# A PROPOSED FRAMEWORK FOR ASPHALTIC CONCRETE PAVEMENT DESIGN FOR TROPICAL SOILS – CASE STUDY OF GHANA

by

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# ABSTRACT

Fundamental differences exist in soil types formed under temperate and tropical climatic conditions. Consequently, their use for road pavement design will require different approaches and standards. The absence of a systematic approach in addressing tropical pavement design requirements and the use of temperate design standards for tropical countries has led to early failure of road pavements in Ghana and other tropical countries. However, few studies have been carried out on developing standards based on field studies to determine key design parameters to address challenges of tropical pavement design.

The aim of this research is to evolve a rational approach using mechanistic-empirical principles to design pavements for tropical laterite soils. The main objectives were therefore to determine the key pavement design parameters for Ghana through empirical studies as well as carry out technical and economic analysis to establish optimum designs for the various climatic zones of Ghana.

Data were collected on newly constructed test sections in two climatic zones through the instrumentation with thermocouples to monitor temperature of all the pavement layers and moisture sensor blocks were fixed in the granular layer of the pavement to monitor the moisture regime. Automatic traffic counters were also installed to collect traffic data. Key design parameters that impact on pavement design such as pavement deflection, layer moduli, pavement temperature, actual traffic data, rainfall and moisture content of granular layers. Effects of seasonal climate on the moisture regime in the granular pavement layers were studied. The relationships between these parameters were established through statistical data analysis.

The research revealed that the annual average daily traffic and the cumulative standard axle loads collected from the test sections were higher than those used for the design of the roads. In addition, the study revealed that average temperature of the asphaltic concrete layers observed from the test sections was in the 35-37$^{o}$C range as against 25$^{o}$C used in the asphaltic concrete mix design. This implies that Ghana‟s asphaltic concrete road pavements are weaker for the operational environment; hence, this partly accounts for the early failure of these pavements. The research further revealed that, Ghana‟s quartzitic laterite soils have engineering properties which meet the specification for selection as crushed stone base course and sub-base material.

Other findings of the research are outlined as follows:

- Moisture content levels of the granular layers are generally low during periods of high rainfall and high during periods of low rainfall and this was observed to be as a result of the evapo-transpiration and temperature levels during these periods in addition to the rainfall;
- The temperature levels of all the layers are high during the wet periods although rainfalls are high and vice versa due to the high sunshine durations during the wet periods;
- Classification of laterite for engineering purposes should be based on experiences in a given climatic zone and must take into account factors such as the geological history, morphological characteristics, genesis, dominant clay mineral type, ion exchange and actual moisture condition;
- Field performance of laterite soils is not totally dependent on its index properties (particle size distribution, liquid and plastic limits) but must take into account the continuous weathering of tropical soils and its impact on engineering properties as well as field performance;
- A strong relationship was established for the ambient temperature and the wearing course or HMA layers; and
- Current design guides used in Ghana are not appropriate as they do not take into account local climatic condition under which the road will be used.

The tropical pavement design framework developed in this study has demonstrated a rational method which gives the pavement engineer control over the design parameters and a better method in the derivation of the candidate roads using the KENLAYER. In order to select the most economical of these candidate sections a life cycle cost module, which used HDM-IV as the analytical tool, was developed to determine appropriate maintenance standards with respect to routine and periodic maintenance.

This framework, therefore, bridges the gap between technical design of pavement and its economic evaluation before selection of a given candidate pavement for construction. The proposed framework arising from this study also eliminates the use of design manuals and monographs from temperate zones which are not suitable for tropical climatic environments.

This study has made a modest contribution to the selection of appropriate laterite materials for use as pavement materials in the various climatic zones of Ghana. In addition, adoption of the proposed framework will result in the development of technically feasible as well as economical and cost effective pavement designs for Ghana and other tropical countries.

The study recommends a follow-up research to develop a package to carry out structural pavement analysis and economic analysis in the selection of optimum pavement design under one framework.

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# TABLE OF CONTENTS

## LIST OF FIGURES

## LIST OF TABLES

# CHAPTER 1

# INTRODUCTION AND STUDY REVIEW

## 1.1 Background

The quest for suitable local soil material with the requisite engineering properties has been the pre-occupation of road engineers in any environment they find themselves designing and constructing road infrastructure.

Lateritic soils, which are forms of residual soils found in tropical parts of the world but principally located in South East Asia, West and Central Africa, Peninsular India and the Northern half of South America have been the main source of material for engineering endeavours. The abundance of laterite soils in these geographic areas makes their use for construction of roadway and airfield highly desirable and economic (De Graft Johnson et al, 1972). It is therefore important to study in detail and understand their properties as a basis to determine the engineering behaviour. Knowledge in this area will ensure development of effective engineering specifications and standards for laterite as road construction material.

There are however, issues associated with laterite soils which make their application for road engineering purposes very challenging and therefore the need to address them to ensure efficiency and effectiveness in the engineering application. The issue of developing universal or appropriate technical specifications as a framework for selection of laterite material is still pending as a result of conflicting definition used by researchers worldwide and therefore making it difficult to agree on a common definition to pave way for developing accepted specification which transcend environmental and geographical boundaries.

Acceptable pavement design methodology is also of paramount interest to engineers practicing in the tropics where laterites abound. There have been many attempts in the past forty years to address these issues by researchers and practicing engineers.

Numerous studies and research done in the past decades to address the above challenges have been very elusive as a result of differences in research methodologies and, to a large extent, the confusing interpretation of individual research initiatives leading to inaction on the way forward.

Currently a universally accepted definition of what constitutes a laterite soil is yet to be achieved as there are biases in the definition with respect to morphology, chemical content and the laterite profile horizon by many researchers. Owing to its wide occurrence, description of the laterite soils stretches from its characteristic hardness through the chemical composition to its soil forming profile as a residual soil. The lack of a standard definition is also hampering the development of the necessary technical specification and design guide (Maignien, 1966). The lack of common definition has also prevented researchers from different tropical areas in the world to accept other initiatives and build upon them.

This has led to an application of laterite material based on local experience in a particular tropical environment without properly understanding its behaviour and hence difficulty in the interpretation of its performance when used as road pavement material. Application of successful laterite road design methodologies based on practical experiences elsewhere on the premise of reproducibility has come with mixed results of successes and failures in different tropical regions in the world.

Information on failures has not been properly documented in order to share globally. Ghana, for example, due to backlash from Government and politicians when such failures occur, are rather confined at country levels. Such failures even though are of high research value in terms of carrying out scientific investigation to identify cause of failure as a guide to future use of the material. Attempts have been made in the past by researchers to consolidate experiences from various countries in terms of engineering application of laterite but responses were not forthcoming as could be seen in unsubstantiated information presented in literature due to non-compilations and improper documentations of work done. Others have also attempted to come out with technical specifications from various countries perspectives but again responses have not been encouraging to make comprehensive analysis and objective propositions (Townsend et al, 1976).

## 1.2 Problem Definition

Laterite soils abound in Ghana and are the main soil material used for construction of road pavements. Cracking, stripping of surface material and rutting are common failures, which have been observed on roads with laterite pavement layers. These specific failures later contribute to the subsequent plastic deformation of the road pavement after few years of being open for use; especially, under heavy traffic loads.

Findings from research and studies carried out in the light of such failures have attributed the problem to inadequate pavement design. It has also been observed that in spite of strict adherence to technical specification and testing standards and control procedures from temperate zones, countries in the tropics continue to experience numerous cases of highway and airfield failures.

These failures have been attributed to the current standard methods for selecting, preparing and testing of laterite soil material in the laboratory. Whereas soils in the temperate zones are considered to be generally stable in character (e.g. particle size distribution of soils are fairly stable and not much affected by the pre-test preparation, testing procedures and pavement construction technology), tropical aggregates, on the other hand laterite soils, change continuously under the influence of chemical weathering and in structure during construction (Gidigasu, 1991). These observed differences, make the current approach to design of laterite road pavements uneconomic due to failure of some of the road.

These design failures are largely due to direct transposition of temperate technical specifications and testing procedures without adequate modifications customisation to make their use relevant to local tropical conditions. The basis for the development of temperate technical specification are on soils that could be referred to as inert- low chemical weathering activity takes place when in use as road pavement material; whereas tropical soils are termed active soils due to continuous chemical weathering activity in the soils. Geographical distribution of weathering types based on rainfall and temperature validates the two general types of soils in the temperate and tropical regions of the World (Strakhov, 1967).

It has been observed and validated through research in Ghana that there are significant differences between the laboratory test results and the field performance of laterites as a road pavement material. The anomalous laboratory and field behaviour of some of these laterite soils appears to stem mainly from lack of suitable engineering evaluation criteria of laterite soil which should take into account not only the soil properties as determined in the laboratory but also the mode of formation (genesis), degree of weathering (decomposition, laterisation and desiccation) as well as unique chemical and mineralogical characteristics (Gidigasu, 1976).

Research works by de Graft-Johnson et al (1969) and Hammond (1970) have also shown that conventional testing methods (e.g. grading, plasticity and CBR) should not be the sole determinants for rejecting or accepting laterite gravel material as road pavement material. Other properties such as the physical and chemical properties must form important evaluation criteria for laterite soils.

Climatic conditions of any given area where the road pavement lies also affect in-service performance of road. The two most important climatic parameters that affect road pavement materials are temperature and moisture. As a result of the effect of temperature of the pavement and the repeated loading of the pavement, textural deformation are experienced on many laterite roadways which then leads to the destruction of the road.

Temperature influences the design and performance of road pavement and this is more so in the tropics where temperatures can be very high. A typical example is the mix design of asphalt pavement course where in accordance with specification, the mix design should be based on softening point of $60^{o}$C.

In some areas of the tropics such as India, daily temperatures could be as high as 50 to $60^{o}$C. In Ghana daily temperature levels are very high (31 to $39^{o}$C) during the dry season of the year. With increasing temperature, the elastic modulus decreases with the attendant increase in traffic stresses imposed on the pavement material below bituminous surfacing or the bituminous road base. Under large surface stresses bituminous materials tend to flow viscously. The elasticity, deformation and fatigue of bituminous materials are all temperature dependent. For proper pavement design and good performance of pavement in service, the influence of temperature on the structural properties is critical.

Some pavement materials contain soluble salts in the form of sulphates of sodium, magnesium and calcium as well as finely divided organic matter. These have important influence on the changes which occur in soils within the granular layers of the pavement structure. The pavement moisture content and the subsequent values also affect the property changes in the soils

In Ghana, extensive work has been done on laterite soils by the Building and Road Research Institute (BRRI) both in the field and the laboratory in isolating some of the key variables contributing to failure of laterite roadways; for example, the rehabilitation of the Anyinam-Kumasi road and Axim-Elubo road all funded by the African Development Bank, and the reconstruction of the Yamoransa-Anwiankwanta road (MRH, 1994) funded by the Overseas Economic Cooperation Fund (OECF), Japan. Research carried out extends from the pre-testing processes (Gidigasu, 1970; Yeboah and Hornsby-Odoi, 1970), testing and the use of aggregate in pavement construction (Gidigasu 1991; Bhatia and Hammond, 1970), construction methods (Gidigasu 1980; 1982), moisture environment within which the pavement is placed (Gidigasu and Appeagyei, 1982), physical and chemical weathering in wet climatic environment (Gidigasu 1971; 1974) among others.

These findings, however useful, have not been properly harnessed to undertake the following:

- Make the appropriate modification to the current standards and technical specification for selecting laterite materials for road pavement;
- Evaluate the subgrade conditions in terms of the stable moisture condition appropriate for building laterite road pavement in Ghana; and
- Evolve a pavement design model to ensure satisfactory field performance of laterite road pavement.

## 1.3 Aim and Objectives

The aim of this research is, therefore, to develop a framework for economic design of asphaltic concrete on tropical laterite soils in Ghana.

In order to achieve this aim, the following objectives are to guide this study:

1. Carry out a detailed literature review of work done on laterite soils occurring in Ghana to understand its formation and behaviour in the different climatic zones of the country;
2. Carry out field studies to determine key pavement design variables;
3. Develop a pavement design framework for Ghana‟s tropical climatic conditions;
4. Determination of optimal pavement design based on technical and economic analysis; and
5. Make appropriate recommendation on design of different pavement type for the various climatic zones in Ghana.

## 1.4 Scope of Study

In order to achieve the set objectives the following tasks will be carried out under each of the objectives:

**Objective 1**:

Carry out a detailed literature review of work done on laterite soils occurring in Ghana to understand its formation and behaviour in the different climatic zones of the country.

**Tasks**

- Identify, locate and study regional distribution and characteristics of laterite soils.
- Consolidate research work and studies carried out on Laterite soils with respect to road construction by Building and Road Research Institute (BRRI) in Ghana.
- Study the geology of West Africa and Ghana from information obtained from Geological Survey Department.
- Obtain pedological information from the Soil Research Institute in Ghana to understand the soil system of Ghana.
- Obtain land system mapping from Centre for Remote Sensing and Geographic Information Services (CERGIS) of University of Ghana and study the relationship of laterite to landform.
- Review meteorological data to ascertain the characteristics of the climatic zones in Ghana and its impact on formation of laterites in Ghana.

- Prepare a report on engineering properties of laterites in Ghana.

**Objective 2**:

Carry out field studies to determine key pavement design variables.

**Tasks**

- Select test sections to monitor the performance of road pavement in the climatic zones of Ghana.
- Prepare a report on the instrumentation of the selected sites.
- Prepare a resume on the test sections based on engineering design reports and as-built reports.
- Prepare a methodology for collection of data from the test sections.
- Prepare a resume on method of data analysis.

**Objective 3:**

Develop a pavement design framework based on Ghana"s tropical climatic condition.

**Tasks**

- Prepare a resume of tropical pavement development experiences with similar climatic condition.
- Carry out evaluation of the principles underlying various approaches in tropical pavement design.
- Establish key parameters influencing tropical pavement designs.
- Review the current practices of road pavement design in Ghana.
- Undertake a comparative analysis of best current practices in tropical pavement design with practices in Ghana and make inferences.
- Development of different pavement designs options for Ghana.

**Objective 4:**

Determination of optimal design standard based on technical and economic analysis

**Tasks**

- Carry out total life cycle analysis using calibrated HDM-4.
- Conduct economic analysis with the alternate pavement designs.
- Establish key pavement design parameters and perform sensitivity analysis.
- Determine key parameters from the technical and economic analysis to determine which parameter impact significantly on performance of road pavement in climatic zones of Ghana.
- Carry out sensitivity and scenario analysis to select set of key parameters.
- Determine the optimized set of parameters for alternate designs.

**Objective 5:**

Make appropriate recommendation on the design of different pavement type for the various climatic zones in Ghana.

**Tasks**

- Develop optimised pavement design guidelines and specification for the four climatic zones
- Prepare recommend design parameters for laterite roads in Ghana.

## 1.5 Structure of Thesis

Research focus and area of contribution: application of lateritic soils in roadway pavement engineering.

### Chapter 1 Introduction

Chapter One of the thesis will comprise the background of the research, explaining the challenges confronting road engineers in the application of tropical soil for engineering purposes and the contribution of the research to overcome the issues in road pavement design. Chapter one will consist the following:

- Background;
- Problem Definition;
- Aim and Objectives of Research;
- Scope of the Study;
- Structure of the Thesis; and
- Research Contribution.

**Chapter 2 Review of Relevant Literature**

Chapter 2 will concentrate on review of relevant literature with respect to engineering properties of tropical laterite soil, current practices and challenges in road pavement designs and their impact on performance of tropical road (the chemical, mineralogical and physical properties as well as their influence on material performance as road pavements), review of relevant literature.

It will focus on the following:

- Materials surveys to identify locate and determine the extent of occurrence of lateritic soils in the following order:

  i. Globally (literature review);
  ii. Africa (literature review);
  iii. West Africa (literature review); and
  iv. Ghana (literature review).

- Field recognition and identification of lateritic soils

  i. Classification of lateritic soils
  ii. Location and sampling of dominant types
  iii. Laboratory identification and classification

- Engineering properties of lateritic soils

  i. Engineering geology of lateritic soils
  ii. Engineering characteristics of lateritic soils

iii. Soil mechanics
iv. Evaluation of lateritic soils for use as civil engineering material
v. Use of lateritic soils as road pavement material

- Road pavement design practices

i. Empirical designs and its appropriateness for the tropics
ii. Challenges and short coming of mechanistic pavement
iii. Methodology for mechanistic–empirical (M-E) design approach
iv. Requirement of M-E approach to ensure satisfactory performance

**Chapter 3 Methodology**

Chapter 2 will outline the approach of the research in terms site selection and instrumentation of the sites, data collection procedures, experimental survey and field study, data processing and analysis and results. More specifically, this chapter deals with the following;

- Site selection and instrumentation;
- Data collection procedure;
- Data processing;
- Data analysis; and
- Results.

**Chapter 4 Experimental Design & Field Study**

The performance of the road pavement in the test sections is assessed in Chapter 4 with respect to temperature regime within the road pavement layers and physical properties on one hand, and the structural capacity of the pavement. The assessment is the key component of the study and will be based on the following:

- Set out the processes of establishing the test sections;
- Carefully carry out field assessment of the material properties in the test sections;

- Measurement of structural capacity performance and deterioration parameter in the road in relation to geological, drainage, traffic and climatic conditions, moisture content and temperature; and
- Review of international test section of similar nature.

**Chapter 5 Data Analysis**

The data requirement for the research will be from the engineering design reports of the test sections, and the as-built field data. Cross sectional data of similar older pavement will be collect to augment the field data. The following will be the major areas of Chapter 5:

- Engineering design data
- As-built field data
- Experimental design data
- Cross sectional data
- Data processing
- Data analysis
- Results

**Chapter 6 Engineering Properties of Lateritic Soils**

The aim of Chapter 6 is to consolidate most of the relevant research work to have a better appreciation of the engineering properties to provide supplementary engineering information to support current standards and specification. The area of focus will be as follow;

- Particle size distribution of Ghana soils
- Plasticity of Ghana soils
- Strength of coarse particle
- Methods of compaction
- Stabilisation of laterite soil to improve bearing capacity

**Chapter 7 Development of Mechanistic and Empirical Pavement Design Methods**

The aim of Chapter 7 is to use the information on the engineering properties and field data to develop pavement design framework relevant to Ghana condition. The work will entail the following:

- Overview of selected M-E pavement design tool;
- Selection of tools for M-E Pavement Design;
- Calibration of pavement design parameters for Ghana;
- Development of model; and
- Model simulation.

**Chapter 8 Calibration and Adaptation of HDM – IV Model for Ghana**

Chapter 8 addresses work due to calibrate and adapt the HDM-4 model to simulate Ghana condition. The chapter discusses procedures and considerations taken to achieving that. The calibration adaptation work entails;

- Configuration of parameters and standards used in Ghana;
- Field studies carried out in data collection for the calibration;
- Calibration of Road Deterioration (RD) sub-model;
- Calibration of Road Works Effect (WE) sub-model; and
- Calibration of Road User Effect (RUE) sub-model.

**Chapter 9 Pilot Study for New Framework of Pavement Design for Ghana**

Economic analysis will be carried out using the developed pavement model. The following will be the main focus of the economic analysis:

- Preparation of catalogue road pavement;
- Determination of pavement failure key parameters;
- Sensitivity on key parameters;
- Selection of optimised parameters;
- Total life cycle analysis;
- Development of optimised pavement design guidelines and specifications; and

- Recommendation of design parameters.

**Chapter 10 Conclusions and Recommendations**

## 1.6 Novelty of the Work

The novelty of the research lies in the use of empirical data from Ghana to design pavement which takes into account material characterisation, loading and actual design method whose input are obtainable locally. This will ensure proper control of resources and improve performance of laterite pavements. These serve as the basis for developing a pavement model and incorporating the findings to improve the design guidelines and specification of laterite pavements in order to ensure economic efficiency in the design.

## 1.7 Deliverables and Benefits/Beneficiaries of the Research

The final delivery of the research programme will be in the following areas:

- Development of a road pavement performance model for laterite road base;
- Optimise design of laterite road pavement by incorporating life cycle performance and economic efficiency; and
- Develop a framework for the design of laterite road pavements in Ghana.

The framework developed for the design of laterite pavement would ensure a wider usage is made of lateritic materials, particularly by highway material consultants who would be mainly concern with pavement design and construction quality control.

# CHAPTER 2

# REVIEW OF RELEVANT LITERATURE

## 2.1 Introduction

To ensure the good performance of any road pavement in a given environmental condition, its design parameters must take into account the characteristics and the properties of the materials used in the construction of the road pavement. The susceptibility of pavement materials to environmental conditions must therefore be properly evaluated to ascertain its reliability before and during the life of the road pavement in carrying traffic loading. For road pavement to be economic, the engineering studies and designs carried out before the construction of the road pavement must take into account all the environmental factors that are responsible for pavement deterioration.

This Chapter discusses the different approaches and challenges that face developing countries in the tropics in their effort to ensure that the pavement design methodologies adopted from temperate countries, take into account the tropical environment factors which influence the formation of soil materials and their engineering properties as well as reviews the merits and demerits of these temperate design methods currently in use in the tropics for road pavement designs. A better appreciation of the various challenges is important as it sets out a clear road map in an attempt to address tropical road pavement design problems.

The focus of the review of literature is to address the issue of tropical pavement designs in the following areas:

- Assessment of challenges in considering local conditions in pavement design;
- Design guides in use in Ghana which are not based on soil material similar to Ghana"s soil;
- Pavement design methodologies and its relevance to tropical condition; and
- Evaluation of key design parameters locally.

Findings from the review provide a deeper understanding of tropical pavement engineering problems and also enable a structure to be developed that conceptualises the research issue and identifies key variables considered significant to the challenges. These key variables then form the basis for addressing the knowledge gap in tropical pavement design.

## 2.2 Limitations in Considering Local Condition in Pavement Design

Many of the empirical studies relating to the development of design guides and methods were based on soils from a given temperate climatic areas. The climate of a region plays a major role in soil formation. The basic process of soil in varying formation involves the weathering of rocks, based on climatic conditions. The speed of chemical alteration of rocks is increased by higher temperatures and wetter conditions. Work done by Strakov (1967) as reported by Gidigasu (1976) established the geographical distribution of weathering from annual rainfall, vegetation, mean annual temperature and evaporation. Also further work done by Morin and Toder, (1969) reported by Charman (1988) developed a schematic relationship between climatic factors (rainfall and temperature) and intensity of chemical weathering as shown in the Figure 2.1.

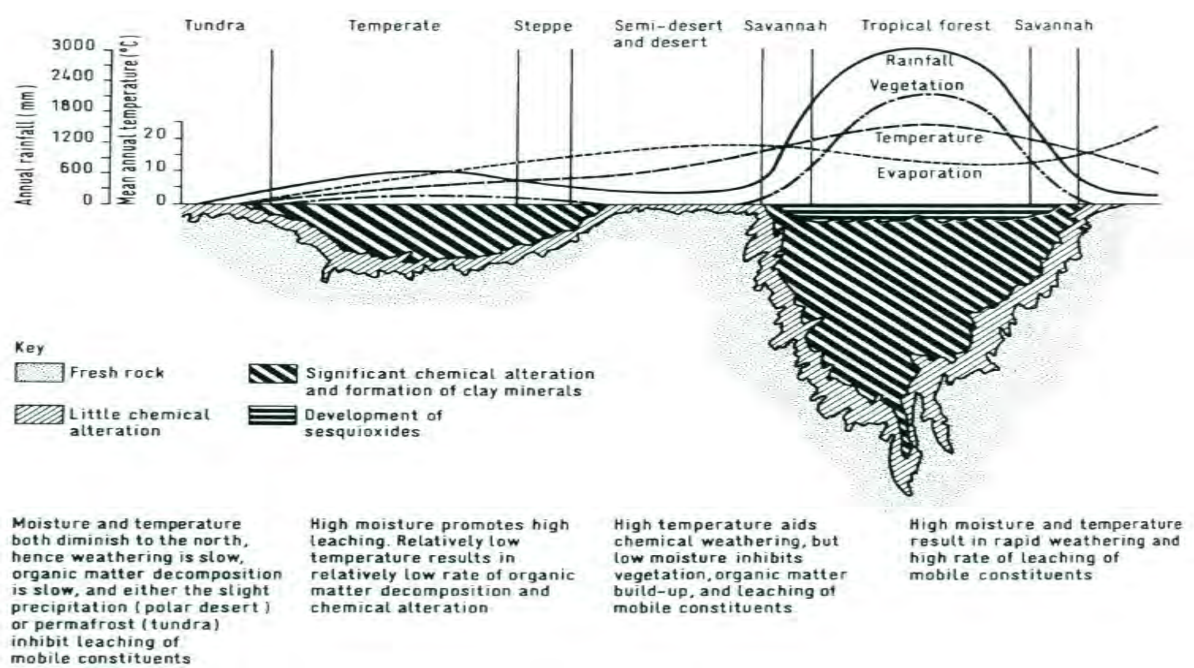



Figure 2.1 - Schematic Relationship between Climate and Weathering (Charman, 1988)

Global locations of zone of high intensity of chemical alteration of the rocks were highlighted as a result of further works by Nixon and Skipp (1957), Quinones (1963) and Saunders and Fookes (1970). A generalized world map in Figure 2.2 shows the distribution of soils within this zone, referred to as Lateritic Soils. Lateritic soils are termed active, as compared to soils in temperate climate zones which are considered to be inert as result of relative intensity of chemical reaction towards their formation.

Laterite should not be confused with lateritic soils which are cultivable and of use to the agriculturalist. The term laterite for engineering work, refers strictly to the tough concretionary (i.e. as hard as concrete) soils which from the engineering point of view is of importance to road construction.

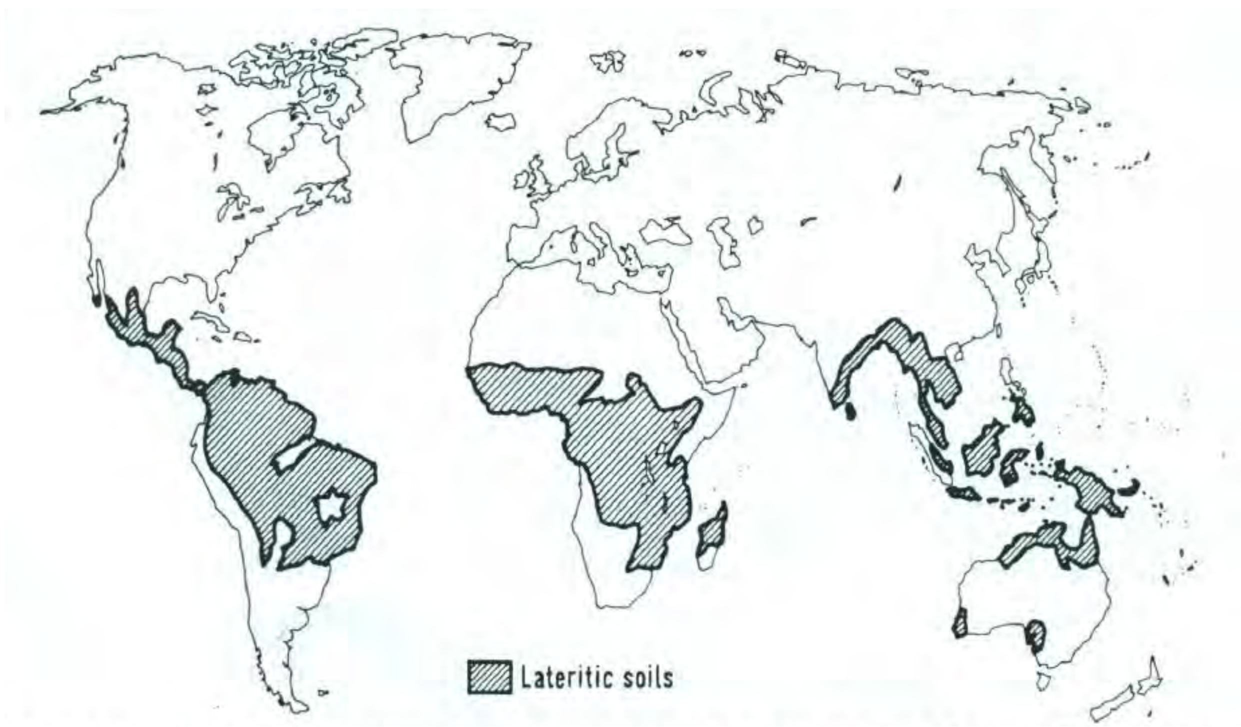



Figure 2.2 - Generalised World Map Showing the Distribution of Laterite Soils (Sanders and Fookes, 1970)

Most of the research works done on laterite soils distribution mainly favour agriculture work under the study of Soil Science - Pedology. Pedological maps developed by researchers from these studies can be summarized in Table 2.1.

Table 2.1 - Summary of Areas Covered By Various Researchers

| Researchers | Year Of Publication | Area Covered |
|---|---|---|
| Prestcott and Pendleton | 1952 | Global Distribution |
| D'Hoore | 1963 | Africa |
| Ray Chandhuri | 1941 | Asia and India |
| Dudal and Moormen | 1962 | South East Asia |
| Neville and Dowhung | n.d. | Malaysia |
| Grant and Hitchson | n.d. | Australia |
| Bramao and Lemos | 1960 | South America |
| Carmargo and Bennema | 1962 | Brazil |
| Marbut | 1932 | South America |

Most road pavements built in developing countries in the tropics are dependent on laterite soils and they are used in one or more layers of the road pavement. Therefore, a better understanding of its geology and the engineering properties thereof is very important to determine its performance as road pavement material. Owing to its wide spread distribution throughout most of the continents with different climatic conditions, the definition of laterite soil is still dependent on the specific climatic condition under which the material is formed.

## 2.3 Definition of Laterites

A drawback in standardization of the engineering properties and performance of laterite soils has been the lack of a universally acceptable definition for laterite soils in the region where they occur. This ambiguity in the definition becomes evident following a review of literature on laterites and commentary on various aspects of the soil. The definition extends from its characteristic hardness, through its chemical composition to its morphology. Some definitions are completely independent of the texture which is of main interest to road pavement engineering.

A typical definition is based on colour as a red residual soil material in the tropical and sub-tropical areas. Such a definition is ambiguous to the extent that it covers all soils reddish in colour and may include occurrences in both temperate and tropical environments considering the fact that temperate areas undergo chemical weathering leading to production of red soils.

### 2.3.1 Definition Based on Hardness

Some definitions have been based on the hardening properties of the soil termed laterite as first observed by Buchanan (1807) in Malabar (India). As a material, which is ferruginous (contains iron), and vesicular in structure, it appears unstratified and porous occurring far below the earth‟s surface. When dug fresh, it has a soft consistency, enough to be cut but rapidly hardens on exposure to air, which then becomes immune to weathering under the influence of climate. The United Nations Educational, Scientific and Cultural Organisation (UNESCO) published a review of research on laterites prepared by Maignien (1966) on the historical background of research works done to address the definition of laterite and cited the following works discussed below:

*Work by Newbold (1846), and Lake (1890) reviewed studies carried out on India laterites pertaining to Geology of South Malabar after Buchanan, as well as associated works by Babington (1821), Benza (1836), Clark (1838), Kelaart (1853), Blanford (1859) and many others established an important criteria that laterite is an alteration product of various materials, including crystalline igneous rock, sediments, detrital deposits and volcanic ash as well as lacustrine deposits. Coverage in terms of definition based on above review became difficult since other subjacent lithomarge exhibit hardness upon exposure to the air (Blanford, 1859).*

*The foregoing reviews were also obscured to indurated occurrences. Talbot (Prescott, 1931) complicated the problem by defining all Australian indurated occurrences containing silica and limestone as laterite. Little (1969) and Quinones (1963) recommended the abandonment of this loosely used terminology. Both Mallet's (1883) and Bauer's (1898) work on more precise definition based on the chemical characteristics was used to narrow the definition by describing laterite as ferruginous and aluminous. It established the relative insignificant content of silica and high content of aluminium in a hydroxide form comparable to the composition of bauxite. Warths (1903) establish that some Indian laterites contain small amount of alumina but rich in iron oxides and vice versa. Work done by Richterfen (1886), Oldham (1893), Bemmelen (1904);*

*Chautard and Lemoine (1908), Arsandeau (1909) were all to define laterites on the basis of chemical and mineralogical content. Fermor (1911) developed a nomenclature of laterite material on the basis of the chemical composition of laterite, using the constituent element (Fe, Al, Ti and Mn). Lacroix (1913) improved on Fermor's work with a similar and more comprehensive classification by dividing laterites into three classes depending on their relative content of the hydroxides. Martin and Doyne (1927, 1930) further narrowed the chemical classification based on silica-alumina ratio ($SiO_2/Al_2O_3$).*

*These ratios were questioned since the original definition based on chemical composition attached importance to the role of iron oxides in laterite rock. Also, the hardening process in laterite soils seems to consist of the crystallization of the amorphous iron and dehydration in laterite soils is also considered the most important factor influencing their engineering properties (Alexander and Cady 1962). The use of silica-sesquioxide ratio ($SiO_2/Al_2O_3 + Fe_2O_3$) was in favour of Martin and Doyne's ratio, but with the same limiting values used. Values greater than 2 indicated non-lateritic, tropically-weathered soils as shown in Table 2.2.*

Table 2.2 - Silica-Sesquioxide Ratios of laterite types (Martin and Doyne, 1927, 1930)

| **Type** | **$SiO_2/Al_2O_3$ Ratio (R)** | **$SiO_2/Al_2O_3 + Fe_2O_3$ Ratio (R)** |
|---|---|---|
| True Laterite | R > 1.33 | R > 1.33 |
| Silicate Laterite | 1.33 < R < 2.0 | 1.33 < R < 2.0 |
| Laterite Clay | R > 2.0 | R > 2.0 |

The chemical definitions based on the above ratios were disputed by other researchers Pendleton and Sharasuvana (1946), Robinson (1949) and Van der Woort (1950). That the value of the ratios were arrived from a combination of alteration processes, neosynthesis, differential migration and mechanical reworking (Magnien, 1961) and therefore gives indication of a presence of laterite rather than in absolute terms to define a laterite soil. As the controversies continued, studies based on chemical and agronomical criteria increased during the period from 1928 and beyond aftermath of the Second World War. This led to establishing types of laterite from Buchanan‟s perspective and in the sense of tropical soils with a $SiO_2/Al_2O_3$ ratio narrower than 2. Kellogg (1949) proposed a criterion based on chemical and physical concepts classified as laterite in four principal forms of sesquioxide-rich material which either harden in situ or is capable of hardening on exposure to the air. The four principal forms are;

a. soft mottled clays that change irreversibly to hard pans or crust when exposed;
b. cellular and mottle hardpans and crusts;
c. concretionary and nodules in a matrix of unconsolidated material; and
d. Consolidated masses of concretion or nodules.

### 2.3.2 Definition Based on Morphology

Further work on the definition from the angle of morphology by Walther (1915), Walther (1916) and followed by Harrassowitz (1930) proposed that, when laterization occurs, as physico-chemical soil forming process converts soil or rock which associates laterite formation with characteristic soil profile development under the tropical savannah, from a fresh zone through alteration to kaolinite than to a lateritic bed, before emerging on the surface zone as ferruginous incrustations and concretion (laterite). This was in line with further work by Pendleton and Sharasuvana (1946) as a profile from an immature horizon, which will develop to a true laterite, if appropriate conditions prevail long enough.

The definitions based on morphological, chemical and physical concepts as proposed by various researchers were summarized by Alexander and Cady (1962) as follows:

“Laterite is therefore defined as a highly weathered material (morphology), rich in secondary oxides of iron, aluminium, quartz and kaolinite or both. It is nearly void of bases of primary silicates but contain quartz and kaolinite (chemical). It either hardens or is capable of hardening on exposure to wetting and drying. Laterite may have hardened either partially or extensively into pisolitic (gravel-like or rock-like masses), it may also have cemented other materials into rock-like aggregates.”

These previous studies have revealed that the laterite soil formation is a chemical weathering process, which involves transformation of primary rock minerals into materials rich in Fe, Al, Ti and Mn. Using the term laterite loosely, may therefore encompass a large group of self-hardening or inclusion of hardened laterite rock or laterite gravel.

The chemical and morphological characteristics of laterite, however, are also well observed in clay soils or material and therefore cannot be restricted to laterites. The extreme heterogeneity of the description of the material confirms the diversity of its occurrence and defying any satisfactory geological, chemical and pedological definition. However, indurations characteristics are a preserve of laterites; that is, its ability to harden under suitable conditions.

Many of the definitions of laterite and lateritic soils are completely independent of its textural and behavioural classification and therefore very challenging to establish its engineering properties from its definition alone.

A compilation by Townsend et al (1976) of laterite terminology as used in different countries is summarized in Table 2.3. This terminology indicates that "laterite" may refer basically to a hard, massive, consolidated crust or individual concretions of hard aggregate.

Further work by Charman (1988) proposed a definition of laterite soils suitable to road engineering work encompassing all forms of highly-weathered natural material formed by the concentration of the hydrated oxides of iron or aluminium. This concentration may be by residual accumulation or by solution, movement and chemical precipitation. In all cases, it is the result of secondary physico-chemical process and not of the normal primary processes of sedimentation, metamorphism, volcanism or plutonism. The accumulated hydrated oxides are sufficiently concentrated to affect the character of the deposit in which they occur. They may be present alone in an unhardened soil, as a hardened layer, or as a constituent such as concretionary nodule in a soil matrix enclosing other materials.

From the literature reviewed, the controversies over a universally accepted definition, expand over more than a century and one half from work done by Maignien (1966) . Therefore, any accepted definition will then have to be supported by some classification and identification systems specifically directed for the intended use based on regional and local knowledge. For engineering purposes, the systems must take into account in-situ appearance; systematic description which allows the ready identification of those laterites which are most suitable for a specific use such as road pavement construction.

Table 2.3 – Definition from Different Countries

| Source | Term | Laterite Terminology | | Nomenclature Used |
|---|---|---|---|---|
| Angola (Novais-Feerira, 1963) | Laterite | Natural concretionary material, pisolitic, or consisting of a crust, vesicular, composed essentially of ferric and aluminium oxides | | Laterite Rock or gravel |
| Thailand (Vallerga & Rananandan, 1969) | Laterite | A hardened material formed by primary weathering or secondary enrichment and cementation. Occurs as a hardened aggregate in a combination of four principal form: (a) consolidated pistolic mass, (b) unconsolidated concretions in a soil matrix, c) consolidated vesicular mass, (d) cemented pre-existing materials in pisolitic of vesicular structure | [a] | Laterite Rock |
| | | | [b] | Laterite Gravel |
| | | | [c] | Laterite Rock |
| | | | [d] | Laterite Rock or gravel |
| British (Kellog, 1949) | Laterite | Materials that are rich in iron and aluminium oxides and poor in bases and combined silica and ore. Either hard or will harden upon exposure. Four principal forms a. soft mottled clays which change irreversibly to hardpans and crust, b. cellular and mottled hardpans and crust, c. concretions in a matrix of unconsolidated material, d. consolidated masses of concretions or nodules | [a] | Laterite Rock |
| | | | [b] | Laterite Rock |
| | | | [c] | Laterite Rock |
| | | | [d] | Laterite Rock |
| Australia (Grant & Aitchison, 1971) | Ferricrete | Breccia or conglomerate-like material in which rock fragments are cemented in a matrix of iron oxides: iron oxides may occur alone, i.e. ferricrete rock, or as a matrix, ferruginous breccia | | Laterite Rock |
| Sierra Leone (Martin & Doyne, 1927) | Laterite Soil | Soil with an $S{:}O_2/R_2O_3$ ratio less than 1.33. Self hardening on exposure | | Laterite Soil |
| Vietnam (USAE School) | Pellet Laterite | Consists of fine grained soils highly iron cemented into pelletized particles: pellets sometimes loosely cemented to for a conglomerate rock or found as uncemented gravelly soil with a high percentage of fines (do not confuse with tropical red gravels). | | Laterite Gravel |
| Angola (Novais-Feerira, 1963) | Laterite (Term Laterite Gravel is used) | Isolated pisolitic concretions of variable strength accompanied | | Laterite Gravel |
| Ghana (DeGraft-Johnson et al., 1972) | Lateritic Gravel | Nodules or concretions in either a consolidated or unconsolidated matrix | | Laterite Rock or gravel |
| Thailand (Vallerga & Rananandan, 1969) | Lateritic (Term Laterite Gravel is used) | Hardened material either as consolidated or unconsolidated concretions | | Laterite Rock or gravel |
| Britain (Kellog, 1949) | Laterite Gravel | Concretions or nodules in a matrix of unconsolidated material | | Laterite Gravel |
| Angola (Novais-Feerira, 1963) | Lateritic Terrain | Earth with a significant amount of laterite; significant amount denotes a quantity which makes the earth behave as a laterite; term applied to a. lateritic soils and also b. materials in a horizon containing laterite lateritic soil; in the case b. being a pisolitic laterite, material designated lateritic gravel | | Laterite Gravel |

## 2.4 Classification of Laterite Soils

The main causes of the poor selection and utilisation of tropical soils is lack of adequate knowledge about the origin, nature, distribution as well as physico-chemical and engineering properties of these soils. The poor field performance is due to lack of a suitable engineering classification system which takes account not only of the soil properties but also the weathering system (associated with a particular environment such as climate, vegetation, parent material, topography, and drainage conditions) and the degree of laterisation (clay mineral coating).The literature on classification of laterite is mainly on agriculture soils through the study of pedology and some attempts by previous researchers to classify laterite for various engineering works and not particularly for road pavement construction

From engineering interest, hardness is an indication of strength and therefore appreciates a definition based on hardness. However, understanding the mineralogy and chemistry will lead to how to handle laterite to maintain its strength integrity. The morphology helps to classify good from bad laterite. Laterites are granular soils which have properties of self-hardening due to the presence of iron, but internal chemical alteration can undermine the strength properties. The Morphological characteristics give an indication of their maturity in terms of strength. The stage of maturity (degree of concretionary developed) governs the engineering performance of laterite. The level of maturity is reflected in the appearance. Weinert (1980) modified work done on classification of calcrete by Netterberg (1969) to develop a scheme appropriate for classifying laterite from its appearance. Weinert‟s maturity classification is shown in Table 2.4.

Table 2.4 - Recommended Classification System for Laterite (Charman, 1988)

| Age | Recommended Name | Characteristic | Equivalent Terms In The Literature |
|---|---|---|---|
| Immature (young) | PLINTHITE | Soil fabric containing significant amount of Laterite material. Hydrated oxides present at expense of some soil material. Unhardened, no modules present, but may be slight evidence of concretionary development. | Plinthite, Laterite, lateritic clay |
| | NODULAR LATERITE | Distinct hard concretionary nodules present as separate particles. | Laterite gravel, ironstone gravel, pisolitic gravel, concretionary gravel |
| | HONEYCOMB LATERITE | Concretions have coalesced to form a porous structure which may be filled with soil material. | Vesicular Laterite, pisolitic ironstone, vermicular ironstone, cellular ironstone, spaced pisolitic laterite |
| Mature (old) | HARDPAN LATERITE | Indurated Laterite layer, massive and tough. | Ferricrete, ironstone, Laterite crust, vermiform Laterite, packed pisolitic laterite |
| | SECONDARY LATERITE | May be nodular, honeycomb or hardpan, but is result of erosion of pre-existing layer and may display brecciated appearance. | |

Note: The recommended names should be used as qualifying terms after a normal soil or rock description and do not replace the need for a full textural, strength and colour description in accordance with recommended practice, e.g.

i. Soft yellowish-brown slightly sandy clay with occasional concretionary zones (up to 10mm in dia.) hard to very weak material (PLINTHITE).
ii. Weak or moderately weak reddish-brown well-cemented porous textured medium gravel sized concretionary HONEYCOMB LATERITE.

### 2.4.1 Engineering Classification of Laterite Soils

The oldest engineering soil classification is based on texture or particle size distribution. The behaviour of any given soil for engineering work is dependent on the constituent (clay, silt, gravel and cobbles) of the soil; hence, the need to carry out grading to determine the various fractions of these constituents and their influence on the behaviour of the soil when in use as road pavement material.

The second classification concept is soil plasticity introduced by Atterberg (Gidigasu 1971) to supplement the textural classification. The plasticity chart was introduced by Casagrande in 1948 and is perhaps the most widely used method classifying soils.

These two criteria are the basis for the development of the Unified Soil Classification System used universally for the classification of temperate zone soils. Temperate soils can easily be classified based on the knowledge of their particle size distribution and plasticity characteristics alone, leading to inferences of the engineering properties and the field behaviour based upon the characteristics of other soils of similar classification. It has also made it possible to exercise very good control of the field performance of temperate materials based on the knowledge of the laboratory test results.

Currently there is no universally accepted classification system for tropically-weathered soils for use by engineers working in these areas of the world. The classification work is confined to one of the important group of tropically weathered soils commonly known as laterite soils. Literature review of classification of laterite soils is based on two main criteria:

1. The first attempt in the classification of laterite soils for engineering purposes was by Buchanan (1807) which was based on the physical property of in-situ hardening.

2. The second criterion was based on the relative content of the so called laterite constituents (Fe, Al, Ti and Mn) in relation to silica (see Table 2.2).

Clare (1957) suggested a typical example of tropically weathered soils according to the parent material and the mode of formation. Soils developed over the same parent rock but under different climatic, vegetation and drainage conditions will possess quite different engineering properties.

Clare and Beaven (1962) concluded that road maintenance problems encountered in the subgrade were closely linked with topographical distribution of soils in Nigeria. Field observations in Ghana have also revealed some correlation between behaviour of the road pavement and local topography (Gidigasu, 1975).

Classification based on climate and vegetation was attempted by Remillon (1967) when soils in French West Africa were divided into ferrallitic and ferruginous (see Table 2.5). Ferruginous soils are formed in dry tropical areas with average rainfall less than 1200 mm per annum under savannah vegetation where evaporation exceeds rainfall and dry season extends over a period of more than 8 months. In humid rain forest areas where rainfall is in excess of 1200mm, precipitation exceeds evaporation and annual dry season last less the 4 months, Ferrallitic soils are formed.

Table 2.5 - Ferrallitic and Ferruginous soils (Remillon, 1967)

| **Climate** | **Ferruginous Soils** | **Ferrallitic Soils** |
|---|---|---|
| | Tropical Dry Zones | Tropical Wet Zones |
| Average Annual Rainfall in (mm) | <1200 | >1200 |
| Hydrological Balance<br>Evaporation (E) – Rainfall (R) | E>R | E<R |
| **Season**<br>Duration of Dry Season in Months | >8 | <4 |
| **Vegetation** | Savannah | Forest |

It is important to consider the degree of weathering as the levels of maturity also contribute in the classification of laterite at each stage in the weathering process exhibiting different engineering properties. Little (1969) proposed a classification system for tropical residual soils based on the degree of weathering. Grading laterite based on level of decomposition from fresh rock to soil with intermediate degrees of weathering ranging from slightly, moderate highly and completely and giving the engineering properties of each grade.

Though there are differences in the classification of its occurrences, types of laterite form depend on climatic factors. Generally high rainfall and temperature; implying either forest or savannah vegetation types, have been found to favour the process of laterisation.

Studies of tropically weathered soils revealed that due to their mode of formation, the physico-chemical and engineering properties differ considerably from soil of similar texture developed under temperate climate. Consequently highway and airfield soil classification systems in use in temperate countries have not found useful application in many tropical countries. In spite of strict adherence to ASTM (American Society for Testing and Materials) grading and plasticity specification and the universality of these classification systems, it is now realized that soil behaviour in the field does not depend on the particle-size distribution and plasticity parameters alone in the case of tropical soils but other factors must be taken into account in the selection of these soils.

Factors such as the geological history, morphological characteristics, genesis, clay mineral type, nature of ion exchanges and actual moisture condition when considered would enhance the use of existing classification systems in the selection of tropical soils. In order to address the engineering classification challenges of laterite soils other than particle-size distribution and plasticity parameters, the formation of laterite soils must be reviewed to better appreciate the role of factors such as the geological history, morphological characteristics, genesis, clay mineral type, nature of ion exchanges and actual moisture condition.

## 2.5 Formation of Laterite Soils

The formation of laterite soil does not differ in any way from the generally known principles of soil formation. The factors affecting tropical soil development equally apply to laterite soil development. The most important factors that control soil formation are *parent material, time, climate, plants* and *animals*, and *slope*. All soils are products of weathering and are directly dependent on the climate of the area.

Different soils are likely to form from the same parent material under different climatic conditions. Other factors, which would contribute to the differences, include nature of the vegetation, the slopes, and the length of time the soils have been forming. Similar soils from different parent materials would result if the above named factors were essentially the same in each situation. Climate is the most important factor in soil formation. Temperature and moisture abundance largely control most of the other weathering variables.

Laterite weathering essentially involves chemical and physic-chemical alteration and/or transformation of primary rock-forming minerals into materials rich mainly in 1:1 lattice clay minerals and its constituents (Fe, Al, Mn and Ti).

Three main stages of laterite soils formation have been identified:-

1. The first stage is the physical and chemical breakdown of rock forming minerals and release of the primary constituent elements ($SiO_2$, $Al_2O_3$, $Fe_2O_3$, CaO, MgO, $K_2O$, and NaO), which appear in ionic forms.

2. The second stage of the weathering process is the leaching of the combined silica and the bases under appropriate drainage conditions leading to the accumulation or enrichment from outside sources, oxides and hydroxides of sesquioxides. The removal of other constituent and the subsequent accumulation of residual soil material as a result of the removal of other constituents. In other cases sesquioxides which have been mobilized may come from the outside soil. Iron for example becomes mobile in two ways; iron in the ferric state ($Fe_2O_3$) is relatively immobile. In situations where drainage is impeded, the soil water fills most or all the soil pores, leading to shortage of soil air. Microorganisms requiring oxygen may be forced to gain oxygen from the $Fe_2O_3$ by reduction (release of oxygen) from the trivalent form to divalent (FeO) which is the ferrous. Ferrous is mobile and may move in the soil solution until it is oxidized to the trivalent state.

3. The third stage, involves partial or complete dehydration of hydroxides of Fe, Mg and Al of the sesquioxide and secondary minerals which lead to hardening. The term secondary mineral is used to include those which have crystallized in situ from atoms and ions not removed by the weathering processes; that is, the residual derivative minerals. The clay minerals and the oxides and hydroxides of the stable elements, namely aluminium, ferric iron and titanium, are the most important in laterite formation. Figure 2.3 depicts a fresh rock through the various stages of weathering to produce residual laterite soil.

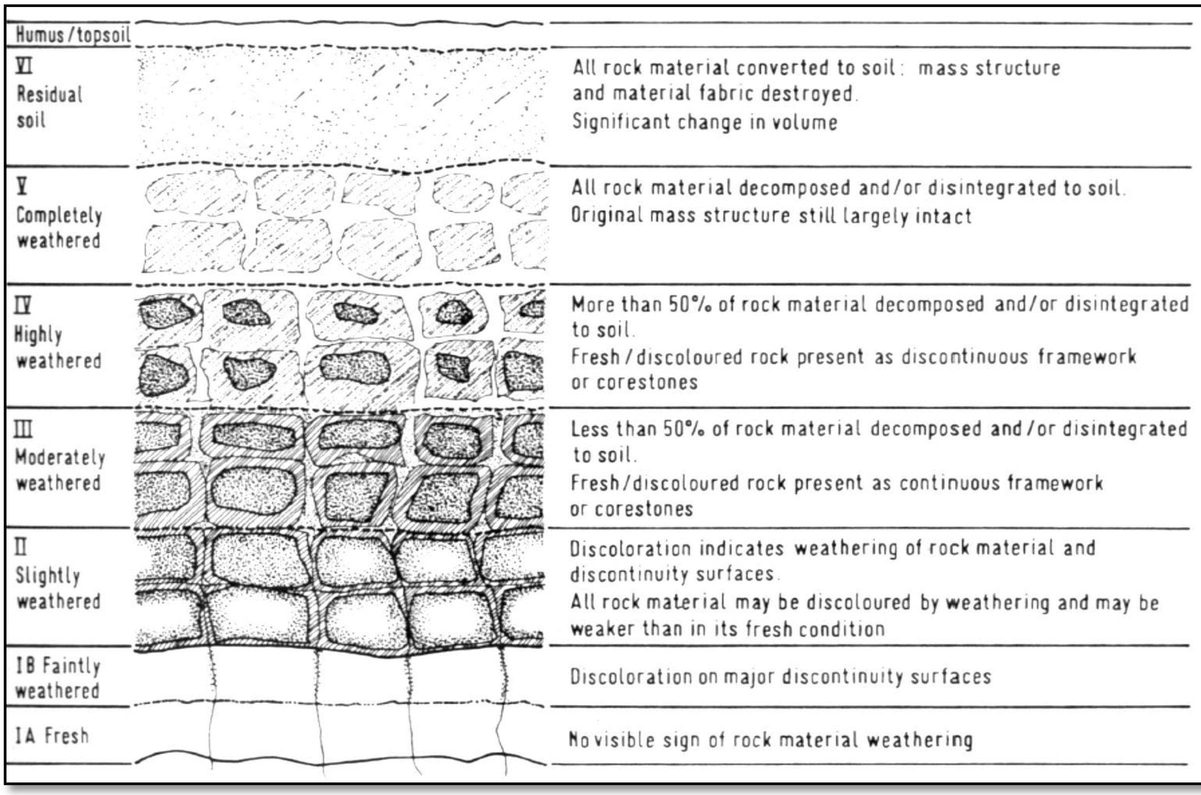



Figure 2.3 - Schematic representation of tropical weathering profiles (Charman, 1988)

From the review so far, it is clear that there is the need to consider various aspects of the material to enable better understanding of the properties of laterite soils. The climatic conditions under which they are formed are key variables for the differentiation and classification of laterite soils. Various definitions are influenced by the climatic conditions under which they are formed. Even though there is enough evidence that laterites are the preserve of tropical and sub-tropical countries. Work done by Townsend et al.(1976) on the geotechnical properties of laterites occurring in different tropical and sub-tropical climatic regions through field examinations, sample collections, and observations of engineering methodology in Brazil, Ghana, Angola, Australia, Thailand and Georgia (USA) among others, concluded, that the common terminology used to describe laterites, as weathered soils rich in secondary oxides of iron and aluminum, possibly containing quartz and kaolinite, and with the property of hardening, was found to be inappropriate. This leads to the need to localize the definition based on regional climatic conditions. This revolution should start from specific country experiences and then to countries with same climatic conditions.

From the review it has also been established that same parent material under different climatic conditions will evolve soils with different soil properties presenting engineering classification challenges. Therefore, laterite soils of similar appearance will exhibit different engineering characteristics when used in pavement construction. Due to the complexity of the definition, work on engineering classification has not advanced at both the regional and global levels. However, most of the works done in laterite soil classification under pedology have established a good and common basis to identify various types of indurated occurrences with respect to the relations of soil-environmental factors, climate, relief and vegetation. It has also enabled differentiation of the term laterite soils for agriculture and engineering purposes.

The classification of laterite should first be based on experiences in a given climatic zone and must take into account factors such as the geological history, morphological characteristics, genesis, clay mineral type, nature of ion exchanges and the actual moisture condition.

