (mm)
Weight of ring		8-15	0		3,424	3424	
Weight of wet sample			10 sec.	0·41	3,214	6638	
			15 sec.	0·50	3,200	9838	
			30 sec.	0·77	3,186	13024	
			1 min.	1·00	3,170	16194	
			2 min.	1·41	3,145	19339	
			4 min.	2·00	3,138	22477	
			8 min.	2·83	3,127	28604	
			15 min.	3·87	3,124	28728	
			30 min.	5·48	3,122	31850	
		9·15	1 hr.	7·75	3,121	34971	
		10·15	2 hr.	10·96	3,106	38077	
		12·15	4 hr.	15·5	3,100	41177	
		2-15	6 hr.	18·97	3,078	44255	
		8·15	12 hr.	26·83	3,064	47319	
		8·15	24 hr.	37-95	3,044	50363	

INITIAL VOIDS RATIO = FINAL VOIDS RATIO =