### 2.5.1 Relation with Environmental Factors

A number of previous studies have established the significant role environmental factors play in formation of laterite soils and useful summary of this work is given by Maignien (1966) as shown in Table 2.6:

Table 2.6 – Studies done on the role of environmental factors in laterite soil formation

| **Researcher** | **Environmental Facts** |
|---|---|
| Richtofen (1886) and Lacroix (1913) | Vegetation |
| Holland (1903) | Bacteria activity |
| Harrison (1910) and Campbell (1917) | Ground-water |
| Maclaren (1906) and Lacroix (1913) | Climate (dry and wet seasons) |
| Lacroix (1913) | Topography |

### 2.5.2 Climatic Factors

Two climatic factors that appear to influence the distribution of laterite soils immensely are temperature and rainfall. Research and studies carried out by Crowther (1930) established that $SiO_2/Al_2O_3$ ratios increase as temperature rises when humidity is constant. Maignien (1966) established that laterites in recent times develop around at about temperature of 25°C but studies in high plateaus of Madagascar established an extremely deep laterite profile at 18-20°C. This is unique since there are other regions with mean temperatures of 18-20°C yet there is no laterisation.

Observations in West Africa by Maignien (1966) established that the limit for laterite formation lies approximately towards the 1200mm isohyets. However, some formation is at levels as high as 950-1000mm. His Studies showed that laterite and its derivatives develop under annual rainfall of between 1100-6000mm depending on, to a large extent, the nature of the parent material. In West Africa, laterisation can occur on basic rocks with precipitation of 1100mm whereas on quartz rich granites, this limit is raised to 1250mm-1300mm annually.

As a tropical country, temperatures in Ghana show very little seasonal variation and range between 26.1°C and 28.8°C (Meteorological Services Agency). The average annual total of rainfall is between 1111mm and 1944mm (see Table 2.7). The higher the rainfall, the greater the leaching effect, which removes free silica, reduces the silica/sesquioxide ratio and therefore increases the proportion of gibbsite.

Table 2.7 - Ranges of temperature and rainfall values for laterite formation

| Climatic Factor | Range Conducive For Laterite Formation | Africa | | West Africa | | Ghana | |
|---|---|---|---|---|---|---|---|
| | | Lower | Upper | Lower | Upper | Lower | Upper |
| Temperature °C | Approx. 25* | | | | | 26.1 | 28.8 |
| Annual Rainfall (mm) | 1100 – 6000 | 950 – 1000 | 1200 | 1100 | 1250 – 1300 | 1111 | 1940 |

*This is for most of the laterite formations except in Madagascar where laterites form around 18 - 20°C.

There are therefore three climatic indices which are generally favourable for laterite formation as summarized in Table 2.8.

Table 2.8 - Climatic Indices

<table>
<tr><th colspan="2">Climatic Indices</th><th>Indices</th><th colspan="2">Ghana Indices</th></tr>
<tr><td colspan="2" rowspan="2">Temperature</td><td rowspan="2">Mean 25°C</td><td>Lower limit</td><td>26.1°C</td></tr>
<tr><td>Upper limit</td><td>28.8°C</td></tr>
<tr><td rowspan="2">Rainfall</td><td>Lower limit</td><td>750mm</td><td colspan="2">1111mm</td></tr>
<tr><td>Upper limit</td><td>1500mm</td><td colspan="2">1940mm</td></tr>
<tr><td colspan="2">Seasonality</td><td>Wet and Dry Periods</td><td colspan="2">4 – 6 month = Wet<br>6 – 8 month = Dry</td></tr>
</table>

### 2.5.3 Topography and Drainage Conditions

The topography controls the amount of water at the various areas of the soil chain. The amount of water available at the various topographic sites defines the drainage characteristics and the depth of weathering for a given soil type. The same parent material that forms a slope in an undulating area will give rise to a soil catena (see Figure 2.4), or a chain of related soils, whose individual characteristics will depend on the particular portions of the slope on which they develop. Thus, on a slope extending from hill top to valley bottom, the soils will show different characteristics of colour, depth, texture, water content, etc. progressing from the top to the bottom of the slope (Alexander and Cady, 1962).

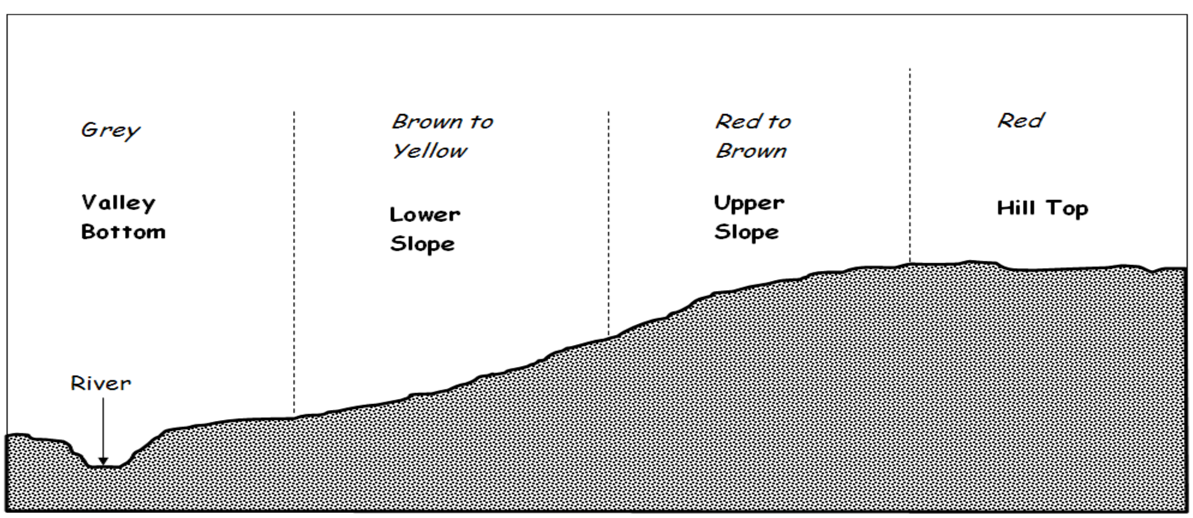



Figure 2.4 - Typical Soil Chain (Alexander and Cady, 1962)

At the uplands of the soil catena the slopes are generally steep leading to greater run-offs, less infiltration and erosion is active. Most of summit soils in West Africa are reddish or reddish brown or brownish red. It denotes the presence of ferric oxide ($Fe_2O_3$) or haematite in the soil. The degree of weathering is not deep.

In the lower slope, drainage is relatively slower than the upland areas. This section of the catena receives soil moisture seeping down-slope from upper soils. Retention of moisture is higher, resulting in an increasing degree of hydration of iron in the soil. The characteristic colour is brown or yellow. The hydrated iron oxides in these soils are mainly goethite ($Fe_2O_3.H_2O$) and limonite ($Fe_2O_3$. $1\frac{1}{2}H_2O$). Limited erosion takes place in this section and long uninterrupted periods of weathering can occur, producing deep soil profiles.

Valley bottoms tend to be relatively flat and the soils have low permeability, resulting in poor drainage. This results in waterlogged ground in the wet season and under these conditions bacteria are forced to derive their oxygen from oxygen-containing compounds by reduction. As a consequence over time the soils become bluish grey, greenish grey and neutral grey, the typical colours of laterite soils. In the dry season, the valley bottom experiences a fluctuating ground water table, leading to the movement of soluble products resulting in the concentration of the minerals of laterite in certain horizons.

## 2.6 Formation and General Characteristics of Laterites and Lateritic Soils of Ghana

Even though laterite and lateritic soils have the same development characteristics, the developments of laterite are conditioned on very wide range of factors. These factors come together to contribute to laterite soil formation in Ghana. These factors and their importance to laterite formation in Ghana are discussed in this section.

The genesis of laterite formation in Ghana is from the parent material and the weathering systems dividing the country into three main vegetation zones; namely Woodland Savannah (Ws), Coastal Savannah (Wc) and the Forest (F). The predominant and most important rocks of these zones are presented in Table 2.9.

Table 2.9 – Vegetational Zones and Their Predominant Rock Types

<table>
<tr><th colspan="2">Vegetation Zone</th><th>Important Rock Types</th><th>Average Temperature °C</th><th>Annual Rainfall mm</th><th>Relative Humidity at 9 hours %</th></tr>
<tr><td rowspan="2">Woodland Savannah</td><td>Guinea Savannah</td><td rowspan="2">Granite Phyllite Sandstone Shale Mudstone</td><td rowspan="2">21.7 – 28.0</td><td rowspan="2">1016 – 1397</td><td rowspan="2">64.7</td></tr>
<tr><td>Sudan Savannah</td></tr>
<tr><td rowspan="2">Forest</td><td>Rain Forest</td><td rowspan="2">Granite Phyllite</td><td rowspan="2">21.2 – 31.6</td><td>1651</td><td>82.2</td></tr>
<tr><td>Semi-Deciduous Forest</td><td>889 – 1651</td><td>84.4</td></tr>
<tr><td rowspan="2">Coastal Savannah</td><td>Coastal Thicket</td><td rowspan="2">Gneiss</td><td rowspan="2">22.4 – 29.7</td><td rowspan="2">1016</td><td rowspan="2">79.6</td></tr>
<tr><td>Coastal Savannah</td></tr>
</table>

### 2.6.1 Geology of Ghana

The types of mineral content of the parent rock greatly affect the nature and properties of the laterite soil formed out of weathering. A study of the geology of Ghana gives better appreciation of nature and behaviour of laterites formed out of different parent rocks.

A simplified geological map of Ghana is shown in Figure 2.5. The more common geological formations associated with gravel formations in Ghana can be divided into the following four groups:

1. Acid igneous (AI) – Granite, Quartzite;
2. Basic igneous (BI) – Basalt , Gabbro;
3. Metamorphic (Met) – Shale, Phyllite, Gneiss, Schist; and
4. Sedimentary (Sed) – Sandstone, Limestone. (Ahn, 1970)

From the perspective of vegetation, the most important rocks are respectively gneisses in the coastal savannah zone, granites and phyllites in the forest zone and granites, sandstones and shales in the woodland savannah zone.

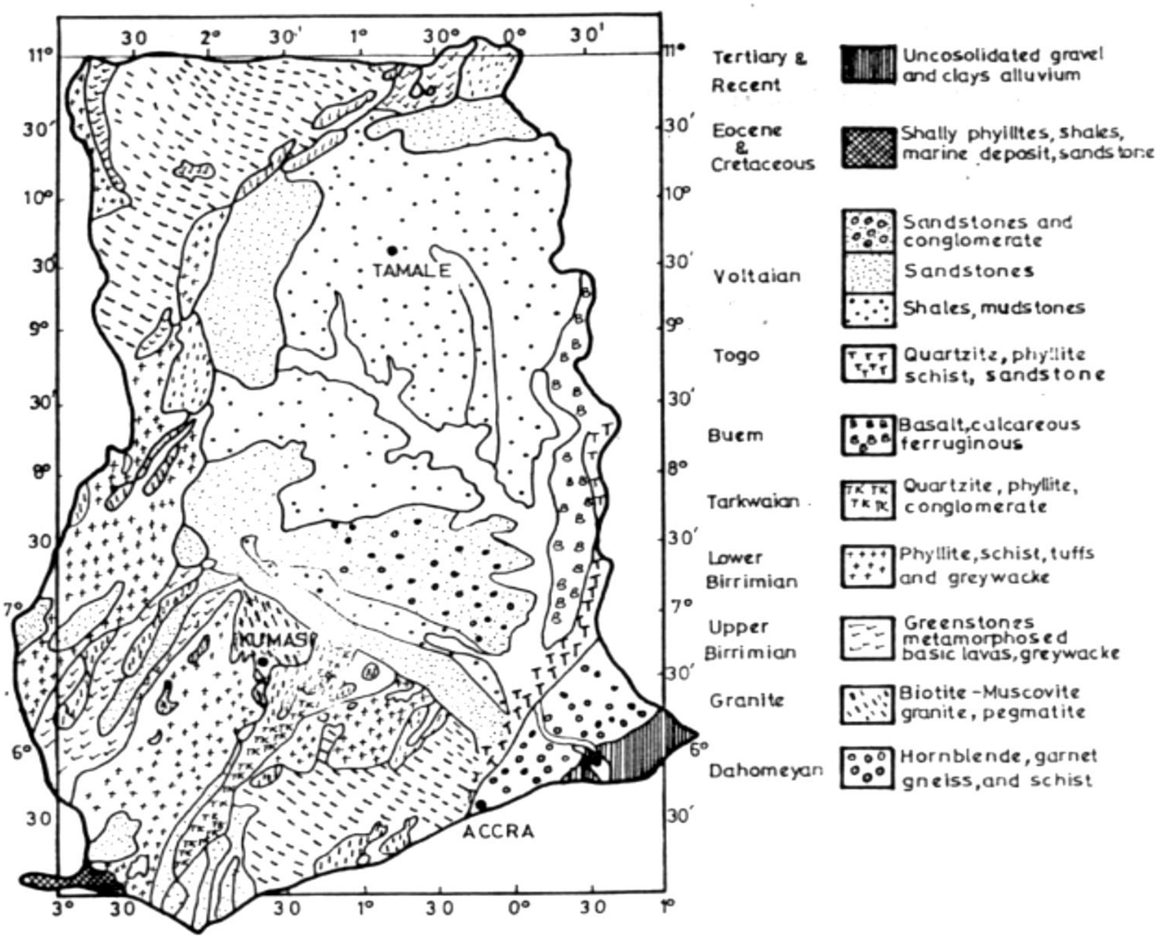



Figure 2.5 - Simplified geological map of Ghana (Bates, 1972)

### 2.6.2 Chemical Evaluation

Chemical analysis carried out by Bates, (1962) on the various rock systems and series in Ghana are summarized in Table 2.10. The summary is based on major constituents which have greater influence on the nature and description of these rocks using the percentage of silica as basis to further classify these formations.

The analysis gives an indication of the state of these rocks and weathering characteristics of these major rock formations and their derivatives. Most of the formation series in Ghana are basic which set a good platform for weathering and subsequent development of laterite soil formation.

Table 2.10 - Geological Formations In Ghana And The Mineral Content Of Their Rock Types (Bates, 1962)

| Formation | Types | % Silica | Rock Type |
|---|---|---|---|
| Dahomeyan | Granite | 43.51 | Ultra Basic |
| | Travertine | 13.42 | |
| Birrimian | Phyllites | 61.18 | Intermediate |
| | Greywacke | 58.93 | |
| | Lava | 50.31 | Basic |
| | Granite | 72.59 | Acidic |
| Tarkwaian | Epidiorite | 47.50 | Basic |
| | Phyllite | 55.51 | Intermediate |
| Buem | Dolerite | 52.50 | Basic |
| | Basalt | 50.52 | |
| | Tuff | 39.46 | Ultra Basic |
| | Limestone | 10.38 | |
| Voltaian | Shale | 65.50 | Acidic |
| | Sandstone | 51.96 | Intermediate |
| | Limestone | 11.64 | Ultra Basic |

### 2.6.3 Geomorphology of Ghana

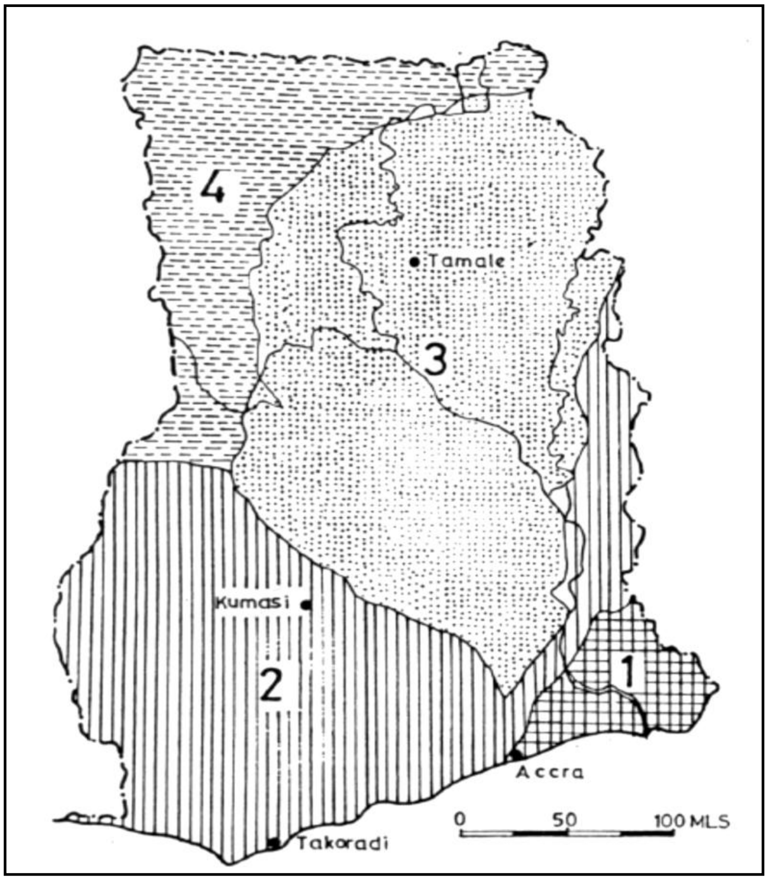
Emanating from the geology is the associated landforms (geomorphology) and in particular of their nature, origin, processes of development and their material composition.

The geomorphology and physical features depend mainly on the geology; most of the hills and ranges consist of hard resistant rocks such as quartzites, whereas the valleys and lower grounds are carved out of softer rocks such as shales, sandstones, phyllites and schists. (Gidigasu 1970)

Geomorphologically, Ghana has been divided into four zones (see Figure 2.6) reflecting common areas of geological and vegetation influence in the landform development. Gidigasu (1980) described these four main zones from the view point of rock type as follows.

1. The Voltaian basin is occupied by the sedimentary formations (**Voltaian formations**) which comprise mainly sandstones, conglomerates, shales, mudstones, limestones, and travertine, etc.

2. The entire south, and all along the western section and extending to the northern region of the country is occupied by the argillaceous metamorphic rocks and volcanics (**the Birrimians**) mainly in the form of phyllites, schists, greenstones, greywackes, and gondites etc. they were later intruded by masses of acidic and basic crystalline rocks such as granites of various kinds, granodiorites, porphyrites, pegmatites, syenites and diorites.




- The Accra-Ho-Keta Plains
- The Forest Zone (The Area of Intermediate Plateau)
- The Voltaian Basin
- The Area of Crystalline Rocks to the north and west of the

Figure 2.6, Map Showing the Geomorphological Regions in Ghana (Gidigasu, 1980)

3. The South-eastern corner of the country is occupied by the so-called **Dahomeyan formations** which comprise mainly schists of various kinds (mica, quartz, and quartz-

mica), amphibole, and para and ortho hornblendic gneisses many of which are garnetiferous.

4. Along the eastern border with Togo are the **complex rock formations** comprising mainly quartzite, phyllite, sandstones, schists, basalt, calcareous sandy and ferruginous shales, etc. the characteristics of these rock systems have pronounced influence of the nature of the soils formed residually over them.

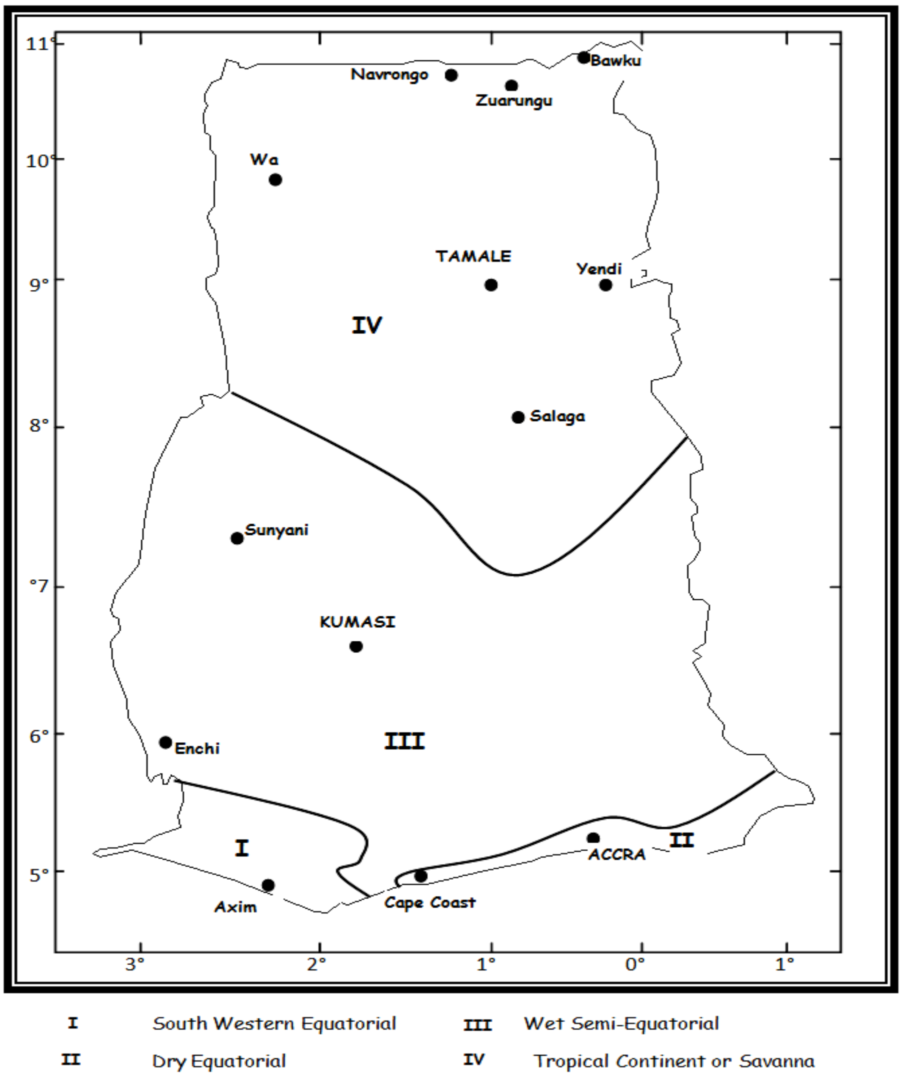



Figure 2.7 - Main climatic zones of Ghana (Dickson and Benneh, 1988)

Based on the rainfall pattern, a correlation has been established with the vegetation types to derive the climatic zones of Ghana (Dickson and Benneh 1988). Ghana is divided into six main vegetation zones (see Figure 2.8) namely;

1. Guinea Savannah
2. Sudan Savannah
3. Moist-Semi Deciduous Forest
4. Rain Forest and
5. Coastal Thicket
6. Coastal Savannah

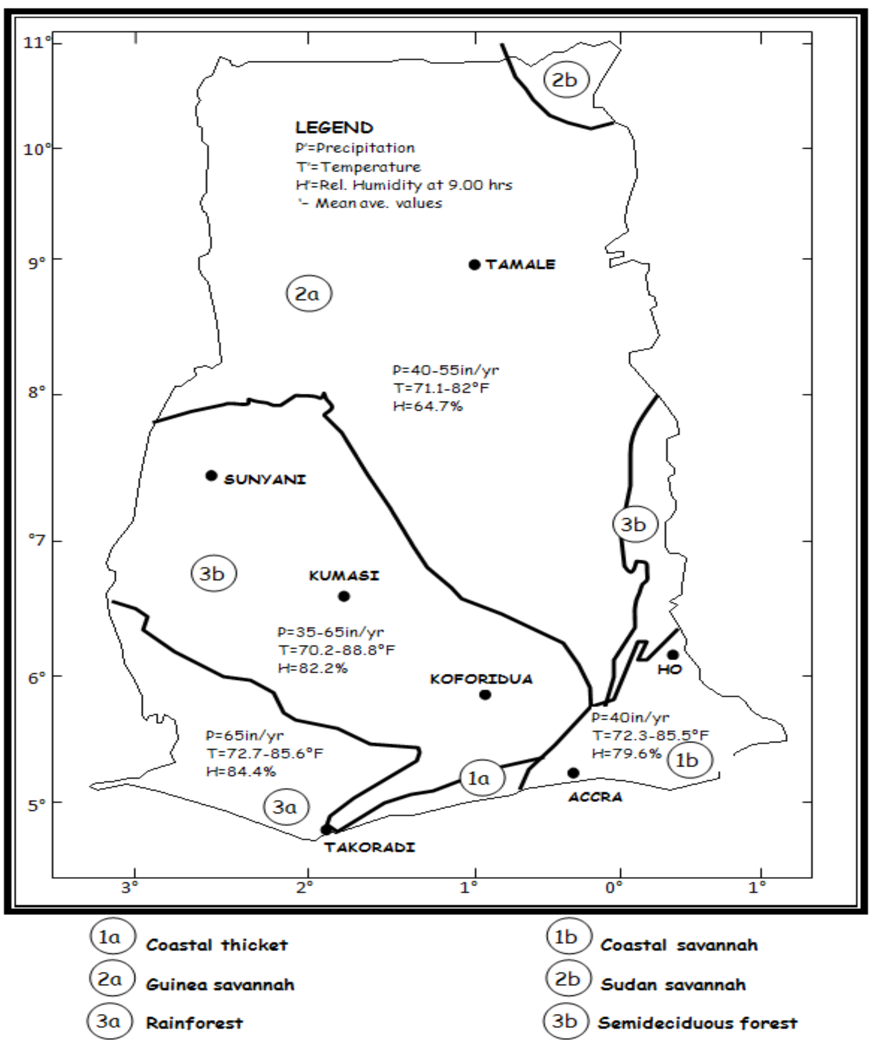



Figure 2.8 - A simplified climatic vegetation map of Ghana (Gidigasu, 1972)

Thornwaite (1948) however proposed another classification of the climatic zones based on moisture index. And from this, the climate of Ghana was classified into four main zones (see Figure 2.9).

The moisture indices for the various climatic zones were obtained from work done by Arulanadan et al (1963). They developed a Moisture Index Map for Ghana using the famous Thornwaite Moisture Index Method (Thornwaite, 1948). The moisture index (see Equation 2.1) is based on rainfall, temperature, vegetation cover, evapotranspiration, and water storage of the soil during the year. In Ghana, distinct wet and dry monthly periods are experienced during the year leading to either water surplus as a result of saturation or soil water deficit due to excessive loss of water and the Moisture Index is a measure of the retention ability of the soil during the year.

$$\text{The Moisture Index, Im} = \frac{100\Sigma D - 60\Sigma d}{\Sigma Ep} \text{………….. Equation 2.1}$$

Where D = monthly water surplus, d = monthly water deficiency (inches) and Ep = monthly potential evapotranspiration (inches)

The derived Moisture Index Map (see Figure 2.8) is fairly consistent with climatic vegetation map and hence justifies the use of the climate and vegetation zones to identify zones of the same soil forming process.

Ghana‟s climatic condition is very conducive for formation of laterite soils through chemical weathering which requires the following as prerequisites:

- Hot Humid Conditions;
- Annual Mean Temperature of 25°C;
- Minimum Annual Rainfall of at least 750 mm; and
- Warm and Wet Periods.

In view of the differences and references used in the definition of the climatic zones of Ghana, Table 2.11 presents the simplified, Moisture Index and Vegetation perspectives. This will ensure consistency in subsequent presentations.

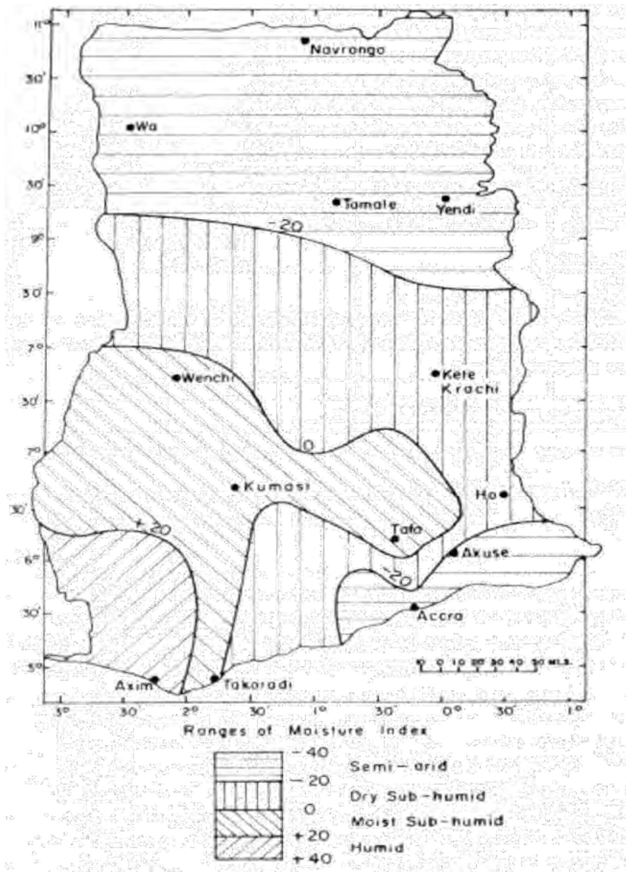



Figure 2.9 - Thornwaite‟s Moisture Index Map of Ghana (Arulanandan et al., 1963)

Table 2.11 – Climatic Zones of Ghana

| **Primary** *(Benneh and Dickson 1988)* | **Moisture Index** **(***Arulanandan et al. (1963)* | **Vegetational** *(Gidigasu 1972)* |
|---|---|---|
| **Tropical - Continental or Savannah** | Dry Sub - Humid | Guinea Savannah |
| | Semi - Arid | |
| | | Sudan Savannah |
| **Wet Semi-Equatorial** | Dry Sub - Humid | Moist-Semi Deciduous Forest |
| | Moist Sub - Humid | |
| **South -Western Equatorial** | Humid | Rain Forest |
| **Dry Equatorial** | Dry Sub - Humid | Coastal Thicket |
| | Semi - Arid | Coastal Savannah |

From the above, it is can be seen that Ghana‟s climatic, environmental and geological conditions are favourable for laterite formation.

### 2.7 Ghana Laterites and Lateritic Soils

Hamilton (1964) divided laterite soils in Ghana into two distinct types as follows:

a. The high level (Primary) laterites; and
b. The low level (Secondary) laterites.

#### 2.7.1 High Level Laterites

High level laterites are formed by normal residual tropical weathering; otherwise called residual or alluvial laterite. They are formed in the uplands because weathering and drying conditions are optimal here. High and exposed positions favour oxidation, dehydration, evaporation and lateral drainage.

#### 2.7.2 Low Level Laterite

They are formed by the filling and saturation of colluvial deposits by iron colloids. The iron colloids come from ferruginous surroundings uplands which flow down the slopes. They are predominantly formed in the lowlands. The supply of iron colloids continues until the whole colluvial mass is totally submerged. The colloids then flocculate, age and crystallize in the same way as High Level laterites. The lowland laterites are dependent on the upland laterite.

### 2.8 Clay Mineralogy

There are two types of primary minerals which are from the nature of the parent rock either acidic or basic. From the Bowen‟s reaction series, stability is a measure of the rock‟s ability to resist weathering. Acidic rocks are more stable and Basic are most unstable. The most stable rock is quartz and least stable is Olivine.

The stability of minerals can be predicted using the Bowen's reaction series. However, in the case of the weathering series this is known as the Goldich Dissolution Series:

Olivine                                        Calcic Plagioclase

Mg Pyroxene                                   Calcic-Alkalic Plagioclase

Mg-Ca Pyroxene          **Increasing stability**          Alkalic-Calcic Plagioclase

Amphibole                                     Alkalic Plagioclase

Biotite                                       Potassium Feldspar

Muscovite

Quartz

Weathering of these rock minerals lead to production of various types of clay minerals. The most common and significant for engineering considerations are kaolinite and montmorillonite.

## 2.9 Soil Distribution

The soils have been grouped into three main zones, namely;

1. **The woodland Savannah Zone:** occupying the Guinea and Sudan Savannah Zones and consists of the ground water lateritic soils, savannah ochrosols and the acid gleisols;

2. **The Forest Zone:** occupying the Moist Semi-Deciduous and Rain Forest Zones and consisting of forest ochrosols, rubrisol ochrosol, lithosols and the oxysols which are found mainly in the Rain Forest Zone; and

3. **The Coastal Savannah Zone:** occupying the Coastal Thicket and Savannah Zones and consisting of the ochrosols, lateritic sandy soils, tropical black clays, tropical grey earths, sodium vleisols and the coastal sandy soils.

Areas around the Accra-Ho-Keta plains are noted for black cotton soils. Black and greyish clays from the coastal and woodland savannah zones contain the expansive clay mineral montmorillonite while the forest zone soils are essentially kaolinitic. Generally upland soils are kaolinitic while the valley soils are montmorillonite and therefore could be potentially expansive though upper slopes would not be. Bhatia (1967) also presented a map showing the distribution of surface soils in Ghana and is shown in Figure 2.10.

The different content of clay mineral determines the water absorption capacity of the soil thus gives an indication of the soils ability to either expand or shrink in the presence or absence of water when in use as road pavement material.

Laterite soils containing high percentage of hydrated halloysite, geolite or gibbsite are known to be problem laterite soils; those containing montmorillonite and illite may have lower strength, high construction pore pressures, high swelling potential, and other undesirable properties that laterite soils with the clay fraction consisting predominantly of kaolinite and chlorite do not have.

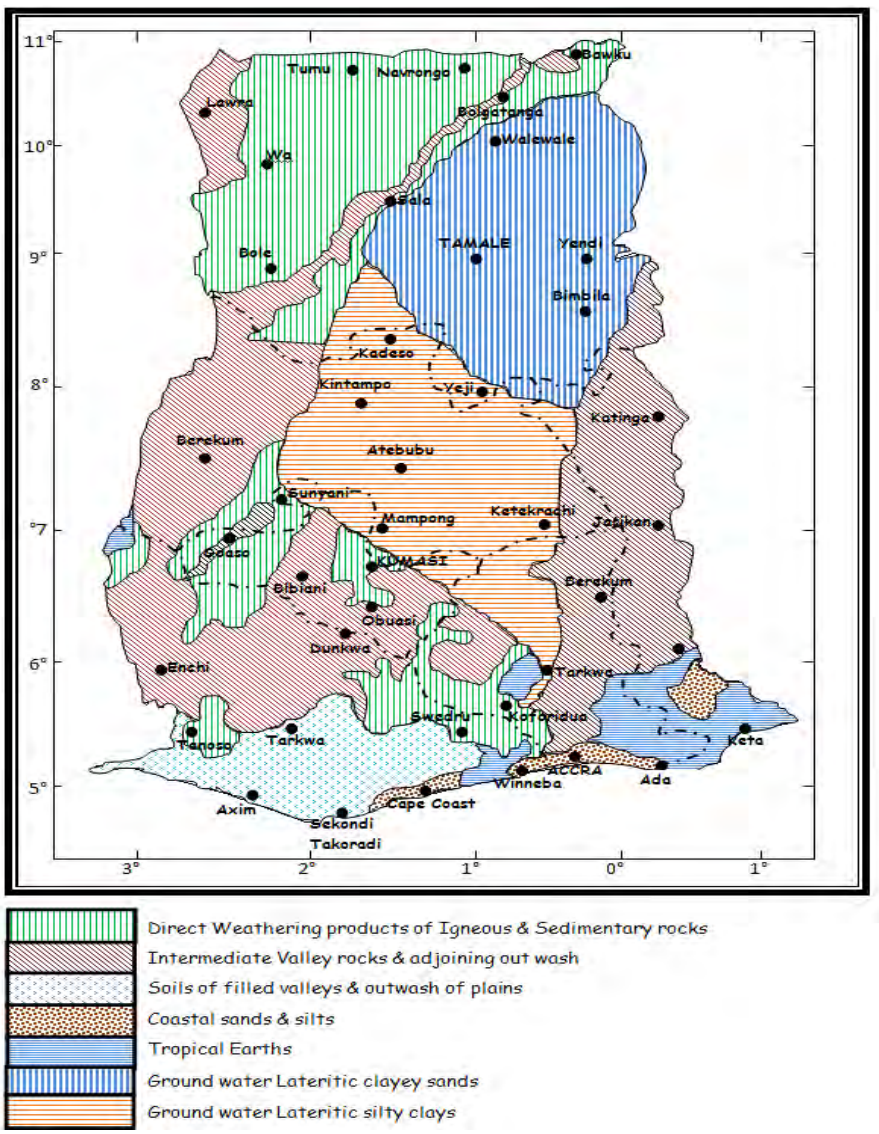



Figure 2.10 - Tentative Surface Soil Engineering Map of Ghana (after Bhatia, 1967)

## 2.10 Classification of Ghana Laterites

Soils formed under similar weathering conditions are known to have similar morphological, mineralogical and geotechnical characteristics (Gidigasu, 1976). Also similar profiles do develop under similar weathering conditions and therefore a useful means of regional description of laterite soils for highway construction would be to group and classify profiles at least, as a first approximation on a regional and local basis according to the similarities or differences among the important soil-forming factors. (USAID, 1971; Gidigasu and Bhatia, 1971)

Gidigasu (1971); Gidigasu and Bhatia (1971), discussed the influence of soil forming factors (climate, vegetation, parent rock, position in relation to local topography), degree of weathering and morphological characteristics on the physico-chemical and geotechnical properties of Ghanaian soils. Since laterite soils have developed under the influence of pedogenic factors, these factors have been found useful in regional identification and classification of laterite soils for engineering purposes.

Table 2.10 summarizes all the geographic and pedological factors that must be known in the determination of the engineering properties of laterite soils in Ghana. Generally coarse in texture, and gravels on basic igneous rocks, such as basalt, gabbro, and dolerite, has a high concentration of calcium-rich feldspar and other minerals are likely to weather quickly to form amorphous hydrous oxide. The absence of quartzite tended to produce plastic materials with fine grading. Such soils had low permeability and high concentration of iron content, which was not readily removed by leaching.

## 2.11 Distribution of Laterite in Ghana

Deposits of various textural groups of laterite materials are associated with characteristic land forms and can be located directly by observation on land surface or indirectly by examination of aerial photographs. Another useful guide for identification of laterite materials in Ghana is based on vegetation. Gidigasu (1972), based on efforts by Morin and Parry (1969); Ruddock (1967); Pedological data evaluated by Brammer (1962) and field studies by Gidigasu (1971) developed a useful guide to identify laterite materials using vegetation and climatic zones as a guide. This is summarized in Tables 2.12 and 2.13.

Table 2.12 – Summary of Factors Classifying Laterites in Ghana (Gidigasu, 1972)

<table>
<tr><th rowspan="2">Climatic Vegetation Zone</th><th rowspan="2">Parent Rock</th><th rowspan="2">Origin - Genetic Type</th><th colspan="2">No. Of Soil Types</th><th rowspan="2">General Topography And Drainage Condition</th><th rowspan="2">Degree Of Leaching And Laterization</th><th colspan="2">Mineral Composition Of The Clay Fraction</th><th rowspan="2">Avg. Temp °C</th><th rowspan="2">Annual Rainfall mm</th><th rowspan="2">Relative Humidity at 9 hours %</th></tr>
<tr><th>Fine Grained Soils</th><th>Gravels/ Gravelly Soils</th><th>Predominant</th><th>Accessory</th></tr>
<tr><td rowspan="4">Forest Zone</td><td rowspan="2">Granite</td><td>Residual</td><td>24</td><td rowspan="2">15</td><td rowspan="2">Gentle undulating to strongly rolling; good drainage</td><td rowspan="2">High</td><td rowspan="2">Kaolinite</td><td rowspan="2">Mica (muscovite)</td><td rowspan="4">21.2 – 31.6</td><td colspan="2">Rain Forest</td></tr>
<tr><td>Non - residual</td><td>5</td><td>1651</td><td>82.2</td></tr>
<tr><td rowspan="2">Phyllite</td><td>Residual</td><td>24</td><td rowspan="2">32</td><td rowspan="2">Lower slopes and valleys; fair to poor drainage</td><td rowspan="2">Low</td><td rowspan="2">Kaolinite</td><td rowspan="2">Mica (muscovite)</td><td colspan="2">Semi-Deciduous Forest</td></tr>
<tr><td>Non - residual</td><td>7</td><td>889 – 1651</td><td>84.4</td></tr>
<tr><td rowspan="2">Woodland Savannah Zone</td><td>Granite, Sandstone</td><td>Residual</td><td>9</td><td rowspan="2">12</td><td>Undulating to gentle rolling; good drainage</td><td>High</td><td>Kaolinite</td><td>Mica (muscovite)</td><td rowspan="2">21.7 – 28.0</td><td rowspan="2">1016 – 1397</td><td rowspan="2">64.7</td></tr>
<tr><td>Shales, Mudstone</td><td>Non - residual</td><td>9</td><td>Very gentle topography to levee; poor drainage</td><td>Low or High</td><td>Kaolinite</td><td>Illite, mica (muscovite)</td></tr>
<tr><td rowspan="2">Coastal Savannah Zone</td><td rowspan="2">Basic and Acidic Gneisis</td><td>Residual</td><td>14</td><td rowspan="2">21</td><td>Mainly lowland levee; depressions</td><td rowspan="2">Low</td><td rowspan="2">Montmorillonite</td><td rowspan="2">Kaolinite vermiculite</td><td rowspan="2">22.4 – 29.7</td><td rowspan="2">1016</td><td rowspan="2">79.6</td></tr>
<tr><td>Non-residual</td><td>9</td><td>Very gently undulating to rolling; poor drainage</td></tr>
</table>

Table 2.13 – Distribution of Soils in Ghana

<table>
<tr><th colspan="2">Climatic zones</th><th>Vegetational zones</th><th>Geomorrphological zones</th><th>Geological formations</th><th>Physiographic regions</th><th colspan="2">Soil groups</th></tr>
<tr><th>(1)</th><th>(2)</th><th>(3)</th><th>(4)</th><th>(5)</th><th>(6)</th><th>(7)</th><th>(8)</th></tr>
<tr><td rowspan="2">Dry Sub Humid</td><td rowspan="6">Tropical - Continental or Savannah</td><td rowspan="4">Guinea Savannah</td><td rowspan="3">Voltaian Basin</td><td>Voltaian Sandstone Conglomerates</td><td>Gambaga Escarpment</td><td rowspan="6">Woodland Savannah</td><td rowspan="4">Groundwater Lateritic Soils</td></tr>
<tr><td>Voltaian Sandstone Shale</td><td rowspan="2">Voltaian Basin</td></tr>
<tr><td rowspan="4">Semi Arid</td><td>Voltaian Shale & Mudstone</td></tr>
<tr><td rowspan="4">Area of Crystalline Rocks</td><td>Tarkwaian</td><td rowspan="3">Savannah High Plains</td></tr>
<tr><td rowspan="2">Sudan Savannah</td><td>Granites</td><td>Savannah Ochrosols</td></tr>
<tr><td>Birrimian</td><td>Acid Gleisols</td></tr>
<tr><td rowspan="2">Dry Sub Humid</td><td rowspan="4">Wet Semi-Equatorial</td><td rowspan="4">Moist-Semi Deciduous Forest</td><td>Phyllites</td><td>Southern Voltaian Plateau</td><td rowspan="5">Forest Zone</td><td>Forest Ochrosols</td></tr>
<tr><td rowspan="7">Forest Zone</td><td>Togo Series</td><td>Akwapim – Togo Ranges</td><td>Forest Ochrosol - Oxysol Intergrades</td></tr>
<tr><td rowspan="2">Moist Sub Humid</td><td>Birrimian</td><td rowspan="3">Forest Dissected Plateau</td><td>Rubisol Ochrosol Intergrades</td></tr>
<tr><td>Granite</td><td>Lithosols</td></tr>
<tr><td>Humid</td><td>South - Western Equatorial</td><td>Rain Forest</td><td>Phyllite</td><td>Oxysols</td></tr>
<tr><td rowspan="3">Dry Sub Humid</td><td rowspan="6">Dry Equatorial</td><td rowspan="3">Coastal Thicket</td><td>Tarkwaian</td><td rowspan="6">Coastal Plains</td><td rowspan="6">Coastal Savannah Zone</td><td>Ochrosols</td></tr>
<tr><td>Buem</td><td>Lateritic Sandy Soils</td></tr>
<tr><td>Cretaceous & Eocene</td><td>Tropical Black Clays or Akuse Soils</td></tr>
<tr><td rowspan="3">Semi Arid</td><td rowspan="3">Coastal Savannah</td><td rowspan="3">Accra-Ho-Keta Plains</td><td>Dahomeyan</td><td>Tropical Grey Earths</td></tr>
<tr><td rowspan="2">Tertiary & Recent</td><td>Sodium Vleisols</td></tr>
<tr><td>Coastal Sandy Soils</td></tr>
</table>

## 2.12 Field Sampling of Laterite Materials in Ghana

Sampling of laterite for road construction evolved by Gidigasu (1972) based on work done by Clare and Baven (1962) for selection of laterite in Nigeria, work done by Ahn (1959 – 1961), Brammer (1962), Stephen (1953), Hamilton (1964) on characteristics of Ghana‟s soils established the following profile for soils in Ghana.

Special attention must be paid to variation in appearance in the vertical direction which indicates the state of weathering of the exposed material. In the study of profiles, attention must be given to topographic site, nature of the various horizons, texture, colour, structure and general morphology, including stone line as well as catenal sequence of soil types.

On the basis of several profiles studied by BRRI from different regions of Ghana, laterites of Ghana fall into the universally accepted division of primary and secondary laterites. These have been further sub-divided into several groups; such as nodular laterites or concretionary laterites, including hard pea gravels, hard pans, ground water laterites, iron stone cap rock and laterite soils. Such a division is useful for the selection of laterite in the field for road construction. Each of the groups has a specific range of physical characteristics.

In Ghana, the following gravels have been used extensively for road construction:

- Nodular or Concretionary Laterites (see Figures 2.11 & 2.12);
- Iron stone hard pans or cap rock (see Figures 2.13 & 2.14);
- Groundwater laterites, with detrital quartz (n.d.)
- Colluvials and terrace laterites (see Figure 2.15)

Figure 2.11 - Nodular Lateritic Gravel (Charman, 1988)

Figure 2.12 - Concretionary Laterite Boulder (Charman, 1988)

Figure 2.13 - Hard Concretionary Laterite (Charman, 1988)

Figure 2.14 - Iron Stone Hard Pans or Cap Rock (cuirasse) (Charman, 1988)

Figure 2.15 - Colluvial Laterites ( Charman, 1988)

From the above literature review, any evaluation of the engineering properties of local laterite to be used as pavement material must take into account the above climatic, environmental and geological factors which underlined the formation of the laterite material. In addition, it will require the use of pavement design method which takes into consideration local factors, application of appropriate design standards and material specification as well as using construction methods which as consistent with standards and specifications.

A subsequent Chapter will discuss the full impact of the engineering properties of the laterite soils when used as road pavement material.

## 2.13 Pavement Design Methods

Primarily, there are two basic design methods currently in use; empirical or mechanistic. In addition, there is a third method which depends on the combination of the two methods and hence it is termed the empirical-mechanistic method.

### 2.13.1 Empirical Design Approach

Empirical methods are evolved from empirical studies and local experience. Review by Bhutta (1999) on present state of knowledge of pavement design method revealed that, the most commonly used empirical design methods are known as the Asphalt Institute Method, American Association of State Highway and Transportation Officials (AASHTO) Method and the California Method. The AASHTO Method is derived from AASHTO road tested carried out in the late 1950s in Ottawa, Illinois. Results from AASHO road tests were included in design charts to compensate for environmental and loading factors. The California Method was evolved from a combination of road tests and local experience, the thicknesses of the layers of the pavement being related to the CBR (California Bearing Ratio) of the subgrade and the type of bituminous materials used.

#### 2.13.1.1 Shortcomings of the AASHTO System of Design

Stoner and Bhatti (1994) reported that these two methods have undergone a series of modifications but still cannot be applied to every situation. Gould (2007) discusses the deficiencies on the use of the AASHTO 1993 Method, when the AASHTO 2002 version was introduced. The major shortcomings of the empirical designs are discussed in this section.

a. The Pavement Distress Models were derived with 1960 traffic levels. Comparisons of these levels to today‟s traffic levels, even in developing countries, far exceed the 1960 developed traffic levels. The ratios of Heavy Goods Vehicles (HGV) to total traffic volume from the 1970 level of 3.2% have increased to about 5%. The original Road Test data collected in the 1950s included less than 1 million equivalent standard axle load (ESAL) which formed the basis of the regression analysis to project the pavement damage equation. Current traffic levels surpass 1 million ESAL in its first year of serviceability. The method is only suitable for new construction or reconstruction projects and does not support rehabilitation interventions considering the economic savings that accrue relative to the cost of reconstruction. The wheel configuration and suspension variations coupled with differing tyre pressures have increased from the 1950s level of 80 psi to 120 psi in today‟s vehicle fleet.

b. Climatic effect was solely based on the testing location and does not include conditions for the specific areas where the new pavement systems are to be constructed. The Road Test was carried out for a period of two years. The results did not investigate the effect of long term climate cycles of material aging. Many roads are designed for 20 to 40 years; therefore a more repetitive, cyclic approach should be used to model the long term effects.

c. Only one type of subgrade was used in the Road Test. Consequently, there is the need to incorporate specific properties of other subgrades to enable the prediction of the performance of the designed pavement system. Only the Hot Mix Asphalt (HMA) surface was used in the AASHTO road test, but currently there are numerous HMA mixes, with varying bitumen grades, aggregate types and asphalt mix designs. These are the base materials used worldwide.

d. No drainage was considered during the test. It is now a standard practice to incorporate a well-drained base in the current designs.

e. There is no systematic arrangement to validate the reliability and performance deficiencies which were either not considered or not validated in earlier versions of the design guides.

f. The empirical relationships developed are limited to the test site environmental conditions, and often require extrapolation for designs outside the range of original test conditions. This restricts the accuracy of the design which is a common disadvantage of all empirical methods.

Of all the empirical methods, the AASHTO and CBR methods, irrespective of their drawbacks are widely used and have been the basis for similar empirical methods, such as the Road Notes, prepared by the Transportation Research Laboratory (TRL) in the United Kingdom.

#### 2.13.1.2 Road Note 31 Design Method

This is based on research and experience in over 30 countries, mainly tropical and sub-tropical. This Note covers a wider range of materials and structures with a catalogue of designs that cater for traffic up to 30 million standard axles. Fixed pavement structures have already been determined for ranges of traffic to be selected. Subgrade moisture condition has been classified based on closeness of the water table to the ground surface. Six strength classes of the subgrade, reflecting the sensitivity of design thickness have been catalogued ranging from S1 (2%) to S6 (30%). For subgrades with CBRs less than 2, a special treatment is required which is not covered in Road Note 31 design guide. The Note specifies the minimum CBR for highest anticipated moisture content of material and in arid and semi-arid areas where the mean annual rainfall can be less than 500mm as well as those of selected subgrade materials and capping layers

#### 2.13.1.3 Road Note 29 Design Method

According to the Guideline-LVSR (Low Volume Sealed Roads), (2003) for the SADC (Southern African Development Community), the Road Note 29 design procedure is of the CBR type. The results of the TRRL"s (Transportation and Roads Research Laboratory) many full scale road trials have been used in the derivation of the design charts and tables contained in the Road Note 29 for UK conditions. The method allows a pavement to be designed for a life of a selected number of years by assessing the cumulative number of commercial vehicles to be carried. With CBR values for the sub-base, the thickness can be determined. The selection charts for the road-base is limited to only two types of asphaltic concrete material; hot rolled asphalt and dense macadam road-bases. In the tropics, the road base materials can be natural gravel and therefore this method cannot be used effectively in tropical conditions. Road Note 29 has however, been used in some tropical conditions where traffic loadings are beyond 30 million ESAL limit covered under Road Note 31.

#### 2.13.1.4 The CEBTP Pavement Design Method

This is a common design method in French-speaking tropical countries. The subgrade strength is assessed on the basis of the CBR, and traffic is categorised into 4 classes. It is essentially a modification of the original CBR design method. The design involves selection of a pavement structure from a list of 4 basic pavements.

#### 2.13.1.5 Other empirical design methods

The Shell Pavement Design Method (1978) which has been developed over the years to incorporate the effect of temperature on bituminous materials is also in use in tropical countries. Hveem Stabilometer, (closed triaxial cell) design method determines the soil strength, and then the flexural strength of the paving materials is determined on the basis of a cohesiometer test and traffic loading, expressed as traffic index (Gichaga and Parker, 1987).

The input requirements of the current empirical pavement design procedures do not make room for empirical information from given local conditions. Climatic factors such as rainfall and temperature have significant influence on the properties of soils in a given geographic area as well as the performance of the road pavement. These are rarely taken into account in the existing pavement design methodologies; except local conditions within which the empirical studies were undertaken or data on observed road performance. These studies are often carried out in temperate regions. The current design guides in use in many tropical countries do not make allowance for an objective assessment of tropical local conditions where the road will be used. The use of pavement design methods and guides evolved from studies carried out on temperate soils without any regard for tropical conditions, may lead to unreliable designs and hence poor performance when used.

The empirical approach employs statistical techniques to explain pavement deterioration with its explanatory variables. Although this approach has the capability to link the pavement performance with the causal variables, the explanatory variables taken are only based on their availability and statistical values. Consequently, this approach suffers from the limitations associated with the scope and range of the available data.

It will, therefore, be misleading to use design guides based on temperate soils properties to support the design of tropical road pavements and thereafter estimate the life of the tropical road pavement on the basis of temperate soil parameters. This is more so as the mode of tropical soil formation is known to be very different from temperate soils. This situation leads to a subjective assessment of pavements designed for use in tropical environments. This incongruous arrangement is a major contributing factor to the road failures experienced in developing countries located in the tropics. In some instances, such earlier failures are counteracted by a high factor of safety, rendering the design functional but uneconomic.

Literature on tropical pavement design methods is almost non-existent as the problem has not received enough attention. Design Guides currently in use in Ghana, such as Road Notes 29 and 31, published by the British TRL are based on some generic assumptions for all tropical countries. These Notes are not prepared to meet specific country needs, such as Ghana‟s pavement soil materials, but have been based on general assumptions. This calls for the need to customize these Design Notes, to suit local tropical conditions. Customization can only be done with reliable road database, which is non-existent in Ghana. In addition to these Road Notes, Pavement Design in Ghana has been greatly influenced by several foreign consultants who use design methods developed by the French, the Japanese, Australians, and the United States.

In order to address the current tropical design challenges, a new pavement design concept will have to evolve which takes into account all the deficiencies and challenges encountered with the use of the empirical design approach. The mechanistic approach is gaining grounds as the current state-of-the-art method for the development of a rational pavement design method that may be a panacea to problems encountered with empirical design approach.

### 2.13.2 Mechanistic Design Approach

The mechanistic design approach is based on the stress state of the pavement layers when it undergoes any deformation as a result of loading. The term „mechanistic‟ refers to the application of principles of engineering mechanics, which provides a rational design process.

#### 2.13.2.1 Stress States

In a cylindrical triaxial test, there are two principal stresses; the total axial stress ($\zeta_1$) and confining stress ($\zeta_3$) as shown Figure 2.16a. With the sample being subjected to a cyclic deviator stress and a confining pressure, the total axial stress ($\zeta_1$) consists of the dynamic deviator ($\zeta_d$) stress and static confining stress ($\zeta_3$) acting in the axial plane. The confining pressure is made of only a static stress component ($\zeta_2 = \zeta_3$) and a zero dynamic stress component in the horizontal direction. The shear stress ($\eta$) component is zero during testing. So the total axial stress ($\zeta_1$)

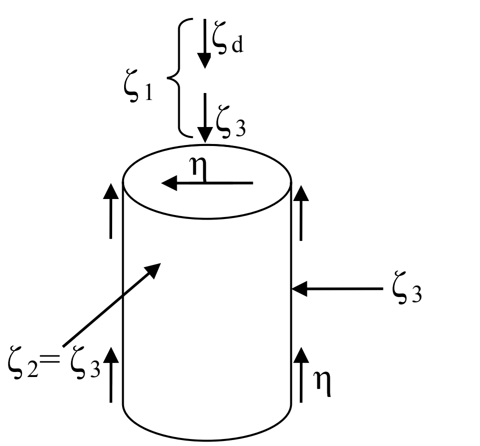
becomes the major principal stress with the minor principal stress being the confining pressure ($\zeta_3$).

Yoder and Witczak (1975) showed that in a multi-layered elastic system, at a given point within any layer, static equilibrium requires that nine stresses exist. These stresses comprise three normal stresses ($\zeta_z$, $\zeta_r$, $\zeta_t$) acting perpendicular to the element faces and six shearing stresses ($\eta_{rt}$, $\eta_{tr}$, $\eta_{rz}$, $\eta_{zr}$, $\eta_{tz}$, $\eta_{zt}$) acting parallel to the faces as shown in Figure 2.16b. Static equilibrium conditions also show that the shear stresses acting on intersecting faces are equal. Thus, $\eta_{rz} = \eta_{zr}$, $\eta_{rt} = \eta_{tr}$, and $\eta_{tz} = \eta_{zt}$. At each point in the system, there exists a certain orientation of the element such that the shear stresses acting on each face are zero. The normal stresses under this condition are defined as principal stresses and are denoted by $\zeta_z$ (major stress), $\zeta_r$ (intermediate), and $\zeta_t$ (minor). Considering this triaxial stress state of any element, the strains may be determined from Equations 2.2, 2.3 & 2.4:

$$\varepsilon z = \frac{1}{E}[\sigma z - \mu(\sigma r - \sigma t)] \ldots\ldots \text{vertical direction} \ldots\ldots.. \text{Equation 2.2}$$

$$\varepsilon r = \frac{1}{E}[\zeta z - \quad (\zeta t - \zeta r)] \ldots\ldots \text{radial direction} \ldots\ldots.. \text{Equation 2.3}$$

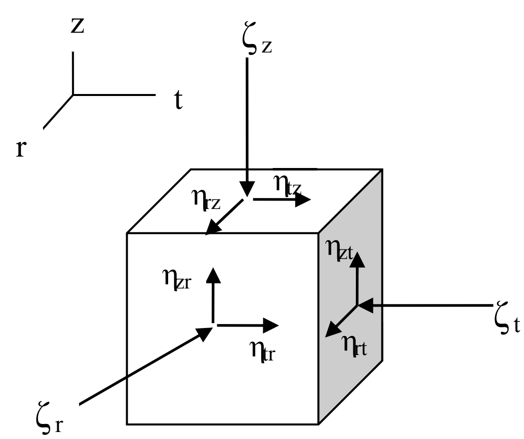
$$\varepsilon t = \frac{1}{E}[\sigma t - \mu(\sigma r - \sigma z)] \ldots\ldots \text{tangential direction} \ldots\ldots.. \text{Equation 2.4}$$




$\zeta_1 = \zeta_3 - \zeta_d$, Bulk stress, $\theta = \zeta_1 + \zeta_2 + \zeta_3$

$\theta = \zeta_d + 3\zeta_3$

Figure 2.16a – Principles of Engineering Mechanics




Bulk stress, $\theta = \zeta_z + \zeta_r + \zeta_t$

Figure 2.16b – Principles of Engineering Mechanics

From an engineering point of view, there is much to be desired from the Mechanistic Approach to pavement design. Yoder and Witczak (1975), who are champions of this design approach, stated that "for the process to be rational three elements are critical and are stated as follows:

- The theory used to predict the assumed failure or distress parameter;
- The evaluation of materials properties to be applicable to the selected theory; and
- The determination of the relationship between the magnitude and the parameter in question to the performance level desired (NCHRP, 2004)."

The basic theory of the mechanistic pavement is based on the layered elastic design method which utilizes primarily engineering mechanics. In analysing the behaviour of pavements subject to wheel load applications, a number of models based on elastic theories have been used to define stresses, strains and the resultant deflection in flexible pavements. These models were primarily derived from Boussinesq‟s theory which states that "the vertical stress at any depth below the earth‟s surface, due to a point load (P) at the surface, is dependent upon the radial distance and the depth from the point load." A diagram depicting the theory is shown in Figure 2.17. This is based on the assumption that the pavement material is isotropic and semi-infinite, and that elastic properties are identical in every direction throughout the material and is a homogenous layer of infinite depth (1-layer pavement)

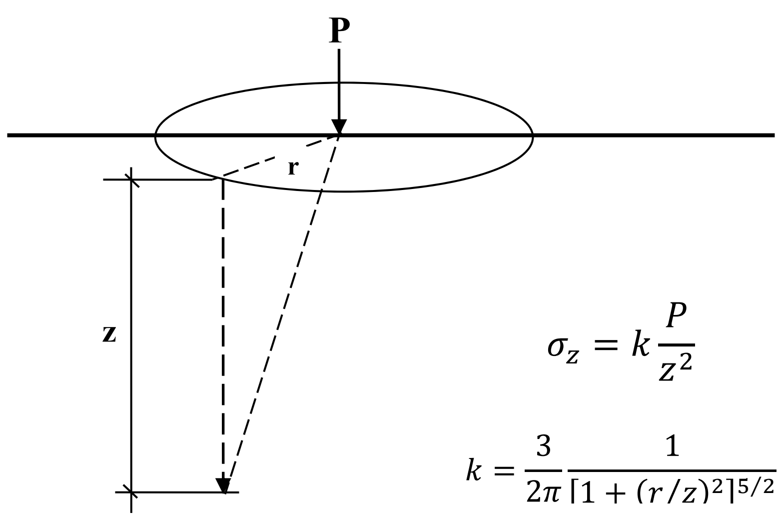



Figure 2.17 – Boussinesq‟s Equation Diagram (Gould, 2007)

To ensure an economic pavement structure, most pavements consist of multiple layers with a primary objective of distributing the applied surface over a greater base. Further works done by Burmister (1943 & 1958), provided an approach that could support the multi-layered pavement system (2-layer) being constructed. The layered elastic model assumes each layer is homogeneous (same composition throughout the body, elastic properties same at every point in the body), isotropic (same elastic properties in all directions) and linearly elastic and holds several assumptions which include the following:

- A surface layer assumed to be infinite in the x-y directions but with finite depth z;
- An underlying layer infinite in all directions, x-y-z;
- The two layers are fully bonded;
- The interface assumed to be either perfectly rough or perfectly smooth; and
- Surface loading also assumed to be uniformly distributed over a circular area.

Both the load and pavement geometrics are symmetrical about a common centreline (axisymmetry). Due to the symmetry in stress analysis, many types of strain were also able to be computed at the bottom of layers 1 and 2 as well as the top of layer 3. Values at these critical locations in the pavement form the basis for development of multi-layered elastic designs.

#### 2.13.2.2 Resilient Modulus in Pavement Design

Resilient modulus is defined as the ratio of the repeated axial deviator or cyclic stress to the recoverable or resilient axial strain. Equation 2.5 shows the definition of the resilient modulus.

$$\mathrm{M_R} = \frac{\sigma_d}{\varepsilon_r} = \frac{\Delta\sigma}{\Delta\varepsilon} \text{ ........................ Equation 2.5}$$

The theory of elasticity is usually used for flexible pavement design. This theory assumes that all materials in the pavement are homogenous, isotropic, and linearly elastic. With these assumptions, only two material properties i.e. Poisson"s ratio ( ) and the elastic modulus (E) would be necessary to calculate stresses, strains and deflections in the pavement layers. According to Anochie-Boateng et al. (2009), the Poisson"s ratio is usually assumed or obtained through the use of correlations and the resilient modulus is used as the modulus of elasticity based on the recoverable strain under repeated loads.

The resilient modulus for most pavement materials is stress dependent. This generally increases with increase in stress (stress-hardening) for granular materials whereas that of fine-grained soils decrease with increasing stress (stress-softening). Many nonlinear models have been proposed over the years for incorporating the effects of stress level on the resilient modulus. A general form for these models can be expressed as follows (Andrei, 1999):

$$M_R = k_1 P_a \left(\frac{\theta}{P_a}\right)^{k_2} \left[\frac{\tau_{oct}}{P_a} + 1\right]^{k_3} \ldots\ldots\ldots\ldots\ldots \text{Equation 2.6}$$

Where, $\theta$ = bulk stress

$\eta_{oct}$ = octahedral shear stress

$$= \left\{\frac{1}{9}\left[\left(\sigma_x - \sigma_y\right)^2 + (\sigma_z - \sigma_x)^2 + \left(\sigma_y - \sigma_z\right)^2\right] + \frac{2}{3}\left[\tau_{xy}{}^2 + \tau_{zx}{}^2 + \tau_{yz}{}^2\right]\right\}^{1/2} \text{Equation 2.7}$$

$= \frac{\sqrt{2}}{3} \Delta\sigma$ For standard triaxial compression loading

Pa = normalizing stress atmospheric pressure = 101.3 kPa

$k_1$, $k_2$, $k_3$ = model parameters obtained from regressional analyses

The above equation combines both the stiffening effect of bulk stress (the term under the $k_2$ exponent) and the softening effect of shear stress (the term under the $k_3$ exponent).

Practical drawbacks in the application of mechanistic design methods are the need to determine several properties in order to find the stress and strains at specific locations throughout the pavement system. Issues with respect to vibrations in cohesionless soils, variations in the stress state in each layer take place causing densification that would cause rutting and changes in material properties. The effect of wheel loads applied close to a crack or a pavement‟s edge, cannot be analysed by use of methods which require axisymmetry.

The Meyerhoff‟s method (Gichaga and Parker, 1987) was developed on the basis of the Burmister‟s Theory, to relate the transient deflection directly to the applied load pavement thickness and the elastic moduli ratio of the pavement to the subgrade.

The multilayer system of analysis is now possible due to the developments in computer techniques and that the required strength properties for each layer of a given thickness (h) must be known before their stress and strain behaviours can be analysed. Two of the input properties are the elastic modulus (E) values and Poisson‟s ratio ( ). Therefore, different materials will give different values reflecting response of material to loading. A description of the problem is shown in Figure 2.18.

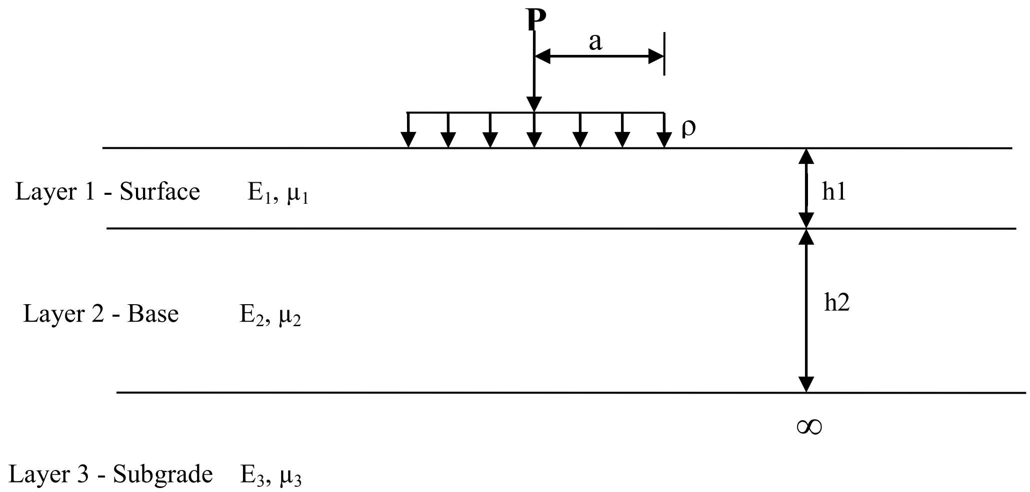



Figure 2.18 – Description of a multilayer system (Gould, 2007)

Hibbeler (2000) has given a concise explanation of strain and stress characteristics of a material. Occurring in two forms; either elastic strain or plastic deformation. Macioce (2002) describes the purely elastic material as one which stores all the energy and releases it upon load removal and has stiffness. Opposite to elastic, is viscous material which does not release any energy upon load removal and it has no stiffness. In between these two extremes of materials, all other materials exhibit viscoelastic characteristics. Some of the energy stored in a viscoelastic material is recovered upon load removal, and the remainder is dissipated. The ability of the material to absorb energy within the elastic limit is referred to as its Resilience (Gere and Timoshenko, 1984).

Most of the bituminous materials used for road pavements are viscoelastic; whose properties are influenced by many parameters includes frequency (loading time), temperature, time effect (creep and relaxation), aging, etc. The most important of these effects are temperature and frequency. It is also important to evaluate how the pavement materials respond to different frequencies or rate of loading, which corresponds to the different traffic speeds pavement could experience in the field. A material‟s elastic modulus is actually an estimate of its elasticity (E). While the modulus of elasticity is stress divided by strain from a slowly applied load, it is known as resilient modulus for repeatedly applied loads, similar to those experienced by road pavement (Muench et al., 2003).

Claros et al. (1990) defined resilient modulus ($M_R$), therefore, as material property that measures the elastic (load-unload) response of a soil under repeated loading. Numerically, it is the ratio of the deviator stress ($\zeta_{d}$) to the resilient or recoverable strain ($\varepsilon_R$) (given in Equation 2.8).

$$M_R = \frac{\zeta_d}{\varepsilon_R} \quad \text{.............................. Equation 2.8}$$

Based on the AASHO Road Test some researchers have concluded that when a load is applied to the pavement surface the resulting deflection is a strong indication of pavement performance (Highway Research Board, 1962). Between 60 to 80% of the measured surface deflection was found to propagate in a form of rut on the surface of the subgrade. Therefore, the resilient modulus test for subgrade soil models is an important part of assessment of performance of flexible pavement (Ksaibati et al, 1994).

Huang (1994) observed that most of the paving materials experience some permanent deformation after each load application. The amount of strain under repeated loading in a material changes over time. In general, considerable increase in the amount of permanent deformation (accumulated plastic strain) is experienced in the beginning and as the load increases, the accumulated plastic strain levels off and the material undergoes essentially, elastic recoverable strain. This phenomenon is depicted in the Figure 2.19.

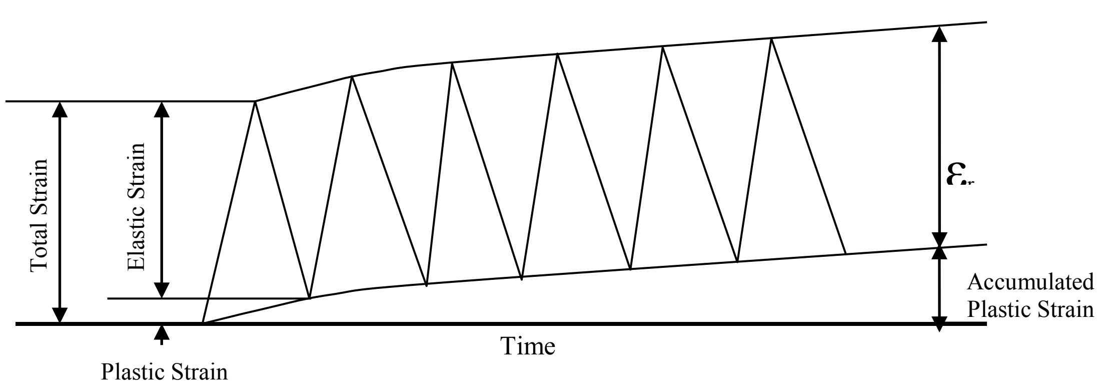



Figure 2.19 - Strain under repeated loading (after Huang, 1994)

In order to address the issue of behaviour of different materials, other than those used in the AASHO Road Test, in 1986, the use resilient modulus ($M_{R)}$ became the basis for the AASHTO Guide for Design of Pavement Structures which replaced the bearing capacity parameters such as CBR, Resistance Value (R-Value) and Soil Support Value (SSV).

According to AASHTO (1993), $M_R$ value has three important advantages over the soil support value used in the previous editions. They are:

- It gives a basic material property which can be used in mechanistic analysis of multi-layered systems for predicting distresses such as roughness, cracking , rutting and faulting;
- It has been recognized internationally as a method of characterizing materials for use in pavement design evaluation; and
- Techniques are available for estimating the $M_R$ properties of various materials in-place from Non-Destructive Tests (NDT).

Seed et al. (1962) developed the test for measuring resilient modulus to reflect several observations in the field and from research projects. Since then, other methods for in-situ estimation of the resilient modulus have been developed. Fall et al. (2008) have investigated the resilient modulus of residual tropical lateritic soils of Senegal using the Uzan-Witczak universal model (Witczak and Uzan 1988), the Andrei model, $k$-Θ model ($M_R = k_3\theta^{k4}$) and the power model ($M_R = k_1\zeta_d^{k2}$) to find suitable model that can best represent the relationship between resilient modulus and stress levels in lateritic soils and established the most influential k-factors which have significant importance on the soil behaviour models. The Uzan-Witczak universal nonlinear model and the Andrei Model are the preferred for the prediction of the resilience modulus. Jimoh and Akinyemi (n.d.) established the regression constants (k-values) for Nigeria"s natural and stabilized (cement and lime) laterite soils (see Table 2.12).

Table 2.14 - $K_1$, $K_2$, Regression Constants For The Lateritic, Lime And Cement Soils (Jimoh and Akinyemi, no date)

| **Laterite soil** | | **Lime soil** | | **Cement soil** | |
|---|---|---|---|---|---|
| $K_1$ MPa | $K_2$ | $K_1$ MPa | $K_2$ | $K_1$ MPa | $K_2$ |
| 1014 | -0.20 | 3350 | -0.38 | 25521 | -0.56 |

Considering the economics and convenience, the NDT methods using computer programs to back-calculate the $M_R$ values are very popular. Several methods currently used and even though these methods provide engineers with a quick method for obtaining $M_R$ values; Uddin (1984) outlines some challenges which must be considered. They include the following:

- The non-uniqueness of the resilient modulus back-calculated from the measured deflection basin;
- Errors due to possible variation in thickness of pavement layers;
- Errors involved in assuming a semi-infinite subgrade;
- Errors in back-calculated moduli because of the non-linear behaviour of granular layers and subgrade; and
- Errors involved in using input values out of the range for which the model is calibrated.

The dilemma among road pavement design practitioners is the total shift to the mechanistic pavement design approach, while there are genuine concerns of the ability of the underlying theory to reliably predict the field behaviour of respective pavement layer materials after design. Pavement materials do not exhibit the simple behaviour assumed in isotropic linear-elastic theory. Nonlinearities, time and temperature dependency, and anisotropy are some examples of the complicated features often observed in pavement materials which will require advanced modelling to predict performance mechanistically. The mechanistic design approach is based on theories of mechanics and relates pavement structural behaviour and performance to traffic loading and environmental influences. Even though there seem to be progress made in isolating pieces of the mechanistic performance prediction problem, the reality is that fully mechanistic methods are yet to be available for practical pavement design.

In the mechanistic approach, the relationship between the structural response (stresses and strains) and the physical causes (traffic loading, type of subgrade, climate and type of pavement material) are described using a mathematical model. Most common models are based on the multi-layered elastic theory and some finite element model applications. Most of the assumptions made do not reflect reality. For example, the surface and underlying layers are not infinite in reality as there is a definite extent of the road pavement in the x-direction; that is, the width of the road which is depicted by the road"s cross-section. The y-direction depicts the length of the road, which in a practical design environment is limited as per contract requirement and therefore cannot be infinite. The assumption that the bonding surfaces are perfectly smooth or rough only exists in theory as in practical terms, shear failure indicts this assumption. Different types of vehicles applying the road surfaces do not distribute loading uniformly. The granular component of the road pavement shows nonlinearity and is time dependent. Consequently, to establish more rational pavement design methods and the construction criteria, it is essential that the response of granular layers under traffic loading be well understood and taken into consideration (Lekarp et al., 2000).

Historically, pavement behaviour has been studied using mechanistic approach based on the physical principles such as soil mechanistic theory, mechanical property of the pavement material under load and multilayer structural analysis techniques. Most of these studies were conducted under limited experimental conditions. Therefore, they need to be validated and calibrated to the full range of real situations before implementing the developed mechanistic models. In addition, most of these models are simple and only represent the material or structural responses in limited situations. Even though the mechanistic approach is regarded as the best to characterize the deterioration process, the development of reliable and acceptable mechanistic models is still at its early stage and requires significant amount of time and effort for continuous studies (Matiko, 2008).

Solutions are yet to be found for the practical challenges confronting empirical and mechanistic pavement design methods. A lot of studies are being done, currently and some in the offing, which address issues relating to field parameters, are different from laboratory assessments. South Africa as part of pavement design review is deepening knowledge in the M-E design method as reported by Theyse et al. (2004) on current work by Pavement Modelling Corporation of South Africa. From the Indian perspective, Amimesh and Pandey (1999) reported on work being done on fatigue and elasticity of pavement material, analysis of field performance data, determination of fatigue and rutting criteria from laboratory and field and the development of thickness design charts for bituminous pavements. Qiang et al. (2011) gives a bird"s-eye view of effort being made in the USA through the NCHRP 1-26, to evolve different M-E method. The USA congress has also supported the establishment of Long Term Pavement Performance Sites (LTPPS) spanning over 20 years to monitor 2,500 in service pavement sections in USA and Canada. The empirical approach cannot be extrapolated with confidence beyond the conditions on which it was based. The effectiveness of any mechanistic method is its efficiency to properly evaluate the behaviour of pavement constituent materials under stress or loading condition (Lekarp et al, 2000).

NCHRP (2004) reported of the NCHRP 1-37A project which has delivered the most recent M-E based method that incorporates nationally calibrated models to predict distinct distresses induced by traffic load and environmental conditions. The NCHRP 1-37A methodology now incorporates vehicle class and load distribution in the design, a step forward from the equivalent standard axle load (ESAL) approach used in the AASHTO design equation. The effect of climate seasonality on behaviour of material has also been computed (Schwartz et al., 2007).

Ullidtz and Peattie (1980) have succinctly and eloquently stated the realistic practical limitations of the accuracy of mechanistic pavement analysis:

> *"The results obtained (from the analysis) may deviate from the exact values. These deviations, however, should be considered in relation both to the simplifications made in the analysis and to the variations of materials and structures with space and time. Real pavements are not infinite in horizontal extent, and subgrade materials are not semi-infinite spaces. The materials are nonlinear, elastic, anisotropic, and inhomogeneous, and some are particulate; viscous and plastic deformations occur in addition to the elastic deformation; loadings are not usually circular or uniformly distributed, and so on. To these differences between real and theoretical structures should be added the very large variations in layer thicknesses and elastic parameters from point*

*to point, and during the life of a pavement structure. Moreover, it is a fact that precise information on the elastic parameters of granular materials and subgrade is in most cases very limited. For most practical purposes, therefore, the accuracy of the method should be quite sufficient"*

To overcome challenges from strict adherence to either empirical or mechanistic methods of design, a compromise design method has now been evolved which considers the input in terms of climate, traffic and then through a modelling process considers the material properties and comes up with distress predictions which is then verified empirically. This is the rationale behind the mechanistic-empirical design approach

### 2.13.3 The Mechanistic-Empirical Design

The mechanistic-empirical approach is a hybrid approach. Empirical models are used to fill in the gaps between the theory of mechanics and the performance of the pavement structures. Simple mechanistic responses are easy to compute with assumptions and simplifications, (i.e. homogeneous material, small strain analysis, static loading as typically assumed in linear elastic theory), but cannot themselves be used to predict performance directly; some types of empirical models are required to make appropriate correlation. Mechanistic-Empirical (M-E) Design Method is considered an intermediate step between empirical and fully mechanistic methods. M-E provides the designer the ability to evaluate the effect of various materials against pavement performance (Gould, 2007).

Structural responses (stresses and strains) are mechanistically calculated based on the materials properties, environmental conditions and loading characteristics. The performance prediction models use the calculated stresses and strains to predict pavement distress using the adopted empirical formulas. The accuracy of these empirical formulas (or models) depends on the quality of the input information and the calibration of empirical distress models to observed field performance (Schwartz and Carvallo, 2007). These forms of distress have to be constrained by other means such as specification of appropriate materials or provision of relevant cross-sections, pavement type and drainage (Queensland Department of Main Roads, 2009). Basically there are two types of empirical models used in the M-E design methods, one type predicts the distress directly (e.g. rutting model for flexible pavement and faulting for rigid); the other type predicts damage which is then calibrated against field measured distress (e.g fatigue cracking for flexible pavements and punch-out for rigid pavements).

Even though the M-E Design provide the opportunity to put pavement design on a more sound basis of science and engineering compared to the empirical approach, Hass et al (2007) outlined future challenges which will have to be dealt with to enhance reliability toward a complete shift from empirical pavement design systems. These include calibration and validation requirement, implementation guidelines, commitment of resources, equipment and training, input data requirement and balancing complexity/comprehensiveness with understandability and practicality. Models need to be verified, calibrated and validated before they can be used with confidence. The calibration of the selected model is to be based on both laboratory and field test data.

The following areas need to be addressed to enhance the M-E"s utility:

- There is the need to establish comparative sensitivity and interactions of factors in the M-E pavement design approach. Randomization of factor values within limits and with repeated runs of the model will evolve the most sensitive factors such as the HDM- model approach. Kim et al (2007) studied the sensitivity of the input parameter of M-E Design Guide and evaluated a total of twenty individual input parameters by studying the effect of each parameter on longitudinal, fatigue cracking, traverse cracking, rutting and roughness. They classified the sensitivity of the parameters into three categories (very sensitive, sensitive and insensitive). It was found that;

- The only very sensitive parameter is the AADT.

- The material coefficient of the subbase and aggregate were insensitive. The model is sensitive to the parameters; layer thickness, tyre pressure, traffic distribution, speed and weather.

- Documentation, dissemination of calibration and validation of results by stakeholders, (e.g. lessons learnt) will avoid future pitfalls.

- Expanding the flexibility of current M-E procedures to go beyond changes in material properties with time and incorporate „self-healing" structures and new construction and maintenance processes.

- Simplify the process, not the M-E method themselves through catalogue designs to serve as a check on designs resulting from M-E analysis.

- Avoiding the tendency to use M-E design packages as a „black box" through guidelines for checks on the reasonableness of the results.

The challenges in pavement engineering do not end with selection of appropriate design method or approach. It is the objective of the road agencies in Ghana to design and construct roads which are economic in terms of initial capital investment required for its construction and thereafter the maintenance requirement during the life span of the road pavement when in use. Economic evaluations of the selected designs considering the initial cost of construction and the maintenance cost during the life of the pavement are used to establish the total life cycle cost. The HDM-IV is a tool which is widely acclaimed to undertake life cycle analysis among other tools. Both editions of the AASHTO Pavement design Guide (1986 and 1993) encourage the use of the life-cycle cost analysis to establish the true cost of any selected design through economic evaluation of alternative pavement design strategies. The reliability of these analyses is the calibration of models to simulate local conditions.

From the foregoing, the critical challenges in this study are summarized as follow:

- Many of the early soil mechanics used in evolving empirical road pavement design guides and procedures were based on European and North American practices. Assumptions used were therefore based on their peculiar temperate environmental conditions. Studies have established that temperate soils are generally inert; compared to tropical soils which are considered active, particularly as they undergo chemical weathering as a result of increasing temperatures and rainfall patterns.

- Current design assumptions and procedures do not take into consideration the local environmental conditions and soil properties. In arriving at pavement layer design thicknesses, the observed behaviour of these tropical soils is not considered. Some of the Design Guides also use the cumulative standard axles estimated to be carried by the road pavement over the design period.

- It has been validated through research and reported by Gidigasu (1976) that there are significant differences between the laboratory test results and the field performance of laterite soils used as road pavement materials in Ghana. The anomalous laboratory and field behaviour of some of the samples of these laterite soils appear to stem mainly from lack of suitable engineering evaluation criteria of local laterite soils. Evaluation of tropical soil properties as determined in the laboratory must take into account the mode of formation (genesis), degree of weathering (decomposition, laterisation and desiccation) as well as the unique chemical and mineralogical characteristics of these laterite soils all of which influence their performance when in use as layers in road pavements.

- Traffic Loading expressed in terms of the equivalent standard axles load (ESAL) is always far higher than what is assumed for the design irrespective of the strict adherence to design standards and specifications. These challenges confront Road Engineers working in most developing countries.

## 2.14 Problem Components

There are four aspects to the problem relating to the current empirical pavement design approaches used in Ghana and other tropical countries, that is, the inability to take into consideration local environmental conditions in the design of pavements; the evolution of Design Guides used in Ghana which are based on temperate soil materials with properties not similar to those of tropical Ghana, differences between laboratory test results and field performance and a wide difference between the designed traffic and actual traffic levels when the road is in use.

### 2.14.1 Inability to Consider Local Conditions in Pavement Designs

Climatic factors such as rainfall and temperature which have significant influence on the properties of soils in a given area and hence the performance of the road pavement these factors are not taken into account in the pavement design. The current design guides used in many tropical countries do not make allowance for objective assessment of local conditions because invariably the design guides evolved from studies carried out on temperate soils and thus may lead to unreliable designs.

It is thus conjectured that use of inappropriate design guides is a major contributing factor in road failures experienced in developing countries located in the tropics. In some instances, such earlier failures are counteracted by a high factor of safety, rendering the design functional but uneconomic. There is no legislation or requirement to subject pavement design to Life Cycle Cost Analysis to determine the cost of a selected pavement design with respect to construction cost, maintenance cost and user cost.

Unfortunately, the economies of many developing countries depending on such design guides are not strong enough to support research to evolve local design guides using data collected from monitoring Long Term Pavement Performance (LTPP) sites.

Ghana‟s current pavement design assessment methodology is not based on any trends ascertained through the monitoring of LTPP sites. Development partners have used pavement design guide conversant to them, regardless of the environmental and climatic conditions. A typical case was the design and construction of the Yamoransa-Anwia Nkwanta road (MRH, 1994) which was funded by the Overseas Economic Corporation Fund (OECF) of Japan. The Japanese consultant used the design standard of Japan‟s trunk road to design Ghana‟s Yamoransa-Anwiakwanta trunk road which in terms of functionality is equivalent to Japan‟s industrial road standard. Japan‟s industrial road standard is higher than their trunk road. This inappropriate design led to early failure of the road when it was open to traffic. It will therefore be appropriate for any effective and efficient pavement design to use design guides or methodologies that take into account the local environmental condition or design guides based on similar tropical conditions.

### 2.14.2 Evolution of Design Guides in Use in Ghana Not Based On Soil Material Similar To Ghana

Design Guides currently in use in Ghana, such as Road Notes 29 and 31, published by the TRL are based on some generic assumptions for all tropical countries. These Notes are not prepared to meet specific country needs, such as Ghana‟s pavement soil materials, but have been based on general assumptions. This calls for the need to customize these Design Notes, to suit local tropical conditions. Customization can only be possible with reliable road database, which is non-existent in Ghana.

In addition to these Road Notes, Pavement Design in Ghana has been greatly influenced by several foreign consultants who use design methods developed by the French, the Japanese, Australian, and the American (Modified AASHTO etc.). It is therefore very important to standardize the Pavement Design Procedure into a single method in Ghana, which takes into consideration the soil properties and methods which are similar to the Ghanaian condition.

### 2.14.3 Differences between Laboratory Test Results and Field Performance

Classification tests for soils in the temperate climate have been standardized from the knowledge of index properties such as particle size distribution and plasticity parameters. These index properties are used to correlate to engineering properties of soils that may be used in design. The use of the temperate zone classification system to characterize tropical soils based solely on their particle size distribution and plasticity parameters has been very disappointing despite the very strict adherence to testing specifications. Gidigasu (1976) reported (on work of researchers) that the use of particle size and plasticity do not yield reproducible result for laterite as they are influenced considerably by the sample pretest preparation and testing procedures (Willis (1946), Hirashima (1948), Winterkorn and Chandrasekhan (1951) and Townsend et al. (1969&1971)). Many tropical countries continue to experience failures in field performance as a result of the adoption of these parameters to classify tropical soils and not taken into account the continuous weathering of tropical material and its impact on the engineering properties and field performance which are influenced considerably by the chemical and mineral contents, genesis, morphology and the environment.

However, despite various attempts, it has been noted that reliable correlation of index properties and engineering properties cannot be developed. This is undoubtedly because the liquid and plastic limit determination and particle size distribution are not sensitive enough tests. It is possible that it is necessary to develop further the range of current index tests to include some chemical tests to better reflect the characteristics of laterite soils.

### 2.14.4 Traffic Estimation for Design

The lack of a reliable system for traffic data collection on major and strategic roads in Ghana has led to a situation where an ad-hoc approach is used in estimating traffic volume for any given road design. This situation has led to the inability of engineers to properly classify and review the functionality of the various roads in the network.

Consultants do not have any reference database to use in the estimation of expected traffic volume during the design phase of a road project. Considering the time required for preparing the pavement design for implementation, limited traffic studies are usually carried out. Generated and diverted traffic estimation is therefore very difficult as lower figures could lead to inadequate pavement design and an over estimation could lead to uneconomic designs and could affect the budget for the project. There is the need to have reliable and continuous traffic data collection system for Ghana‟s road agencies which will enable engineers to use the data as a reference in the estimation of the traffic volume for the different road types (trunk, feeder and urban), its composition and accurate estimation of standard axles over the design life of road pavements in Ghana.

## 2.15 Summary

This chapter has presented literature on the main subject of the research. It has briefly explained the challenges and the difficulties in using laterites soil material in road pavement and also the previous work done in the pavement design methods in order to identify the good principles to govern this study, and the weaknesses to be improved by the current study. It has also discussed the major terms, models, principles and practices to be used in the proceeding chapters.

The literature review has shown that lateritic soils are very complex and their properties are dependent on the regional climate. It has also been shown that standard index tests do not truly reflect the behaviour of soils. Further development is needed in this area. The current design guides are not appropriate as they do not take full account of local conditions. There is also inadequate traffic data. Hence, lack of suitable information about traffic, soil properties and poor design guides are leading to most failure of roads.

# CHAPTER 3

# RESEARCH APPROACH AND METHODOLOGY

## 3.1 Introduction

This Chapter presents the process followed in this research to develop the Pavement Design Assessment Framework (PDAF). It discusses the different components of the PDAF and the principles governing the various specifications used in the development of the PDAF. It also discusses the data that were used in estimating the parameters of the PDAF and the methodology used to estimate the various parameters of the framework, and how the results were verified. Consequently, the chapter is organized along these major schematic areas.

## 3.2 Problem Component

The objective of developing the new Pavement Design Assessment Framework is intended to further enhance the existing pavement design methods by addressing some of the challenges being encountered by Road Pavement Engineers working in Ghana. These challenges can be identified as follows:

- The validity of the use of technical specifications developed for climatic conditions which are not consistent with tropical environmental conditions in Ghana.
- Shortcomings of the current empirical design methods and the unreliability of the full adoption of mechanistic pavement design approach.
- Lack of an economic assessment of the designed pavements, using the Whole Life-Cycle approach, and a calibrated economic analytic tool.

The development of a new pavement design framework for Ghana took the following into account:

- Effect of the tropical environmental conditions on the characteristics and the properties of the pavement materials;
- The adaptation of appropriate design method which allows inputs related to local conditions; and
- Optimization of the designed pavement through the use of "whole life-cycle" analysis.

Three main tasks were therefore completed in order to achieve the objectives of this Research in the development of the PDAF and they are defined as follows:

- Preparation of engineering properties of Ghanaian soils to support current specification;
- Development of a mechanistic-empirical design procedure applicable to the Ghanaian condition; and
- Whole Life-Cycle assessment of a pavement designed on the basis of mechanistic-empirical methods.

Therefore, the research methodology addresses how the three main tasks have been achieved under this research.

### 3.3 The General Research Design

The overall approach used in this research was to employ existing knowledge and findings from previous research and publications, principles and theories underlying road pavement design and through field studies of pavement performance trends pertaining to Ghana to develop the PDAF. Data on key variables relevant to the development of the PDAF were collected through the establishment and instrumentation of test sections, field and laboratory test results and climatic data from the Ghana Meteorological Agency"s weather stations in four climatic zones in the country.

Statistical methods were used to establish relationships between the dependent variable (pavement performance indicator e.g. pavement condition index, or individual performance indicators (e.g. rutting, cracking, etc.) and one or more independent variables representing pavement structural strength (resilience modulus), traffic loading and environmental condition (temperature, moisture content, rainfall, etc.) as well as among the independent variables.

After these exploratory analyses, in order to ascertain trends for Ghana, a pavement model had to be defined and adopted. This strategy to adopt other tropical M-E pavement models is to make up for the current lack of an objective pavement model in Ghana.

The optimal performance of the designed pavement is achieved by the use of the optimization tool. In order not to confound the output of the selected optimization tool, calibration of the tool was carried out to ensure that simulated trends are consistent with observed trends in Ghana. The optimization is based on the definition of the objective function with a set of constraints that control the optimal solution. The objective function to be used in this research seeks to minimize the total life cycle cost of the designed pavement during its design life.

Figure 3.1 depicts the schematic presentation of the research methodology and approach. The methodology is implemented through six main stages: Literature Review to establish the gaps in Tropical Pavement Design, Experimental Design and Field Studies, Data Collection and Analysis, Development of Pavement Assessment Model and Optimization.

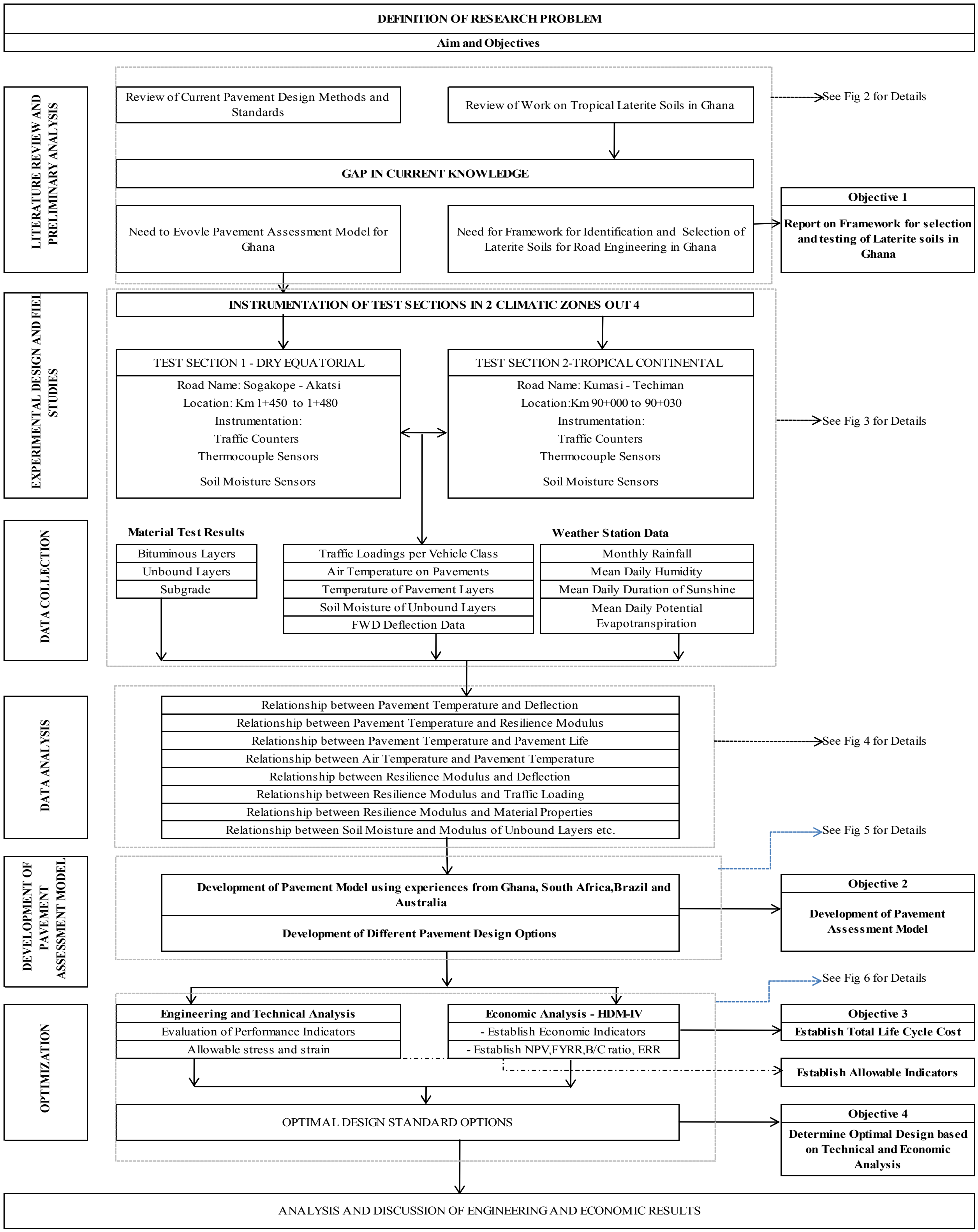



Figure 3.1 - Schematic Presentation of research methodology

### 3.4 Methodology for Preparation of Specifications based on Ghanaian Soils

The need to evolve a specification based on the engineering behaviour of naturally occurring regional construction materials within tropical regions has been advocated by Millard (1990). It has also been established that there is a radical difference in the engineering behaviour of tropical and sub-tropical soils and rocks and those of the temperate regions where the majority of the research and development of current materials standards, specification and construction procedures emanated from. It has also been observed that in the tropics and sub-tropics, roads tend to have non-standard responses to the impacts of the environment and the traffic, unless the approach to investigation and assessment of construction materials is specifically tailored to that environment (Cook et al., 2001). Studies done in Southern Africa by Bradbury showed that roads built with laterite and calcrete non-standard road building materials can carry particularly heavy traffic loadings. These provide important exceptions to the principle that laterite is of a marginal quality when evaluating against traditional specification. In Ghana, it has been observed from the field that road cracks are eminent on surface dressed roads with standard crushed stone base, while natural occurring nodular laterite of the same thickness performs well and show no crack with the same traffic loading.

Cardoza (1987) reported of satisfactory performance at several airfields built with lateritic gravel sub-bases and bases in Brazil. Greenstein (1987) described two airfields in a semi-arid region of Peru built with lateritic gravel that failed to meet conventional airfield base and subbase criteria but performed well for years under Boeing-727 and 737 traffics but only failed dramatically after flooding due to a major storm.

The Overseas Road Note 31 (ORN 31) which is widely used in developing countries in the tropics does not provide any details relevant to the selection, testing and appropriate use of these tropical materials. Neither does it present information on alternative nor standard approaches to material assessment since it only gives one design method.

It can be inferred from the foregoing that the poor performance notwithstanding poor construction quality of roads in the tropics is due to poor selection and utilization because of lack of adequate knowledge about the origin, nature and distribution as well as physical and chemical behaviour of tropical soils. The behaviour of tropical soils differs from that of temperate soils because of the weathering process. While chemical weathering is relatively intense in the tropics due to high temperature and rainfall, weathering of temperate soils is predominantly mechanical; thus, relatively stable (inert) compared to tropical soils which are active as result of continuous weathering of the material under tropical conditions.

It is therefore imperative for tropical countries to undertake research and develop specifications and standards that are pertinent to their particular rock and soils materials for road engineering. Modifications to the testing procedures developed for temperate soils necessary to better reflect the behaviour of tropical materials limits. The preparation of the Engineering Properties Report in this research was based on a detailed review of completed BRRI Research and projects reported by other international research bodies and consultants in the general field of road construction materials in tropical regions. Even though BRRI has embarked on a lot of research towards the preparation of Technical Specifications for the use of laterite soils in Ghana, there have been very few attempts to harness individual efforts of researchers into a consolidated document for road engineering use. This is one of the reasons why Ghana continues to rely on specifications from other countries (sources) which are not fully applicable to these types of soils.

The Engineering Properties Report will be prepared from the geological background of the rocks and soils from the four climatic zones of Ghana. This approach allows a better appreciation of the processes that led to the formation of these soils and provides the framework for identifying material sources and understanding of the chemical composition and mineralogy which influence the material properties and their likely behaviour patterns in an engineering context. The distinct nature of the climatic zones of Ghana and knowledge from the geological history were used as the basis to classify and characterize rocks and soils in each climatic zone.

The classification helped to establish the extent of potential utilization of the rocks and soils in each climatic zone. From the classification, the various findings and recommendations from previous researchers in Ghana and other publications were reviewed and evaluated to arrive at standards for each of the current engineering parameters used in the selection of materials for road engineering work (e.g. particle size distribution, Atterberg"s limits, aggregate crushing value, 10% fines aggregate crushing value, aggregate impact value and Los Angeles abrasion test). The merits of the prepared standards are compared with the current Technical Specification for Road Works and Bridges (2006) used in Ghana. However, the final acceptance of the new specifications and standards will involve their application in the road industry over a period of time and an evaluation of the performance of roads which were based on these new standards.

The Flow Chart in Figure 3.2 outlines the approach adopted.

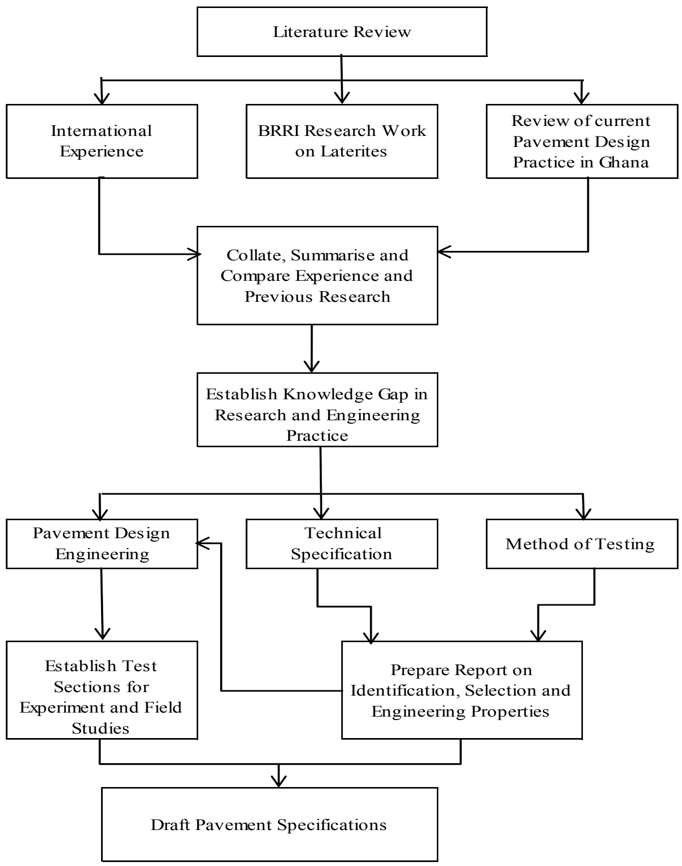



Figure 3.2 – Flow Chart for Field Experimentation and Report Preparation on Engineering Properties

## 3.5 Development of the Mechanistic-Empirical Design Procedure Applicable to Ghanaian Conditions

One of the fundamental limitations of an empirical design procedure is that it does not take account of local conditions. This limits the effectiveness of the approach if the local conditions differ from those in which the empirical method is based.

The Mechanistic Design Approach on the other hand, is based on theories of mechanics and relates pavement structural behaviour and performance to traffic loading and environmental influences. Even though there seems to be progress made in isolating pieces of the mechanistic performance prediction problem; only simple mechanistic responses are easy to compute with assumptions and simplifications, (i.e. homogeneous material, small strain analysis, static loading as typically assumed in linear elastic theory). However, they cannot themselves be used to predict actual pavement performance directly as the pavement materials do not behave elastically. According to mechanics theories, asphaltic concrete is a nonlinear visco-elastoplastic material of which rutting of asphaltic pavement depicts the permanent deformation (plastic) under wheel loads (Peng et al., 2006). The granular layers of the pavement are not elastic and therefore behave nonlinearly. Empirical models are used to fill in the gaps between the theory of mechanics and the performance of the pavement structures and indeed some types of empirical models are required to make appropriate correlations.

The Mechanistic-Empirical (M-E) Design Method which is considered as an intermediate step between empirical and fully mechanistic methods provides the designer with the ability to evaluate the effect of various materials against pavement performance (Gould, 2007).

The distinct positive features of the M-E Design Approach are summarized as follows:

- It can be used for both new pavement construction and pavement rehabilitation;
- It makes use properties of pavement materials that relate to the actual pavement rehabilitation;
- It can effectively accommodate changing load types;
- It allows the use of different types of pavement materials irrespective whether it conforms to empirical standards or not, leading to a better utilization of the existing materials and can accommodate new types of materials; and
- It gives more reliable performance predictions since the state of stress is determined from the engineering property of the individual pavement and soil layers.

Unlike regression-based empirical procedures, mechanistic concepts are generally applicable in such a way that, a full range of future enhancements can readily be developed and implemented. Therefore, the M-E Procedure will not become obsolete with changes in construction materials, traffic patterns, vehicle types or tyre type and configuration, as well as changes in environmental conditions.

The M-E method is an iterative process used in the prediction of the performance of the selected pavement structure which is compared to the design criteria. The structure and the material parameters are adjusted until a satisfactory design is achieved. A step-by-step description is given as follows:

- Definition of a trial design for a specific site"s subgrade support, material properties, traffic loading and environmental conditions;
- Definition of the design criteria for acceptable pavement performance at the end of the design period, (i.e. acceptable levels of rutting, fatigue cracking, thermal cracking and roughness);
- Selection of reliability level for each one of the distresses considered in the design;
- Calculation of monthly traffic loading and seasonal climatic conditions (temperature gradients in asphalt concrete layers, moisture content in unbound granular layers and subgrade);
- Modification of materials properties in response to the local environmental conditions;
- Computation of structural responses, (stresses, strains and deflections) for each axle type and load throughout the design period;
- Calculation of predicted distresses, (e.g. rutting, fatigue cracking) at the end of each time step throughout the design period, using the calibrated empirical performance models; and
- Evaluation of the predicted performance of the trial design against the specified reliability level. If the trial design does not meet the performance criteria, the design (thicknesses or

material selection), must be modified and the calculations repeated until the design is acceptable.

The Flow Chart depicting the above steps as presented by Schwartz and Carvalho (2007) is shown in Figure 3.3.

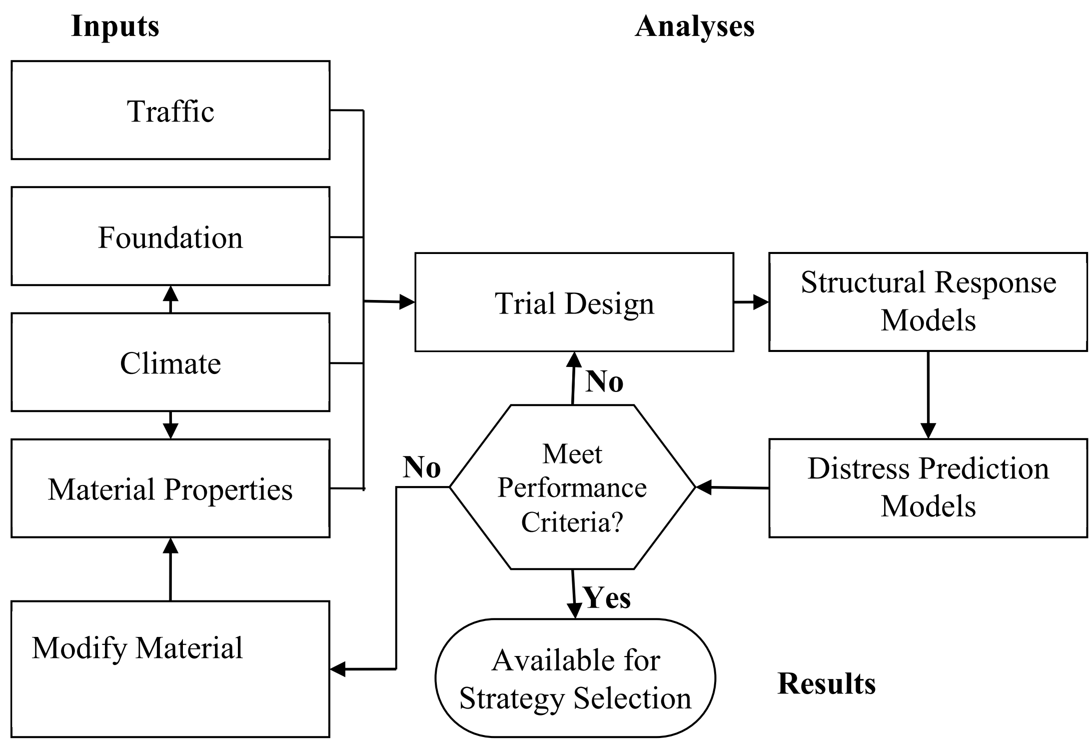



Figure 3.3 – Flow Chart for M-E Pavement Design Method

The M-E Design Procedure developed under this Study is based on the existing techniques. The design is based on an input such as traffic loading to an output such as pavement response. The response is then used to predict pavement distress which is the pavement performance. The design process consists of three major phases:

- Phase 1 is aimed at developing input values and evaluation;
- Phase 2 involves the structural analysis of trial designs, including performance modelling; and
- Phase 3 involves the evaluation of viable alternatives, including life cycle cost, culminating in a final strategy or design selection.

### 3.6 Phase 1: Input Requirement and Evaluation

The Resilient Modulus ($M_R$) is a key input property of pavement geomaterials in the mechanistic-empirical pavement design approach. The properties of the geomaterials to withstand stress (with limited deformation) are characterized through the determination of its $M_R$. The input requirement for the estimation of $M_R$ depends on the method used. There are two basic methods; the destructive and the non-destructive. The destructive method has not been widely accepted since the pavement is disturbed during testing. Another shortcoming is that it is difficult to accurately measure $M_R$ in the laboratory, by simulating actual field loading conditions (Seed, 2000). During this study, the non-destructive testing (NDT) was used. The NDT evaluation of pavement follows one of the two main techniques: surface deflection or wave propagation. For this study, the NDT approach used is the surface deflection (NDOT Research Report RDT 91-025).

Measurements of surface deflections are obtained in the field test sections through the Falling Weight Deflection (FWD) method, using the Dynatest 800 equipment. The deflection data were then evaluated mechanistically (using back-calculation) to determine each layer‟s in situ $M_R$ (Seed, 2000). The ELMOD5 computer programme was used to do the back calculation to obtain $M_R$. The flow chart for the determination of $M_R$ is given in Figure 3.4.

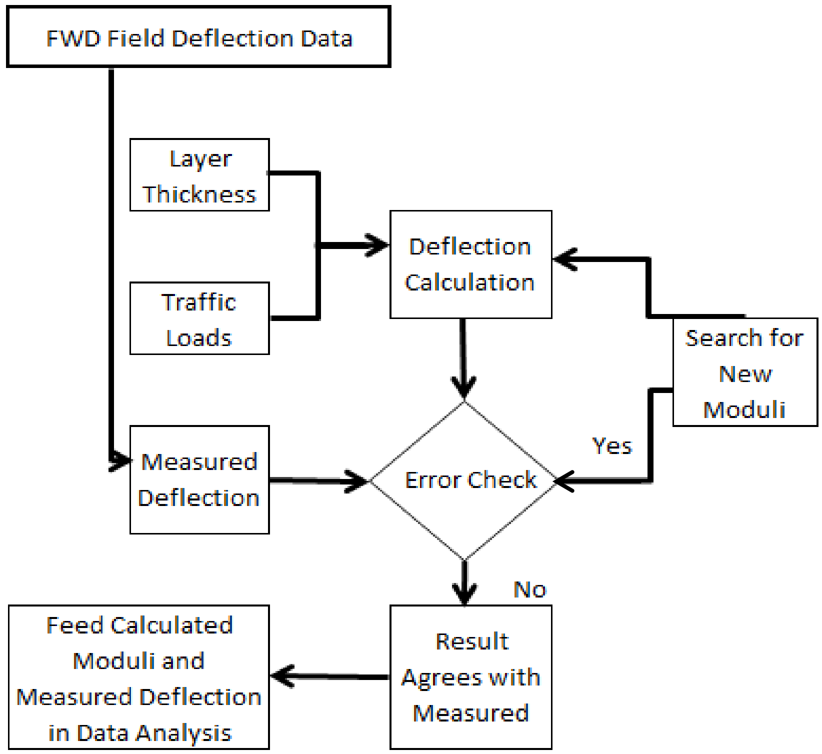



Figure 3.4 – Flow Chart for the Calculation of Resilient Modulus

The use of surface deflection as a distress parameter must be evaluated within the context of elastic theory. Field studies were therefore undertaken to establish the pavement (test section) distress using surface deflection data from the FWD as the distress parameter. The key independent variable must be identified and use to establish its relation with surface deflection. According to Kim et al. (2007), of all the classified sensitive parameters for Mechanistic-Empirical Pavement Design Guide (MEPDG), Actual Annual Daily Traffic (AADT) is the only very sensitive parameter. However, owing to lack of proper traffic database in Ghana, the surface deflection could not be related to AADT. To overcome this shortcoming, a 24-hour continuous collection of traffic data was carried out through the installation of permanent traffic counters. With the installation of traffic counters, the AADT was determined, which was used to accurately estimate the expected cumulative standard axles to be carried by the road pavement over the design life. Additional to the field traffic data, tyre pressures of the representative vehicle class were undertaken as substitute to axle load survey and as input requirement for the mechanistic pavement structural analysis.

Over a period of 12 – hour daily measurements of surface deflections and corresponding traffic loadings for a period of one-week, were undertaken for the dry seasons on the test sections. The surface deflection (subgrade deformation) and the cumulative standard axles were used as input to explain the elastic behaviour of the road pavement. Evaluation of surface deflection based on time data from the field studies and traffic data from permanent traffic counters were used to validate the elastic theoretical behaviour of the asphaltic pavement, as presented by Huang (1994). The surface deflection (total strain) has two components, the elastic and the plastic. In this study, the plastic strain was estimated and used to calibrate the M-E design model.

### 3.6.1 Temperature Considerations

In flexible pavement, the asphaltic concrete layer is a viscoelastic material that is closer to being elastic at low temperature and viscous at high temperature. At low temperature, the pavement has the tendency to contract, tensile stresses develop and friction between Asphalt layer and base layer resist this contraction. If the build-up of internal stresses exceeds the strength of the asphaltic concrete layer, micro cracks develop on the surface and the edge which penetrate to full depth under the action of repeated temperature cycle. At high temperature, the asphalt layer softens and permanent deformation develop under loading condition, the modulus of temperature sensitive layers are reduced significantly by increased temperature which in turn reduce the strength of the whole pavement structure.

The most important environmental factor affecting the surface deflection of flexible pavements is the temperature of the asphaltic layer (Kim and Lee, 1995; Shao et al., 1997; Park et al., 2002) and hence its resilience modulus. A lot of research works continue to be carried out for the determination of actual asphalt temperatures from LTTP databases. From the literature review, there are several equations established to determine the asphalt temperature based on the pavement surface temperature.

Temperature is one of the most dominant climatic factors that impact on the variation of the modulus of the asphaltic concrete layers. An accurate relationship between the temperature and the modulus is necessary to successfully characterize the asphaltic concrete pavement. Variations in modulus are typically related to the average temperature of the layer. The temperature gradient within a layer also plays a role in this relationship. Knowledge of temperature effects is essential for the determination of the frequency and the type of maintenance required throughout the pavement‟s service-life. It also has a tremendous influence on the mix design. Most of the mix design is based on a standard temperature of 25°C. Under this study, the appropriateness or otherwise of this mix design will be ascertained through the field studies.

Road Agency Organizations, such as the Federal Highway Administration (FHwA) of the USA, have established a long-term pavement performance (LTTP) monitoring programme and have seriously focused on this issue for some time.

Hot Rolled Asphalt (HMA) pavements are termed viscoelastic; the structural load carrying capacity of such pavements varies with temperature. Thus to accurately determine in situ strength characteristics of flexible pavements, it is necessary to predict the temperature distribution within the HMA layers, taking into consideration the influence of the ambient temperature and seasonal changes.

Most of these attempts to predict the asphalt layers temperature are however not very accurate. Under this Study, all the pavement layers were fitted with thermocouples to determine the actual temperature in two of the climatic zones all year round and information was collected at 30 minutes intervals for two and half years. The temperature data were used to validate figures from the thermometer. The stiffness of the asphalt layers were designed based on the expected temperature regime.

Since road pavement layers consist of unbound granular materials besides the asphaltic concrete, its impact on the performance of the road pavement must be evaluated and its contribution to the overall performance of the road pavement appreciated. The moisture content regimes in the granular layers due to seasonal climatic changes have an effect on the resilient modulus of the road pavement. Under this Study, the moisture regimes in the unbound granular layers were studied through the installation of moisture gypsum blocks and daily recordings taken.

### 3.7 Phase 2: Structural Analysis

The fundamental outputs of mechanistic analysis can be based on four theories, namely; linear elastic, non-linear elastic, viscoelastic or plastic. The mechanistic part is directed to calculating one or more responses in the pavement structure as a function of material properties, layer thicknesses, loading conditions and the impact of the environment. These response(s) must then be related to the observed performance criteria. According to Bhutta (1999), there are three major performance criteria, namely:

- Compressive vertical strain at the surface of the subgrade which controls the permanent deformation of the subgrade;
- Horizontal tensile strain in the HMA layer, generally at the bottom, which controls the fatigue cracking of the layer; and
- Permanent deformation (rutting) of the asphalt layers.

Gedafa et al. (2009) reported that the AASHTO algorithm for back-calculation of the subgrade modulus established that deflections measured at the pavement surface are due to subgrade deformation only. In this study, the structural response is related to the deflection measured on the pavement surface which is due to this deformation at the surface of the subgrade. The field measured surface deflection will be used to calibrate the selected mechanistic model for the pavement structural analysis.

For the damage analysis of the pavement, the compressive vertical strain at the top of the subgrade and the horizontal tensile strain under the HMA layer will be used.

There are several computer programmes that use mechanistic approach to determine the structural response of a road pavement upon the input of vehicular loading, materials and environmental interactions. Many developed countries have adopted mechanistic design methods at various levels to evolve a catalogue of pavement designs based on traffic loading, subgrade type, environment and standard material specification. Bhutta, (1999), in his thesis, gave a brief description of the research work done in the area of development of various programme to undertake structural analysis. The KENLAYER is used under this research to carry out the pavement structural analysis. The KENLAYER was developed by Huang (1994). The method of analysis is similar in theory to the other programmes. However, it has the following extra capabilities which are appropriate for the research objectives:

- More materials models can be used (linear-elastic, non-linear elastic, viscoelastic, and a combination thereof);
- Different material parameters may be entered for each seasonal period;
- More flexibility exists with regard to the sub-division of the seasonal variations;
- Detailed characterization of traffic loading with respect to speed, etc;
- A total of 19 pavement layers can be examined; and
- Users can specify the parameters of the critical failure criteria.

The challenges in pavement engineering do not end with the selection of an appropriate design method or approach. It is the objective of the Road Agencies to design and construct roads which are economic in terms of the initial capital investment required for its construction and thereafter the maintenance requirement during the life span of the road pavement. The concept of Total or Whole Life Cycle Cost is used to establish the total cost of the selected designs considering the initial cost of construction and the maintenance cost during the life of the pavement.

The HDM-IV is a tool which is widely acclaimed to undertake life cycle analysis among other tools. Various editions of the AASHTO Pavement Design Guide (1986 and 1993) encourage the use of the life-cycle cost analysis to establish the true cost of any selected design through the economic evaluation of alternative pavement design strategies (Kansas Department of Transport, 2002). The reliability of these analyses depends on the calibration of models to simulate local conditions. A Flow Chart for the Life-Cycle analysis is presented in Figure 3.5.

After the pavement structural analysis, the final acceptance of a given design must be subjected to A Whole Life Cycle Analysis, in order to appreciate the total cost implication of selecting a given pavement design with respect to the cost of construction, maintenance and rehabilitation costs over the entire design life of the pavement.

## 3.8 Phase 3: Evaluation of Viable Alternatives through Life Cycle Analysis

All road pavements material properties undergo changes during the construction phase and when they are finally subjected to traffic loadings. Therefore, a pavement"s life starts from when the pavement is built, how its condition changes over time, and how different forms of interventions in terms of maintenance, rehabilitation and reconstruction can affect this change process. The selection of optimal maintenance, rehabilitation and reconstruction interventions must not be arbitrary but must be based on objectivity, and the use of scientific tools to arrive at decisions.

The process of pavement design optimization leads to the selection of alternate designs that minimize the entire cost of the pavement through its design life.

The scope of this research does not cover in full, the merits and demerits of each of the above models. The HDM-IV model was used to carry out the optimization of the various pavement design alternatives. The HDM-IV model was calibrated for Ghanaian conditions through field studies, and those of the cross-section data analysis technique in order to ensure that model"s key parameters simulate observed trends in Ghana. The model was also configured to ensure that its standards are consistent with Ghanaian conditions. A flow chart for Life – Cycle Analysis is presented in Figure 3.5.

## 3.9 Summary

The ensuing chapters will address the input data described in this chapter. The Data Collection Approach is fully given in Chapter 4 which gives details of establishment of experimentation site to collect traffic data, pavement temperature, and moisture content require. Other supplementary data such as the deflection measurement which was used to back-calculate the respective Resilient Modulus of the pavement layers is given.

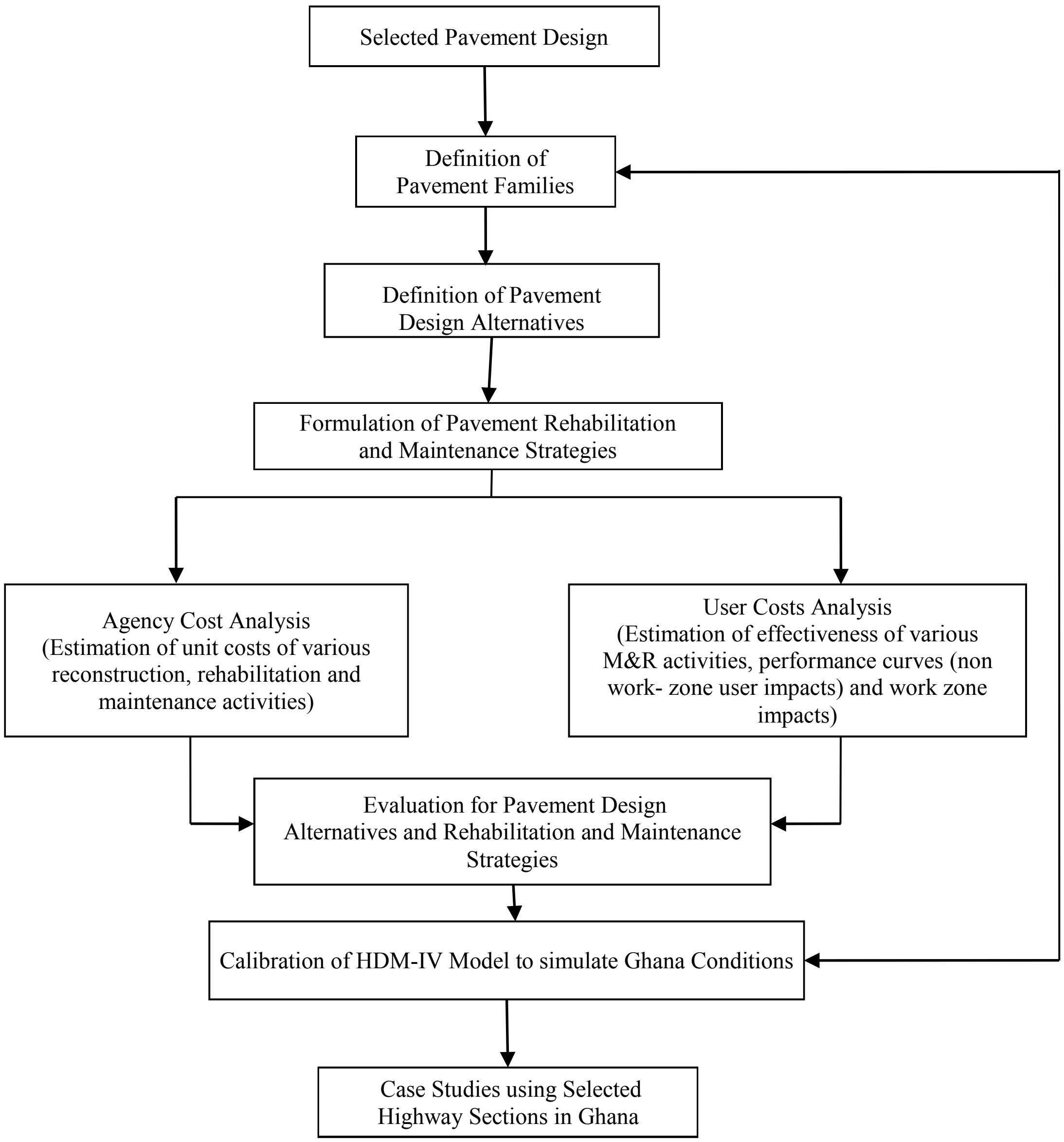



Figure 3.5 – Flow Chart for Life Cycle Analyses (Modified after Lamptey et al., 2005)

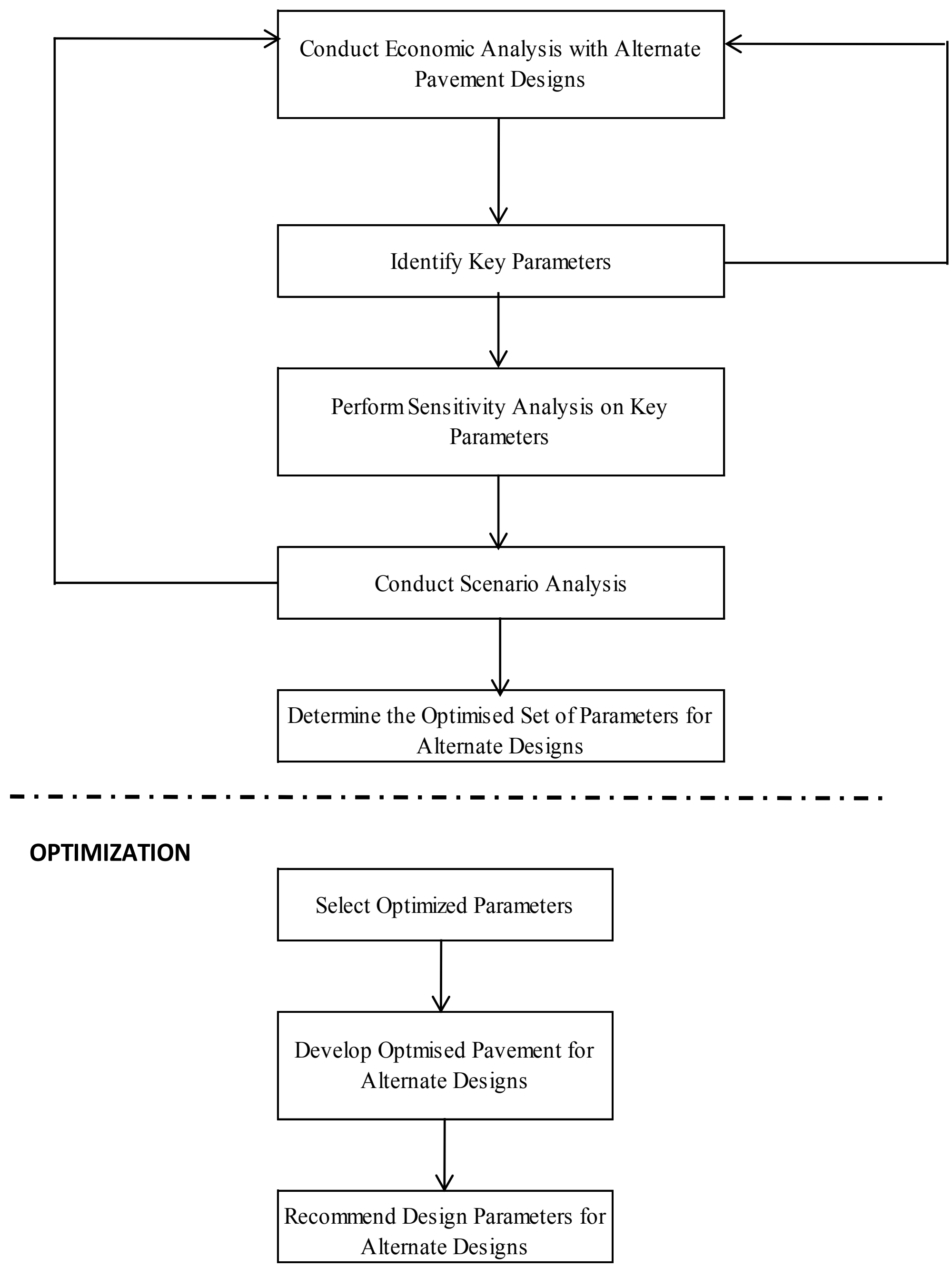



Figure 3.6 – Flow Chart for Economic Analysis and Optimization (HDM – IV)

# CHAPTER 4

# EXPERIMENTAL DESIGN, FIELD STUDIES AND DATA COLLECTION

## 4.1 Introduction

This section of the study outlines the design of experiments for data collection undertaken as part of the research. It gives the outline of the data collection, the type of field data collection, procedure for installation of instruments and equipment at the test section. The criteria used in the selection of the test sections in the two climatic zones of Ghana. A description and profile of the study zones of the field studies are also provided.

In order to address the issue of the data required to develop the frame work for Mechanistic-Empirical design studies were undertaken to collect field data which were used to calibrate computer models. Two sites, Akumadan in the Moist Sub Humid Zone and, Sogakope in the Semi-Arid Zone were instrumented (see Figure 4.1).

## 4.2 Akumadan Test Site

The Akumadan Highway is classified as a National Road (N10) based on the Ghana Highway Authority (GHA) functional classification scheme produced by Centre for Remote Sensitivity and Geographic Information Services of the University of Ghana and also forms an integral part of the Trans-West Africa Highway. It connects major population centres in the southern and northern parts of Ghana, namely Kumasi, Accra, Tema, Cape Coast and Takoradi all to the south and Tamale and Bolgatanga to the north. The road also provides a vital link between Ghana and her neighbouring countries; Burkina Faso to the North and Togo to the South.

### 4.3 Sogakope Test Site

The Sogakope Highway forms an integral part of the ECOWAS Trans-West African Coastal Highway Network System. The road traverses from the East to the West and forms part of the National Road Network System, designated Route N1. The road provides a vital link between Ghana and her neighbouring countries; La Cote D‛Ivoire to the West and Togo to the East and serves the international communities in the sub-region.

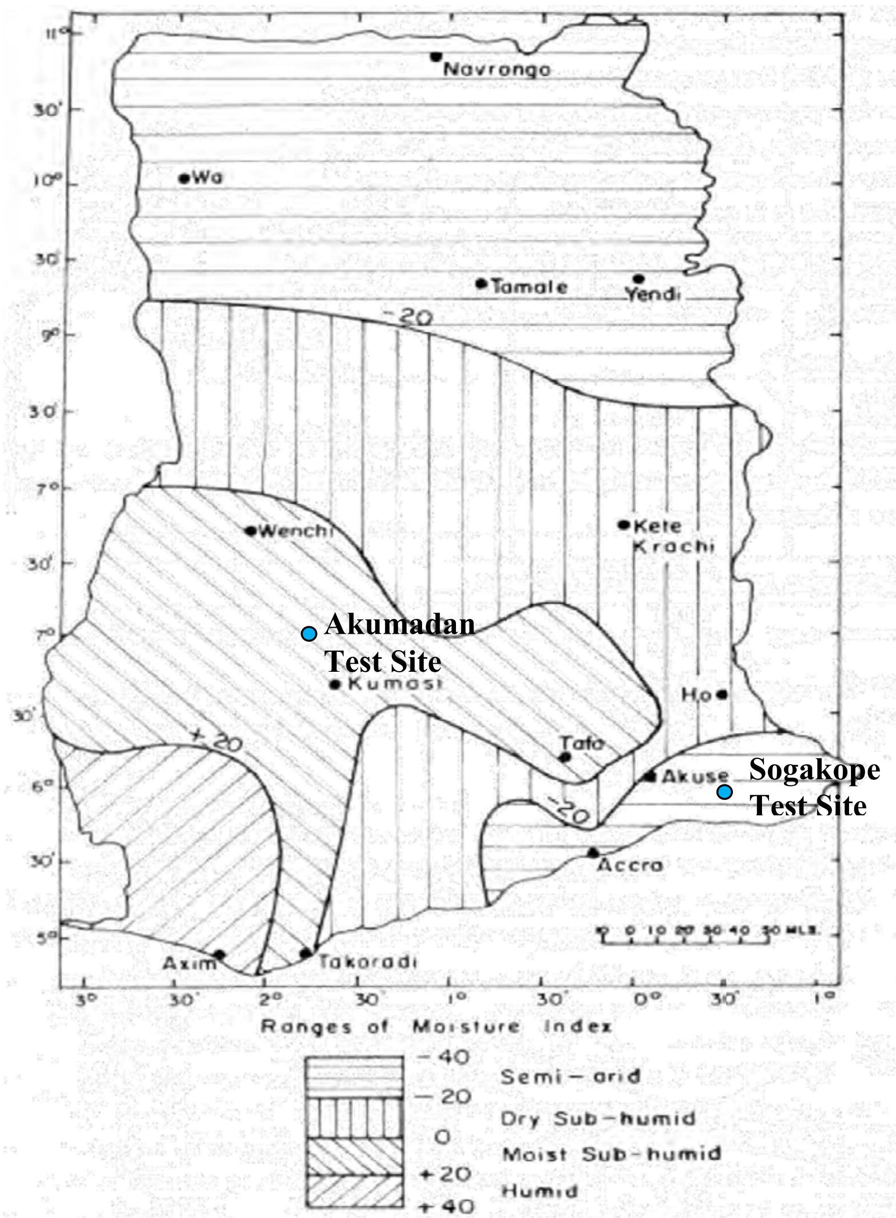



Figure 4.1 – Climatic Map of Ghana showing Test Site (Arulanandan et al., 1963)

## 4.4 Data requirement

The objective of the field studies was to obtain the test data required for the research. The data collected with respect to the pavement temperature, moisture content and continuous traffic data using field installed automatic counters is pioneering work in Ghana.

Data collected from the field studies provided actual condition of the road pavement and the real traffic loading the pavement sections are subjected to. It provides important information as vindication of differences with respect to tropical and temperate environmental conditions. It also gives important information on the residence behaviour of the asphaltic concrete layer as a visco-elastic material, in a tropical environment and validation of theoretical behaviour of material which is a key assumption used in the mechanistic approach to pavement design.

The instrumented test sections made it possible to determine the temperature of the various layers. It also provided information on the state of moisture content in the granular layers. The field studies carried out in this research is to get information on actual behaviour of the road pavement under loading in a tropical environmental condition. Details of the data variables collected during the field study required for assessment of pavement performance in the tropics have been identified. The information collected can be classified into the following areas;

1. Pavement Material Test Data (Appendix A);
2. Weather Station Data (Appendix B); and
3. Field Studies Data (Appendix D.

The flow chart for the data collection is given in Figure 4.2.

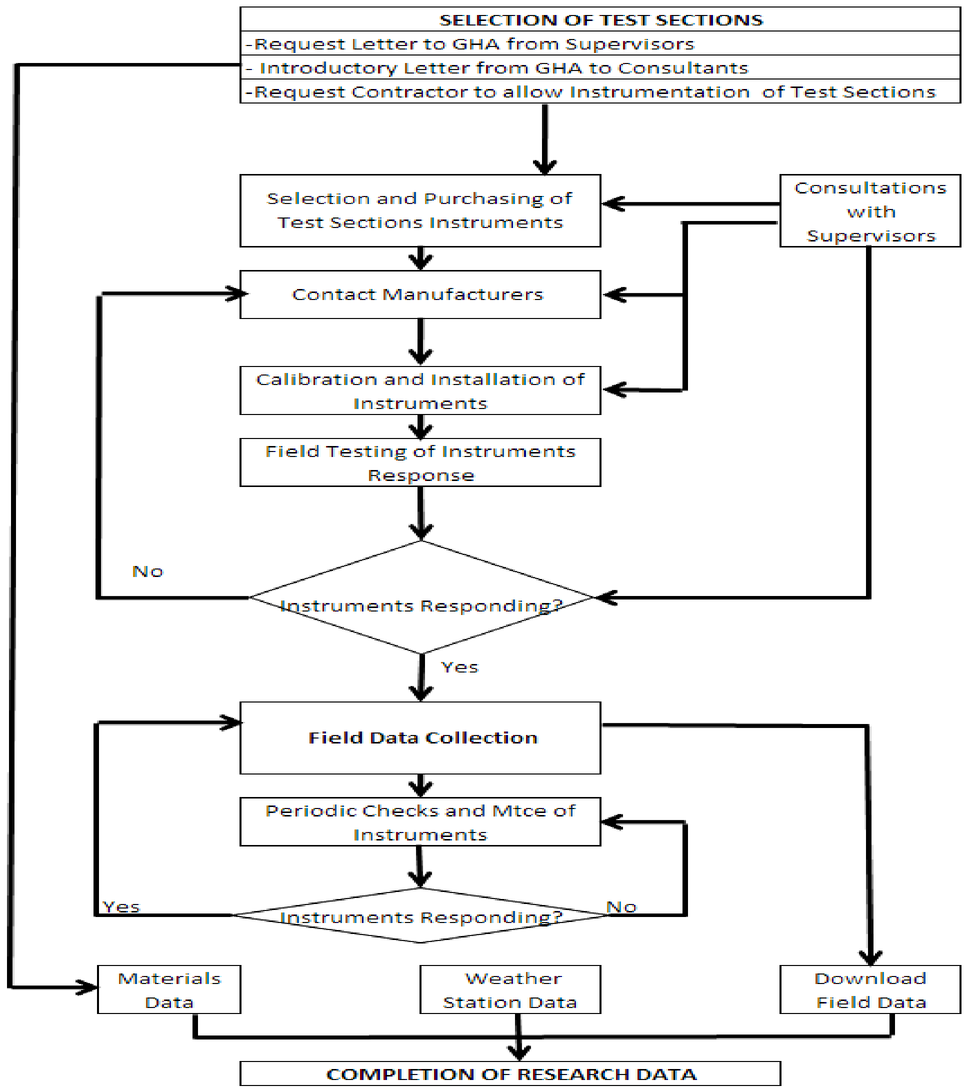



Figure 4.2 – Flow Chart for Data Collection

From the flow chart, the three types of data were collected during the three phases of pavement life for the respective pavement layers as shown in Figure 4.3. The phases are;

- Before Construction;
- During Construction; and
- Monitoring of Pavement Performance.

Table 4.1 – Classification of Data

<table>
<tr><th colspan="2">Material Data</th><th>Weather Station Data</th><th>Field Study Data</th></tr>
<tr><td rowspan="5">Subrade</td><td>Grading</td><td rowspan="14">Monthly Rainfall<br><br>Mean Daily Temperatures (Maximum and Minimum)<br><br>Mean Daily Relative Humidity (at 6.00 and 15.00 Hours)<br><br>Mean Daily Duration of Bright Sunshine<br><br>Mean Daily Potential Evapotranspiration<br><br>Mean Daily Vapour Pressure (at 6.00 and 15.00 Hours)</td><td rowspan="14">Pavement Temperatures<br><br>Moisture Content (Base and Subbase)<br><br>Traffic<br><br>Deflection<br><br>Resilient Moduli</td></tr>
<tr><td>Plasticity Properties [1]</td></tr>
<tr><td>Laboratory Compaction Properties [2]</td></tr>
<tr><td>CBR</td></tr>
<tr><td>Natural Moisture Content</td></tr>
<tr><td rowspan="4">Subbase & Base Courses</td><td>Grading</td></tr>
<tr><td>Plasticity Properties [1]</td></tr>
<tr><td>Laboratory Compaction Properties [2]</td></tr>
<tr><td>Field Control Parameters [3]</td></tr>
<tr><td rowspan="5">Asphalt Courses</td><td>Marshal Test Parameters [4]</td></tr>
<tr><td>Mixing and Compaction Temperatures</td></tr>
<tr><td>Grading</td></tr>
<tr><td>Aggregate Properties [5]</td></tr>
<tr><td>Creep Compliance</td></tr>
<tr><td colspan="4">(1) - Liquid Limit (LL), Plastic Limit (PL), Plasticity Index (PI), Plasticity Modulus (PM) and Swell</td></tr>
<tr><td colspan="4">(2)- Maximum Dry Density (MDD) and Optimum Moisture Content (OMC) (Modified Compaction)</td></tr>
<tr><td colspan="4">(3)- Average Thickness, Field Density and Moisture Content and Relative Density</td></tr>
<tr><td colspan="4">(4)- Bulk S.G., Bitumen Content, VMA, VOID, VFB, Stability and Flow</td></tr>
<tr><td colspan="4">(5)- Specific Gravity (S.G.), Relative Density (SSD), Water Absorption, LAAV, ACV, AIV, Flakiness, Sand Equivalent, Stripping, Chloride Content, Sulphate Content and 10% Fines (Wet and Dry)</td></tr>
</table>

It must be noted that instrumentation trials were carried at the Sogakope test site first to ensure that all the challenges encountered during the instrumentation trials were addressed before the instrumentation of the second test site. Challenges encountered will be described later. Instrumentation of each site was the same: Moisture profiles in the granular layers of the pavement, using a soil moisture meter, temperature profile of the respective pavement layers, using thermocouples and installation of traffic counters to collect information on traffic.

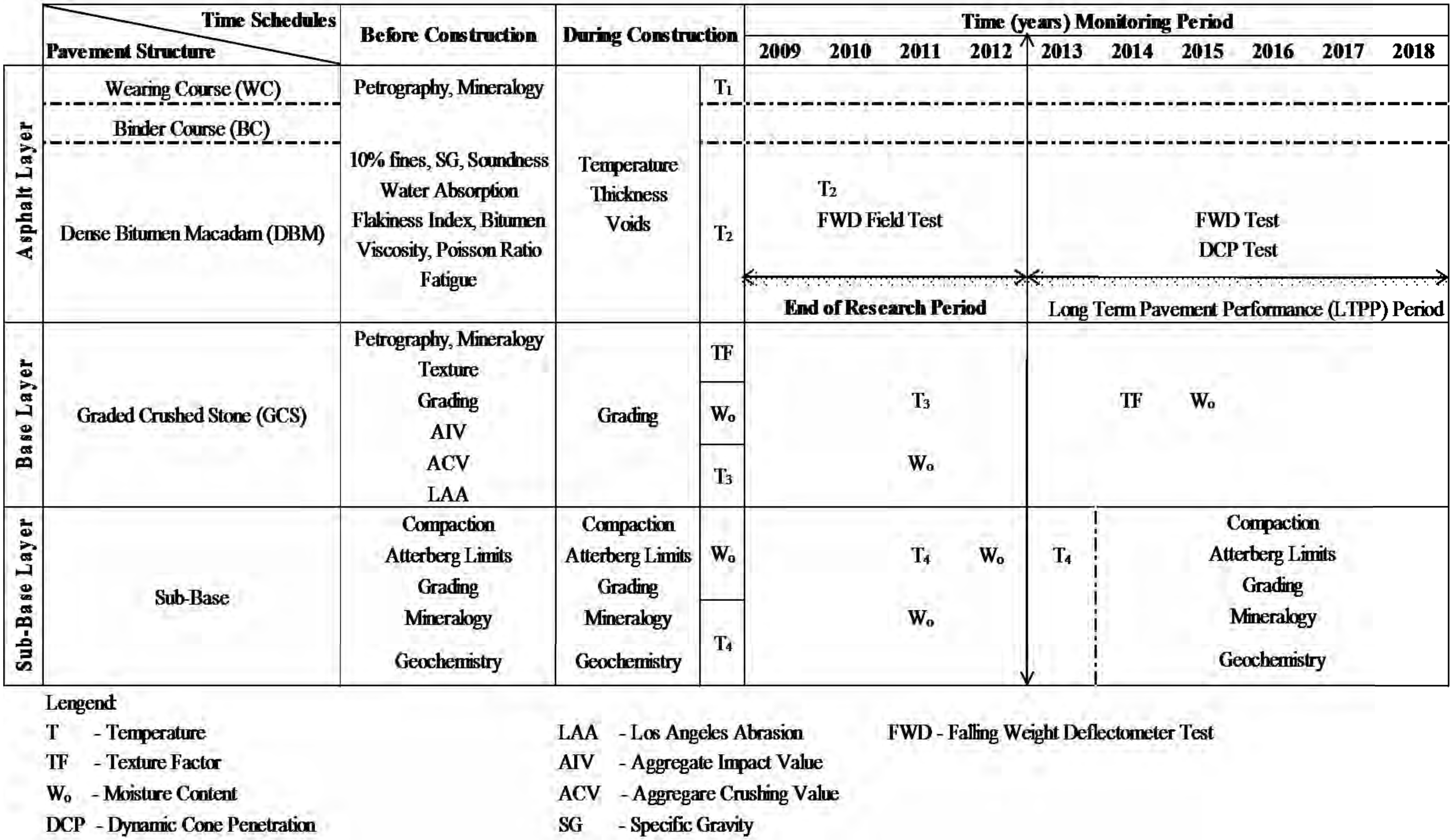



Figure 4.3 - Outline of Field Studies

## 4.5 Procedure for Data Collection

### 4.5.1 Installation of Traffic Monitoring Equipment

The following sets of equipment were installed to carry out the monitoring of the Pavement Sections:

#### 4.5.1.1 Setting-Up Marksman M660 Traffic Counter

Monitoring of traffic loading in the Test Section was done with the Marksman M660 Traffic Counter manufactured by Golden River Limited, Bicester in the United Kingdom (See Figure 4.4).

The installation of the Marksman M660 followed strictly in accordance with the manufacturer‟s instructions. However, serious challenges were encountered in setting up the traffic equipment. This led to considerable exchanges with the Equipment Manufacturers over a period of 12 months. The problem was only resolved after a visit to Bicester, UK and presentation of the list of equipment and accessories received in Ghana. Inspections of the freight list revealed that a wrong marksman-computer cable was supplied and was the main cause of the installed software‟s inability to pick signals from the Marksman 660.

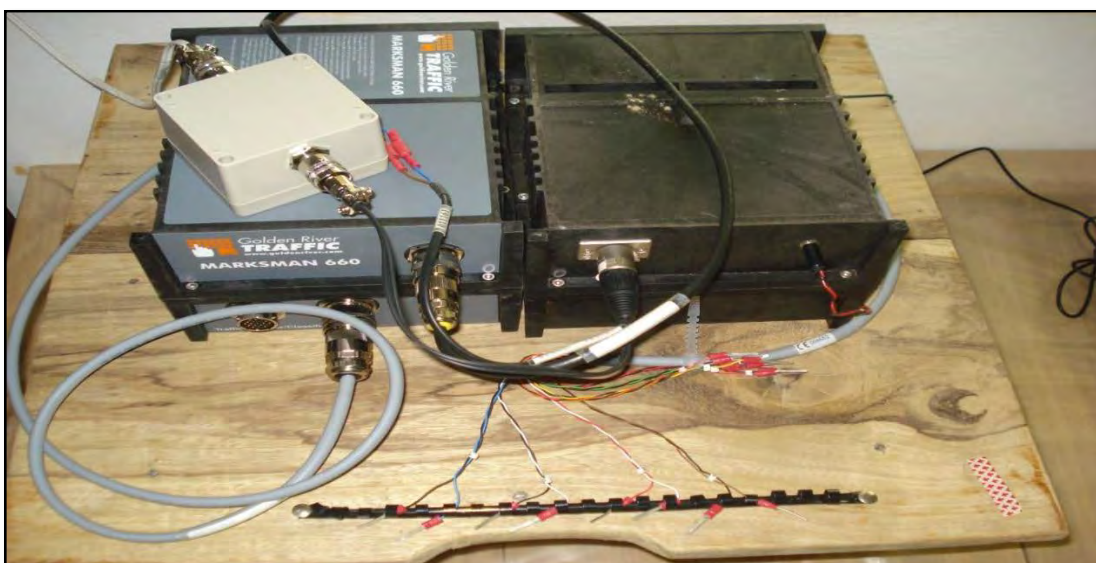



Figure 4.4 – Presentation of Marksman M660 Traffic Counters

#### 4.5.1.2 Traffic Loops and Marksman Sensors Connections

The sensors enabled the Marksman 660 to determine traffic presence and the characteristics of vehicles or other events. Several types of sensors are used by the Marksman 660. These include the Tube, Loop, Piezo, WimStrip and Pollution detectors. These Sensors can be combined in several ways to perform specified tasks like counting vehicles, lengths and speeds of vehicles, axle counting, class, headway, gap, wheelbase, weight and checking pollution.

Loop sensors are the most commonly used, since they have no moving parts and are long-lasting. A loop-loop configuration will help in determining the following:

- Speed of Vehicle;
- Length of Vehicle;
- Gap between Vehicles;
- Headway between Vehicles;
- Direction of Vehicle;
- Number of Vehicles passed; and
- Class of Vehicle.

The loops were laid in the road-way as directed in the Marksman 660 Manual as shown in Figure 4.5.

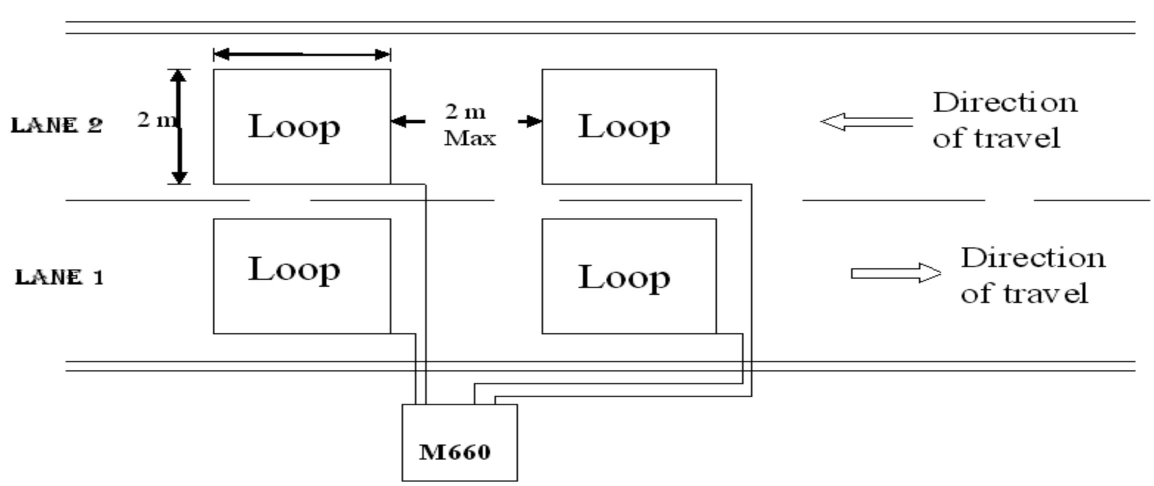



Figure 4.5 – Specification for Arranging the Induction Loops

The slots to receive Loop sensors were cut in accordance with the manufacturer‟s specifications. The outline of the chambers (2m x 2m) was marked on the road. Adjacent Loop chambers in each road lane are separated by 2m. Using an Asphalt cutter, each chamber was cut to a depth of 50mm shown in Figure 4.6.

Figure 4.6 – Field Construction of Induction Loops Site

The Loop chambers were then sprayed with bitumen before the Loop Cables were placed in them (see Figures 4.7 and 4.8). The cable is held firmly to the ground while asphalt is laid and compacted over to cover the cables as shown below.

Figure 4.7 - Field Preparation for Installation of Induction Loop

Figure 4.8 – Field Installation of Induction Loops

For the purpose of this research and the level of accuracy required, only the Loop Detector and a Loop-Loop configuration were used. A Loop Detector consists of a coil of wire (6.0 $mm^2$ electrical cable) buried in the road. As the installation did not give any indication of the number of turns required for a vehicle presence to be picked by the sensor, one turn was used and during the trials the sensor was not picking signals. This was resolved with the manufacturers who directed that a minimum of four turns was required to detect presence of a vehicle as shown in Figures 4.9 and 4.10.

So, typically, a Cable Loop will be rectangular in shape and will have 4 turns of wire and measure about 2m across the width of a lane and 1–2m along the direction of travel before vehicle presence could be registered by the Marksman traffic equipment.

Figure 4.9 – Preparation of Induction Loops on Site

Figure 4.10 – Specified Turns of Induction Loops

#### 4.5.1.3 Connecting Loops to Marksman

The Detector Card was connected to the loops to the Marksman through its marked terminal strip in the order given in the sensor layout (loop-loop). The lead marked „A"", was connected first and the rest followed in sequence. Loop connections were simply in pairs with no special polarity requirement. Sensor readings will not be recorded correctly unless the sensors are connected to the Marksman 660 in the sequence that the instrument has been programmed in accordance with the SENSOR command of the computer programme.

The terminal strip of the Detector Card has paired leads marked from „A"" to „J". The first loop in lane 1 (in the direction of travel), was connected to lead „A"" and the second, to lead „B"" (see Figure 4.10). The first loop in lane 2 (in the direction of travel), was connected to lead „C""while the second was to lead „D"" of the terminal strip. The interface connection between the Traffic Loops and the Marksman 660 Sensors are depicted in Figure 4.11.

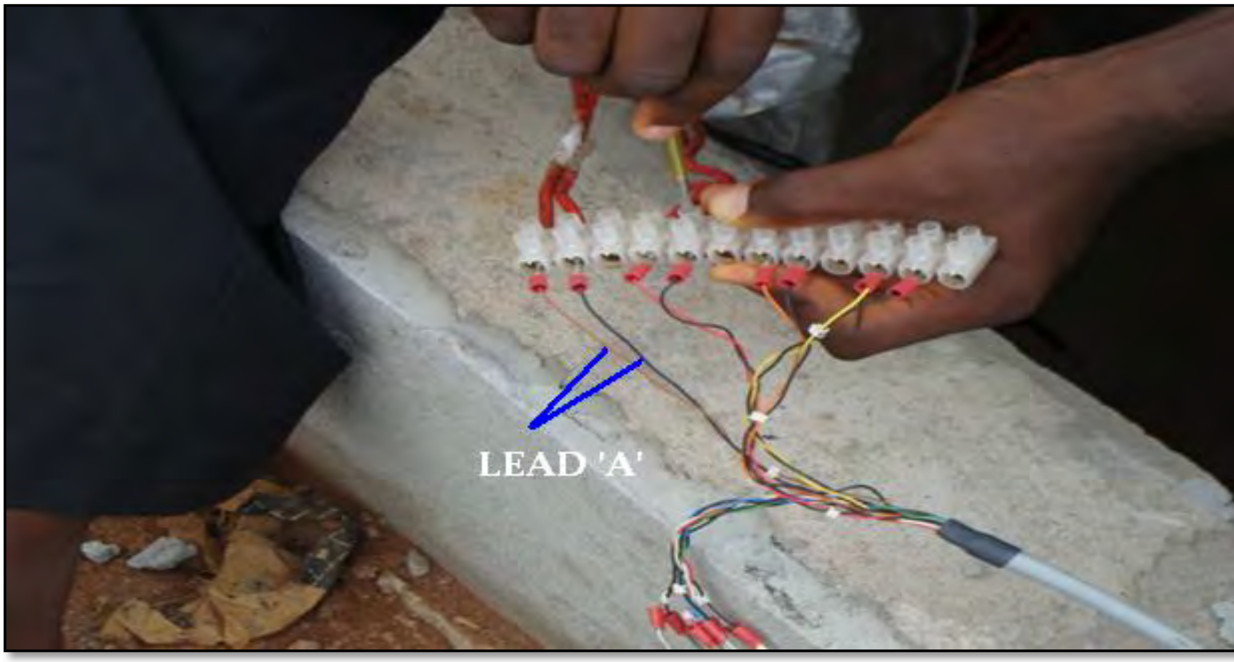



Figure 4.11 – Connection of Sensors to Marksman 660 Traffic Counter

The Detector Card in the Marksman 660 energizes the loop with a pulse of oscillating current. This creates a „zone of detection" extending beyond the periphery of the Loop (about 50 cm from the edges of the Loop). When the metal part of a vehicle enters the „zone of detection" of the loop, it affects the current in the Detector, and triggers the Detector output signal. A car enters the zone earlier, but leaves later than a truck (see Figure 4.12). Thus, the length of high-bodied vehicles is mostly underestimated.

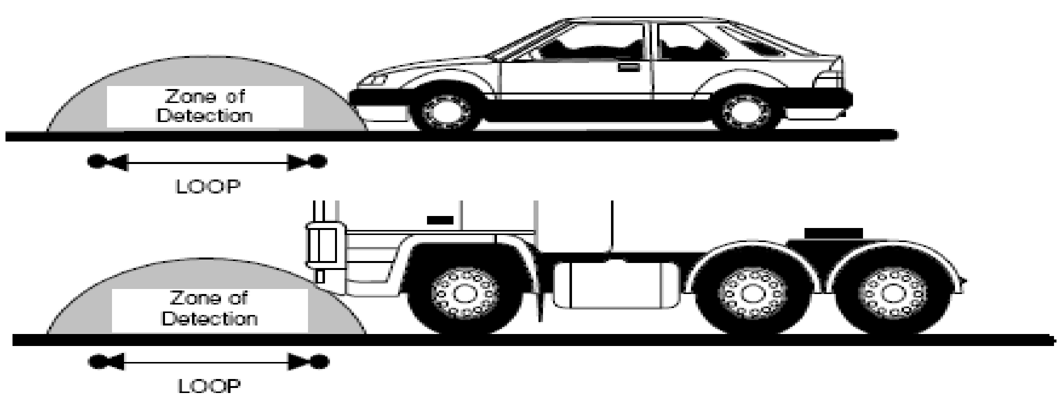



Figure 4.12 – Demonstration of Operation of Traffic Sensors

In a layout incorporating several loops connected to the same detector card, each loop is pulsed in sequence while all the others are inactive. To record a valid „detect" event, (vehicle arrived) or „un-detect" event (vehicle departed) a number of successive scans of the same loop must all agree.

The Marksman 660 measures the strength of the signal of a vehicle above the Loop Sensor to distinguish between various types of vehicles. Detection sensitivity is greatest along the sides of the Loop (as shown in Figure 4.13), that are parallel with the direction of travel, and least sensitive along the sides running across the road. Hence a narrow vehicle like a motorcycle would give a higher level of signal when travelling close to the edges of the lane and least signal, when travelling down the middle of the lane.

Figure 4.13 – Completed Sites for Traffic Counters

### 4.5.2 Vehicle Classification

Vehicle classifications were based on Class (EUR 6) of vehicles. EUR 6 Classification has been customised to meet Ghana"s Vehicle Classification, shown in the Table 4.2. The equivalence between the EUR 6 classification and Ghana classification was established from actual field study.

Table 4.2 - Vehicle Classification Equivalence

| Ghana Vehicle Classification | Vehicle No. | EUR Equivalent |
|---|---|---|
| | | EUR 6 Classification |
| Car | 01 | CS2 |
| Taxi | 02 | |
| Pickup | 03 | |
| Light Truck | 04 | CS4 |
| Medium Truck | 05 | |
| Heavy Truck | 06 | |
| Articulated Truck | 07 | CS5 |
| Small Bus | 09 | CS6 |
| Medium Bus | 11 | |
| Heavy Bus | 10 | |
| Motorcycle | 12 | CS1 |
| Car + Trailer | 08 | CS3 |

### 4.5.3 Traffic and Axle load survey

#### 4.5.3.1 Objective of the Survey

The objective of the survey is to determine the traffic flow characteristics in terms of volume, vehicle type and flow pattern on the project road and to provide the necessary data for the geometric and pavement design of the proposed project.

#### 4.5.3.2 Classified Manual Traffic Count

A 12-hour classified manual traffic counts were carried out for seven consecutive days at established points along the Akumadan and Sogakope Highways. The classified traffic counts were collected to conform to the Ghana Highway Authority (GHA) approved traffic categories.

The Average Daily Traffic (ADT) obtained for the Akumadan and Sogakope sections of the project road are as indicated in Table 4.3;

Table 4.3 - Traffic Count Results

| Census Station No. | Road Section | Census Point | 12hr Count | Factored 24hr Count | ADT |
|---|---|---|---|---|---|
| AK– N10-08 | Kumasi-Akumadan-Techiman | Akumadan | 18,642 | 23,116 | 3,302 |
| VK–N1–09 | Tema-Sogakope-Aflao | Sogakope | 39,751 | 48,644 | 6,949 |

The deterioration of paved roads caused by traffic is due to both the magnitude of the individual wheel loads and the number of times these loads are applied. For pavement design purposes, it is therefore necessary to consider not only the total number of vehicles that will use the road but also the vehicle wheel or axle loads. Hence, both traffic count and axle load information are essential for pavement design.

Axle load survey is carried out to determine the axle load distribution of vehicles using the road. The survey data are then used to calculate the mean number of Equivalent Standard Axles.

The survey was conducted at Sogakope and Akumadan trunk roads. A portable weigh pad equipment was used for the study. The survey lasted for a period of two days; from 6am to 6pm each day. The vehicles were weighed either loaded or empty and were recorded axle by axle.

### 4.5.4 Setting Up of Temperature Monitoring Equipment

The set-up for the installation of the thermocouple equipment is schematically shown in Figures 4.14a and 4.4b, following a procedure in the Manual supplied by the Manufacturer.

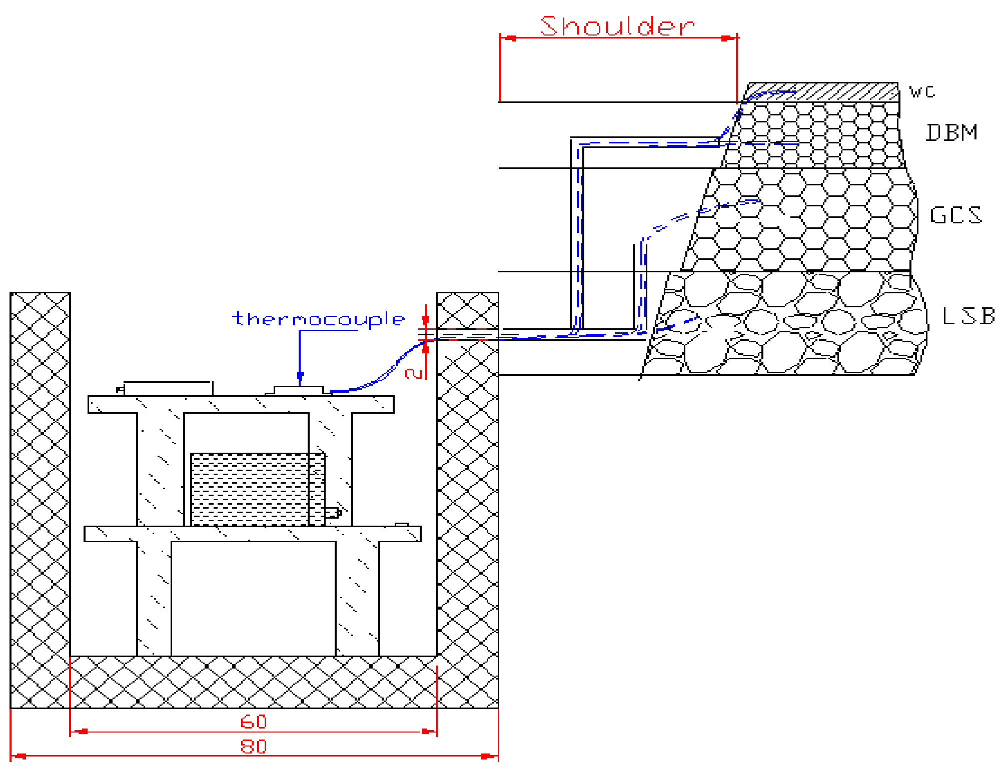



Figure 4.14a – The General Installation Setup Showing the Thermocouple and Position of Sensors

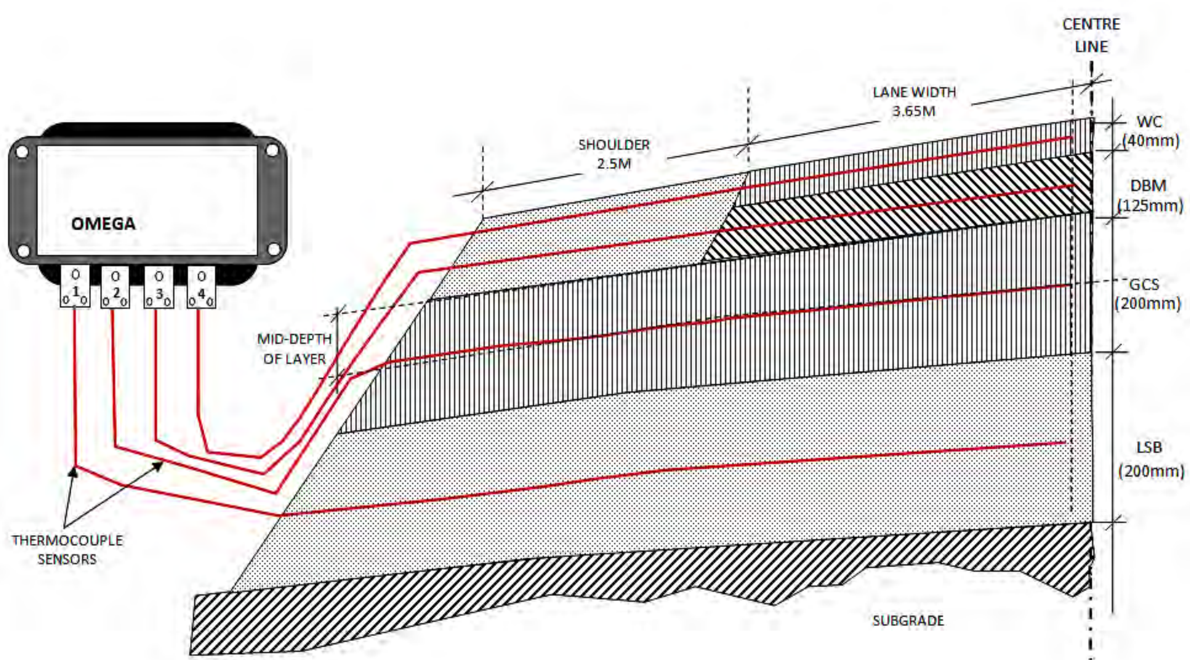



Figure 4.14b – Schematic Preparation of Temperature Monitoring Equipment

The temperature regime in the various layers of the pavements was monitored using the thermocouple equipment (OM-CP-QUADTEMP) manufactured by Omega Laboratories in USA. The installation process was straightforward. The equipment has the following components, a thermocouple recorder, temperature sensors and USB Data Logger interface.

The OM-CP-QUADTEMP has four channels for measuring temperature (TC1, TC2, TC3 and TC4) as shown in Figure 4.15. Each port is connected to a Sensor numbered 1, 2, 3 and 4 which monitor temperature in each of the road pavement layers in which the sensor is installed. Table 4.4 shows layers whose temperature the various sensors record.

Table 4.4 – Sensors and their corresponding layers

| Thermocouple Sensor | Pavement Layer |
|---|---|
| 1 | Subbase (LSB) |
| 2 | Graded Crushed Stone (GCS) |
| 3 | Dense Bitumen Macadam (DBM) |
| 4 | Wearing Course (WC) |

The temperature range capacity of the equipment for ambient condition is from -20 to +60°C

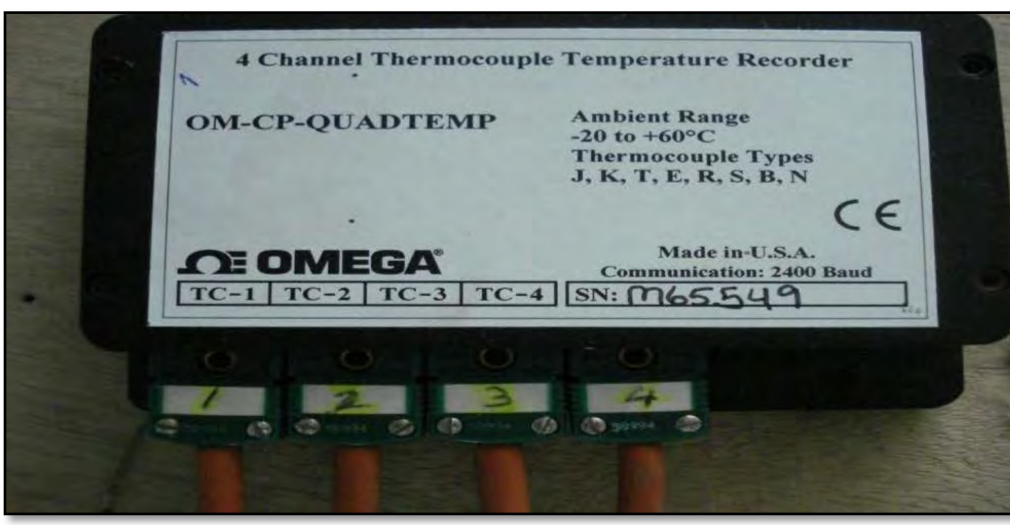



Figure 4.15 – Presentation of Temperature Channels for Storing Data

#### 4.5.4.1 Thermocouple Calibration

Thermocouple equipment was calibrated to improve the accuracy and reliability of the output so that a comparison could be made with the observed temperature readings obtained using a digital thermometer. The USB Data Logger was connected to the computer and the Thermocouple Recorder. The method of calibration is outlined below:

The temperature sensors were connected to the USB Data Logger and put into a water basin on fire. A digital thermometer was also put into the water basin. From the Device Menu, the "Real Time Chart Recording" and "Start Recording" were clicked. At a given time, the temperature of the water recorded by the digital thermometer and the thermocouple recorder was observed and noted.

From the Device menu, Calibration was selected. The Calibration Wizard Screen showed up and the ambient temperature was set. The temperature recorded by the digital thermometer was entered against the Thermocouple 1 reading on the Thermocouple Recorder. This was repeated for Thermocouple 2 up to 4. The Calibration Wizard automatically calculated the offsets and gains of the Thermocouple Recorder. The difference in readings between digital thermometer and the thermocouple equipment was negligible (See Table 4.5).

### 4.5.5 Setting Up of Soil Moisture Equipment

The set-up of the Soil Moisture equipment is shown in the Figures 4.16, 4.17 and 4.18. The installation was essentially the same as that for the thermocouple.

.. Whilst ideally moisture movement in the subgrade should have been measure, this was not possible as the contractor who was responsible for the test sectionwas not prepared to take responsibility in the event of damage to the pavement if instrumentation was located the subgrade.

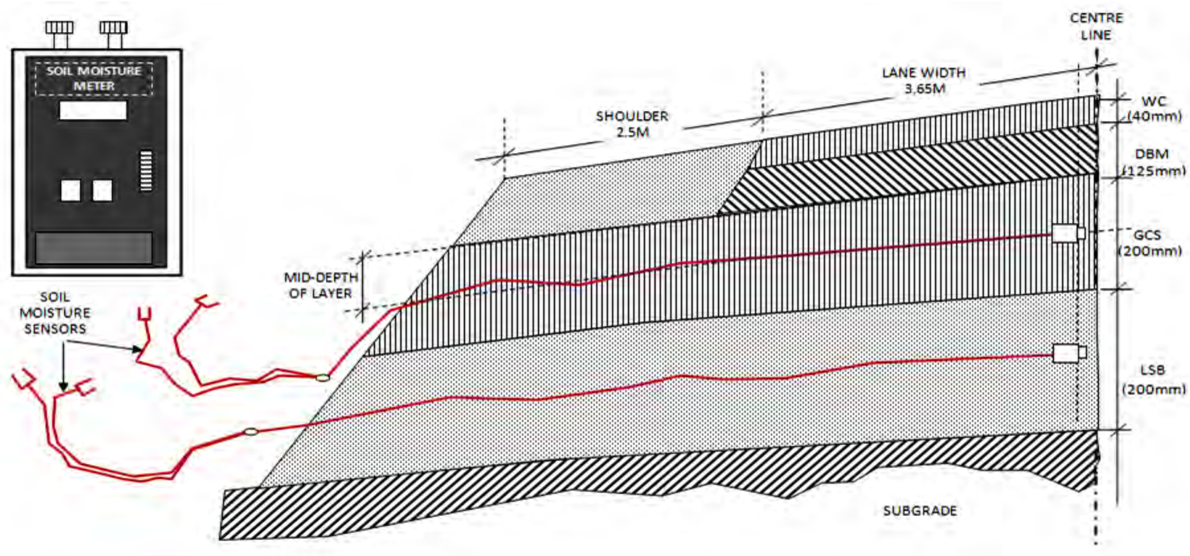



Figure 4.16 – Schematic Presentation of Soil Moisture Equipments

Table 4.5 - Calibration of Thermocouples

| Reading Number | Thermometer | Ambient Temperature (°C) | Thermocouples (°C) | | | |
|---|---|---|---|---|---|---|
| | | | 1 | 2 | 3 | 4 |
| 1 | 60.2 | 30.2 | 51.84 | 52.32 | 51.43 | 51.48 |
| 2 | 59.1 | 30.1 | 52.09 | 52 | 52.73 | 52.45 |
| 3 | 55.6 | 30.4 | 49.92 | 49.51 | 50.41 | 50.05 |
| 4 | 52.9 | 30.4 | 47.6 | 47.11 | 48.01 | 47.79 |
| 5 | 45.9 | 30.6 | 42.59 | 43.18 | 43.05 | 42.36 |
| 6 | 44.7 | 30.8 | 40.84 | 40.76 | 41.74 | 41.25 |
| 7 | 44.0 | 31 | 40.95 | 41.86 | 41.32 | 40.91 |
| 8 | 42.7 | 30.8 | 40.25 | 41.02 | 39.94 | 40.12 |
| 9 | 41.90 | 30.6 | 38.91 | 38.87 | 39.46 | 38.68 |
| 10 | 39.20 | 30.6 | 36.9 | 36.67 | 37.13 | 36.54 |
| 11 | 38.30 | 30.4 | 36.06 | 36.61 | 36.47 | 36.39 |
| 12 | 37.20 | 30.5 | 34.74 | 35.07 | 35.66 | 35.39 |
| 13 | 35.60 | 30.5 | 34.06 | 33.6 | 34.38 | 34.61 |
| 14 | 35.20 | 30.4 | 34.64 | 34.05 | 34.33 | 34.28 |
| 15 | 34.10 | 30.2 | 32.71 | 32.44 | 33.12 | 32.94 |
| 16 | 33.80 | 30.4 | 32.36 | 31.87 | 33.27 | 32.55 |
| 17 | 33.40 | 30.2 | 32.39 | 32.12 | 32.08 | 32.21 |
| 18 | 32.70 | 30.2 | 31.34 | 32.71 | 31.93 | 31.39 |
| 19 | 32.40 | 30 | 31.96 | 31.28 | 31.24 | 30.73 |
| 20 | 32.10 | 29.9 | 31.59 | 29.76 | 31.68 | 31.37 |
| 21 | 31.80 | 30 | 31.6 | 30.5 | 31.47 | 31.78 |


Figure 4.17 – Soil Moisture Meter Reading Equipment

Figure 4.18 – Field Installation of Moisture Blocks

#### 4.5.5.1 Field Set-up of Thermocouple and Soil Moisture Equipment

Figures 4.19 and 4.20 depict the field setup of thermocouple equipment in the road pavement layers. A trench was made from the equipment chamber to the centre of the roadway and the thermocouple sensors were placed in the mid-depths of each pavement layer based on the results of regression models developed by Marshal et al. (2001) to relate AC modulus to the mid-depth pavement temperature which gave excellent correlations. . A typical asphaltic concrete pavement comprises the Wearing Course (WC), Dense Bituminous Macadam (DBM), Graded Crushed Stone (GCS) and Lower Sub Base (LSB).

Figure 4.19 – Excavation of Site for Installation of Thermocouple and Soil Moisture Equipment

Figure 4.20 – View of Laterite Excavation

The sensors of the thermocouple and the soil moisture were guided through half-inch PVC pipes and placed in the trench to the equipment chamber. Four sensors for the temperature readings were placed in the pavement layers. Two sensors for moisture reading were also placed in the granular layers of the pavement. Steps taken in the field installation are shown in the next pages

### 4.5.6 Installation of Thermocouple Sensors in the DBM and Wearing Course Layers

**Steps**

1. A ditch is cut through the shoulder to the side of the DBM.
2. Two Sensors are guided through a ½" pipe to the DBM.
3. One of the Sensors is extended 1 meter into the DBM and laid in the ditch (See Figure 4.21**).**
4. The ditch is filled and the DBM is compacted.
5. The other Sensor is also extended a meter into the Wearing Course, just before the Wearing Course is compacted.

Figure 4.21 - Installation of thermocouple Sensor in Asphaltic concrete layer

### 4.5.7 Installation of Moisture Block and Thermocouple Sensors in the Sub base and the GCS

**Steps**

1. A trench was dug through the shoulder to the sub base and GCS.
2. Two holes were dug into the side of both the sub base and GCS.
3. A moisture block was guided through a ½ in pipe into one of the holes in the Sub base.
4. The thermocouple sensor went through the same pipe into the other hole in the sub base.
5. It was then covered with sub base materials and rammed. Step 3, 4 and 5were repeated for the GCS.

Figure 4.22 – Installed Moisture Block

### 4.6 Pavement Deflection Measurement

Pavement deflection measured with a non-destructive test (NDT) device, Falling Weight Deflectometer (FWD). The FWD is an impulse loading device used to simulate moving wheel loads and measure the corresponding pavement response. It was used to measure pavement deflection at one hourly interval from 6a.m. to 6p.m. from 20$^{th}$ February, 20011 to 26$^{th}$ February, 2011 (Seven days) at the test sections.

The test section has two loops on both the Kumasi bound and Techiman bound lanes. Each loop has a dimension of 2m x 2m. The distance between the two loops in each direction was 2.3m.

During the deflection measurement, the load was dropped at 25m from the edge of the first loop, the mid-distance of the two loops and finally at 25m from the edge of the second loop in each lane. Refer to Figure 4.23 for details on the plan of the Test Section.

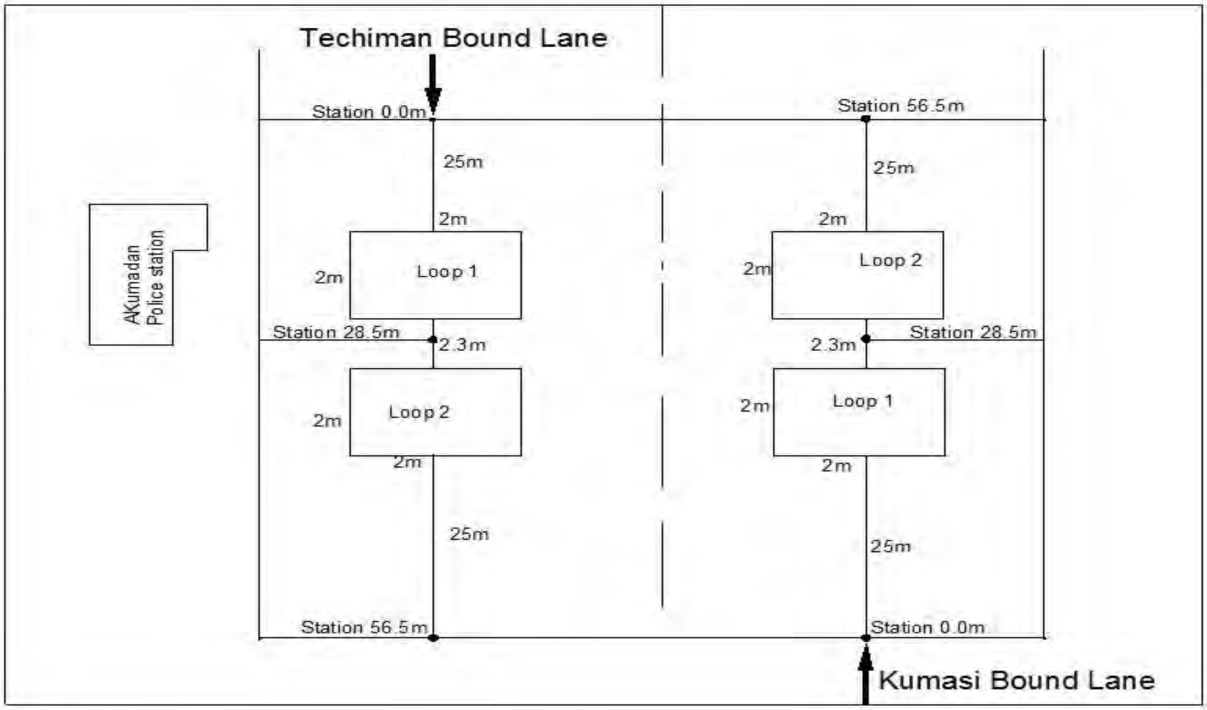



Figure 4.23 - Plan for FWD Deflection Data Collection

Figures 4.24 to Figure 4.27 are sample pictures showing the FWD in operation at test site.

Figure 4.24 – Operation of FWD on Site


Figure 4.25 – Safety Control of Test Section


Figure 4.26 – The FWD Ready to Start Testing

Figure 4.27 – The FWD in Operation

The information about each of the variables was compiled and formed into a database for use in the pavement assessment. A summary of this data is provided in Appendix D.

## 4.7 Summary

This chapter has defined the experimental design and the data collection techniques used in this study. The effectiveness of the procedures was tested in the pilot studies, the results of which were used to help create the detailed field studies. The information collected from this field studies together with other secondary sources were processed and used to create a database, the purpose of which was to use actual data not customization to develop a new framework for the pavement design. The analysis of the information collected is presented and discussed in the next chapter.

Apart from the data collected from the experimental test section, rainfall and sunshine data have been collected from secondary sources such as Meteorological Services Authority over the last 40 years. The purpose of not using data based on projection forecast and default values in the development of the tropical pavement design procedure is to improve the reliability of the new method and establish a case that current technology gives opportunity for tropical countries to develop local capacities to evolve a pavement design method relevant to their peculiar tropical condition.

# CHAPTER 5

# DATA ANALYSIS

## 5.1 Introduction

In this section, results of data collected at the two test sites were used to assess pavement performance against the design. Statistical analyses were conducted on the data using the SPSS software and the results discussed. Details of the sample size collected in this study are provided in another section of the thesis. The flow chart for the data analysis is shown in Figure 5.1. The analysis is undertaken in two stages depending on what data were collected. The data collected from the installed equipment were analyzed first, and then those collected simultaneously with the FWD test results. A report on the traffic and axle load survey during data collection has been included in this chapter as part of the analysis.

For the data collected from the installed equipment, the objectives of the analyses were to examine the hourly, daily, weekly and monthly temperature variations and to establish minimum and maximum values. An attempt has been made to establish relationships between all the temperature groups. Also, relationships between the ambient temperature and the pavement layers were established and an attempt was made to predict the pavement temperatures with the ambient temperature through correlations and regression analyses. Some of the data for the months could not be collected as a result of battery failure of the thermocouples.

Trends and variations were also established for the moisture regimes in the base and subbase layers at each section. An attempt was made to establish some trend and relationship in the base and subbase moisture content values between the two test sections.

For the second part of the analysis, the main objective is to establish relationships between the FWD data (thus deflection and moduli), as well as the temperature and traffic data that were collected at the same periods. Trends and differences based on the hourly variations of the modulus and temperature of each pavement layer for each site were established. Changes in the modulus due to cumulative traffic have also been established from their hourly variations with time. From these relationships, an attempt has been made to predict the modulus using the traffic and temperature as predictors.

The same analyses done for the modulus was also done for the deflection; thus, relating it to temperature and traffic load. From scatter diagrams plotted, establishment of the general relationships between parameters for both sites combined has been attempted. The relationships between the various parameters and the layers have been defined in Table 5.1.

Table 5.1 - Definition of Parameters

| **Layer** | **Moduli** | **Deflection** | **Temperature** |
|---|---|---|---|
| HMA Layer | E1 | Maximum Deflection | Thermocouple 4 (TIT) |
| Base | E2 | | Thermocouple 3 (T2T) |
| Subbase | E3 | | Thermocouple 2 (T3T) |
| Subgrade | E4 | | Thermocouple 1 (T4T) |

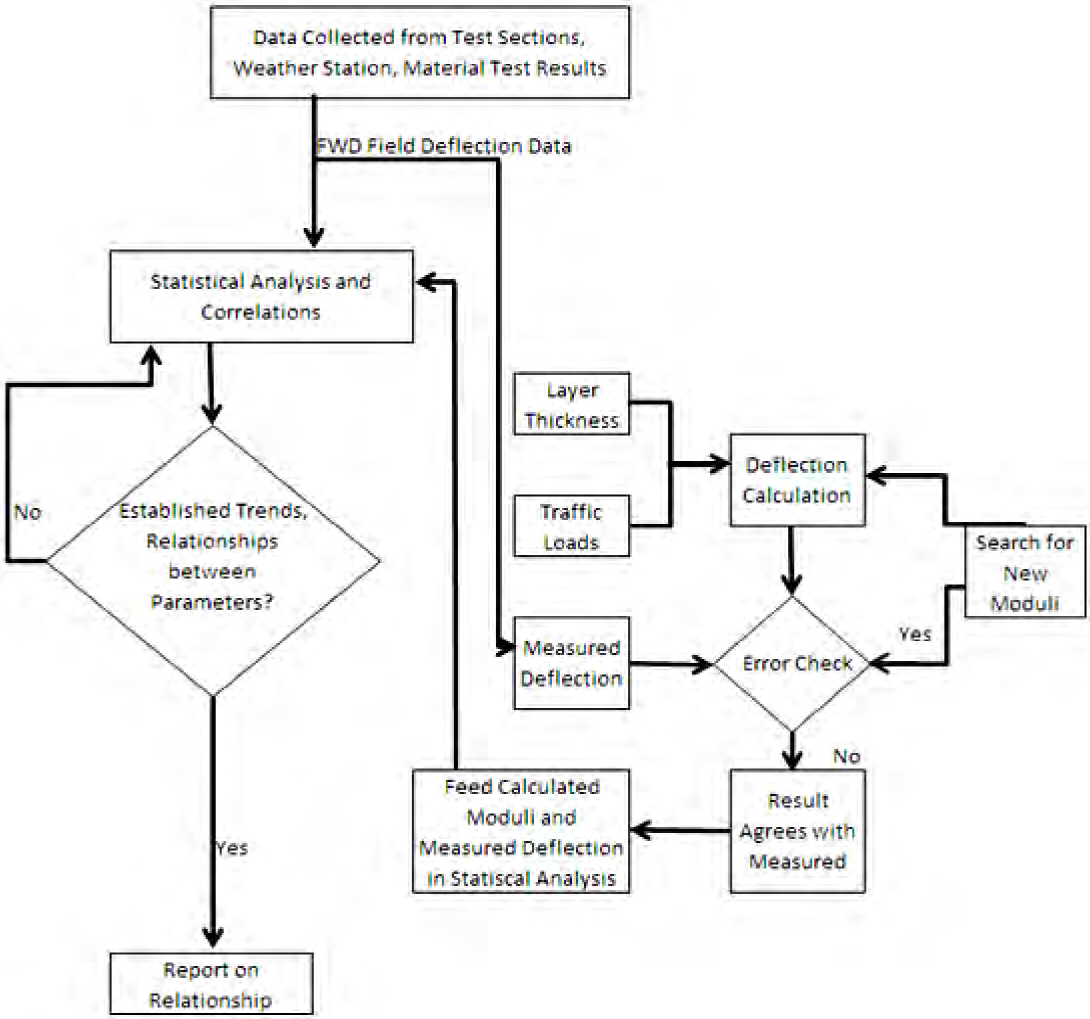



Figure 5.1 – Flow Chart for Data Analyses

## 5.2 Establishing trends and variations

### 5.2.1 Hourly Temperature variation

To examine temperature variations, a time series plot is obtained to aid in the description of the temperature variations. This will also help us to determine the times (hour, day, week or month) that the minimum and maximum temperatures occurred.

Figures 5.2 & 5.3 depict the hourly temperatures during the day for both sites. From the figures, it can be seen that for all the curves, the temperature falls initially to the minimum and it then rises to the maximum where it starts to fall again. Except for thermocouple 4 (HMA layer temperature), which showed entirely different trend of variation for the Sogakope site, attaining maximum and minimum hourly temperatures recorded for all sensors in the various layers. The rising and falling times of the temperatures have been summarized in Table 5.2. The thermocouple 1 (sub-base temperature) does not vary that much with respect to time as it assumed a smooth pattern making the values difficult to ascertain especially for Akumadan site since the different hour temperatures seems to be closer to each other.

Table 5.2 – Summary of Temperature Variation in Pavement Layers

| **Temperature** | **Ambient** | | **Thermocouple 1** | | **Thermocouple 2** | | **Thermocouple 3** | | **Thermocouple 4** | |
|---|---|---|---|---|---|---|---|---|---|---|
| | SGK | AKM | SGK | AKM | SGK | AKM | SGK | AKM | SGK | AKM |
| Rising Time (Hourly Min.) | 7:00 GMT | 7:00 GMT | 14:00 GMT | - | 10:00 GMT | 9:00 GMT | 7:00 GMT | 8:00 GMT | 6:00 GMT | 7:00 GMT |
| Falling Time (Hourly Max.) | 15:00 GMT | 14:00 GMT | 24:00 GMT | - | 9:00 GMT | 17:00 GMT | 14:00 GMT | 16:00 GMT | 13:00 GMT | 14:00 GMT |
| SGK – Sogakope AKM - Akumadan | | | | | | | | | | |

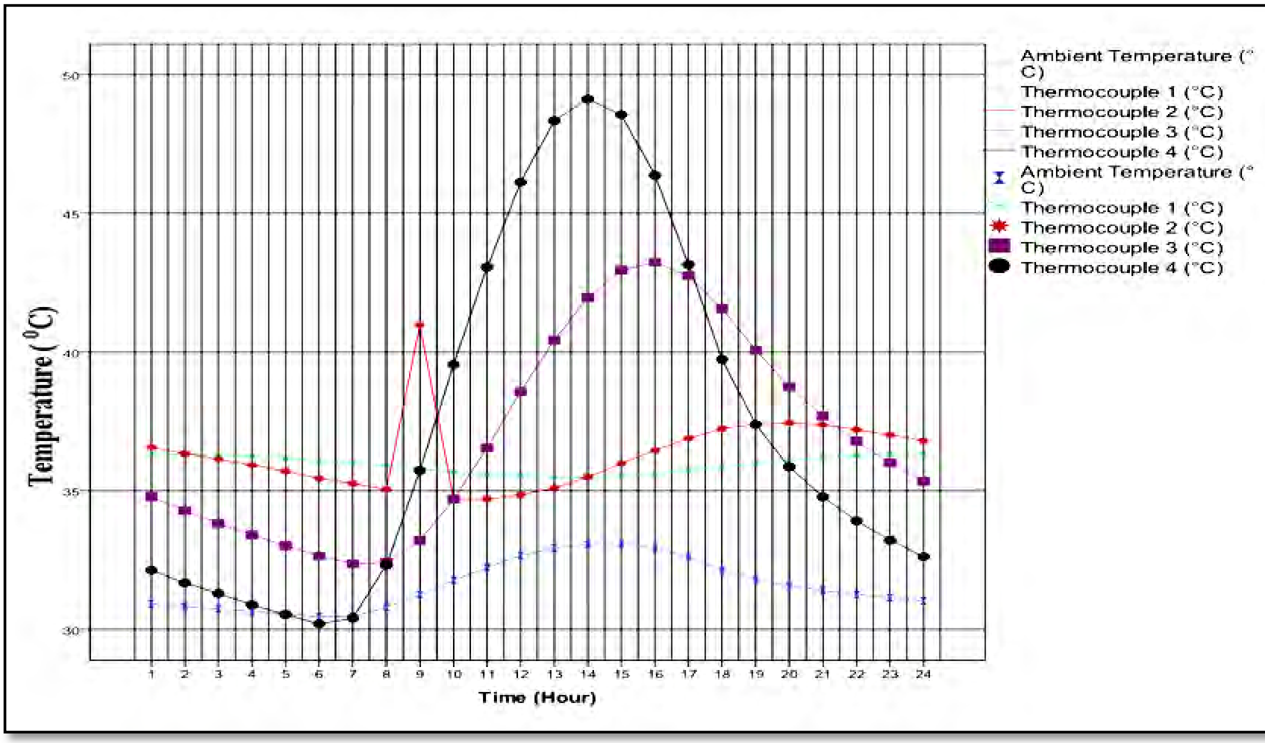



Figure 5.2 – Hourly Temperature Trend for Sogakope

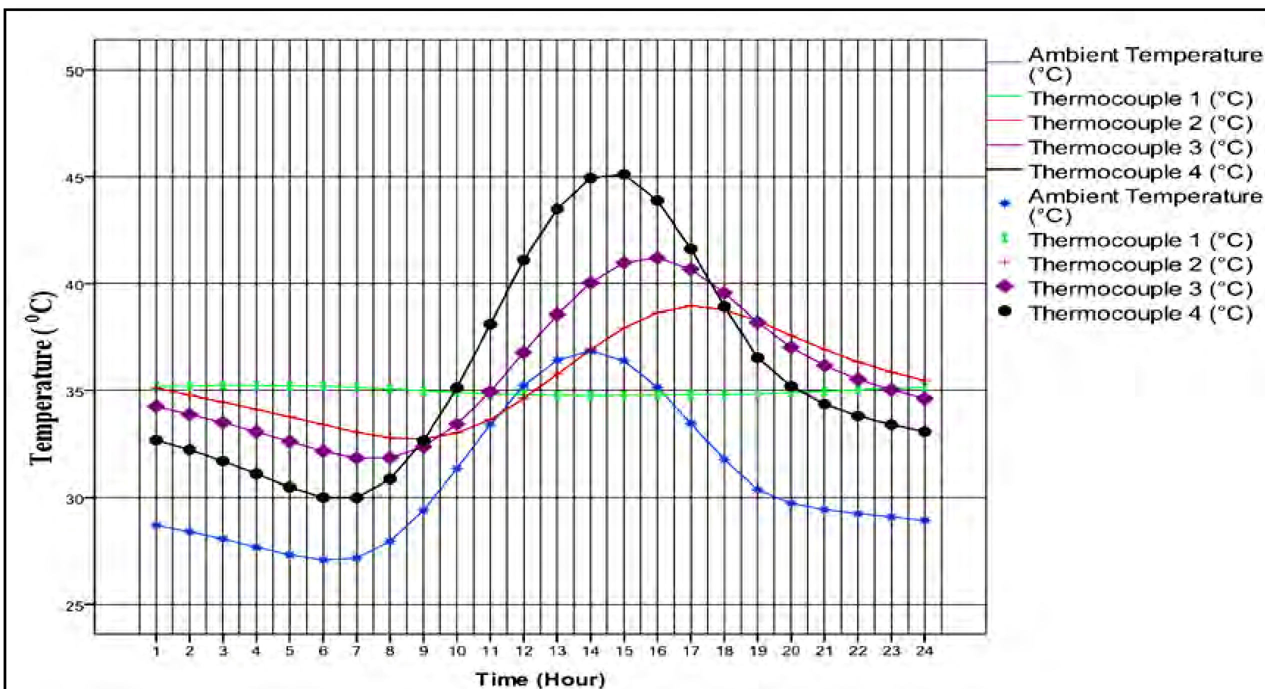



Figure 5.3 – Hourly Temperature Trend for Akumadan

### 5.2.2 Daily Temperature Variation

In general, the various temperatures during the week, taking days of the week as the time line, showed some oscillating trend as shown by Figure 5.4. It can also be realized that all the various temperatures assume maximum value on Saturday except thermocouple 1 and 2 which assumed their maximum value on Thursday. Also, with the exception of thermocouple 3 and 4, the rest show that temperature falls on Monday, rises on Tuesday, and falls again on Wednesday, in that order. Monday recorded the minimum temperature for ambient temperature and thermocouple 1. Thermocouple 2 has it minimum temperature on Wednesday, thermocouple 3, on Tuesday and thermocouple 4 on Monday.

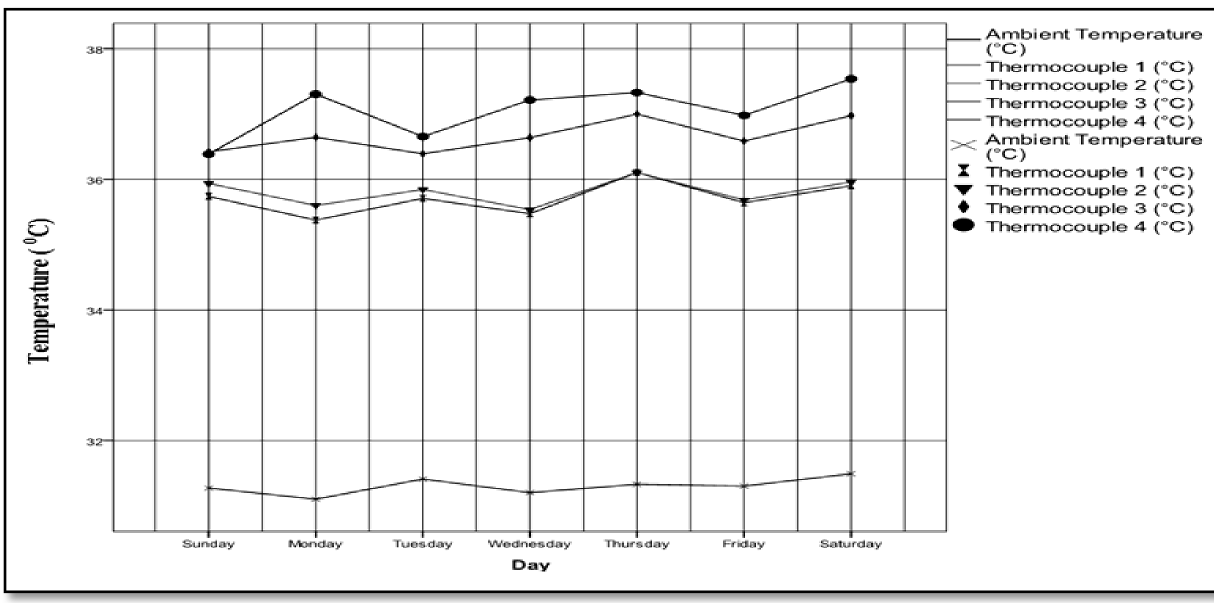



Figure 5.4 – Daily Temperature Trend

### 5.2.3 Weekly Temperature Variation

From Figures 5.5 & 5.6, the ambient temperature, thermocouples 2, 3, and 4 assumed similar pattern of temperature variation throughout the period with the exception of thermocouple 1 which assumed a different pattern. Studying the Ambient, thermocouple 2, 3 and 4 temperatures carefully, it can be seen that the pattern in temperature seem to repeat itself after two weeks. Even though the pattern seemed to recur every two weeks, it is difficult to say categorically that a particular week in the month assumed a minimum temperature and another assumed a maximum temperature.

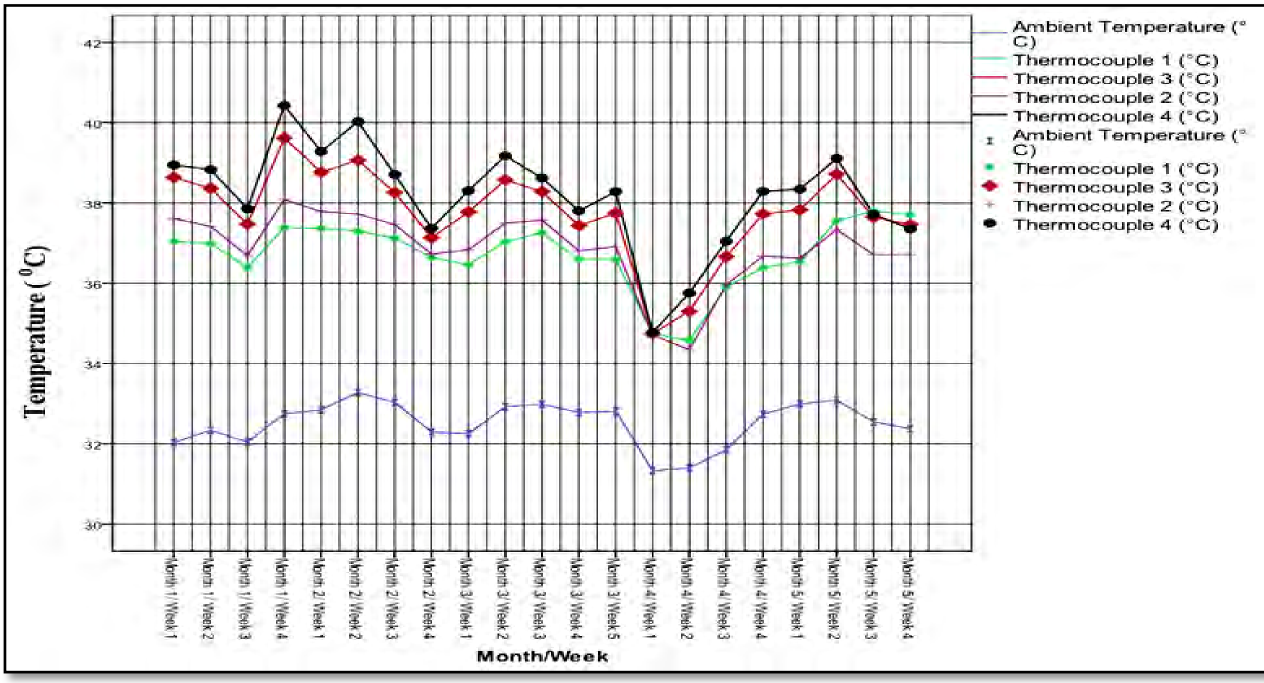



Figure 5.5 – Weekly Temperature Trend for Sogakope

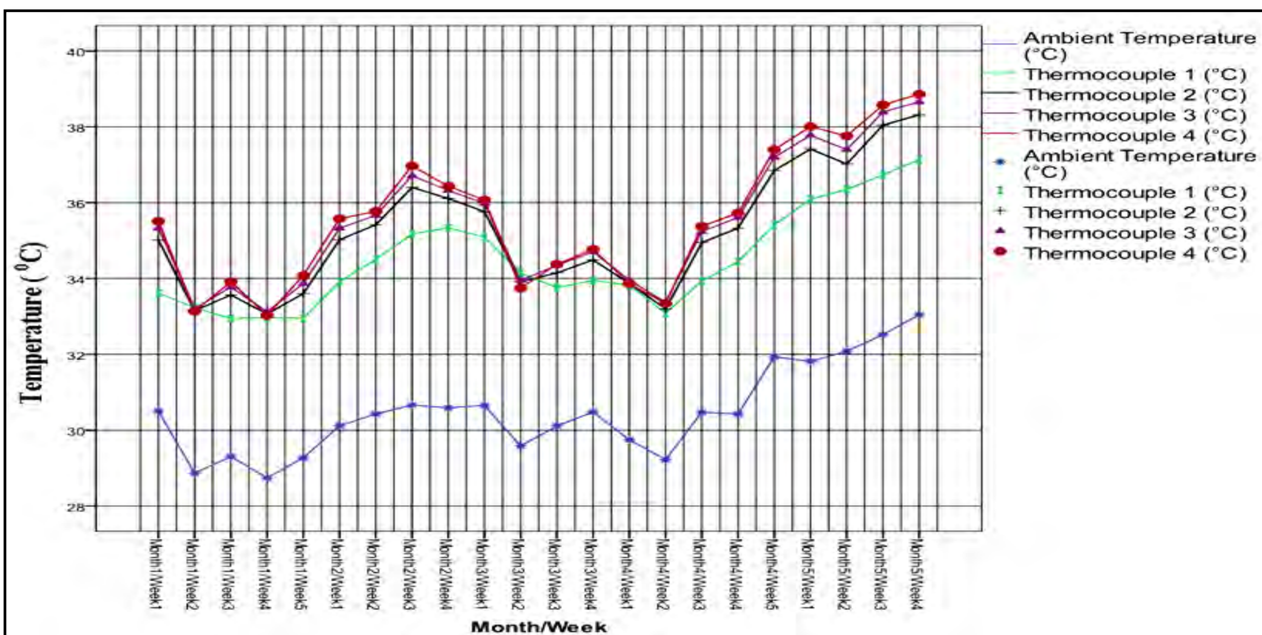



Figure 5.6 – Weekly Temperature Trend for Akumadan

### 5.2.4 Monthly Temperature Variation

Figures 5.7 & 5.8 depict the monthly temperature variations for the various layers within a year. Studying the figures carefully revealed that, the ambient temperature and the temperature of the four (4) layers assumed a similar pattern of variation. It can be seen that all the temperatures for the different levels seems to have the same pattern. For the Sogakope site, they all seem to have started increasing from the beginning of January until March. Thermocouple 3 and 4 started declining till July when it declined sharply in August. They then started to increase until they both attained a maximum temperature in October. The ambient temperature and the temperature of the four (4) layers assumed a maximum temperature around the April and a minimum temperature around August for the Akumadan site.

It should be noted that the values plotted on the time graph above are average values of the periods in question.

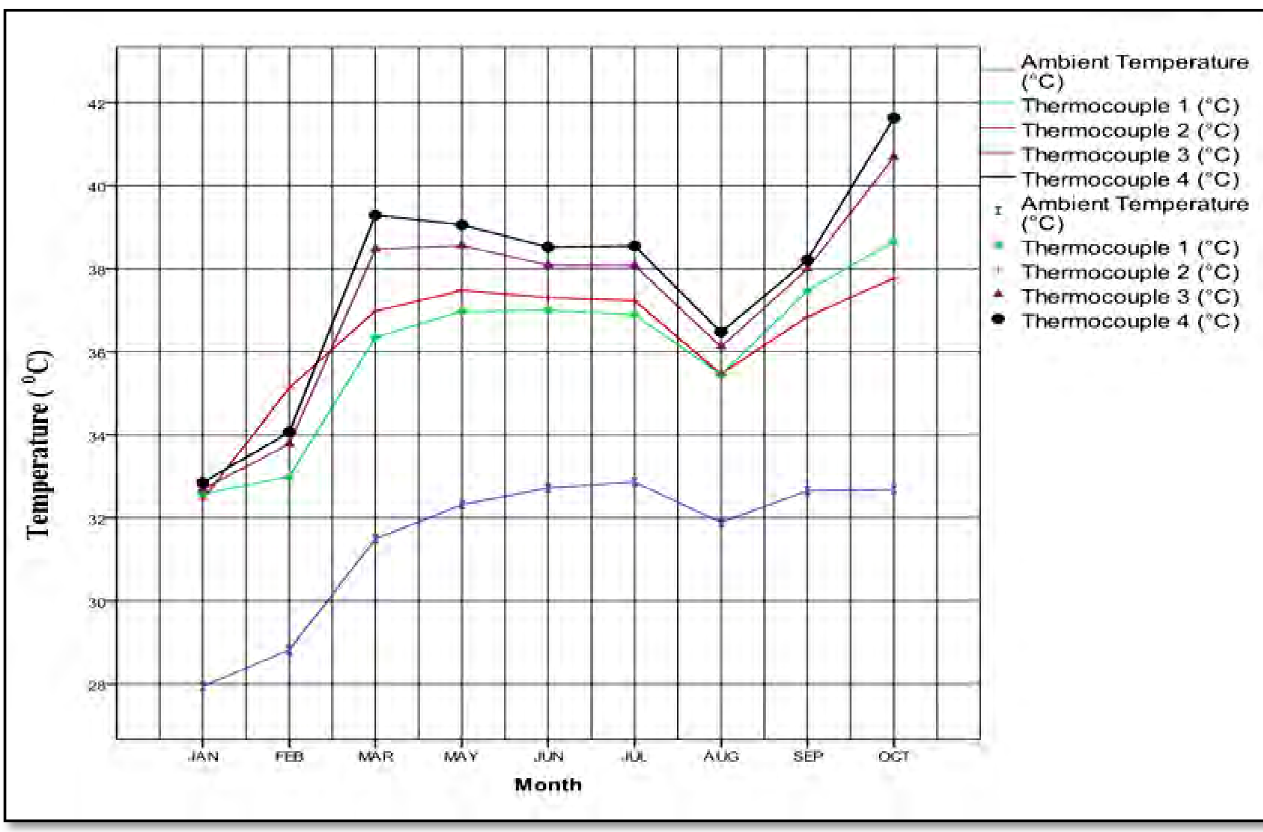



Figure 5.7 – Monthly Temperature Trend for Sogakope

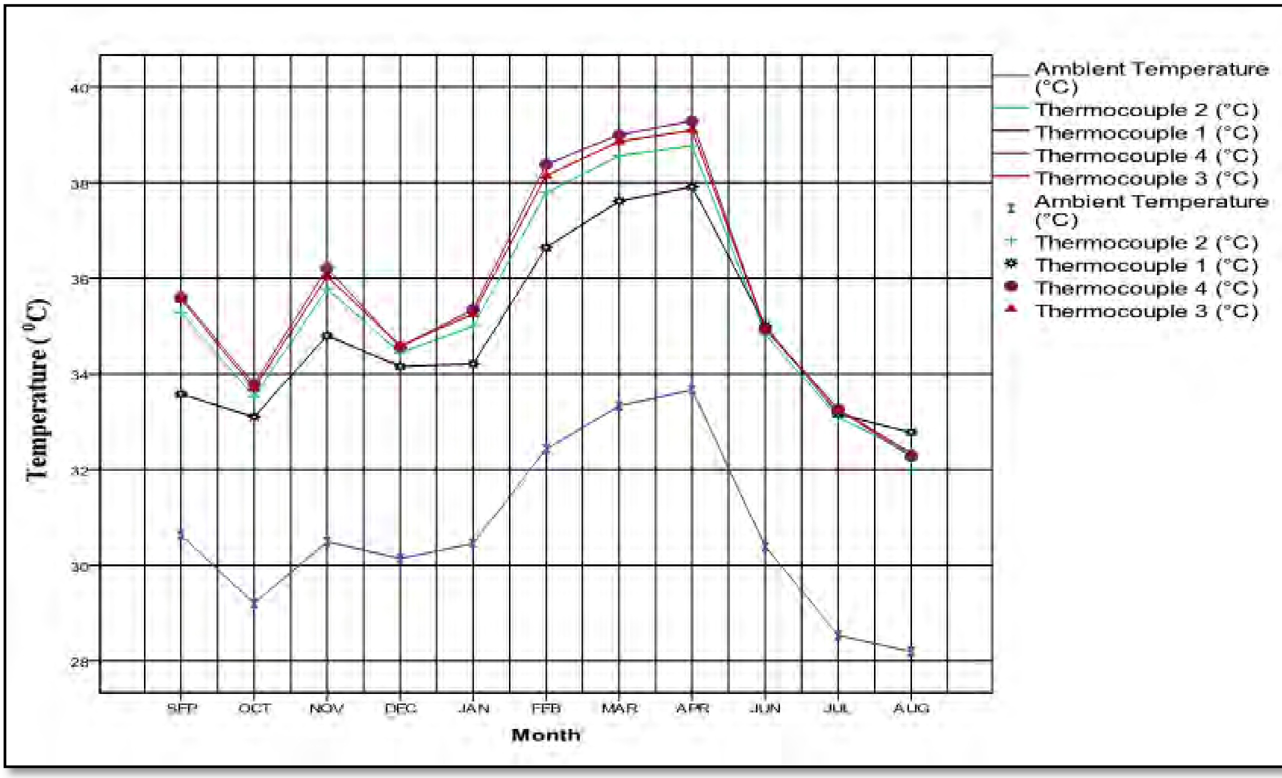



Figure 5.8 – Monthly Temperature Trend for Akumadan

In order to verify the variability of the temperature, the mean, standard deviation and the coefficient of variation were computed. But since it has been revealed that the deviation tends to increase with an increase in the mean, the coefficient of variation (COV) was used to determine the temperature variations. It is generally accepted that COV values below10 % indicates low variability, and values above 10 % indicate that significant variability exists. Tables 5.3 and 5.4 summarize the mean, standard deviation, and the COV of the temperatures on annual and monthly basis respectively for the two sites; Akumadan and Sogakope.

From Table 5.3, it can be seen that the ambient, thermocouple 1 and thermocouple 2 for the Sogakope site and thermocouple 1 for the Akumadan site exhibited low variability of temperature values. The monthly temperature values for the data collected have been provided in Appendix C and from these, it can be noted that most of the COV values are higher than 10 % especially the Akumadan site indicating significant temperature variability at the site.

Table 5.3 - Annual Temperature Values (Mean, Standard Deviation and Coefficient of Variation)

| | Sogakope Site | | | Akumadan Site | | |
|---|---|---|---|---|---|---|
| **Layer** | **Mean** | **Standard Deviation** | **Coefficient of Variation** | **Mean** | **Standard Deviation** | **Coefficient of Variation** |
| Ambient | 31.19 | 2.09 | 6.72 | 30.97 | 4.82 | 15.57 |
| Subbase | 35.67 | 2.27 | 6.38 | 35.37 | 2.04 | 5.76 |
| Base | 35.47 | 2.48 | 7.01 | 35.76 | 3.84 | 10.74 |
| DBM/Binder Course* | 36.15 | 4.39 | 12.16 | 35.98 | 5.00 | 13.90 |
| Wearing Course | 36.9 | 6.71 | 18.43 | 36.07 | 7.24 | 20.08 |

Source: Calculated from Field Data

*DBM for Sogakope and Binder Course for Akumadan

## 5.3 Establishing Relationships between Parameters

### 5.3.1 Relationship between the Various Temperature Layers and the Ambient Temperature

From correlation analysis, the data revealed the strength of relationship between the pavement layers and the ambient temperature and within the layers themselves. A Pearson correlation coefficient approaching zero (0) signifies a weak relationship and a coefficient approaching one (1) signifies a strong relationship.

From Table 5.4 for the Sogakope site, it can be seen that there is a moderate relationship between the ambient temperature and thermocouple 4, but very low relationship between the ambient temperature and thermocouple 2. The relationship between thermocouple 1 (T1T) and the other layers is very low. Also, there is no relationship between thermocouple 2 (T2T) and 3, and 4. The only relationship that exists between the layers is thermocouple 3 and thermocouple 4. This relationship is a very strong one.

Table 5.4 - Correlations (Sogakope Site)

| | | AMT (°C) | T1T (°C) | T2T (°C) | T2T (°C) | T4T (°C) |
|---|---|---|---|---|---|---|
| **AMT (°C)** | Pearson Correlation | 1 | .718** | .076** | .743** | .676** |
| | Sig. (2-tailed) | | .000 | .000 | .000 | .000 |
| **T1T (°C)** | Pearson Correlation | .718** | 1 | .101** | .386** | .157** |
| | Sig. (2-tailed) | .000 | | .000 | .000 | .000 |
| **T2T (°C)** | Pearson Correlation | .076** | .101** | 1 | .060** | .016 |
| | Sig. (2-tailed) | .000 | .000 | | .000 | .241 |
| **T3T (°C)** | Pearson Correlation | .743** | .386** | .060** | 1 | .844** |
| | Sig. (2-tailed) | .000 | .000 | .000 | | .000 |
| **T4T (°C)** | Pearson Correlation | .676** | .157** | .016 | .844** | 1 |
| | Sig. (2-tailed) | .000 | .000 | .241 | .000 | |
| | N | 5216 | 5216 | 5216 | 5216 | 5216 |
| NB: **Correlation is significant at the 0.01 level (2-tailed) | | | | | | |

Source: Calculated from Field Data

And from Table 5.5 for the Akumadan site, it can be verified from the table that, the ambient temperature has very weak relationship with thermocouple 1 temperature, moderate relationship with thermocouple 2 temperature and very strong relationship with thermocouple 3 and 4 temperatures. Within the layers themselves, it can be seen from the table that, thermocouple 1 and 2 has moderate relationship, thermocouple 2 and 4, moderate. However, there exist very strong relationship between, thermocouple 2 and 3, and thermocouple 3 and 4. The relationship between thermocouple 1 and 3 and thermocouple 1 and 4 is a very weak one.

Whenever the relationship between two variables is weak, predicting one using the other will not give any good result, but if there exist a strong relationship, one can predict the other with a greater percentage of the variation in the dependent variable explained by the independent variable (predictor).

Based on the parameters with strong relationships, regression analyses were performed on them and their relationships were established as shown in Table 5.5.

Table 5.5 - Correlations (Akumadan Site)

| | | T1T (°C) | T2T (°C) | T2T (°C) | T4T (°C) | AMT (°C) |
|---|---|---|---|---|---|---|
| **T1T (°C)** | Pearson Correlation | 1 | .540** | .366** | .199** | .249** |
| | Sig. (2-tailed) | | .000 | .000 | .000 | .000 |
| | N | 6395 | 6395 | 6395 | 6395 | 6395 |
| **T2T (°C)** | Pearson Correlation | .540** | 1 | .921** | .696** | .611** |
| | Sig. (2-tailed) | .000 | | .000 | .000 | .000 |
| | N | 6395 | 6395 | 6395 | 6395 | 6395 |
| **T3T (°C)** | Pearson Correlation | .366** | .921** | 1 | .916** | .850** |
| | Sig. (2-tailed) | .000 | .000 | | .000 | .000 |
| | N | 6395 | 6395 | 6395 | 6395 | 6395 |
| **T4T (°C)** | Pearson Correlation | .199** | .696** | .916** | 1 | .975** |
| | Sig. (2-tailed) | .000 | .000 | .000 | | .000 |
| | N | 6395 | 6395 | 6395 | 6395 | 6395 |
| **AMT (°C)** | Pearson Correlation | .249** | .611** | .850** | .975** | 1 |
| | Sig. (2-tailed) | .000 | .000 | .000 | .000 | |
| | N | 6395 | 6395 | 6395 | 6395 | 6395 |
| NB: **Correlation is significant at the 0.01 level (2-tailed). | | | | | | |

Table 5.6 - Regression Models For Predictions

| | | T1T | T2T | T3T | T4T | AMT |
|---|---|---|---|---|---|---|
| **T1T (°C)** | Sogakope | | | | | ***TIT*** = 14.946 + 0.665 ***AMT*** |
| | Akumadan | | | | | |
| **T2T (°C)** | Sogakope | | | | | |
| | Akumadan | | | | | |
| **T3T (°C)** | Sogakope | | | | | ***T3T*** = -15.124 + 1.649 ***AMT*** |
| | Akumadan | | ***T3T*** = -7.732 + 1.224 ***T2T*** | | | ***T3T*** = 9.251 + 0.862 ***AMT*** |
| **T4T (°C)** | Sogakope | | | ***T4T*** = -13.153 + 1.367 ***T3T*** | | ***T4T*** = -9.552 + 1.475 ***AMT*** |
| | Akumadan | | | ***T4T*** = -13.044 + 1.367 ***T3T*** | | |
| **AMT (°C)** | Sogakope | | | | | |
| | Akumadan | | | | | |
| The relation between **x** and y is given by **y = B + Ax**<br>where A is the intercept on the y and B is the slope of the line | | | | | | |

## 5.4 Moisture Content Analysis

### 5.4.1 Sogakope Site

From the moisture variation (Subbase Layer) chart shown in Figure 5.9, it can be seen that the moisture content for the base layer is high for January. It then starts falling till it attains its lowest in March. From March, it then starts rising again until it attains it highest in September where the level falls gradually to the end of the year. For the Base layer as shown by the graph (Figure 5.10), it can be seen that variation pattern is almost the same as that of the Subbase layer. The only difference is in the month April where the Base moisture attains its lowest level and rises sharply up to May where this level is maintained to August and then rises again to attain its maximum in September where the level is maintained again to the end of the year.

Between March and August which can be regarded as the wet season (see Figures 5.11 & 5.12), it can be seen as displayed by the graph that, moisture levels were generally low during this period and vice-versa during the dry season.

The next task was to compare the trends in temperature (Figure 5.13), sunshine duration (Figure 5.14) and evapotranspiration levels (Figure 5.15); and their relationship with moisture content trend (Figure 5.9) for the year. It can be observed that from the beginning to the middle of the year as rainfall rises to higher levels, temperature is also seen to be behaving the same way. This behaviour can be explained as the sunshine duration levels are also high during this period. The rise and fall trend in the sunshine duration is more similar to that of the temperature. The moisture content has almost an opposite trend of the rainfall and temperature; reducing to low levels from the beginning to the middle part of the year as rainfall and temperature rise during this period. This behaviour in the moisture content can be explained considering the high levels of evapotranspiration during this early part of the year. The evapotranspiration falls to lower levels as the rainfall attains higher levels and during this period the moisture content is seen to be rising.

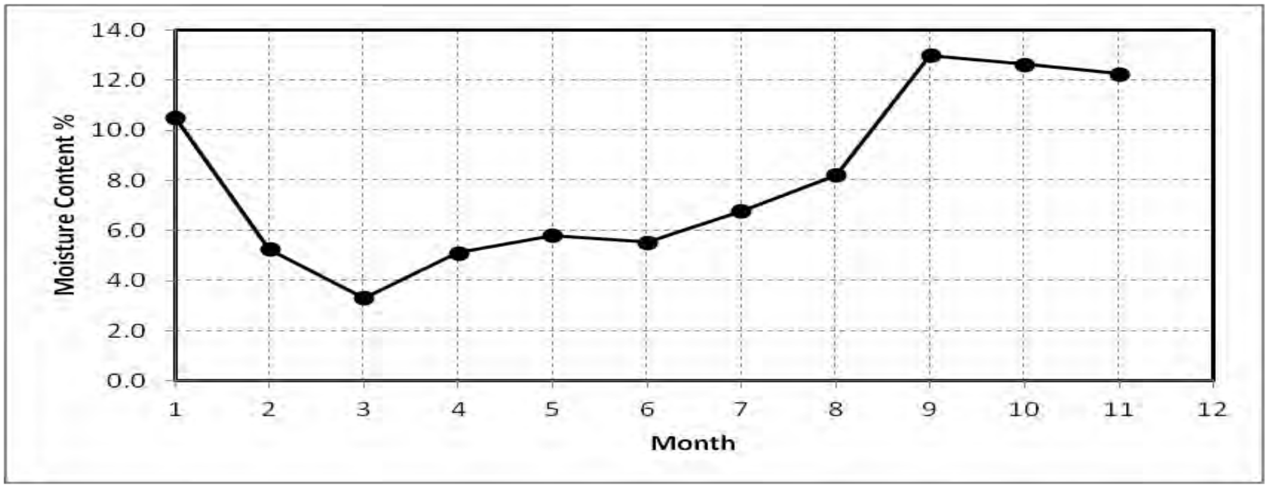



Figure 5.9 – Moisture Content Variation for Subbase Layer (Sogakope Site)

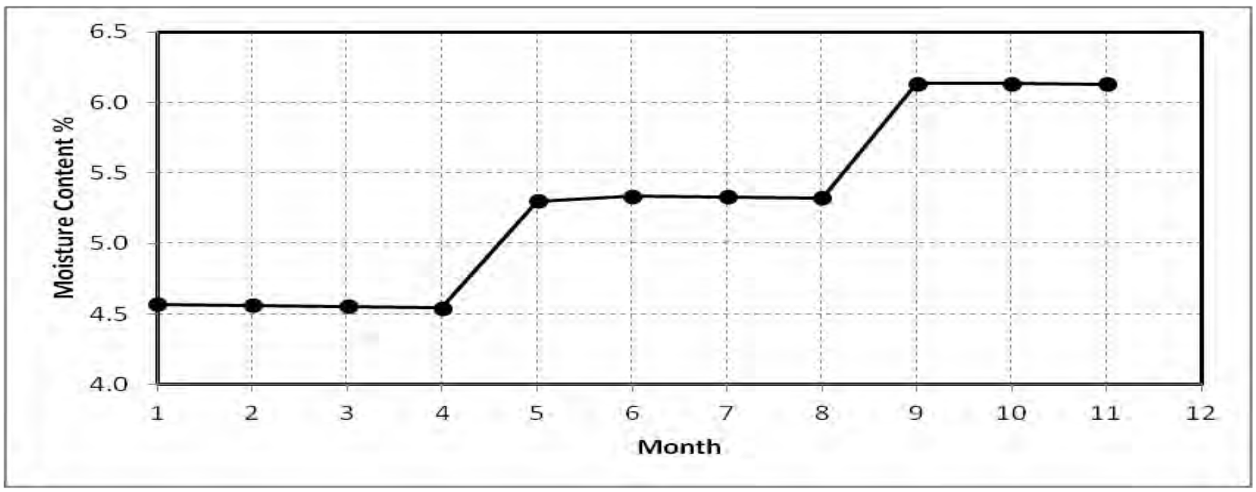



Figure 5.10 – Moisture Content Variation for Base Layer (Sogakope Site)

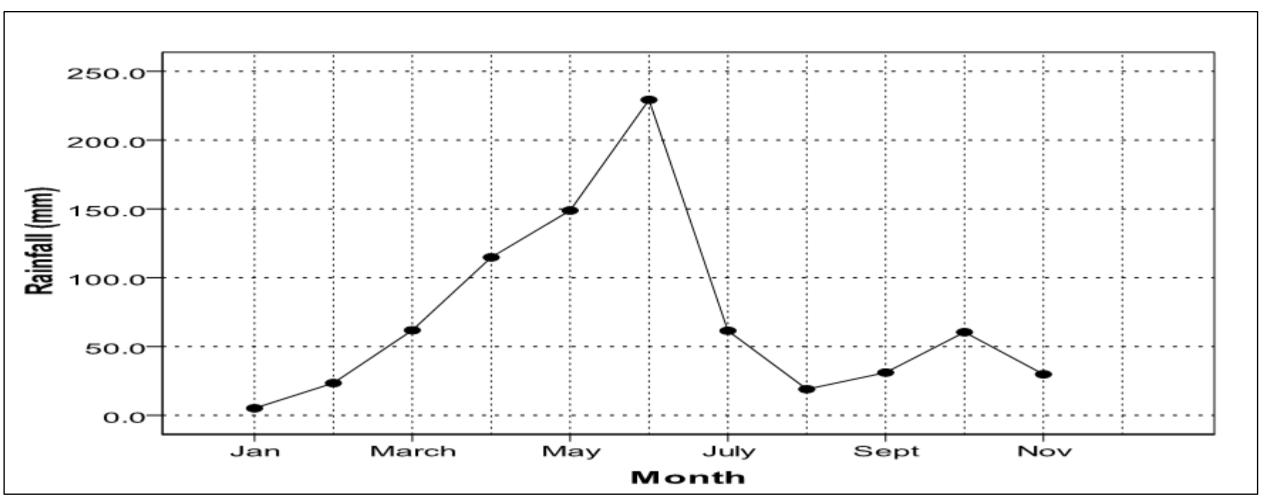



Figure 5.11 – Average Monthly Rainfall (Sogakope Site)

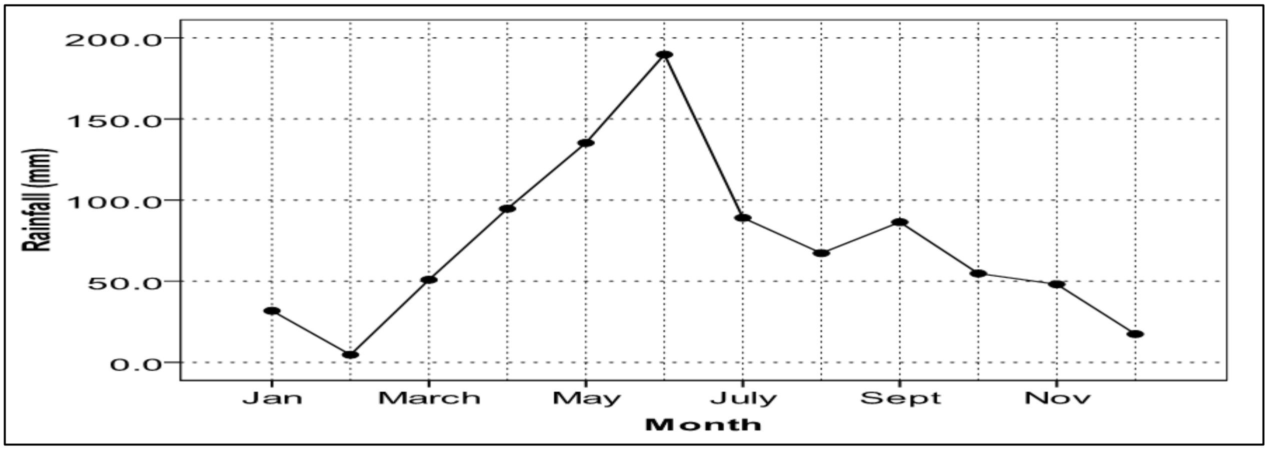



Figure 5.12 – Average Monthly Rainfall (National)

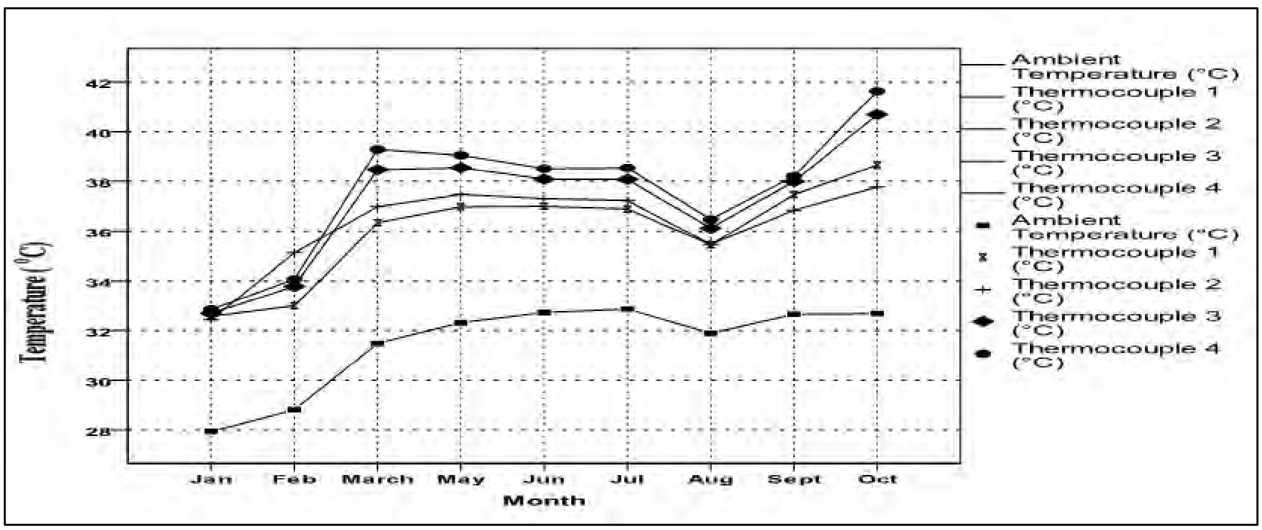



Figure 5.13 – Temperature Variations for the Months (Sogakope Site)

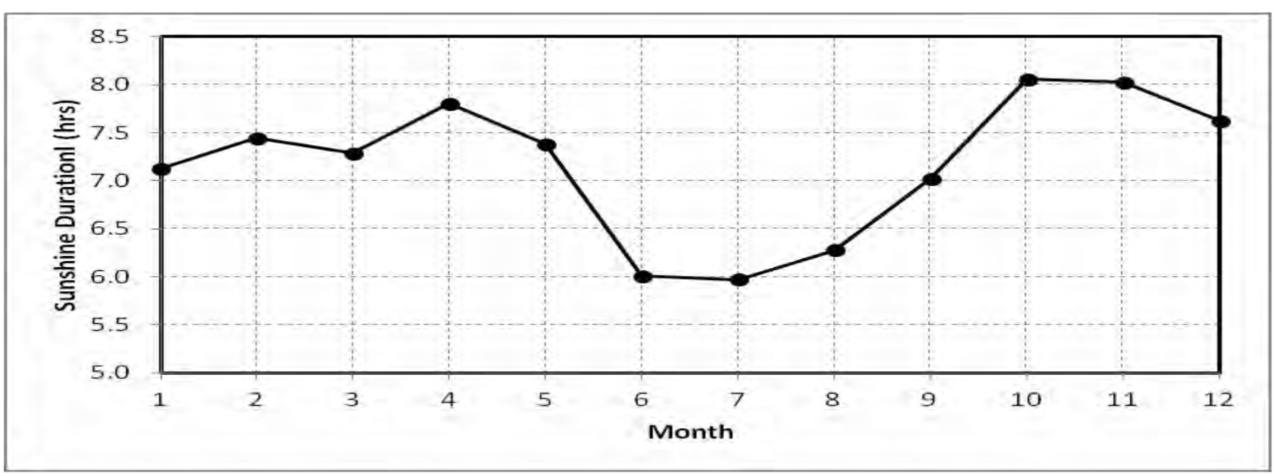



Figure 5.14 – Sunshine Durations for the Months (Sogakope Site)

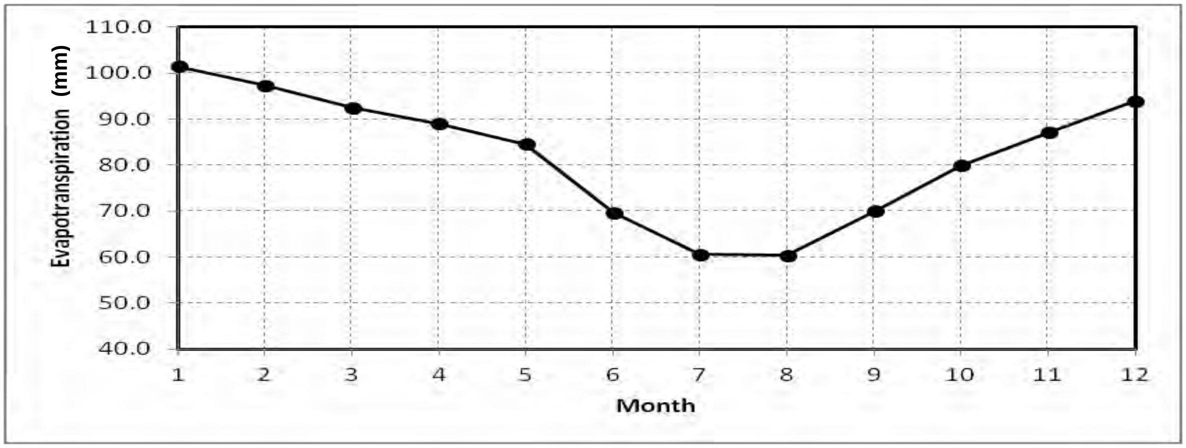



Figure 5.15 – Evapotranspiration Levels for the Months (Sogakope Site)

The research also seeks to find out if there is any relationship between the Base layer and the Sub Base layer. To find this, a correlation was run and the result shown in Table 5.7. From the correlation table (Table 5.7), it can be revealed that there is some kind of moderate positive relation between the Base and the Sub Base layer.

Table 5.7 - Correlations – Sogakope

| | | **Base** | **Subbase** |
|---|---|---|---|
| **Base** | Pearson Correlation | 1 | .648** |
| | Sig. (2-tailed) | | .000 |
| | N | 280 | 280 |

**. Correlation is significant at the 0.01 level (2-tailed).

### 5.4.2 Akumadan Site

Figure 5.16 is a chart showing the moisture content variation on the site during 2009/2010 for the Subbase layer. From the chart, the moisture content started to fall from the beginning of the year attaining it lowest value in April. It then begins to rise, attaining it highest in July and fall again by August. It then rises gradually to the end of the year. Figure 5.19 shows the variations of the moisture content in the Base layer on the same site. The Base layer generally showed the same rising and falling trend but attaining its maximum in August and falling up to October before rising again gradually to the end of the year.

At this site, there seem to be more rainfall throughout the year. From the monthly rainfall (Figure 5.16), it can be observed that the rainfall rises at the beginning of the year up to May where it falls gradually up to August. It rises again to September where it attains its maximum and drops from October to the end of the year where it attains its minimum in December.

Comparing Figures 5.16, 5.18, 5.20 and 5.21, it can be observed that the temperature has a similar trend as the rainfall throughout the year. Both of them rise to high levels from the beginning to the middle of the year where they drop in August to rise and fall again to the end of the year. This is so due to the high sunshine durations during the wet periods and low durations during the dry periods as can be seen, in April where sunshine duration is maximum, temperature too is maximum and in August where sunshine duration is minimum, temperature too is minimum.

Moisture content on the other hand falls at the beginning of the year to its minimum in April where rainfall rises to a very high level. Around the middle of the year moisture content rises to its high levels whiles rainfall falls to its lowest levels. This opposite behaviour can be explained to be due to the temperature variation which is also almost the opposite of the moisture trend and also by observing Figure 5.22 where evapotranspiration levels are very high at the early months of the year and then falling to its minimum in August where the moisture content then attains its maximum although rainfall too is at its minimum.

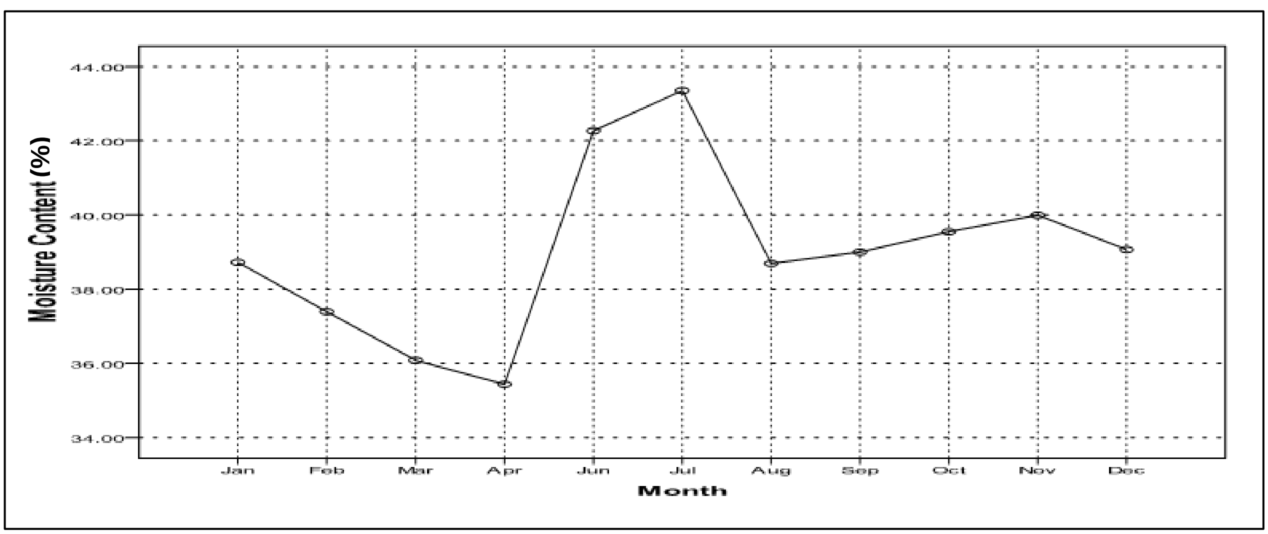



Figure 5.16 – Moisture Variation for Subbase Layer (Akumadan Site)

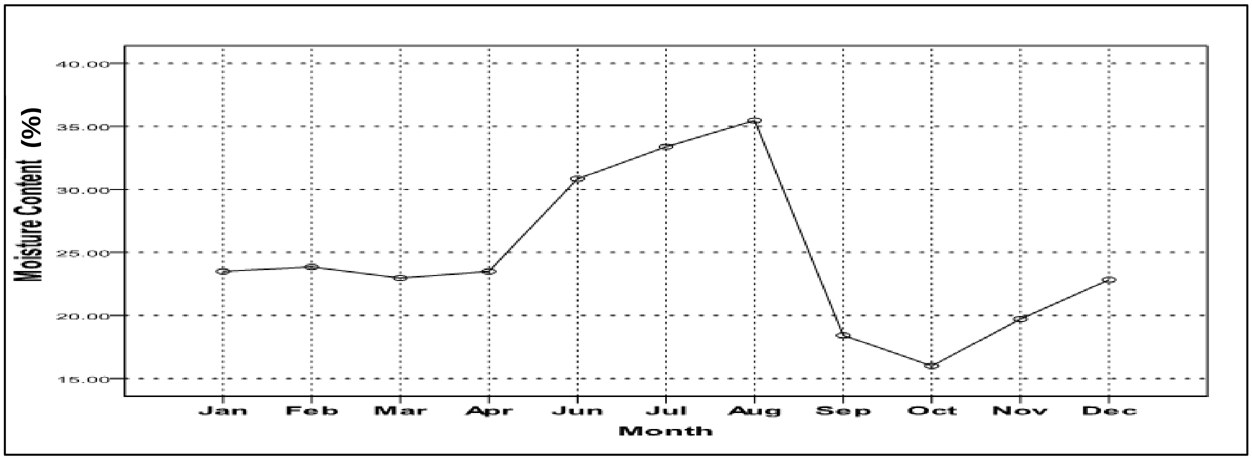



Figure 5.17 – Moisture Variation for Base Layer (Akumadan Site)

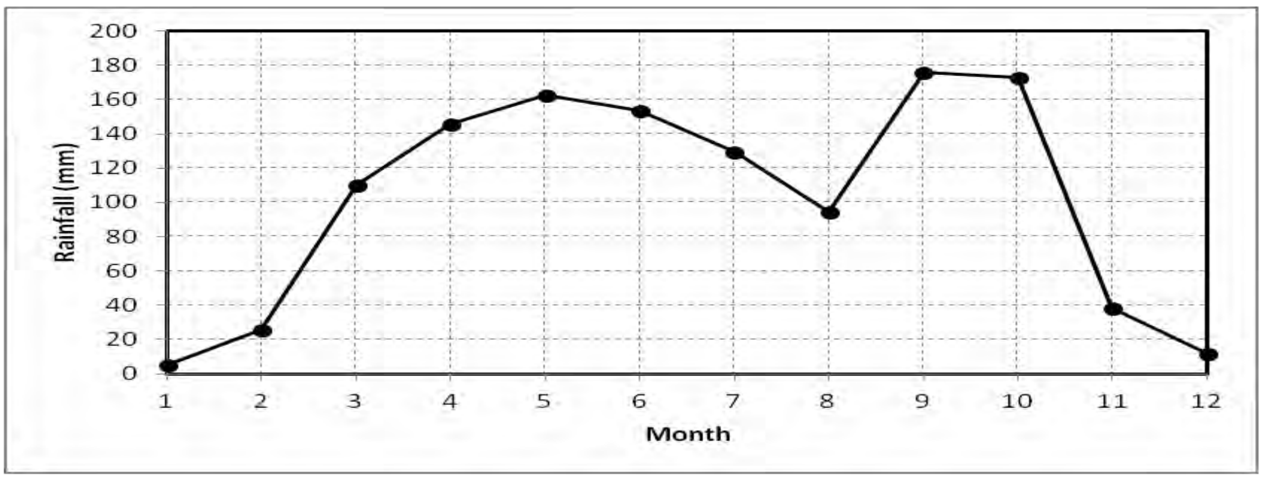



Figure 5.18 – Average Monthly Rainfall (Akumadan Site)

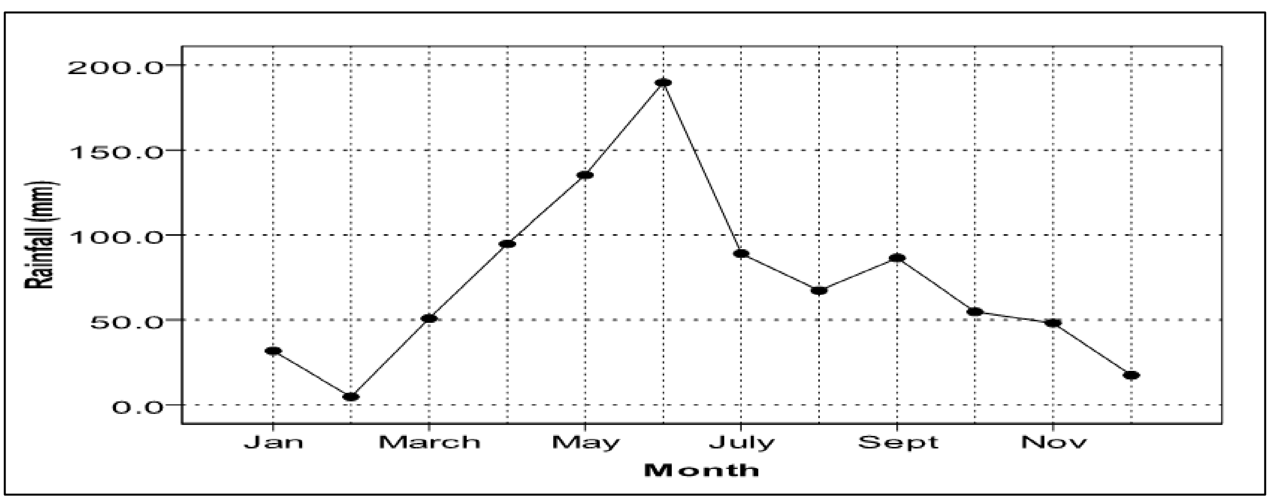



Figure 5.19 – Average Monthly Rainfall (National)

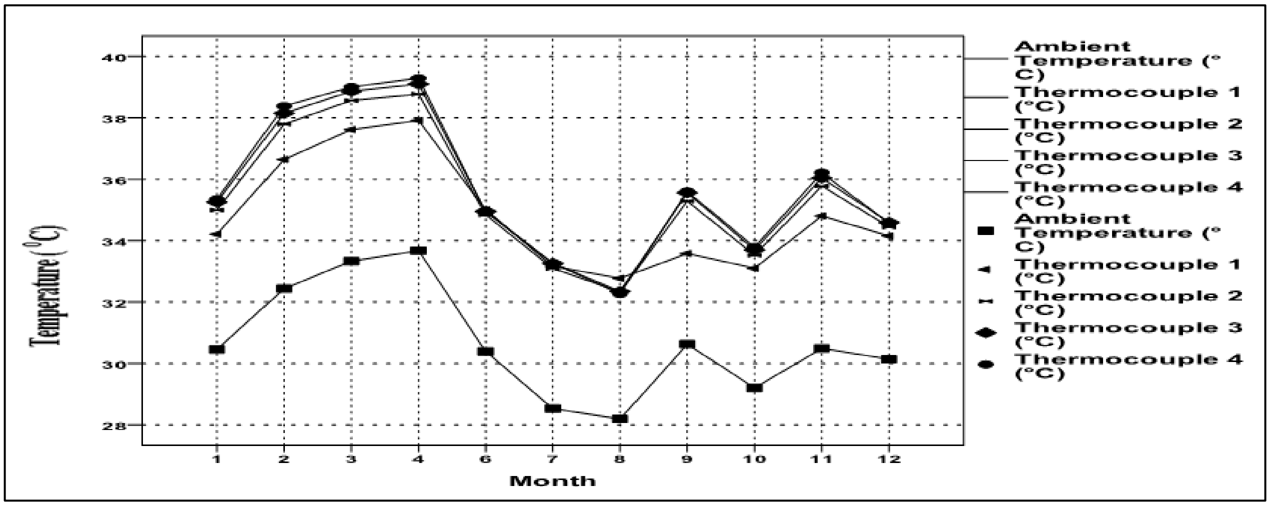



Figure 5.20 – Temperature Variations for the Months (Akumadan Site)

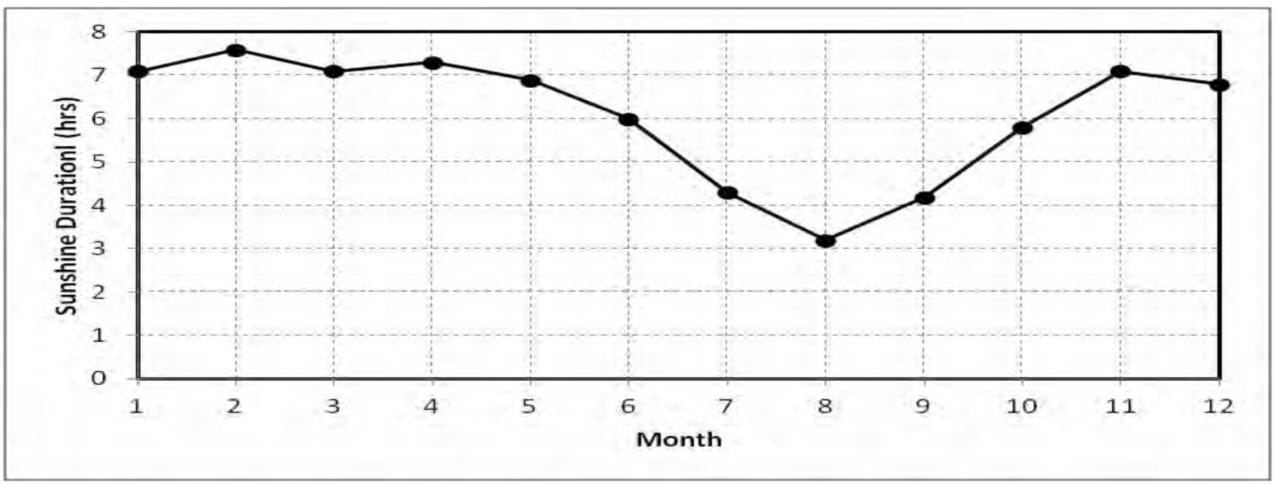



Figure 5.21 – Sunshine Durations for the Months (Akumadan Site)

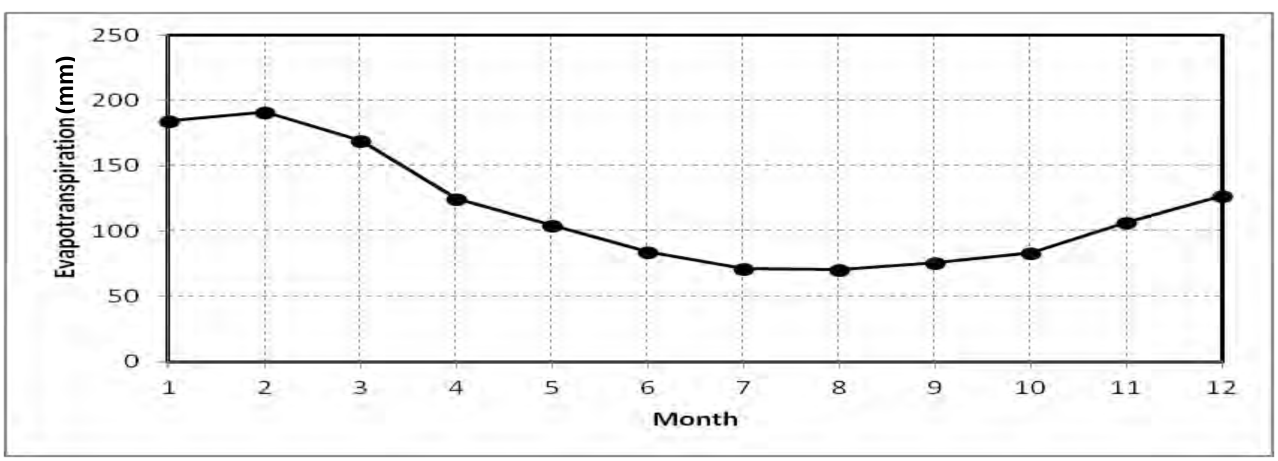



Figure 5.22 – Temperature Variations for the Months (Akumadan Site)

From Table 5.8, it had been revealed that there is a weak positive relation between the Base and the Sub Base layer for the Akumadan Site.

Table 5.8 - Correlations - Akumadan

| | | Base | Subbase |
|---|---|---|---|
| **Base** | Pearson Correlation | 1 | .456** |
| | Sig. (2-tailed) | | .000 |
| | N | 355 | 355 |

**. Correlation is significant at the 0.01 level (2-tailed).

## 5.5 Modulus and Temperature Variation

Figure 5.23 displays a typical variation of temperature with moduli with time for the Akumadan site. The timescale is the hourly periods for the daily data collected from 6am to 6pm. The pattern was the same for the Sogakope site also (see Figure 5.24) except that surprisingly, an unexpected pattern pops up with regards to moduli E4 and thermocouple 1. The pattern observed here deviated from the usual one. Here, both the moduli and the thermocouple rise at the same time and fall as well. From the figure, it can be observed that at a particular time period, when the temperature attains it maximum, the moduli on the other hand attains it minimum and vice versa. This trend repeats itself for the seven (7) days that data were gathered. It can also be observed that on day four, the pattern seems not to have repeated itself.

Analysis of all the other layers showed the same pattern between temperature and moduli with time as stated above. From this pattern, we can infer that, if a relationship is established, it will be a negative one. Later in the analysis, we will try to establish a regression to enable us predict the likely value of the moduli given a particular value of the temperature.

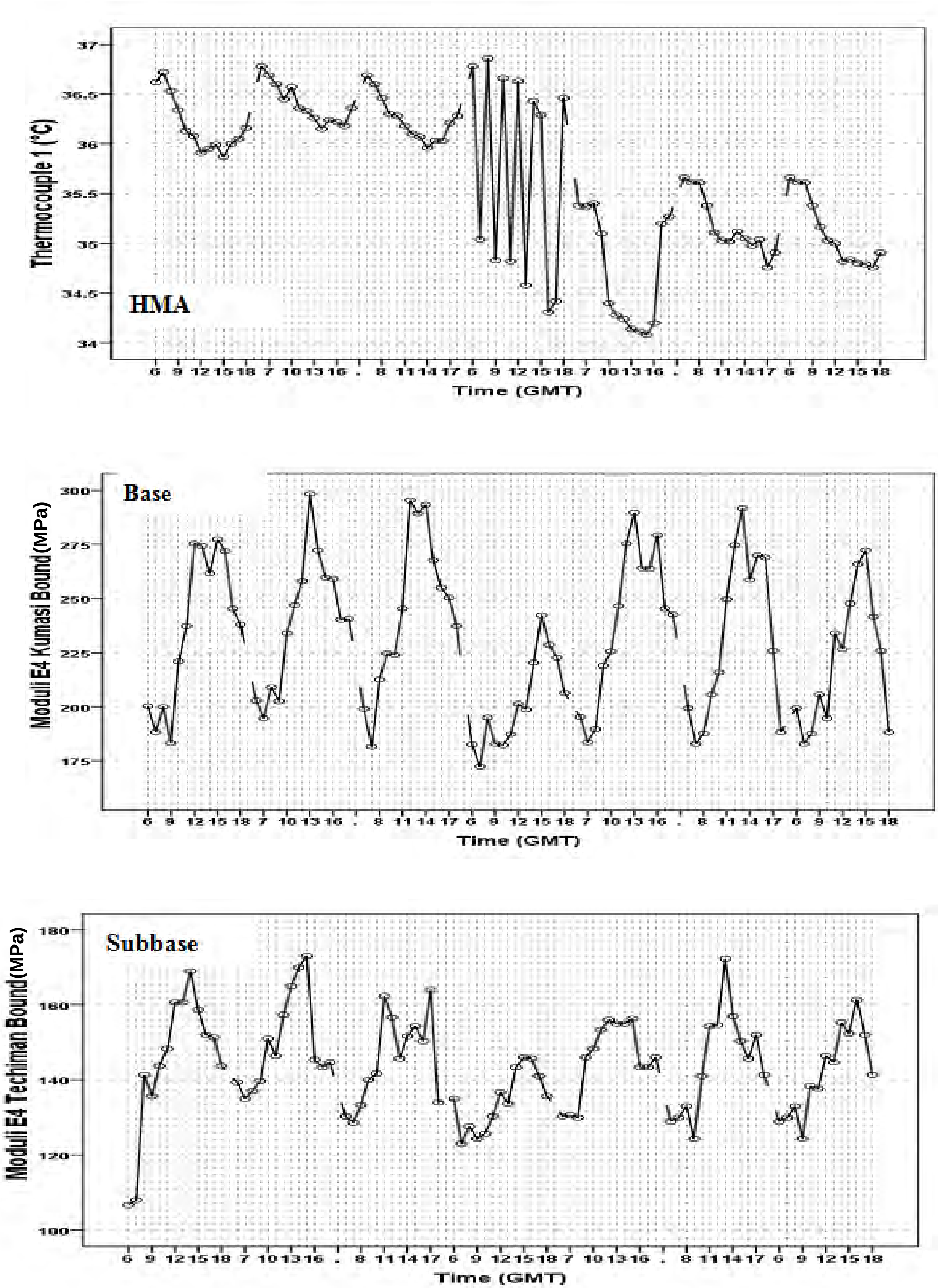



Figure 5.23 - Temperature and Moduli Variation with Time (Akumadan Site)

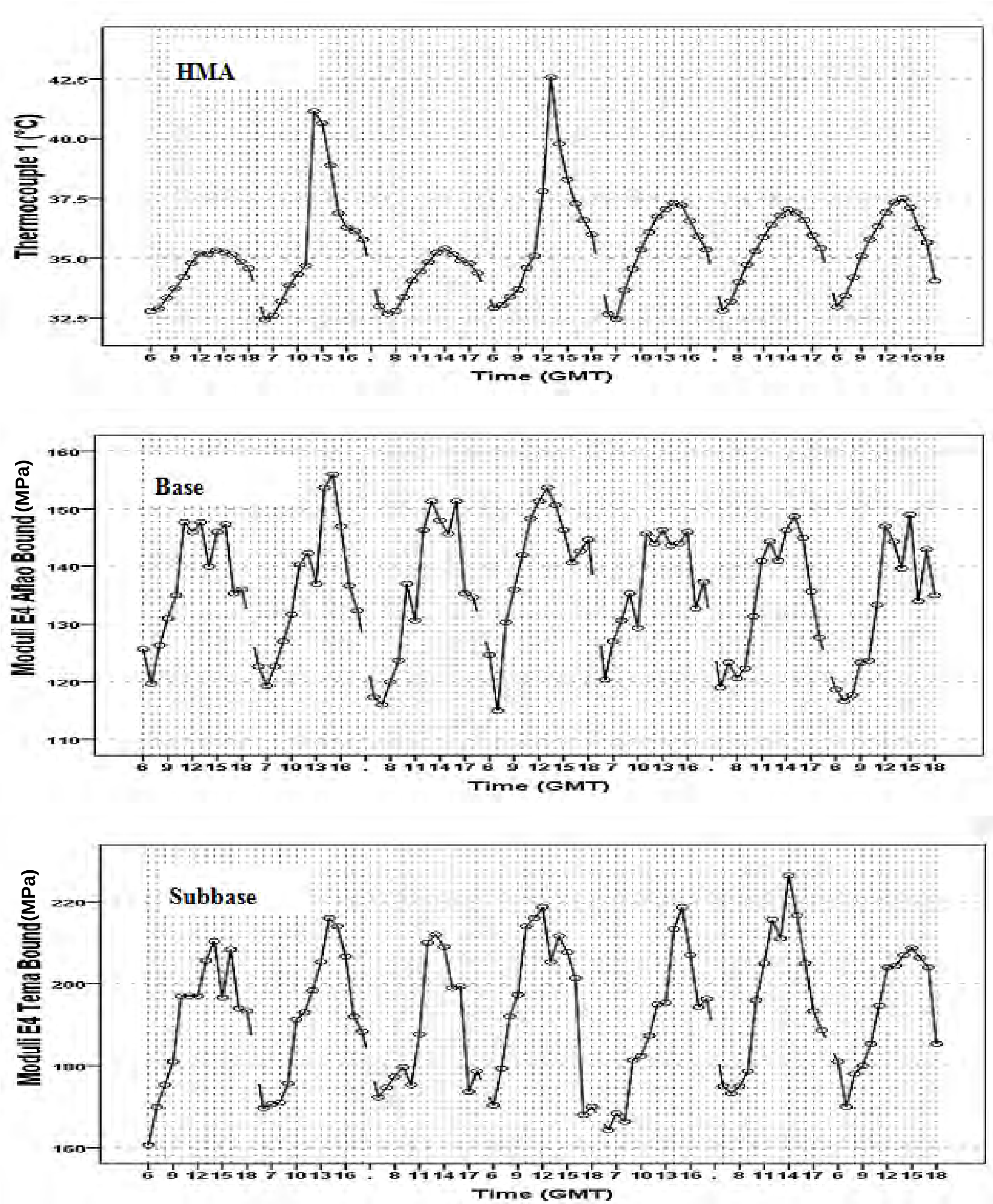



Figure 5.24 - Temperature and Moduli Variation with Time (Sogakope site)

## 5.6 Cumulative Traffic – Moduli Variations

Figures 5.24 & 5.25 show the typical examples of how the various moduli vary hourly with cumulative traffic from the various sites.

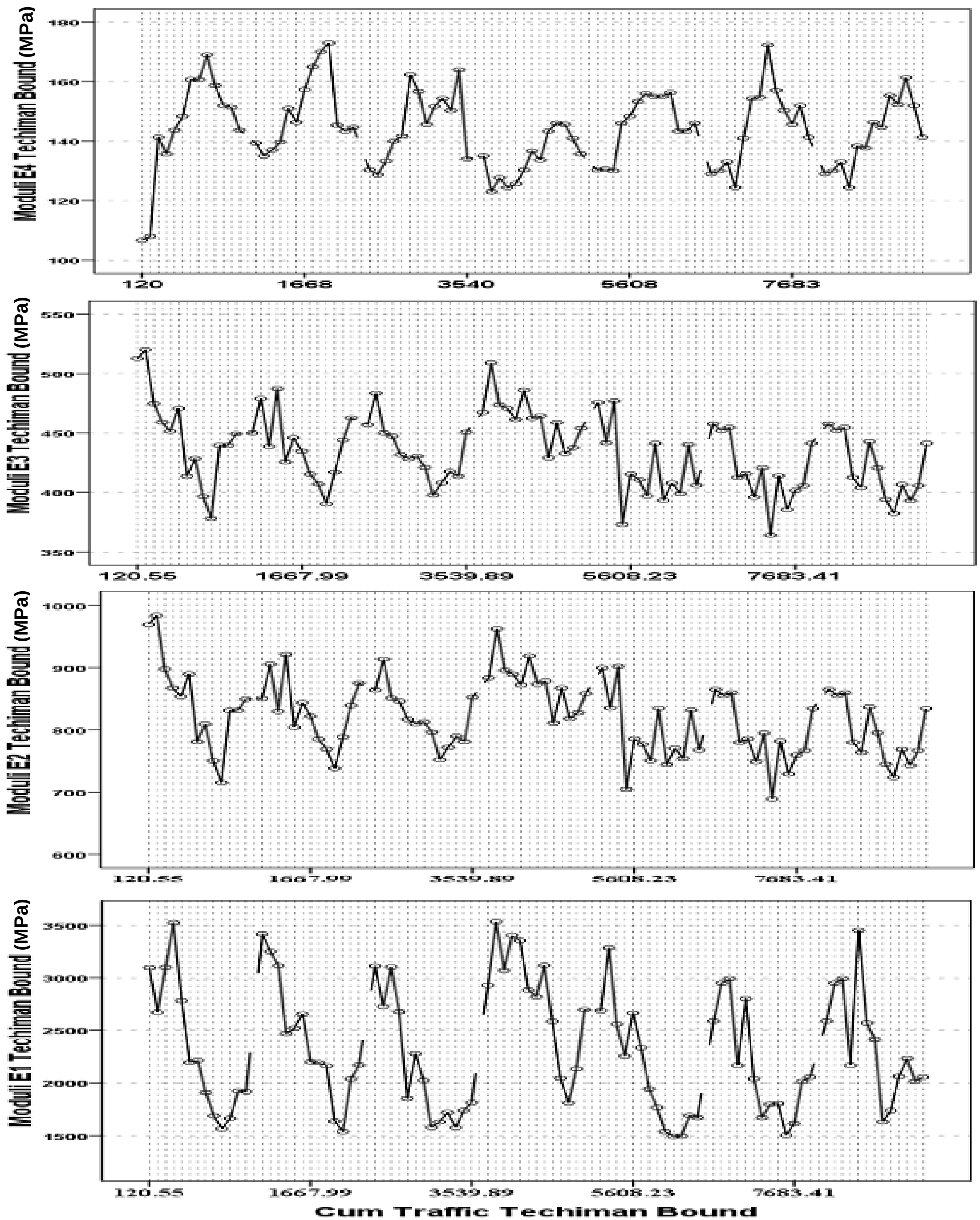



Figure 5.25 - Hourly changes in moduli due to cumulative traffic in each pavement layer (Akumadan Site)

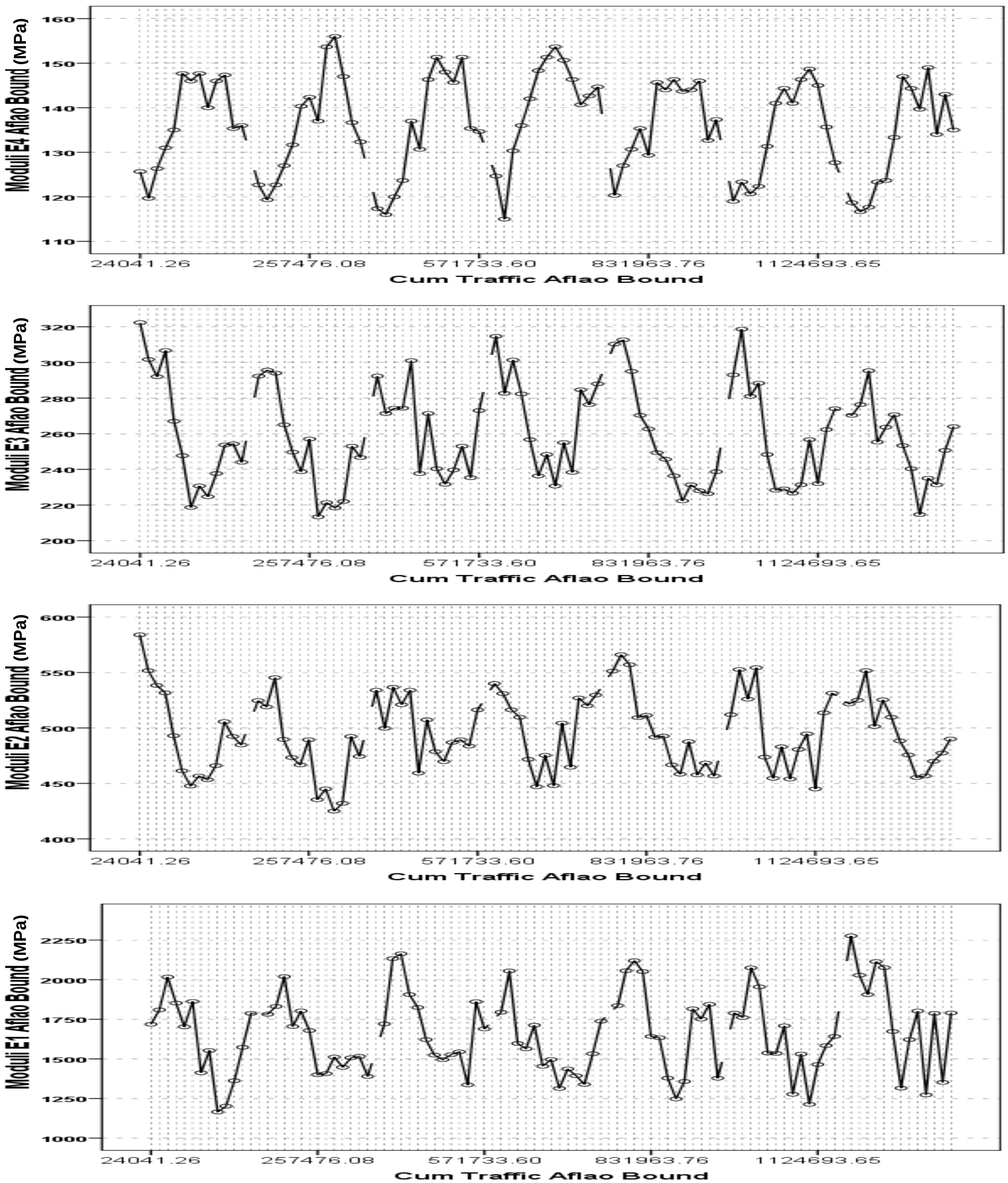



Figure 5.26 - Hourly changes in modules due to cumulative traffic in each pavement layer (Sogakope Site)

## 5.7 Influence of Temperature, Traffic on Moduli Variations

It is believed that temperature and traffic have some influence on the modulus. With this information, we want to determine a multiple linear regression equation, with the assumption that the relationship between them is linear. To check whether the regression equation generated is representative of the data, we will use $R^2$ to verify that. If the $R^2$ is closer to zero, then the equation does not represent the data. If the $R^2$ is closer to one (1), then the equation does represent the data. The equation will be of the form

$$Y = a + B_0 X_0 + B_1 X_1$$

Where Y represents the modulus, $X_0$ represents the temperature, with coefficient $B_1$, and $X_1$ represents the cumulative traffic, with coefficient $B_2$

Table 5.9 - Regression Equation Output For Predicting Modulus Using Temperature And Traffic As Predictors, Akumadan Site

| **Dependent** | **Independent variable** | | **Coefficients** | | | **$R^2$** |
|---|---|---|---|---|---|---|
| **Modulus** | **Temperature** | **Cumm. Traffic** | **a** | **$B_0$** | **$B_1$** | |
| E1 Techiman Bound | T4T | Techiman Bound | 5001.210 | -63.07 | -.038 | .755 |
| E2 Techiman Bound | T3T | Techiman Bound | 1122.115 | -6.891 | -.009 | .545 |
| E3 Techiman Bound | T2T | Techiman Bound | 614.443 | -4.316 | .001 | .402 |
| E4 Techiman Bound | T1T | Techiman Bound | 399.809 | -6.994 | -.001 | .091 |
| E1 Kumasi Bound | T4T | Kumasi Bound | 6680.038 | -89.966 | -.008 | .837 |
| E2 Kumasi Bound | T3T | Kumasi Bound | 1255.726 | -9.847 | -.008 | .593 |
| E3 Kumasi Bound | T2T | Kumasi Bound | 690.972 | -6.081 | -.005 | .389 |
| E4 Kumasi Bound | T1T | Kumasi Bound | 975.345 | -20.310 | -.005 | .113 |

Table 5.10 - Regression Equation Output For Predicting Modulus Using Temperature And Traffic As Predictors, Sogakope Site

| Dependent | Independent variable | | Coefficients | | | $R^2$ |
|---|---|---|---|---|---|---|
| **Modulus** | **Temperature** | **Cumm. Traffic** | **a** | $B_0$ | $B_1$ | |
| E1 Aflao Bound | T4T | Aflao Bound | 3999.725 | -68.277 | .000 | .437 |
| E2 Aflao Bound | T3T | Aflao Bound | 698.509 | -4.927 | 7.263E-6 | .517 |
| E3 Aflao Bound | T2T | Aflao Bound | 500.454 | -6.111 | -5.284E-6 | .075 |
| E4 Aflao Bound | T1T | Aflao Bound | -4.404 | 4.072 | -4.952E-6 | .523 |
| E1 Tema Bound | T4T | Tema Bound | 9002.720 | -188.323 | .000 | .642 |
| E2 Tema Bound | T3T | Tema Bound | 1184.795 | -11.381 | -3.397E-6 | .749 |
| E3 Tema Bound | T2T | Tema Bound | 699.302 | -7.856 | -8.714E-6 | .072 |
| E4 Tema Bound | T1T | Tema Bound | -2.626 | 5.526 | -1.721E-7 | .456 |

## 5.8 Deflection – Temperature variations

Figure 5.27 & 5.28 show the simultaneous variation in maximum deflection and temperature with respect to time. Form the figures, it can be observed that the maximum deflection rises with a rise in the temperature and falls with a fall in the temperature. This trend depicts some kind of positive relationship between the two variables and it is the same for all the deflection-temperature variations for both sites.

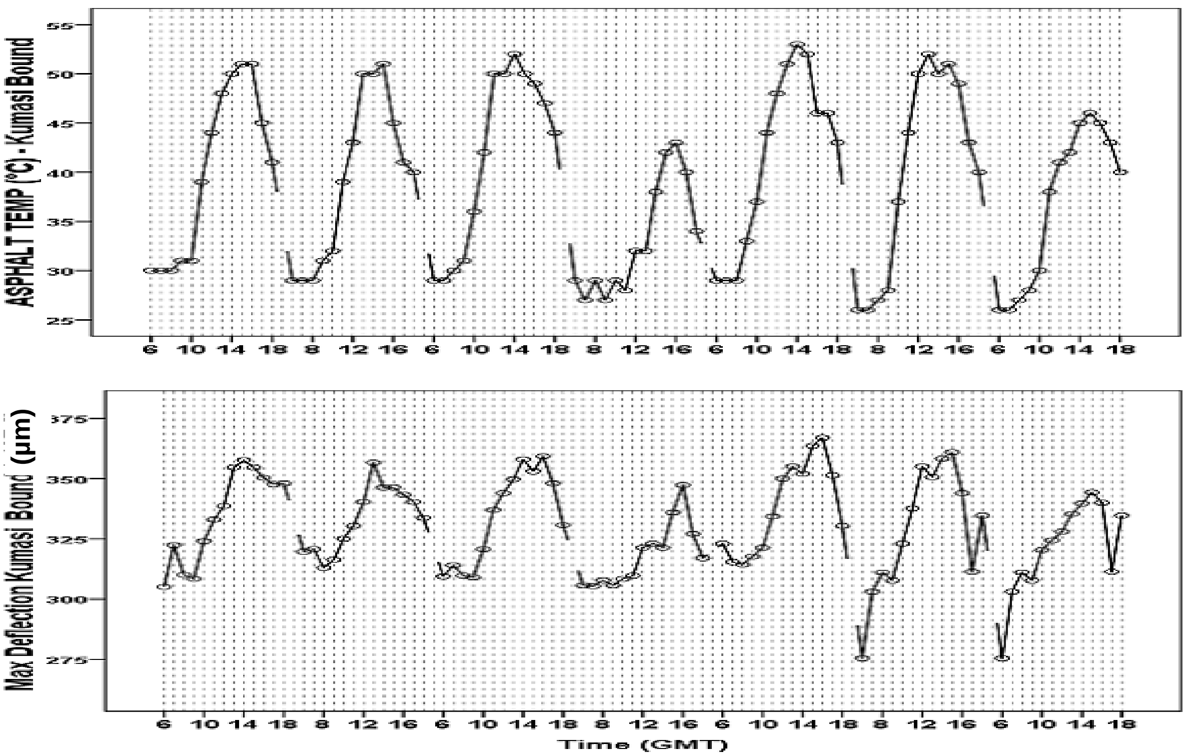



Figure 5.27 - Maximum Deflection and Temperature with time (Akumadan Site)

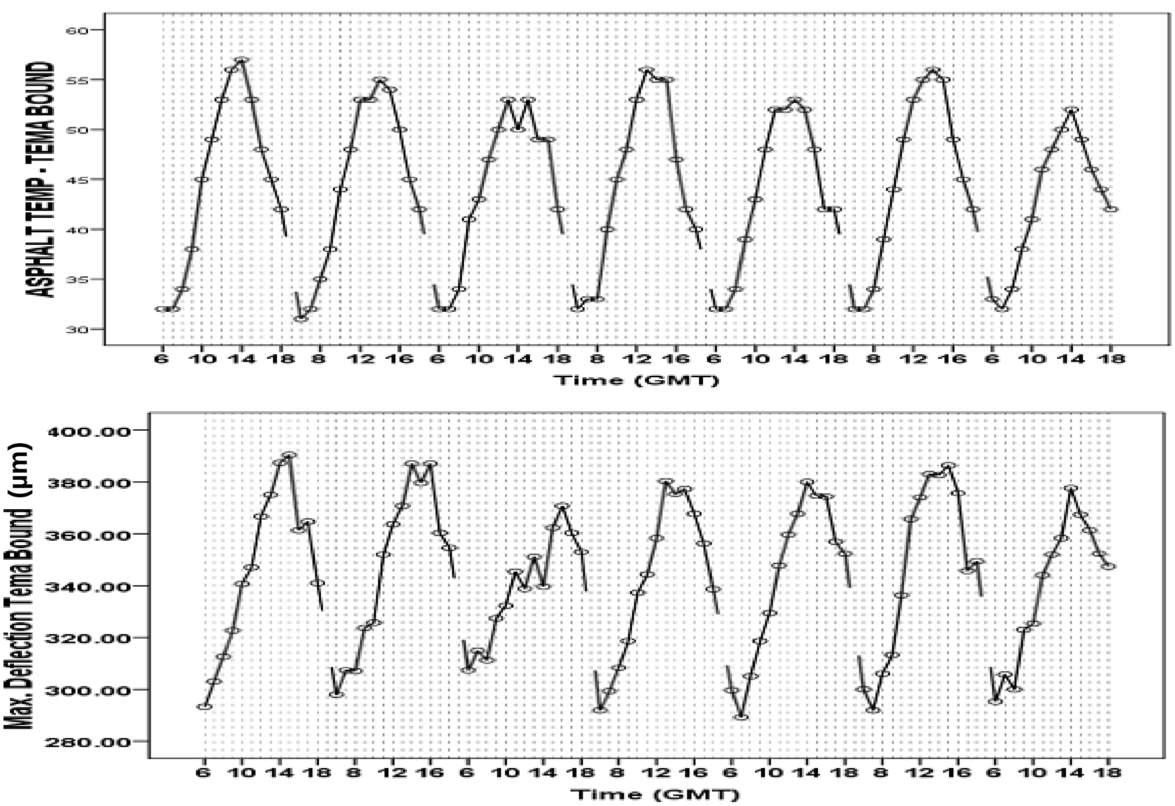



Figure 5.28 - Maximum Deflection and Temperature with Time (Sogakope Site)

## 5.9 Deflection - Traffic Variations

Figures 5.29 & 5.30 depict how the maximum deflection and cumulative traffic vary simultaneously with time for all the bounds at both sites.

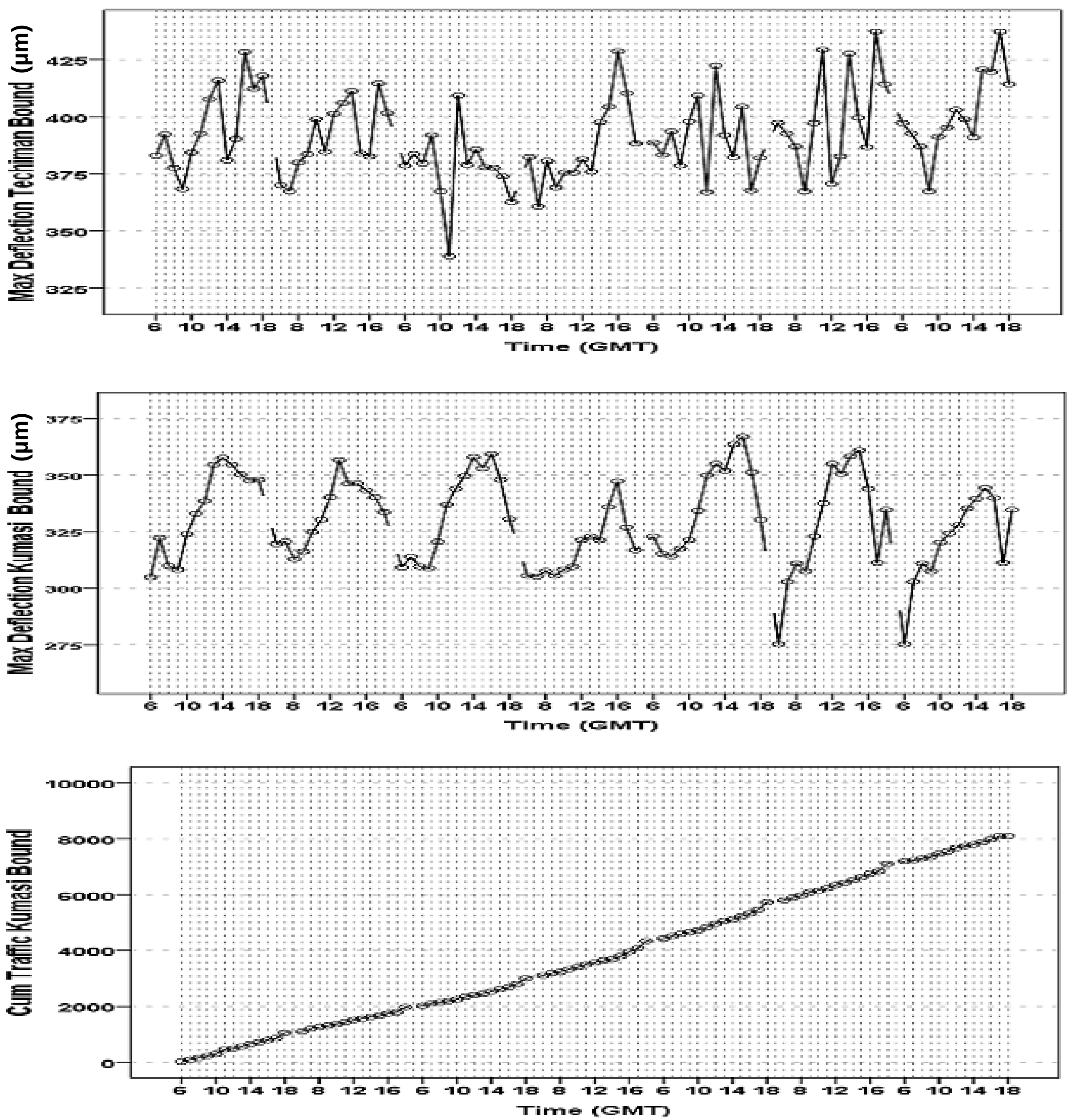



Figure 5.29 - Deflection and Traffic Variation with time (Akumadan Site)

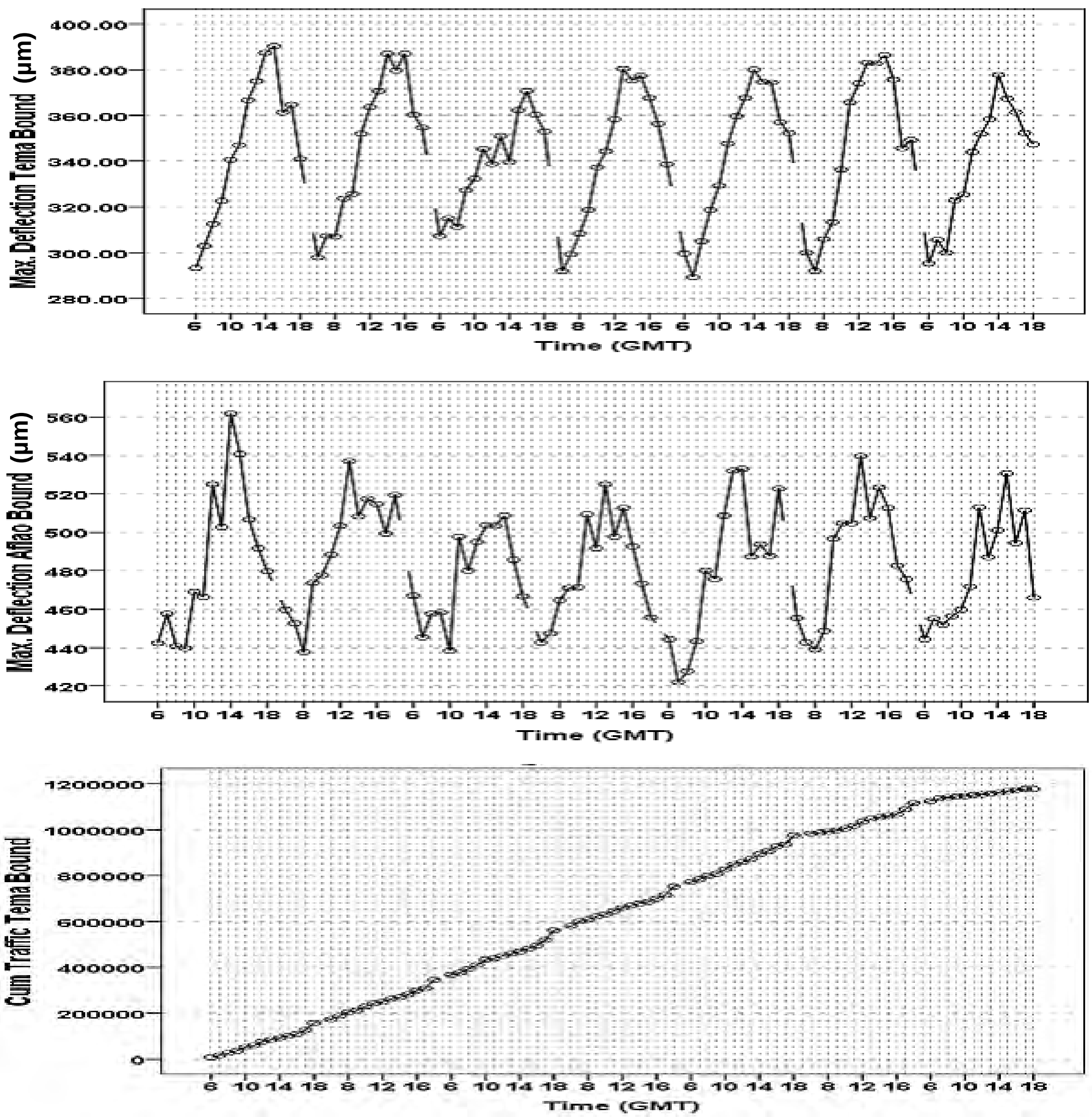



Figure 5.30 - Deflection and Traffic Variation with time (Sogakope Site)

### 5.10 Modulus – temperature relationship

In developing a modulus – temperature relationship, a scatter plot was used to determine the relationship between the four moduli and their corresponding temperatures for the two test sites. In plotting the scatter plot, we assume both linear and exponential relationship, but surprisingly, the coefficient of determination ($R^2$) obtained in both instances are the same, so we adopted a linear model. From Figure 5.32, the Kumasi bound and Aflao bound have somehow moderate $R^2$; 0.532 and 0.51 respectively. From Figures 5.33 and 5.34, the coefficient of determination obtained for the Techiman bound, Aflao bound and Tema bound are closer to zero, hence unreliable for prediction. From the coefficient of determination obtained for Figures 5.33 and 5.34, the coefficient of determination ranges between 0.07 and 0.495. These values indicate that a linear relationship when build will not be appropriate for prediction.

However, the Tema bound from Figure 5.32 and the Kumasi and Techiman bound form Figure 5.31 have very good coefficients of determination; 0.749, and 0.836 and 0.722 respectively. Regression model can then be built for predicting Moduli E2 using Thermocouple 3 for the Tema bound. Also, regression model for predicting Moduli E1 using Thermocouple 4 for both Kumasi and Techiman bound can be built. The issue however is that, the model built for a particular bound can only apply to that specific bound but not to all the other bounds. The simple linear regression equation of the form;

$$Y = b_0 + b_1X$$

Where Y represents the Moduli, X is the corresponding temperature, $b_0$ is the intercept, and $b_1$ is the slope.

Table 5.11 - Estimated Coefficients of the Linear Function; Moduli-Temperature Relationship

| **Site** | **Dependent variable** | **Independent variable** | **$b_0$** | **$b_1$** | **$R^2$** |
|---|---|---|---|---|---|
| Akumadan – Kumasi Bound | Moduli E1 | Thermocouple 4 | 6645.152 | -89.867 | 0.836 |
| Akumadan –Techiman Bound | Moduli E1 | Thermocouple 4 | 4803.135 | -62.395 | 0.722 |
| Sogakope –Tema Bound | Moduli E2 | Thermocouple 3 | 1183.246 | -11.405 | 0.749 |

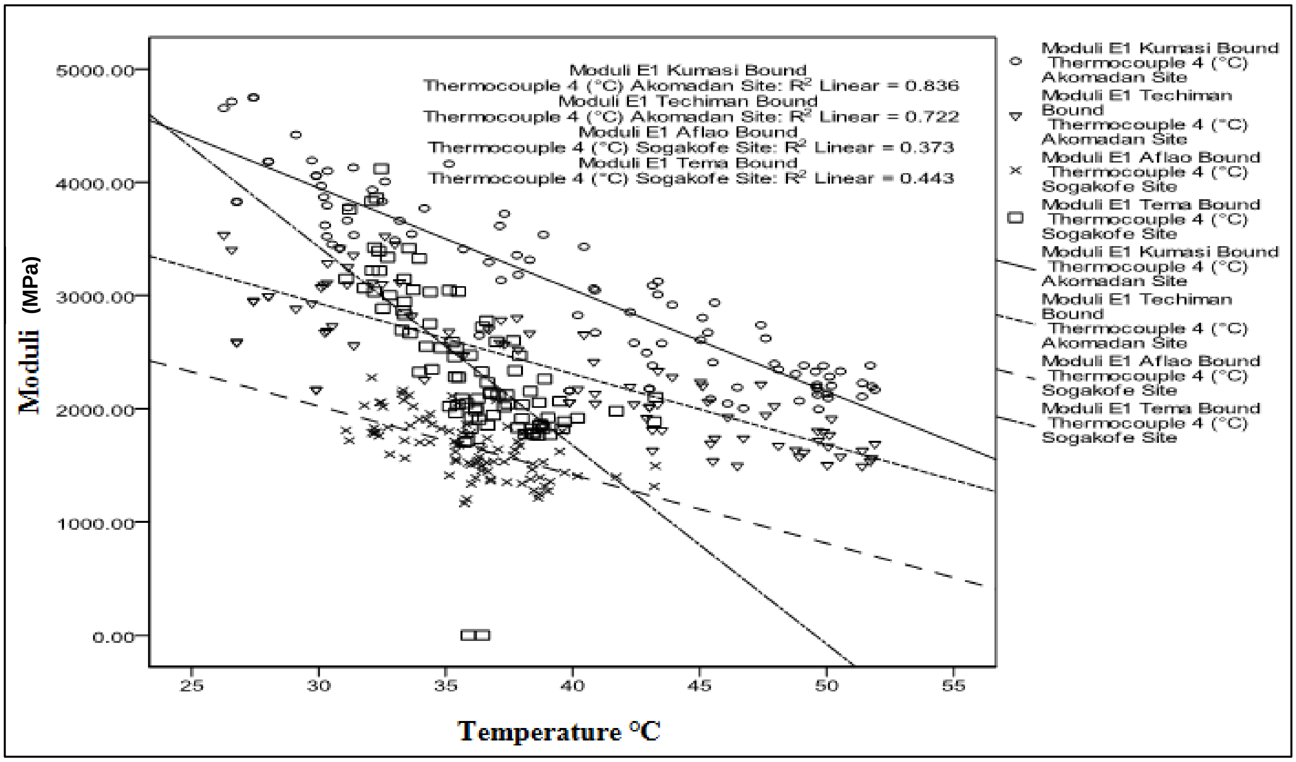



Figure 5.31 - Modulus E1– Temperature (Thermocouple 4) relationship for both sites

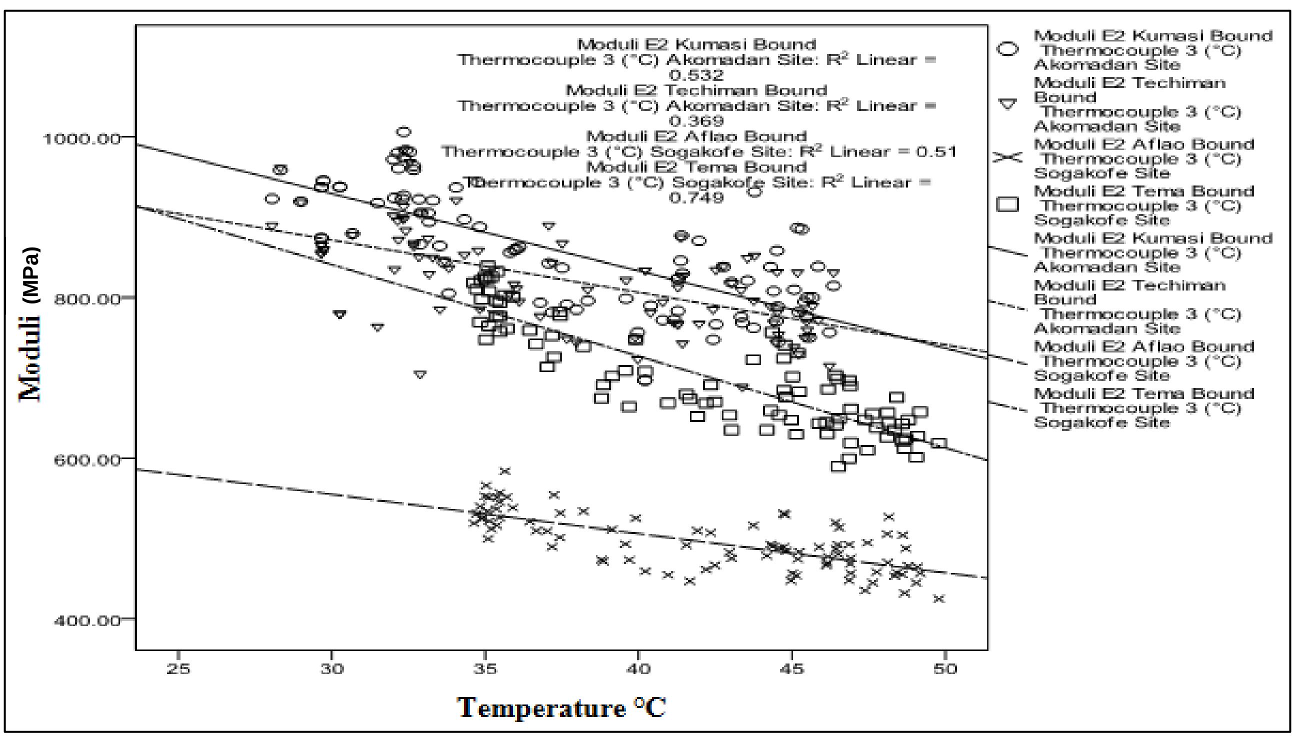



Figure 5.32 - Modulus E2– Temperature (Thermocouple 3) relationship for both sites

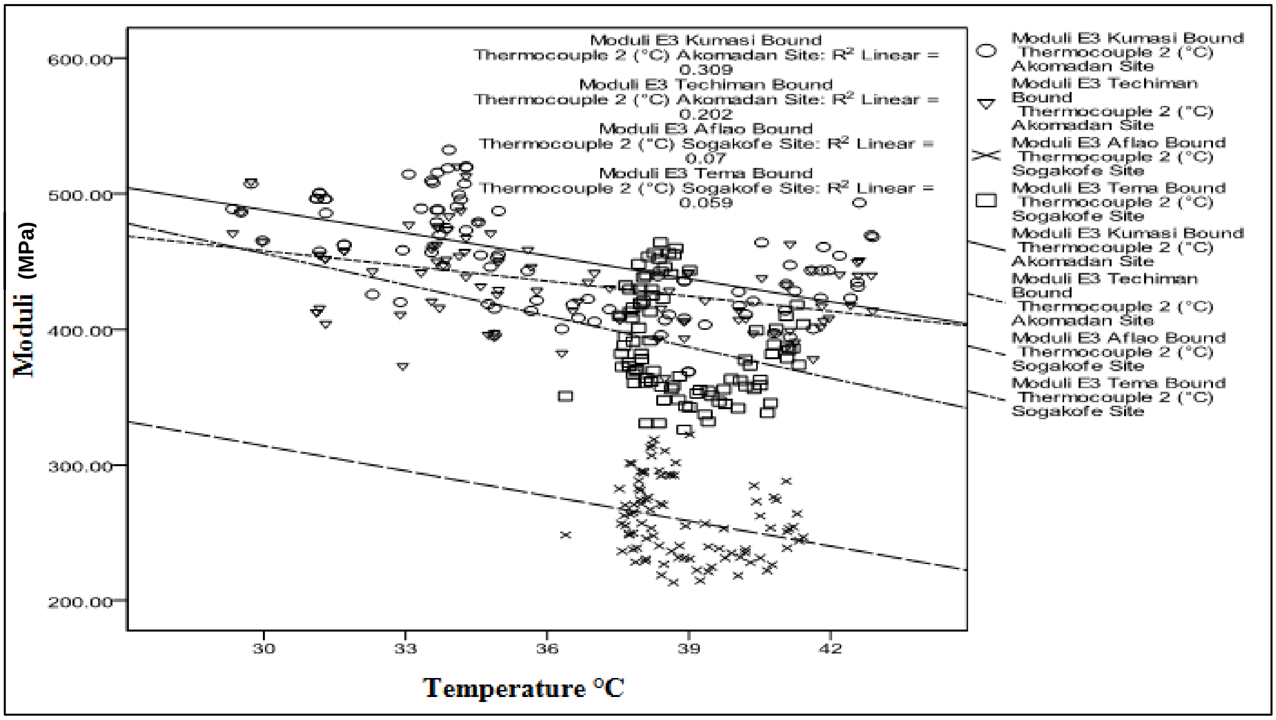



Figure 5.33 - Modulus E3– Temperature (Thermocouple 2) relationship for both sites

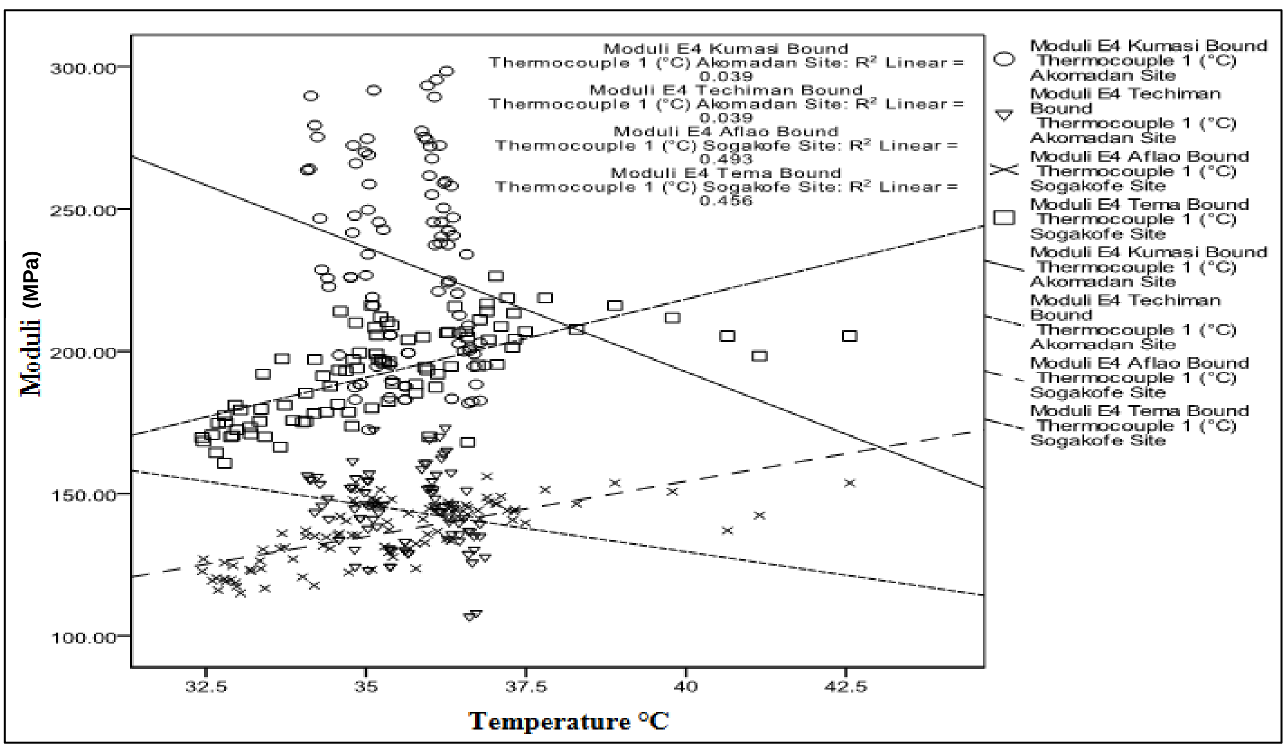



Figure 5.34 - Modulus E4– Temperature (Thermocouple 1) relationship for both sites

## 5.11 Moduli - Cumulative traffic relationship

In trying to find the kind of relationship that existed between the modulus and the cumulative traffic, a scatter diagram was first plotted. A closer look at the scatter diagram revealed that the very weak relationship between these two variable; refer to Figure 5.35. This is also evident from the very low $R^2$ obtained. Due to this, a regression model for prediction will not yield any good result.

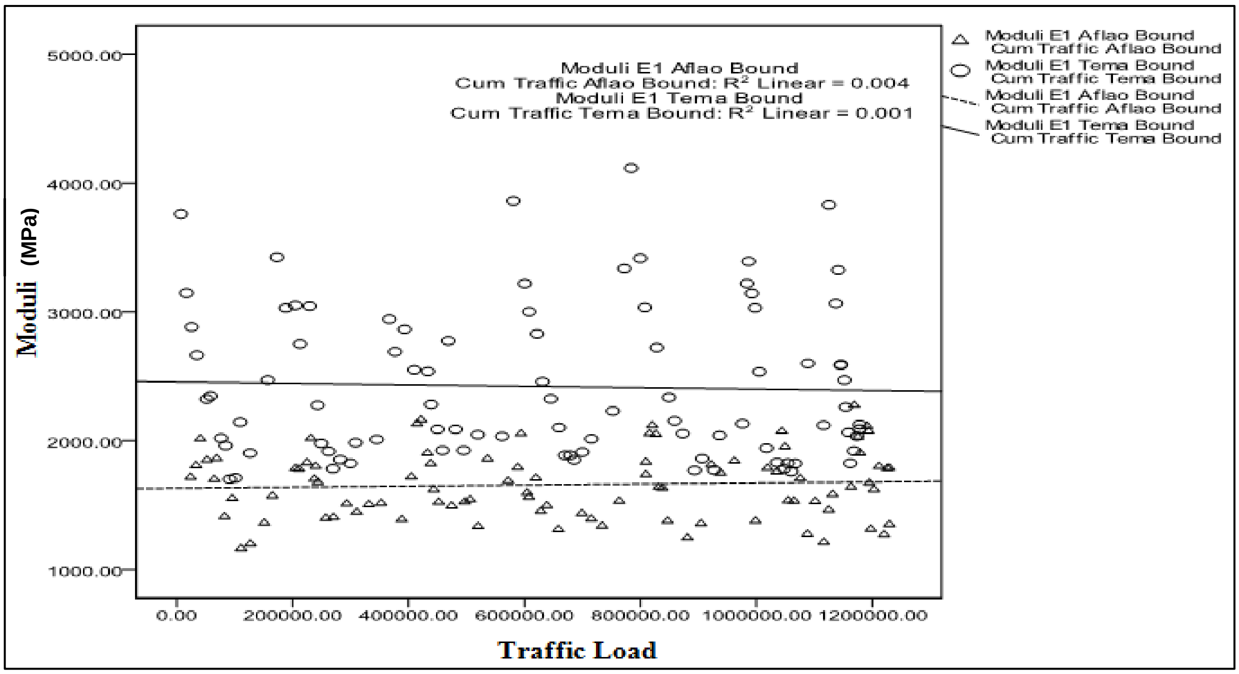



Figure 5.35 - Moduli-Cumulative Traffic relationship

## 5.12 Deflection – Temperature Relationship

Again, a scatter plot helped determine the likely relationship between the deflection and temperature. From Figure 5.36, it can be observed that a linear relationship can be established with the exception of the Techiman bound of the Akumadan site which recorded a very low $R^2$, 0.112. The Kumasi bound, Aflao bound and Tema bound respectively recorded $R^2$ of 0.822, 0.701 and 0.803, respectively; an indication of a good model.

Table 5.12 shows the estimated regression coefficients for the Kumasi bound, Aflao bound and Tema bound.

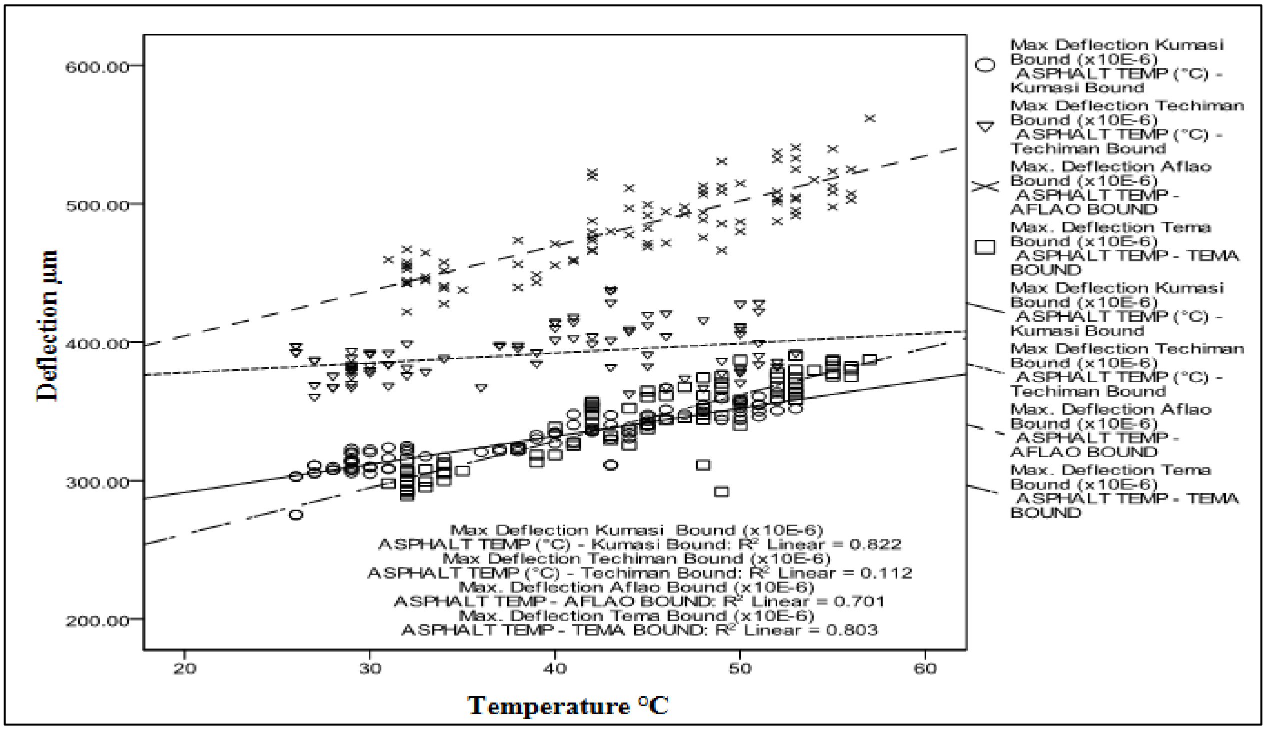



Figure 5.36 - Deflection – Temperature Relationship

Table 5.12 - Estimated Coefficients of the linear function (Max. Deflection-Temperature relationship)

| Site | Dependent variable | Independent variable | $b_0$ | $b_1$ | $R^2$ |
|---|---|---|---|---|---|
| Akumadan– Kumasi Bound | **Max. Deflection** | **Asphalt Temperature** | 251.088 | 2.023 | 0.822 |
| Sogakope –Aflao Bound | **Max. Deflection** | **Asphalt Temperature** | 339.627 | 3.251 | 0.701 |
| Sogakope –Tema Bound | **Max. Deflection** | **Asphalt Temperature** | 193.98 | 3.364 | 0.803 |

## 5.13 Deflection – Traffic relationship

From Figure 5.37, the scatter diagram shows that there is a very weak relationship between deflection and the cumulative traffic. This is also evident from the low $R^2$ values obtain between these two variables. Hence, a regression model when developed will yield miss leading results.

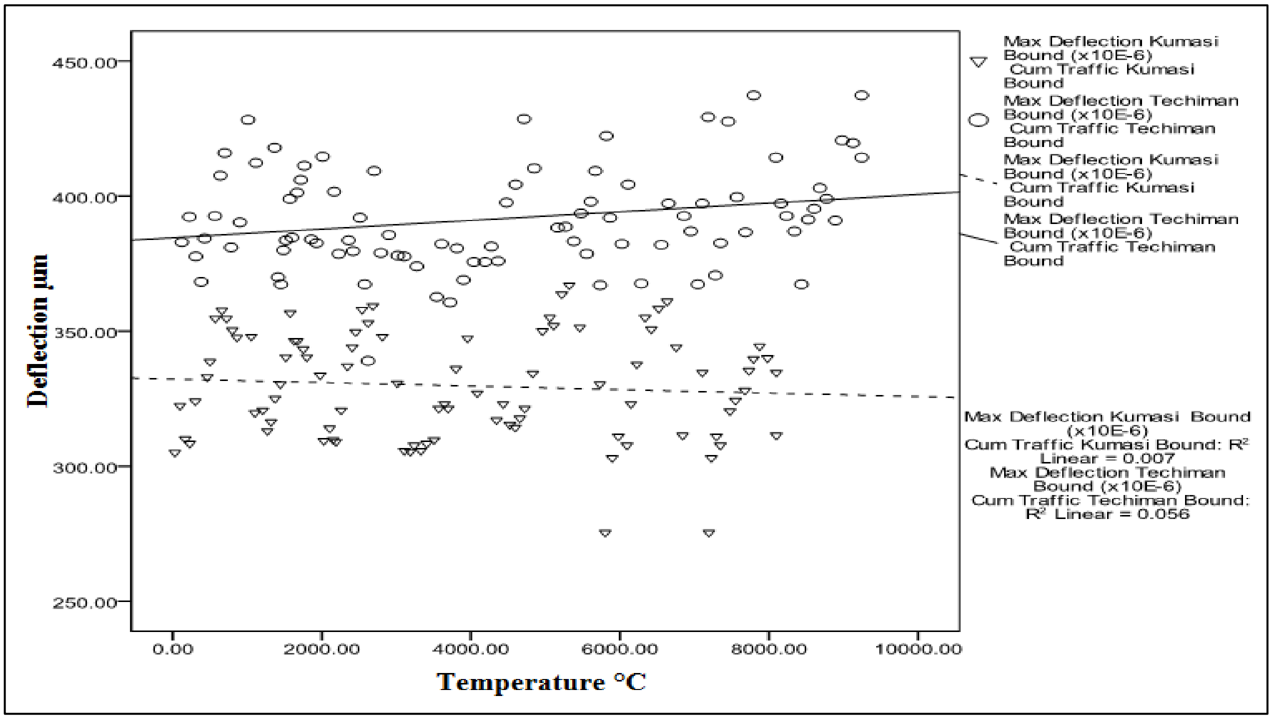



Figure 5.37 - Deflection – Cumulative Traffic Relationship

## 5.14 Traffic Analysis

The 12hr counts were converted to 24hr counts by applying GHA daily conversion factor of 1.24 for the highway sections under consideration.

The highlight of the traffic counts on the main road is as follows:

### 5.14.1 Kumasi-Akumadan-Techiman

The ADT on this section was 3,302 with small buses being the most predominant vehicle category – 26.30%; followed by motor bike – 15.12%. Table 5.13 shows the vehicle class distribution and the hourly distribution for the Kumasi-Akumadan-Techiman Section.

Light vehicles formed 77.36% of the traffic surveyed. Medium category accounted for 13.69% and heavy category – 8.95% (see Figure 5.38). The lane distribution of vehicles was as follows;

- **Northbound** – 49.7%
- **Southbound** – 50.3%.

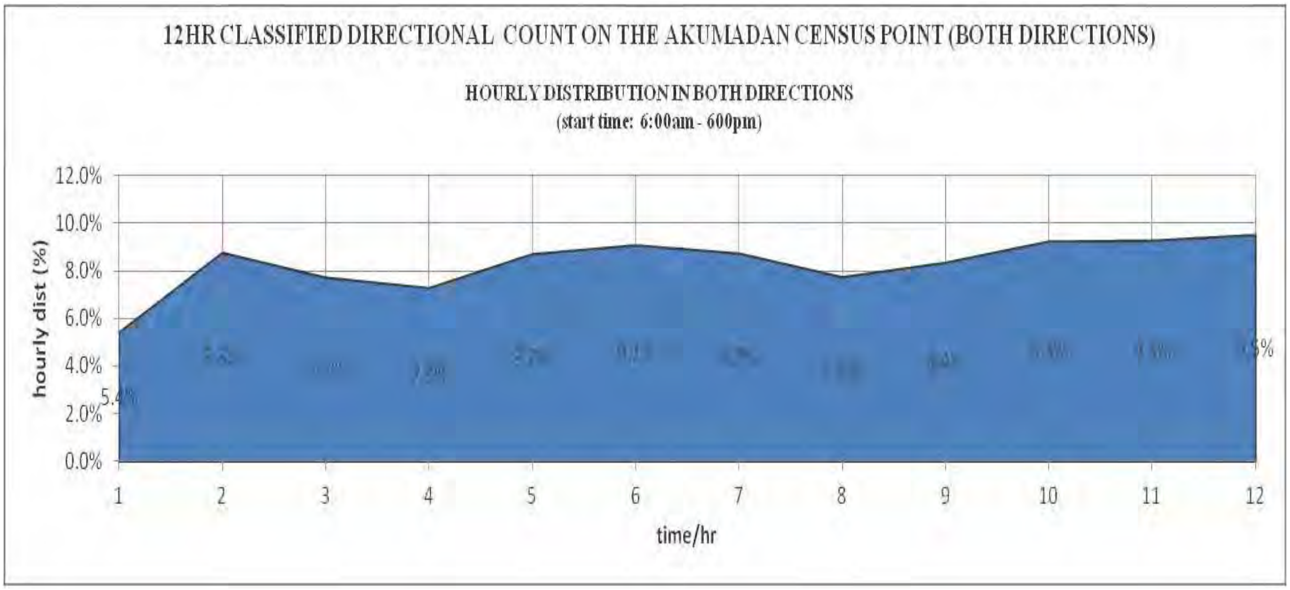



Figure 5.38 – 12 Hour Classified Directional Count on the Akumadan Census Point (Both Directions)

Table 5.13 – Summary of Daily Traffic Count for the Akumadan Census Point

**Census point:** **Akumadan**
**Direction: both directions**
**Station number** **Days** **Monday - Sunday**
**Date: 21/02/2011** **Comment**
**Start time: 6:00am** **Factor to convert 12hr to 24hr =1.240**

| Vehicle type | 6:00 - 7:00 am | 7:00 - 8:00 am | 8:00 - 9:00 am | 09:00 - 10:00 am | 10:00 - 11:00 am | 11:00 - 12:00 pm | 12:00 - 13:00 pm | 13:00 - 14:00 pm | 14:00 - 15:00 pm | 15:00 - 16:00 pm | 16:00 - 17:00 pm | 17:00 - 18:00 pm | Total 12 hrs | Fact'd 24hrs | Veh. Class Dist'n |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| **Motor bike** | 63 | 128 | 105 | 108 | 149 | 123 | 112 | 103 | 135 | 157 | 129 | 124 | 1436 | 1781 | 15.13% |
| **Cars** | 37 | 65 | 49 | 50 | 75 | 88 | 76 | 80 | 76 | 71 | 83 | 83 | 833 | 1033 | 8.78% |
| **Taxis** | 36 | 107 | 89 | 94 | 106 | 93 | 199 | 81 | 68 | 100 | 108 | 102 | 1183 | 1467 | 12.47% |
| **Pick-up/van** | 59 | 80 | 90 | 81 | 115 | 179 | 139 | 122 | 146 | 131 | 122 | 119 | 1383 | 1715 | 14.57% |
| **Small bus** | 106 | 171 | 190 | 162 | 211 | 224 | 222 | 214 | 234 | 252 | 240 | 239 | 2465 | 3057 | 25.97% |
| **Med bus/mummy wagons** | 2 | 9 | 7 | 2 | 4 | 6 | 3 | 3 | 5 | 8 | 8 | 8 | 65 | 81 | 0.68% |
| **Large bus** | 22 | 49 | 39 | 28 | 22 | 26 | 28 | 25 | 28 | 67 | 45 | 46 | 425 | 527 | 4.48% |
| **Light truck** | 16 | 25 | 13 | 7 | 16 | 14 | 18 | 17 | 22 | 19 | 19 | 21 | 207 | 257 | 2.18% |
| **Medium truck** | 29 | 37 | 43 | 32 | 52 | 46 | 53 | 44 | 50 | 52 | 57 | 83 | 578 | 717 | 6.09% |
| **Heavy truck** | 5 | 4 | 0 | 1 | 1 | 0 | 0 | 0 | 1 | 5 | 5 | 4 | 26 | 32 | 0.27% |
| **Semi-trailer (light)** | 18 | 23 | 22 | 14 | 9 | 23 | 7 | 18 | 11 | 16 | 13 | 28 | 202 | 250 | 2.13% |
| **Semi-trailer (heavy)** | 6 | 10 | 14 | 12 | 5 | 5 | 4 | 9 | 14 | 10 | 8 | 5 | 102 | 126 | 1.07% |
| **Truck trailer** | 22 | 31 | 23 | 21 | 17 | 19 | 25 | 15 | 20 | 19 | 16 | 30 | 258 | 320 | 2.72% |
| **Extra large truck & others** | 24 | 36 | 11 | 17 | 24 | 29 | 30 | 24 | 33 | 38 | 33 | 28 | 327 | 405 | 3.45% |
| **Total** | 445 | 775 | 695 | 629 | 806 | 875 | 916 | 755 | 843 | 945 | 886 | 920 | 9490 | 11768 | 100% |
| **Hourly dist'n** | 4.7 % | 8.2 % | 7.3 % | 6.6% | 8.5% | 9.2% | 9.7% | 8.0% | 8.9% | 10.0 % | 9.3% | 9.7% | 100.0 % | **ADT=1681** | |

### 5.14.2 Tema-Sogakope-Aflao

The ADT recorded was 6,949 vehicles per day. Motor bikes formed the chunk of the volume (33.36%) followed by cars (18.59%), Pick-up/van (16.85) and small buses (15.27%).

Light vehicle category accounted for 91.34%. Medium class accounted for 5.16% and heavy vehicle category formed 3.50% (see Figure 5.39). The lane distributions of vehicle were as follows;

- **Eastbound** – 51.4%
- **Westbound** – 48.6%

Table 5.14 shows the vehicle class distribution and the hourly distribution for the **Tema-Sogakope-Aflao section.**

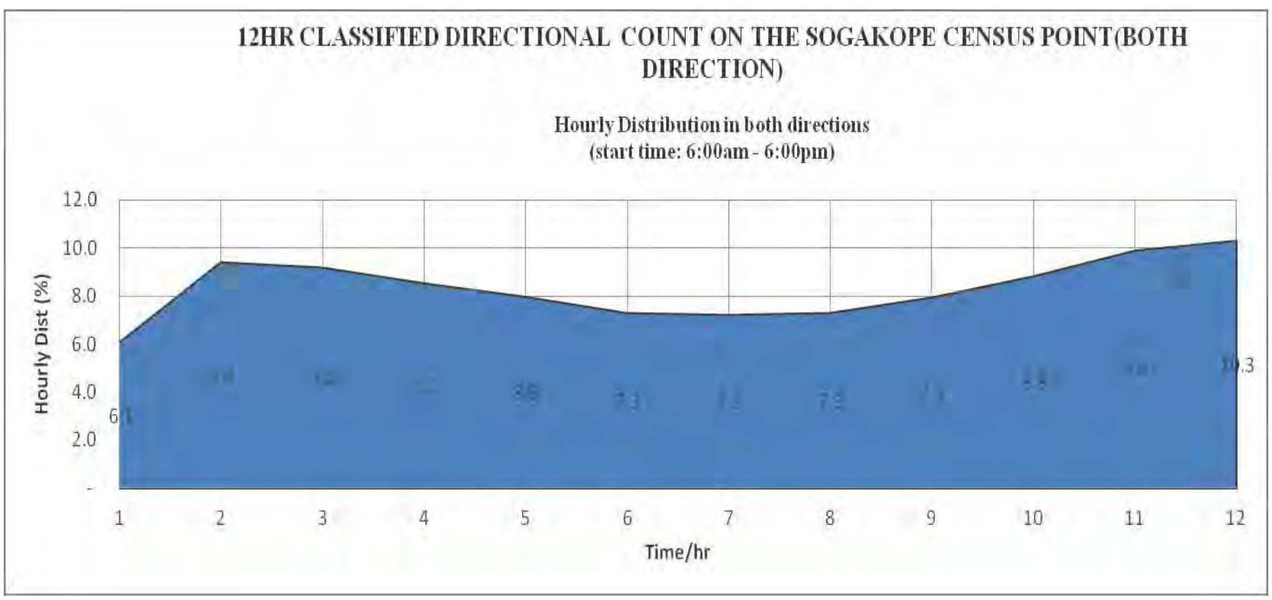



Figure 5.39 – 12 Hour Classified Directional Count on the Sogakope Census Point (Both Directions)

Table 5.14 – Summary of Manual Traffic Count for the Sogakope Census Point

**Census point:** **Sogakope**
**Direction: both directions**
**Station number** **Day:** **Monday - Sunday**
**Date: 02/04/2011** **Comment:**
**Start time: 6:00am** **Factor to convert 12hr to 24hr (avg. Of daily factors) = 1.224**

| Vehicle type | 6:00 - 7:00 am | 7:00 - 8:00 am | 08:00 - 09:00 am | 09:00 - 10:00 am | 10:00 - 11:00 am | 11:00 - 12:00 pm | 12:00 - 13:00 pm | 13:00 - 14:00 pm | 14:00 - 15:00 pm | 15:00 - 16:00 pm | 16:00 - 17:00 pm | 17:00 - 18:00 pm | Total 12hrs | Fact'd 24hrs | Veh. class dist'n |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| **Motor bike** | 869 | 1353 | 1,289 | 1,218 | 1,058 | 974 | 919 | 828 | 904 | 1,097 | 1,318 | 1,434 | 13261 | 16,228 | 33.36% |
| **Cars** | 376 | 737 | 720 | 648 | 619 | 551 | 515 | 567 | 610 | 660 | 684 | 702 | 7,389 | 9,042 | 18.59% |
| **Taxis** | 118 | 277 | 342 | 266 | 221 | 196 | 173 | 165 | 218 | 246 | 311 | 357 | 2,890 | 3,537 | 7.27% |
| **Pick-up/van** | 360 | 536 | 518 | 578 | 542 | 515 | 538 | 568 | 607 | 661 | 651 | 625 | 6,699 | 8,198 | 16.85% |
| **Small bus** | 432 | 569 | 504 | 425 | 456 | 435 | 452 | 485 | 505 | 537 | 652 | 616 | 6,068 | 7,425 | 15.27% |
| **Med bus/mummy wagons** | 1 | 9 | 10 | - | 3 | 7 | 5 | 4 | 4 | 6 | 3 | 3 | 55 | 67 | 0.14% |
| **Large bus** | 23 | 23 | 33 | 39 | 41 | 21 | 24 | 23 | 29 | 36 | 42 | 41 | 375 | 459 | 0.94% |
| **Light truck** | 4 | 8 | - | 2 | 2 | 9 | - | - | - | 2 | 1 | 1 | 29 | 35 | 0.07% |
| **Medium truck** | 113 | 127 | 154 | 123 | 118 | 101 | 140 | 133 | 147 | 152 | 136 | 149 | 1,593 | 1,949 | 4.01% |
| **Heavy truck** | - | 2 | - | 1 | - | 2 | - | 1 | 2 | - | - | - | 8 | 10 | 0.02% |
| **Semi-trailer (light)** | 13 | 17 | 12 | 13 | 10 | 9 | 12 | 23 | 14 | 18 | 19 | 27 | 187 | 229 | 0.47% |
| **Semi-trailer (heavy)** | 28 | 21 | 24 | 35 | 41 | 30 | 33 | 22 | 24 | 22 | 32 | 50 | 362 | 443 | 0.91% |
| **Truck trailer** | 24 | 18 | 31 | 21 | 22 | 28 | 18 | 25 | 28 | 33 | 30 | 39 | 317 | 388 | 0.80% |
| **Extra large truck & others** | 53 | 44 | 17 | 25 | 37 | 23 | 42 | 59 | 67 | 39 | 56 | 56 | 518 | 634 | 1.30% |
| **Total** | 2414 | 3741 | 3,654 | 3,394 | 3,170 | 2,901 | 2,871 | 2,903 | 3,159 | 3,509 | 3,935 | 4,100 | 39751 | 48,644 | 100.0% |
| **Hourly dist'n (%)** | 6.1 | 9.4 | 9.2 | 8.5 | 8.0 | 7.3 | 7.2 | 7.3 | 7.9 | 8.8 | 9.9 | 10.3 | 100.0 | **ADT =6,949** | |

### 5.14.3 Daily Variations in ADT

As previously indicated, 12-hour manual classified counts were carried out for seven (7) days at census points on the various road sections of the project road. It would therefore be necessary to determine the daily variation among the seven days. Table 5.16 shows the daily variation in traffic as recorded along the various sections of the road.

It can be seen from the graphical representation of the daily traffic volumes obtained that there were two peaks. One peak was on Tuesday and the other on Saturday. The higher peak was observed on Saturday, which can be attributed to trips made mostly on Saturdays to attend funerals which are normally held on this day in the Country.

Table 5.15 - Daily Traffic Values, January, 2006

| | **Mon.** | **Tues.** | **Wed.** | **Thu.** | **Fri.** | **Sat.** | **Sun.** | **Total 12 hours** |
|---|---|---|---|---|---|---|---|---|
| **Akumadan (unfactored)** | 2442 | 2785 | 2582 | 2756 | 2895 | 2814 | 1934 | 18208 |
| **Akumadan (factored)** | 3028 | 3453 | 3202 | 3417 | 3590 | 3489 | 2398 | 22578 |
| **Daily Distribution** | 13.4% | 15.3% | 14.2% | 15.1% | 15.9% | 15.5% | 10.6% | 100.0% |
| **Sogakope (unfactored)** | 5,755 | 5,175 | 5,006 | 5,303 | 6,258 | 7,116 | 5,138 | 39,751 |
| **Sogakope (Factored)** | 7,042 | 6,333 | 6,126 | 6,489 | 7,658 | 8,708 | 6,287 | 48,644 |
| **Daily distribution** | 14.5% | 13.0% | 12.6% | 13.3% | 15.7% | 17.9% | 12.9% | 100.0% |

## 5.15 Axle load analysis

The method of analysis was based on the use of a simple excel spreadsheet program. Axle loads were converted to ESAs using the equation $EF = (L_i/SL)^{4.5}$.

Where, EF = no. of equivalent standard axles, $L_i$ = axle load in tonnes, i = 1 to n, n = no. of axles

SL = Standard single axle load of 8.16 tonnes

The EF of each axle of each vehicle was first evaluated and then, the EFs for a vehicle were summed over all axles. The average EF values were also computed for the various vehicle classes to obtain the EF factor, for each vehicle class.

A total of 333 and 873vehicles were weighed during the survey at Akumadan and Sogakope respectively. Table 5.16 & 5.17 show the various classes of vehicles and their Average EF/vehicle for both sites.

Table 5.16 – Average Equivalent Factors for the various Classes of Vehicles (Akumadan Census Point)

| **Class** | **Vehicle** | **No. Of axles** | **Total weighed** | **(%)** | **Avg. EF Per vehicle class** |
|---|---|---|---|---|---|
| 1 | Cars/Taxis | 2 | 47 | 13 | **0.000031** |
| 2 | Vans, Pick-ups | 2 | 13 | 4 | **0.000012** |
| 3 | Small Buses | 2 | 50 | 14 | **0.00028** |
| 4 | Mammy Wagon/Medium Buses | 2 | 5 | 1 | **0.054** |
| 5 | Large Buses | 2 | 23 | 6 | **2.583** |
| 6 | Light Trucks | 2 | 46 | 13 | **1.296** |
| 7 | Medium Truck | 2 | 8 | 2 | **3.490** |
| 8 | Heavy Truck | 3 | 40 | 11 | **16.349** |
| 9 | Semi-Trailers (Light) | 4 | 51 | 6 | **4.1288** |
| 10 | Semi-Trailers (Heavy) | 4 | 14 | 4 | **4.129** |
| 11 | Truck-Trailers | 5 | 75 | 21 | **3.046** |
| 12 | Large Truck and Others | 6 | 41 | 11 | **4.425** |

Table 5.17 – Average Equivalent Factors for the various Classes of Vehicles (Sogakope Census Point)

| Class | Vehicle | No. Of axles | Total weighed | (%) | Ave. EF Per vehicle class |
|---|---|---|---|---|---|
| 1 | Cars/Taxis | 2 | 47 | 5 | 0.000014 |
| 2 | Vans, Pick-ups | 2 | 13 | 1 | 0.000015 |
| 3 | Small Buses | 2 | 50 | 6 | 0.00032 |
| 4 | Mammy Wagon/Medium Buses | 2 | | | 0.0540976 |
| 5 | Large Buses | 2 | 7 | 1 | 2.5832 |
| 6 | Light Trucks | 2 | 1 | 0 | 1.2956 |
| 7 | Medium Truck | 2 | 51 | 6 | 3.4897 |
| 8 | Heavy Truck | 3 | 81 | 9 | 16.3493 |
| 9 | Semi-Trailers (Light) | 4 | 60 | 7 | 5.6399 |
| 10 | Semi-Trailers (Heavy) | 4 | 51 | 6 | 4.1288 |
| 11 | Truck-Trailers | 5 | 216 | 25 | 3.0457 |
| 12 | Large Truck and Others | 6 | 304 | 35 | 4.4250 |

## 5.16 Summary

1. Each pavement layer attains its maximum and minimum temperature at different times of the day and at different degrees.
2. The temperature levels reduce from the HMA layers to the Subbase.
3. Any period of the day, the temperature of the pavement layers are higher than the ambient.
4. A strong relationship was established for the ambient temperature and the wearing course or HMA layers.

5. Finally, clear trends were established when the pavement temperature and moisture regime in the granular layers of the pavement monitored at the test sections were compared with rainfall, sunshine duration and evapotranspiration data obtained from the Meteorological Services Authority.
6. The temperature levels of all the pavement layers were found to be high during the wet periods as a result of high sunshine durations experienced during these periods.
7. Moisture content levels of the granular layers are generally low during periods of high rainfall as a result of the high levels of evapotranspiration and temperature during these periods which leads to high loss of moisture.
8. During periods of low rainfall, temperature and evapotranspiration are equally low leading to less loss of moisture. The effect of this is increase in seepage of moisture into the lower layers of the pavement.
9. The moduli of the layers are low at high temperatures and explain why high deflections were also observed during these periods.

# CHAPTER 6

# ENGINEERING PROPERTIES AND DEVELOPMENT OF TECHNICAL SPECIFICATIONS FOR LATERITE SOILS IN GHANA

## 6.1 Introduction

Townsend et al. (1976) working on laterites in different tropical and sub-tropical climatic regions concluded, that the common terminology used to describe laterites, as weathered soils rich in secondary oxides of iron and aluminium, possibly containing quartz and kaolinite, and with the property of hardening, was found to be inappropriate. The evaluation and classification should primarily be based on local climatic and environmental conditions. It was also established that classification of laterite for engineering use must take into account factors such as the geological history, morphological characteristics, genesis, clay mineral type, nature of ion exchanges and the actual moisture condition; since these factors vary significantly from one climatic region to the other and have immense influence on the engineering properties of local soil formation. In this Research, an attempt is made to review factors which influence engineering properties of laterite soils in Ghana so that specification for road design and construction can be developed, taking into consideration regional climatic and environmental conditions.

## 6.2 Engineering Properties of Laterite Soils in Ghana.

Engineering characteristics of laterite gravels exhibit considerable variations. A significant proportion of laterite soils in Ghana are known to be mechanically weak and tend to break down as a result of weathering and repeated loading due to traffic. This characteristic applies to laterites that are not fully matured. According to Gidigasu et al (1980), with the exception of a few locations (parts of Central Region, Afram Plains and Northern and Upper Regions) in Ghana of good quality quartzitic gravels that meet the existing specification for imported base and subbase material. The rest of the country is covered mainly by sub-standard lateritic gravels when judged by these specifications. Considerable cost savings can be made if these materials, some regarded as marginal, can be used.

### 6.2.1 Textural Classification

The term „soil texture" relates to the relative percentages of sand, silt, and clay in a soil. The feel of the soil gives some indication of these percentages (Ahn, 1970). Grain size or texture may also be related to the nature of the parent material, (Alexander and Cady, 1962) and other weathering factors.

Textural classification of residual laterite soils is important as it is the fundamental basis for a preliminary grouping of laterite soils. De-Graft Johnson (1972) established that Ghana"s laterite soils are formed over granite and gneiss. Based on the content of the gravel, sand and fine (silt/clay) fractions, Gidigasu (1972) grouped laterite gravels and gravelly soils in Ghana into six textural classes which appear to have similar geotechnical characteristics (see Table 6.1).

Table 6.1 - Proposed Textural Chart for the Classification of Laterite Gravels and Gravelly Soils (after Gidigasu, 1972)

| Classification | Gravel % | Sand % | Silt /Clay % | Remarks |
|---|---|---|---|---|
| Gravel | 50-100 | 0-50 | 0-20 | - |
| Loamy Gravel | >50 | 0-30 | 20-40 | Silt < Clay < 30% |
| Gravelly Sand | 10-50 | 30-90 | 0-20 | Sandy Silt + Clay |
| Gravelly Loam | 10-50 | 10-50 | 20-40 | Clay < 30% |
| Gravelly Sandy Loam | 10-30 | 50-70 | 20-40 | Silt < Clay < 30% |
| Gravelly Clay | 10-50 | 0-50 | 40-90 | Clay = 20 - 40% |

For the Wooded Savannah, the texture is that of silty or sandy loam, if the soils are developed over the Voltaian shales or coarse sandy-loam or if they are developed over the granites. Soils from the rainforest, or the moist semi-deciduous forest, are porous, well-drained and generally loamy.

The texture alone will not be enough to describe accurately and determine the proportion of the various sizes for engineering use. Further analysis is needed to determine or ascertain the particle size distribution.

In order to determine the engineering properties of any given material, Fossbery (1963) in his presentation on "Gravel Roads – The Performance and Testing of Materials," listed the following material properties as the most relevant to ensure satisfactory performance of natural gravels in road pavements:

- Particle Size Distribution;
- Plasticity;
- Strength of Coarse Particles; and
- Compaction and Bearing Capacity.

### 6.2.2 Particle Size Distribution

The purpose of particle size analysis is to determine the proportional volume occupied by the particles of different sizes (texture). Almost all the other engineering properties of the soils are dependent on the particle size distribution of the soil. For every engineering use of soil, proportions of the various particle sizes to serve a particular purpose or perform a particular function are required. Liquid limits, plastic limits, plasticity index and swelling, depend on the amount of fines while shear strength, specific gravity and other mechanical properties of the soils depend on the coarser fraction of the soil. It is important to establish the nature of the fines. Clay fraction (size < 0.002mm) will have different behaviour from rock dust of the same size.

Distribution of particles in their natural soil matrix can be described as; well graded, uniformly graded, poorly graded and others, depending on the type of classification system adopted. Well graded soils are always preferred in road works as maximum densities can be achieved during compaction. This results in both increased density and strength and generally leading to reduction in permeability and volume change (i.e. volume stability).

Most temperate zone material specifications, including the AASHTO Specification, adopted their grading limits from the approximation of the formula proposed by Fuller and Thompson (1907):

$$\% \text{ Passing Any Sieve, (P)} = 100\sqrt{\frac{\text{Apperture size (d)}}{\text{Size of largest Particle (D)}}}$$

In a review by Gidigasu (1975), a critical evaluation of the formula in a more generalised form; i.e., $P = 100\ (\frac{d}{D})^n$ in relation to performance of gravel roads in tropical Ghana. Gidigasu concluded that in the case of Ghana;

- Where n = 0.25, the fines content is excessive and the soil material lacks stability, particularly in wet weather; and
- Where n = 0.5, the soil material is stony and porous and usually requires additional soil binder to ensure satisfactory behaviour.

In terms of use of laterite materials in road pavement, Charman (1988) also highlighted some anomalies with respect to specific gravity and proposed how to address them when evaluating laterite materials. These are listed as follows:

- The general assumption that the specific gravity of the particles is constant over the range of sizes is valid for most soils. But for soils like laterites, as there can be large differences between the specific gravities of the coarse and fine fractions, a correction to the grading curves is necessary by using grading based on volume proportions to modify the conventional grading based on mass proportions;

- The handling process during testing and construction can affect the specific gravity and may be corrected by ensuring that the coarse fractions do not fracture and the fine particles are removed by means of dispersion from the coarse particles; and

- Calculation of the specific gravity based on spherical particles could lead to unreliable results as true clay minerals are flaky.

De-Graft Johnson et al (1969) established the envelopes of grading for the four main types of gravel extensively used in road construction in Ghana (see Table 6.2). It was based on the mode of formation and physical properties of the soils, by focussing on gravels considered less susceptible to weathering and mechanical degradation.

Bhatia and Yeboah (1970) carried out a follow-up study to link the proposed grouping to geology, rainfall, topography and drainage. The following key observations were made with respect to the influence of geology and climate on texture and grading of laterite soils:

- Although there were variations of different fractions of the total samples studied, some similarities exist in the grain size distribution of samples, from similar geological and climatic areas; and

- Using geology and climate as guidelines, all lateritic gravels of Ghana fall into four distinct grading envelopes.

Therefore, a slight modifications were made to the original grading envelopes developed by de-Graft Johnson et al. (1969).

Table 6.2 – Groups of Laterites in Ghana (after de-Graft Johnson et al., 1969)

| **Type of Material** | | **Group 1** | **Group 2** | **Group 3** | **Group 4** |
|---|---|---|---|---|---|
| | | **Nodular or Concretionary Laterite** | **Iron Stone Hard Pans or Cap Rock** | **Group Water Laterite with Detrital Quartz** | **Colluvial and Terrace Laterites** |
| **GRADING ENVELOPES** | **Size (mm)** | | | | |
| | 0.075 | 2 – 14 | 2 – 10 | 4 – 16 | 23 – 36 |
| | 0.425 | 6 – 26 | 6 – 16 | 16 – 28 | 24 – 40 |
| | 2 | 10 – 45 | 10 – 35 | 24 – 44 | 28 – 48 |
| | 5 | 16 – 56 | 30 – 72 | 36 – 60 | 50 – 76 |
| | 10 | 20 – 72 | 52 – 93 | 60 – 94 | 70 – 92 |
| | 20 | 44 – 84 | 100 | 84 – 100 | 84 – 100 |
| | 25.4 | 58 – 100 | | 100 | 92 |
| | 37.5 | 80 | | | 100 |
| | 75 | 100 | | | |

Particle size distribution envelopes for laterites from the various climatic zones, geological formations and types of parent rocks in Ghana are summarised in Table 6.3 (see Gidigasu 1970, 1971 and 1980).

Table 6.3, Grading envelopes of laterites from various parent rocks in Ghana (Gidigasu, 1970, 1971 & 1980)

| **Climatic Zone** | | Coastal Savannah | Forest Zone | Forest Zone | Woodland Savannah | Woodland Savannah | Forest Woodland Savannah | Woodland Savannah | Coastal Savannah |
|---|---|---|---|---|---|---|---|---|---|
| **Geological Formation** | | Dahomeyan | Granites | Lower Birrimian Tarkwaian | Voltaian | Voltaian | Granites | Voltaian | Tarkwaian Togo Series |
| **Type of Material** | | Coastal Savannah Gneiss | Forest Granites | Forest Phyllites | Woodland Savannah Sandstone | Voltaian Sandstone | Decomposed Granite And Pegmatite | Quartz Drift Gravels | Weathered Quartzite |
| **GRADING ENVELOPES** | **Size (mm)** | | | | | | | | |
| | **0.075** | 26 – 52 | 17 – 53 | 46 – 80 | 20 – 66 | 16 – 60 | 0 – 20 | 2 – 18 | 9 – 19 |
| | **0.425** | 54 – 92 | 38 – 90 | 60 – 100 | 60 – 100 | 78 – 94 | 2 – 30 | 6 – 24 | 16 – 32 |
| | **2** | 80 – 100 | 75 – 100 | 78 | 84 | 96 – 100 | 12 – 54 | 14 – 44 | 32 – 50 |
| | **5** | 92 | 92 | 96 | 96 | 100 | 28 – 72 | 33 – 60 | 44 – 64 |
| | **10** | 100 | 100 | 100 | 100 | | 52 – 85 | 52 – 82 | 46 – 72 |
| | **20** | | | | | | 72 – 100 | 72 – 95 | 64 – 88 |
| | **25.4** | | | | | | 74 | 76 – 96 | 68 – 90 |
| | **37.5** | | | | | | 79 | 86 – 98 | 76 – 100 |
| | **75** | | | | | | 100 | 100 | 100 |

A key observation made by Bhatia and Yeboah (1970) that all Ghanaian laterite gravel soils, from climate and geology perspective, fall in one of the 4 groups proposed by De-Graft Johnson (1969). Using this observation, plots made established that the following natural gravel materials are not good for road construction due to their poorly graded nature unless they undergo improvements through stabilization;

- Forest Zone Granite;
- Forest Phyllites;
- Woodland Savannah Sandstone;
- Voltaian Sandstone; and
- Coastal Savannah Gneiss.

Figures 6.1 to 6.4 show the positions of the grading curves of these rejected gravels on the groups (1-4) grading band.

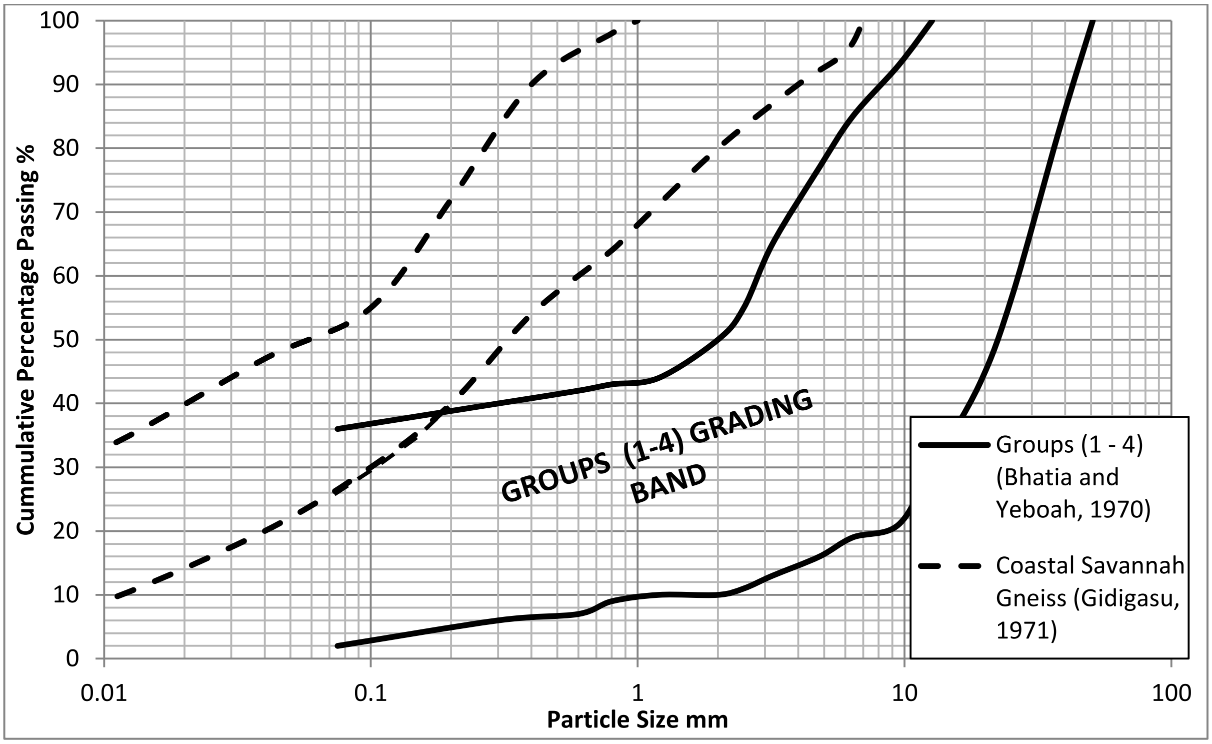



Figure 6.1 – Position of Coastal Savannah Gneiss Grading on the Groups (1-4) Grading Band

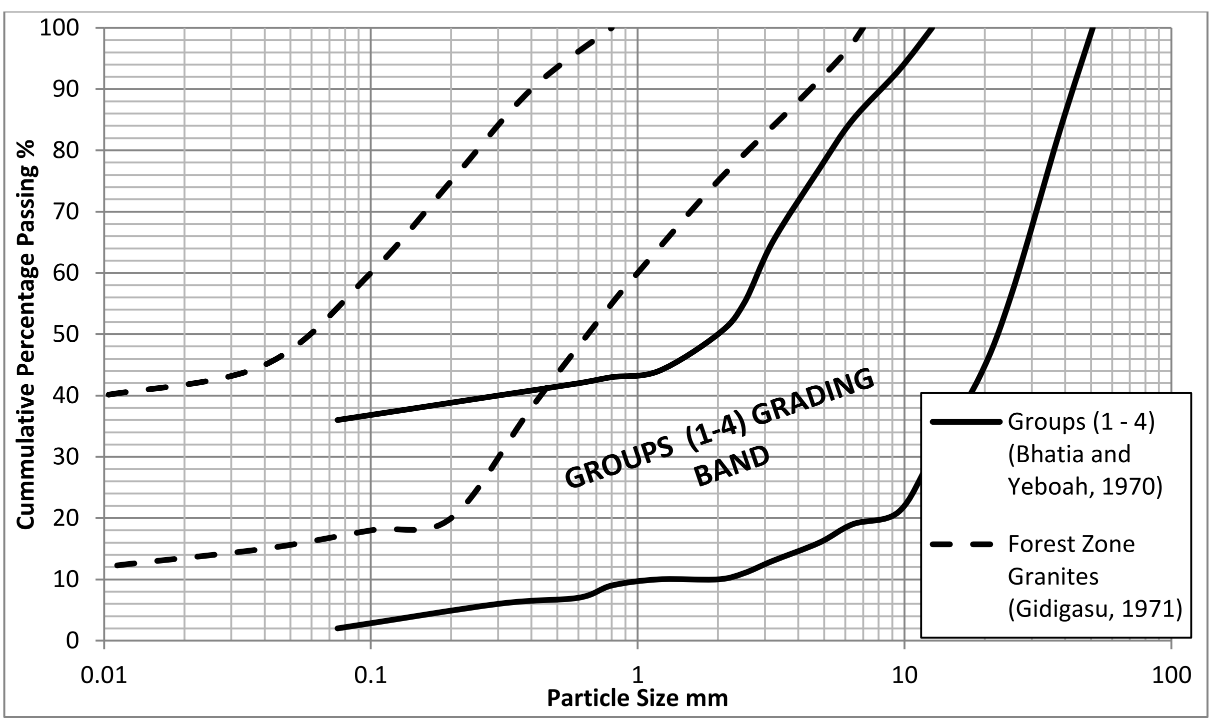



Figure 6.2 – Position of Forest Zone Granites Grading on the Groups (1-4) Grading Band

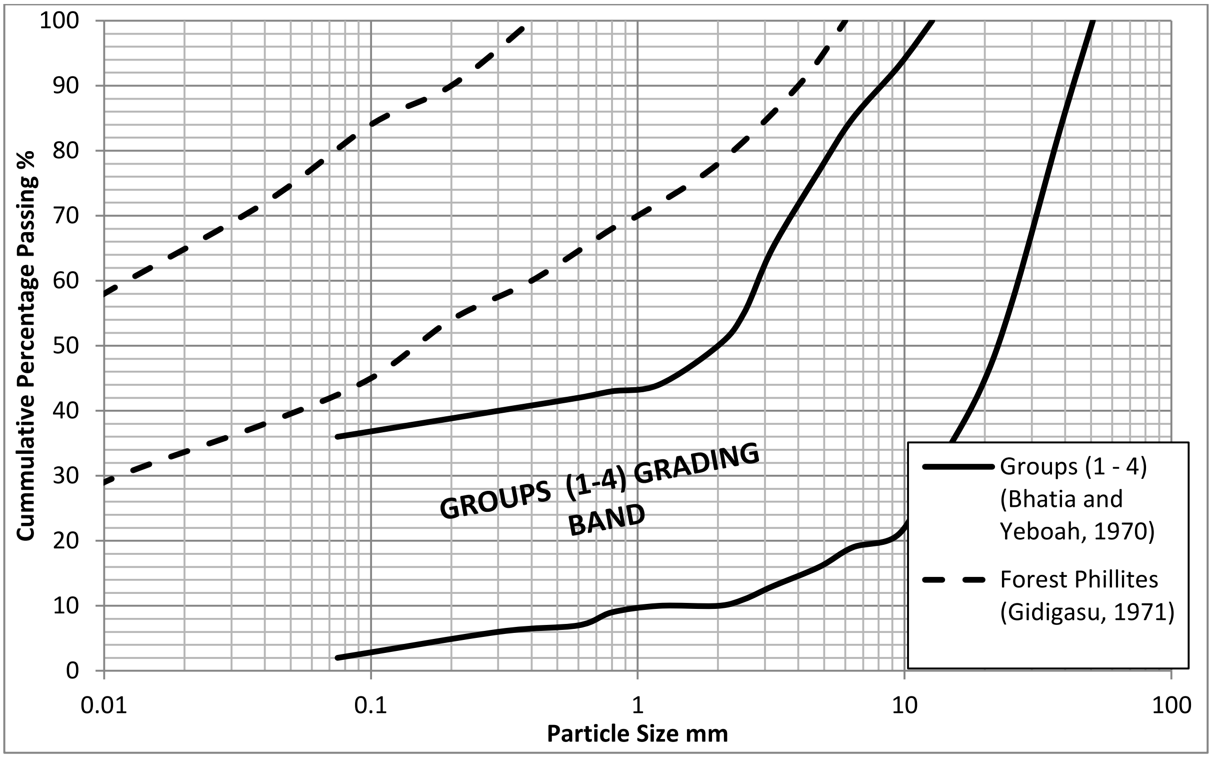



Figure 6.3 – Position of Forest Phillites Grading on the Groups (1-4) Grading Band

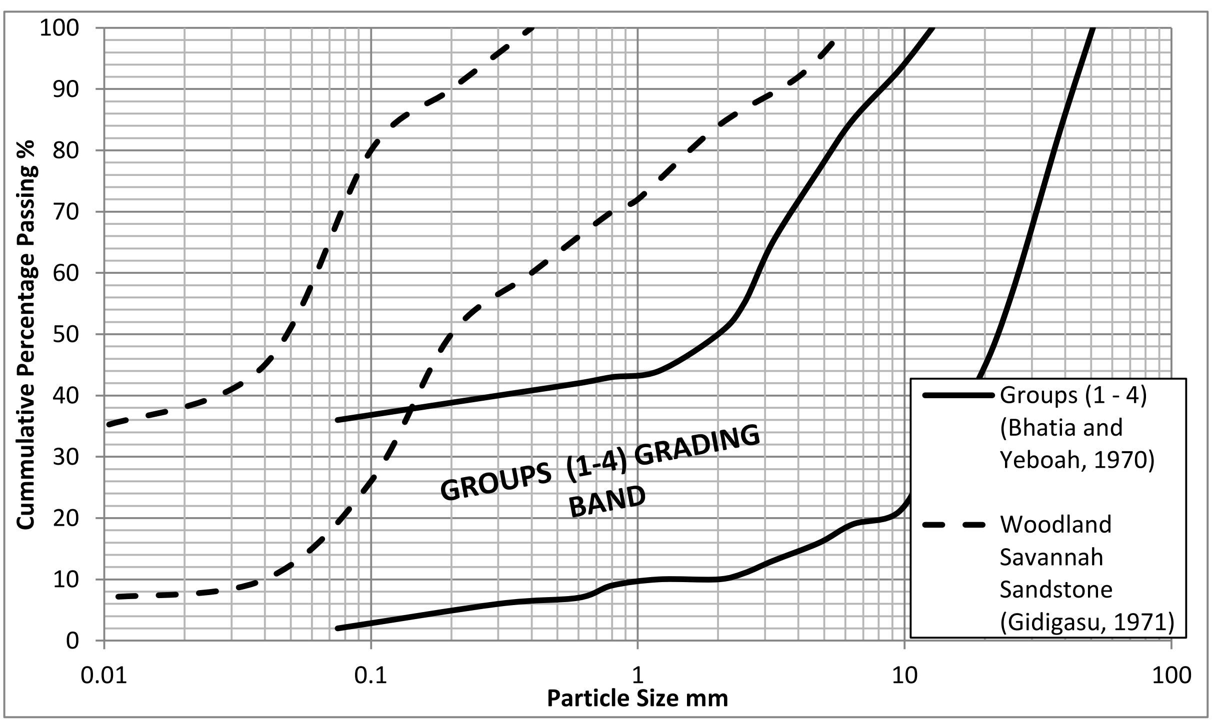



Figure 6.4 – Position of Woodland Savannah Sandstone Grading on the Groups (1-4) Grading Band

Their unsuitability for use as road construction material can be explained from how the constituent minerals forming these materials are easily weathered as compared to the suitable materials which are described as quartzitic. Quartzite is extremely resistant to chemical weathering hence their high degree of stability. In the case of Coastal Savannah Gneiss and Forest Phyllites, these are basically metamorphic rocks which are formed due to changes in sedimentary rock or metamorphosis of pre-existing igneous rocks. Very characteristic of metamorphic rock is an orientation of the constituents to give a band effect. Ahn (1970) established that the very fine forms of the rock are phyllites and the coarse gneisses (which are roughly band). Sandstones are forms of sedimentary rocks. Those without quartz sand will break down on weathering to the original sand giving rise to a very poor sandy soil. If the sandstone contains some feldspar or sand other than quartz sand, then weathering may result in the formation of some clay and the soil will be less light-textured. Ahn (1970)

This means that only Granites, Voltaian Quartz and Drift Gravels and Weathered Quartzites from Forest Woodland Savannah, Woodland Savannah and Coastal Savannah regions respectively, as shown in Table 6.3 are suitable for road construction (see Figures 6.5 to 6.7). Other grading plots have been provided in Appendix E.

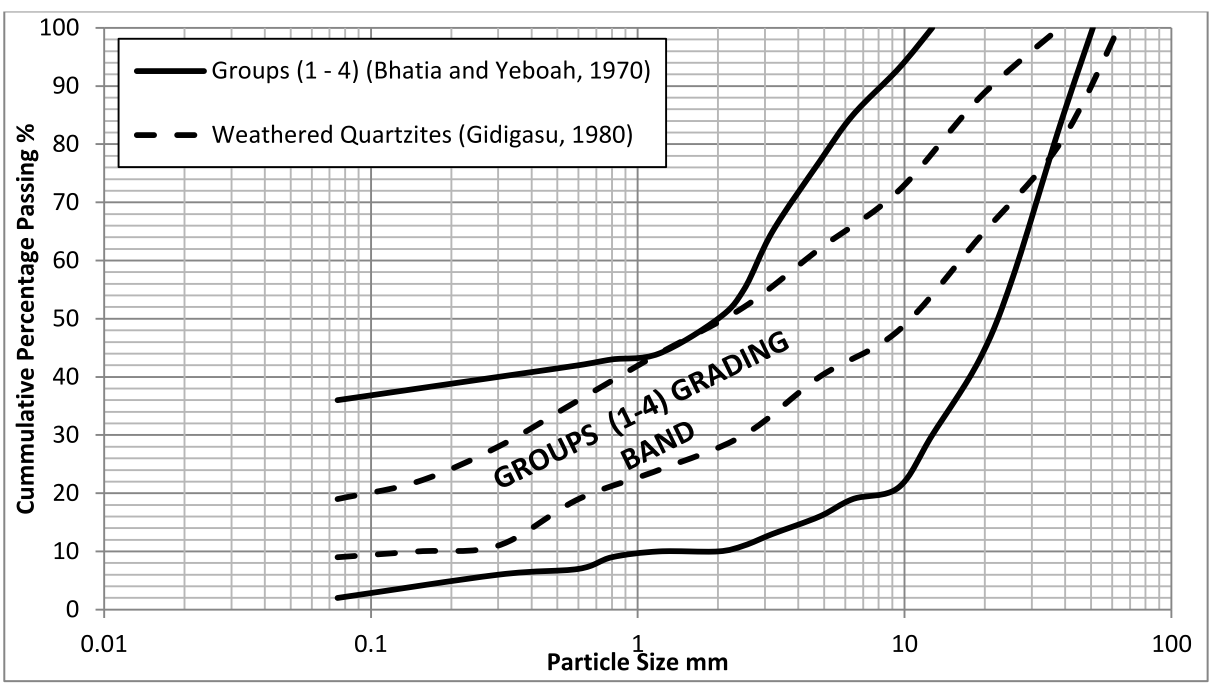



Figure 6.5 – Position of Weathered Quartzites grading on the Groups (1-4) Grading Band

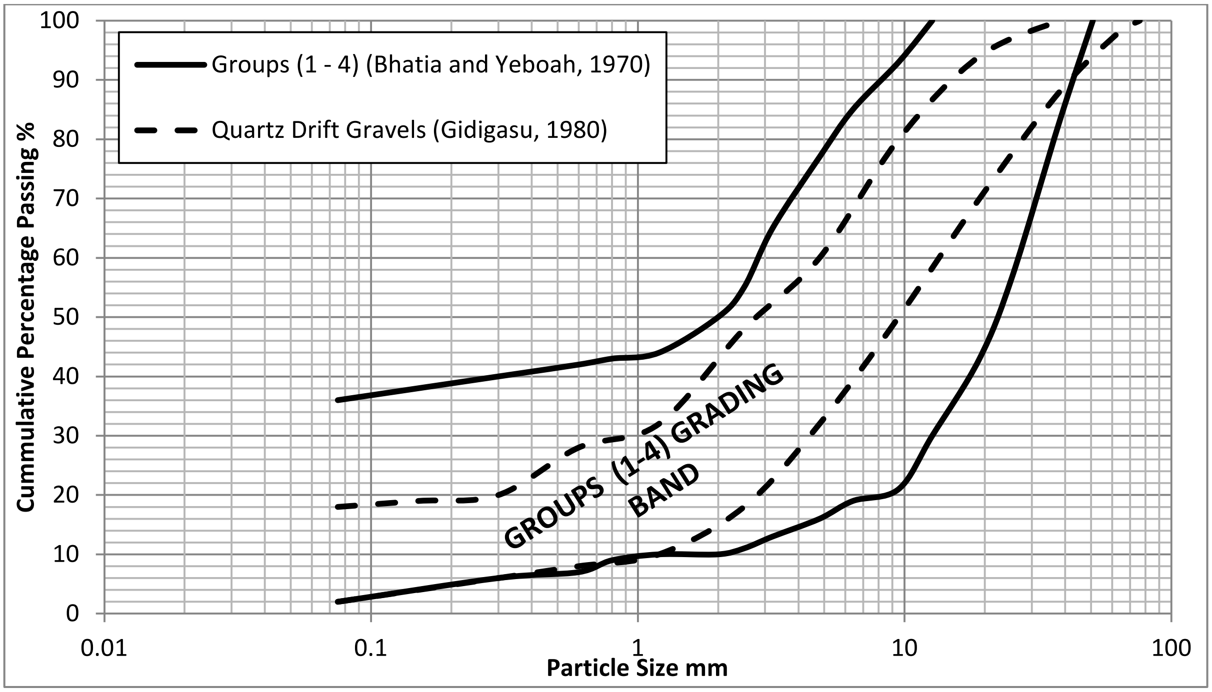



Figure 6.6 – Position of Quartz Drift Gravels grading on the Groups (1-4) Grading Band

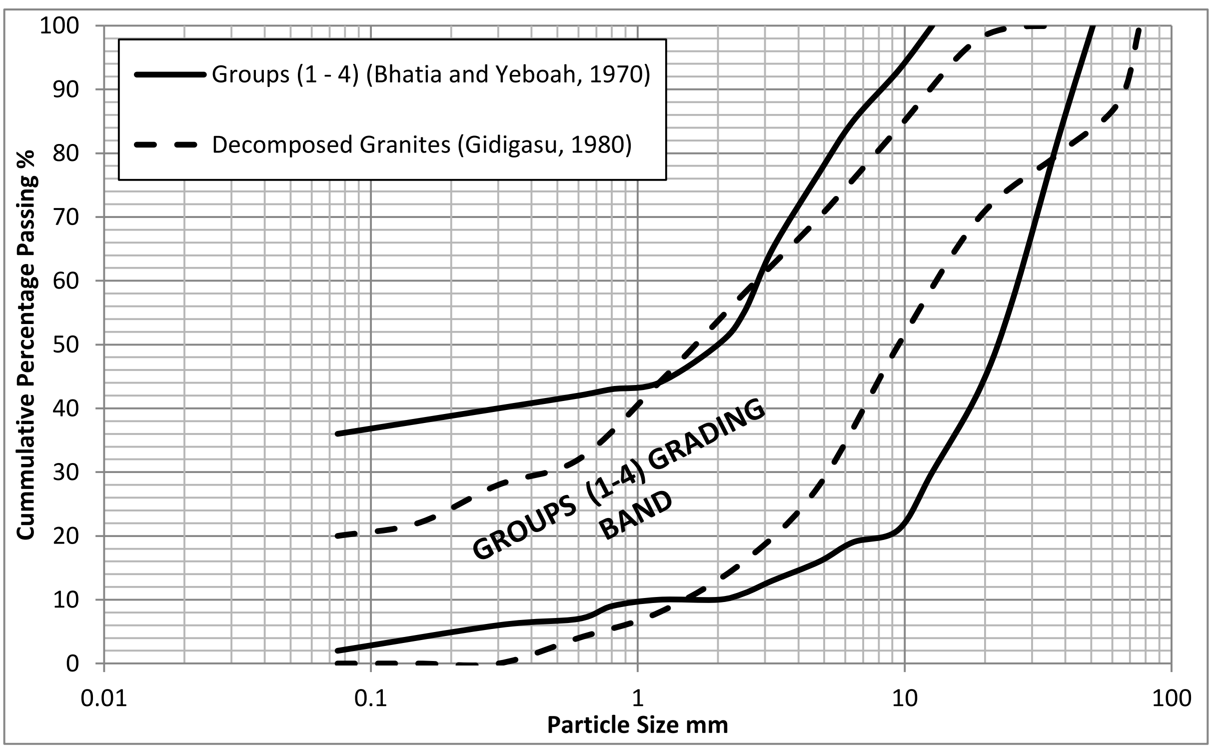



Figure 6.7 – Position of Decomposed Granites grading on the Groups (1-4) grading band

With respect to their use as pavement layer material, the work by Gidigasu (1972), on geotechnical characteristics of laterite materials in Ghana was used to establish their suitability as Sub base, Base or Surface Course. Table 6.4 gives the grading derived by Gidigasu for pavement layers.

Table 6.4 - Grading Specifications and Criterion for Selecting Quartzitic Laterite Gravelly Soils for Road Construction (Gidigasu, 1972)

| Type of Material | | Subbase | Base 1 | Base II | Surface Course |
|---|---|---|---|---|---|
| GRADING ENVELOPES | Size (mm) | | | | |
| | 0.075 | 24 – 42 | 0 – 15 | 16 – 26 | 16 – 30 |
| | 0.425 | 32 – 46 | 3 – 28 | 18 – 32 | 18 – 42 |
| | 2 | 36 – 56 | 7 – 36 | 20 – 37 | 26 – 52 |
| | 5 | 56 – 76 | 20 – 76 | 36 – 60 | 60 – 86 |
| | 10 | 72 – 90 | 32 – 91 | 48 – 76 | 80 – 100 |
| | 20 | 86 – 100 | 52 – 100 | 70 – 100 | 100 |
| | 25.4 | 91 | 62 | 78 | |
| | 37.5 | 100 | 80 | 90 | |
| | 75 | | 100 | 100 | |

Based on the grading specification above, the recommended uses of the various gravel materials were derived by superimposing the material grading, using geology and climate, on the grading for the various pavement layers. And as shown in Figures 6.8 to 6.10, Table 6.5 gives the recommended use of the accepted gravels based on the grading specifications proposed by Gidigasu (1972) in Table 6.4.

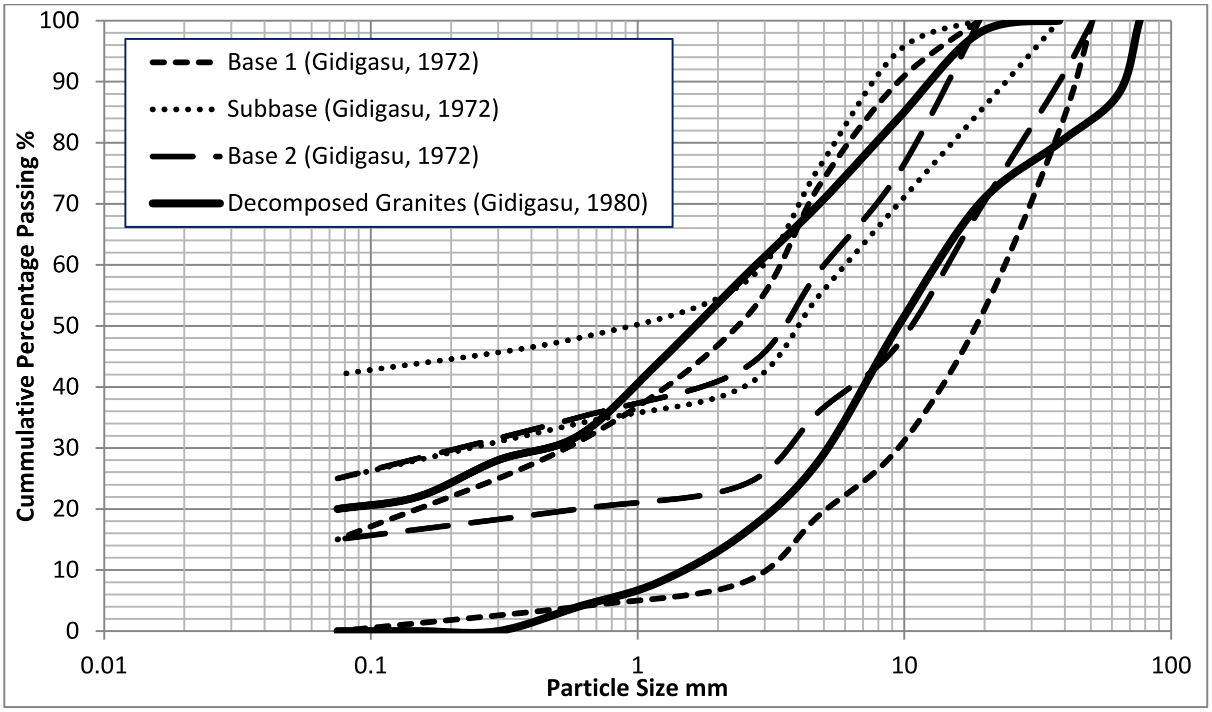



Figure 6.8 – Decomposed Granites Grading and the Recommended Grading Specifications

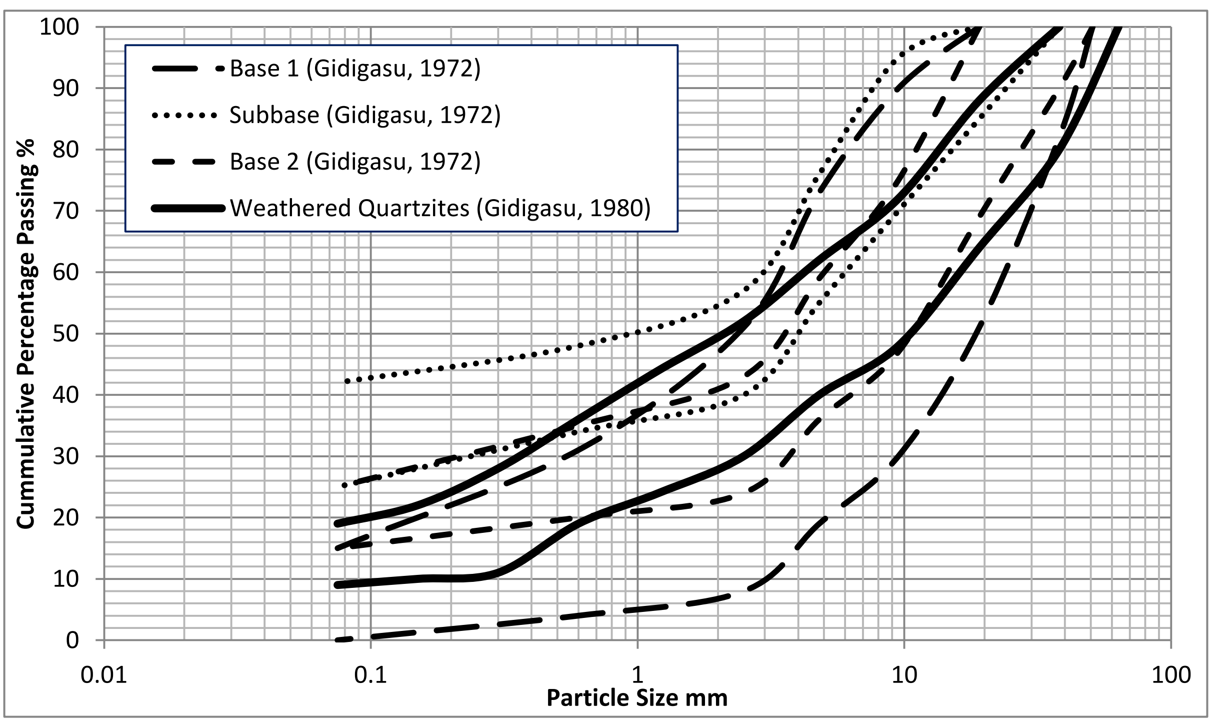



Figure 6.9 – Weathered Quartzites Grading and the Recommended Grading Specifications

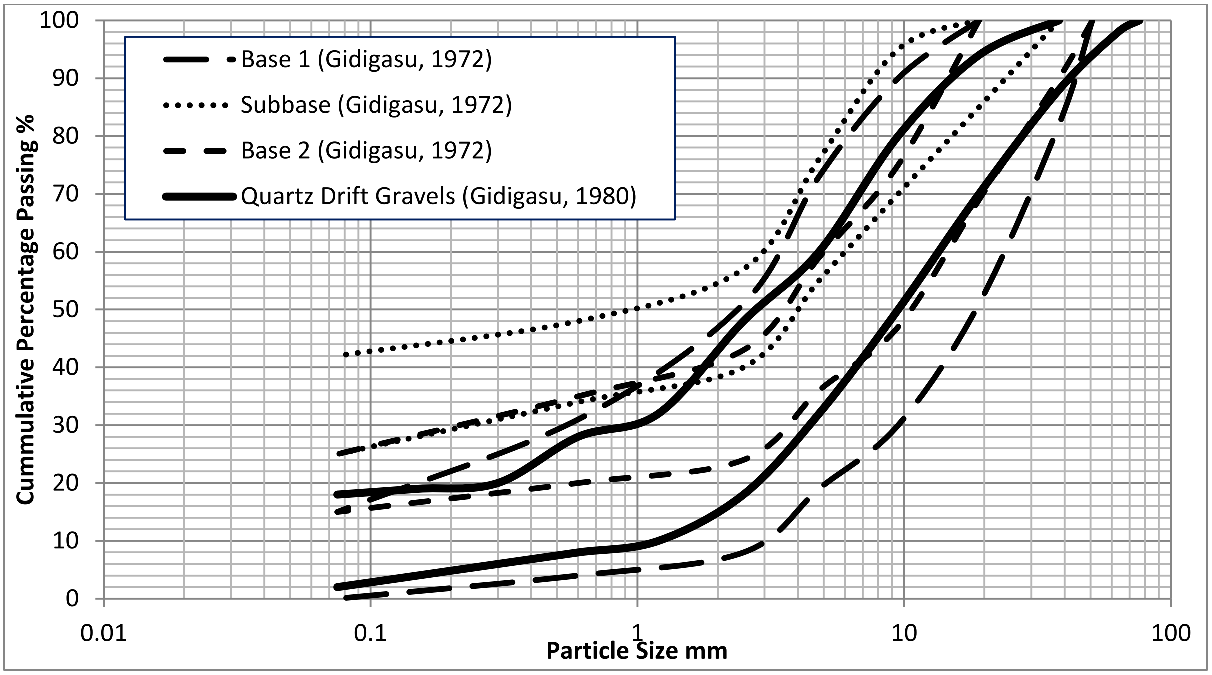



Figure 6.10 – Quartz Drift Gravels Grading and the Recommended Grading Specifications

Table 6.5 - Recommended Use of Material for Pavement Layers

| **Material Type** | **Recommended Use** | | |
|---|---|---|---|
| | **Sub Base** | **Base I** | **Base II** |
| Decomposed Granite and Pegmatite | X | X | X |
| Quartz Drift Gravel | X | X | X |
| Weathered Quartzite | X | | X |

The grading requirements of natural aggregates used as sub-bases and bases (See Table 6.6) have been specified by the Standard Specification for Road and Bridge Works for Ghana (2007). Requirements of each material class are based on their typical use, thus;

G80 – Base course

G60 – Base course for low traffic roads

G40 – Base course for sealed rural access roads and subbase

G30 - Subbase

Table 6.6 - Requirements for Natural Gravel Materials for Base and Sub-base (Standard Specification for Road and Bridge Works, 2007)

| **Material properties** | **Material Class** | | | |
|---|---|---|---|---|
| | **G80** | **G60** | **G40** | **G30** |
| Grading | | | | |
| % Passing Sieve Size (mm) | | | | |
| 75 | 100 | 100 | | |
| 37.5 | 80 - 100 | 80 - 100 | | |
| 20 | 60 - 85 | 75 - 100 | | |
| 10 | 45 - 70 | 45 - 90 | | |
| 5.0 | 30 - 55 | 30 - 75 | | |
| 2.0 | 20 - 45 | 20 - 50 | | |
| 0.425 | 8 - 26 | 8 - 33 | | |
| 0.075 | 5 – 15 | 5 – 22 | | |
| Grading Modulus (min) | 2.15 | 1.95 | 1.5 | 1.25 |
| Maximum size (mm) | 53.0 | 63.0 | 75.0 | $2/3^{rd}$ layer thickness |
| Grading Modulus (GM) = 300 – (%passing 2.0 + 0.425 + 0.075mm sieves) x 100 | | | | |

Comparing the recommended uses of the accepted materials as shown in Table 6.5 with the current specification for Ghana, the following conclusions can be made (see Figures 6.11 to 6.16);

1. Both Base 1 and Base 2 qualify as base materials as they all fall or a significant portion of their bands fall within the G60 and G80 grading limits.

2. Critical examination of the plots will show that Base 1 corresponds more to G60 (see Figure 6.11) than Base 2 which also corresponds to G80 (see Figure 6.14).

3. The subbase lies almost completely outside the G80 band (see Figure 6.16) but a significant portion lies inside the G60 band (see Figure 6.12) signifying the standard of the material being lower than that of a base and hence its recommended use.

4. The Subbase grading band can serve as the grading standard for subbase materials which the current specification lacks for such materials.

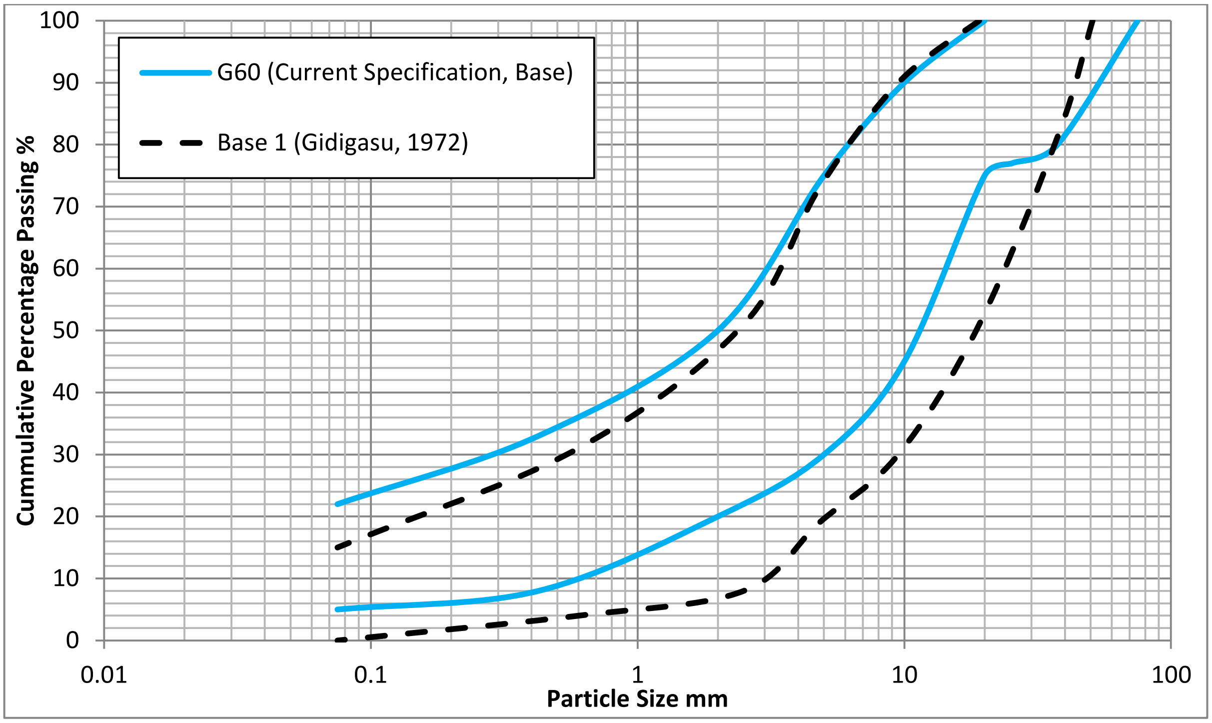



Figure 6.11 - G60 and Base 1 Grading Curves

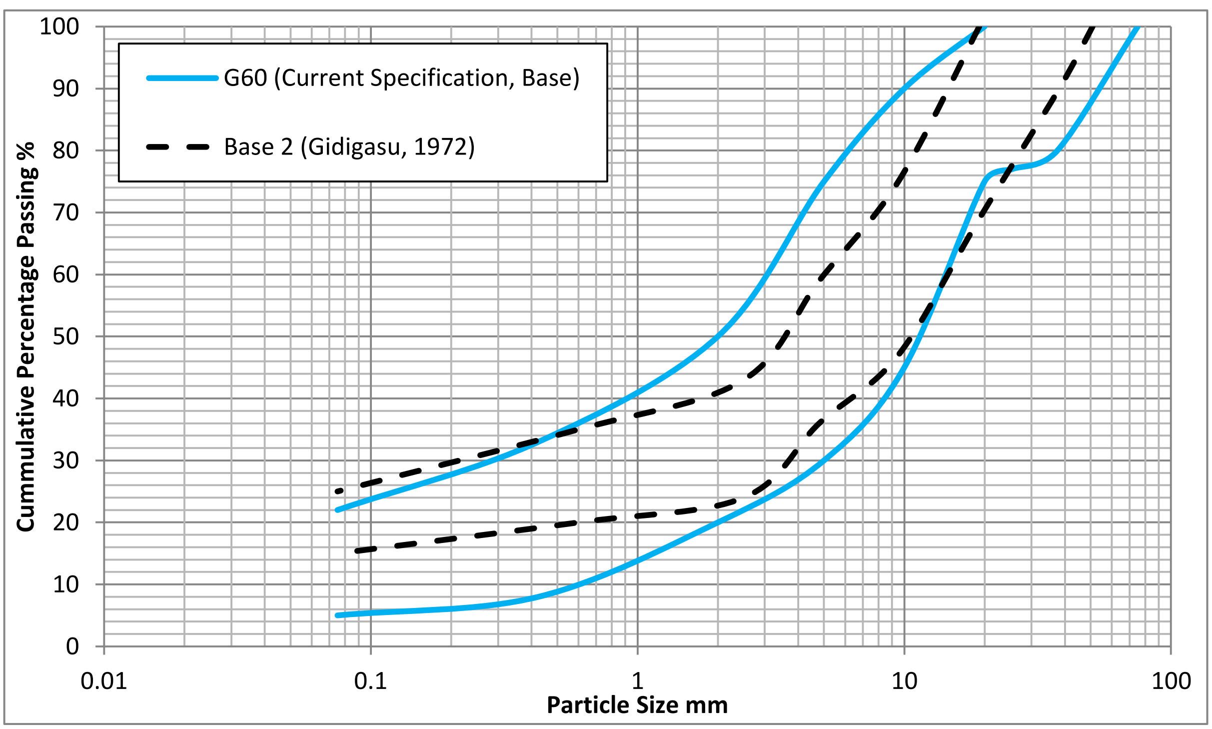



Figure 6.12 - G60 and Base 2 Grading Curves

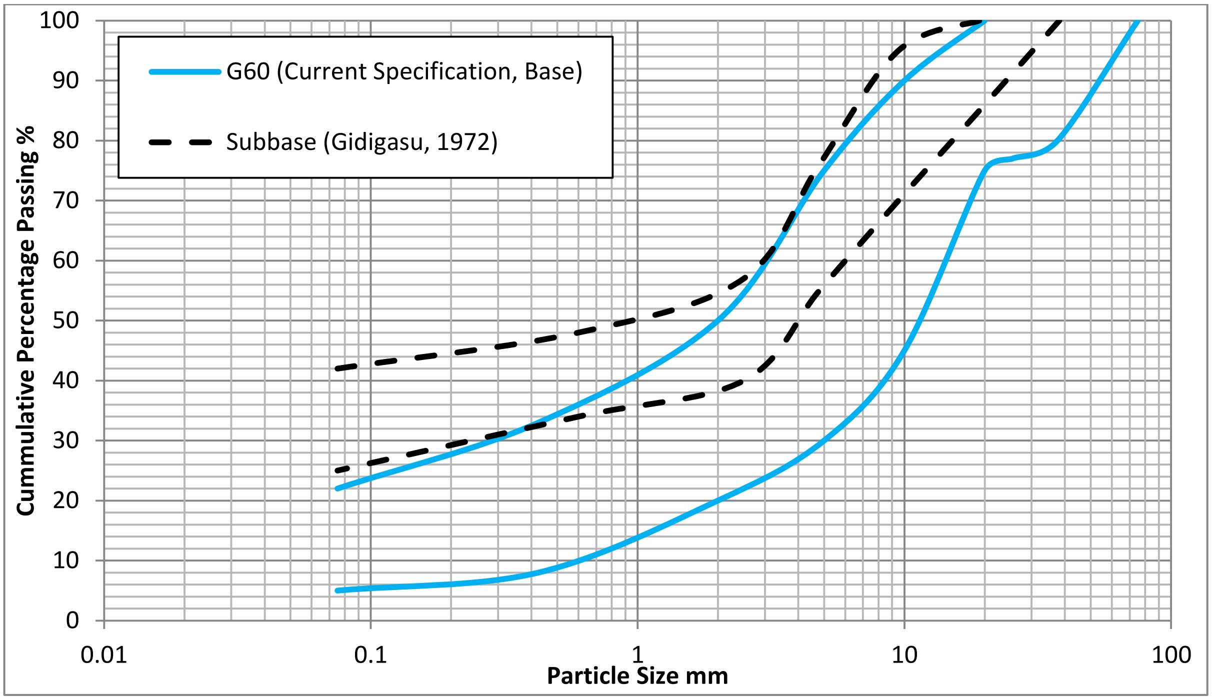



Figure 6.13 – G60 and Subbase grading curves

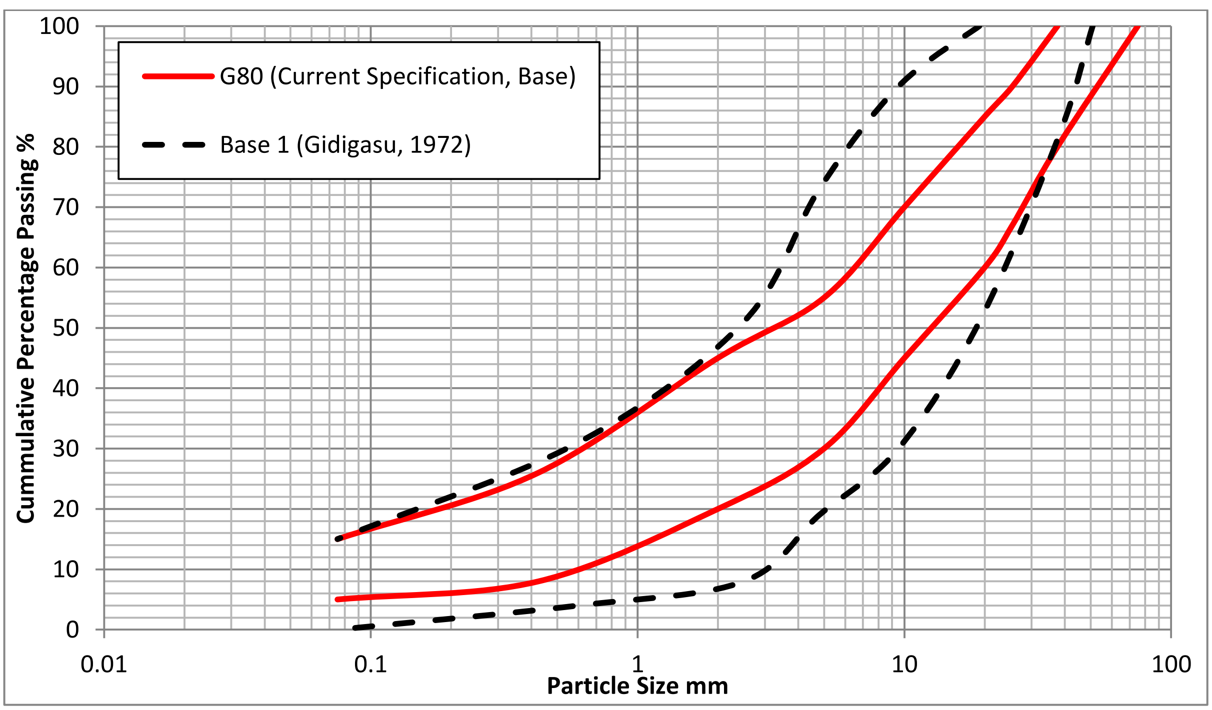



Figure 6.14 – G80 and Base 1 Grading Curves

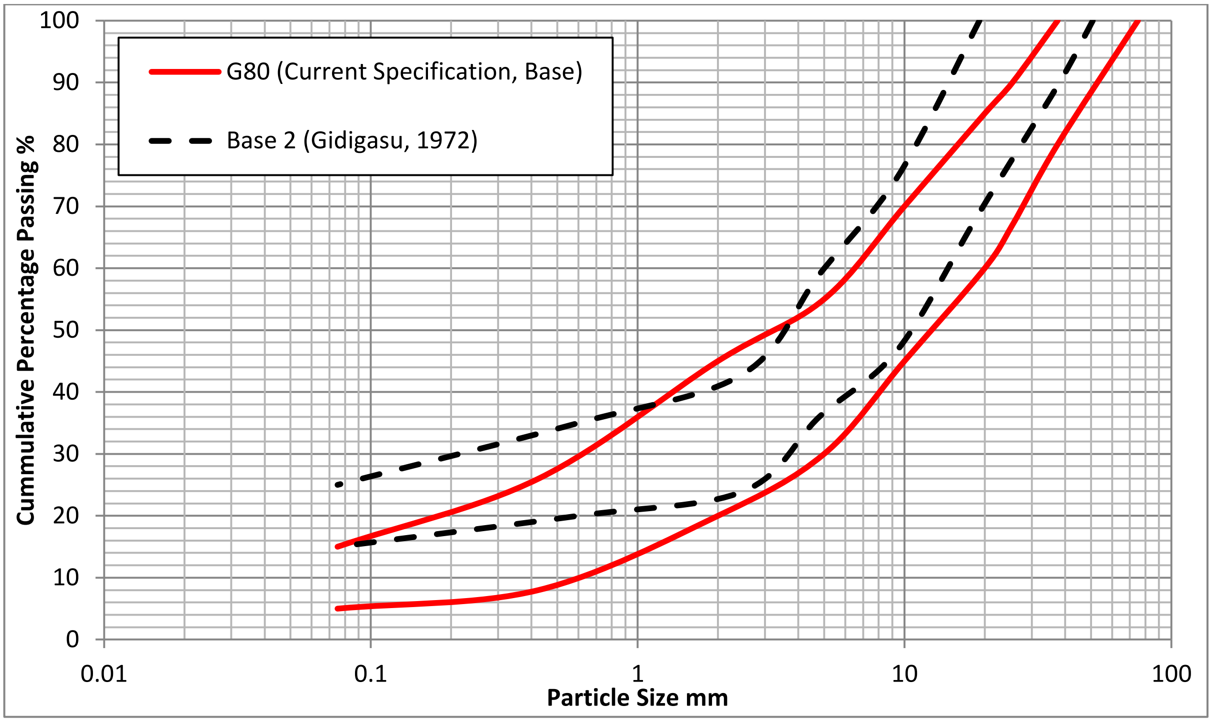



Figure 6.15 – G80 and Base 2 Grading Curves

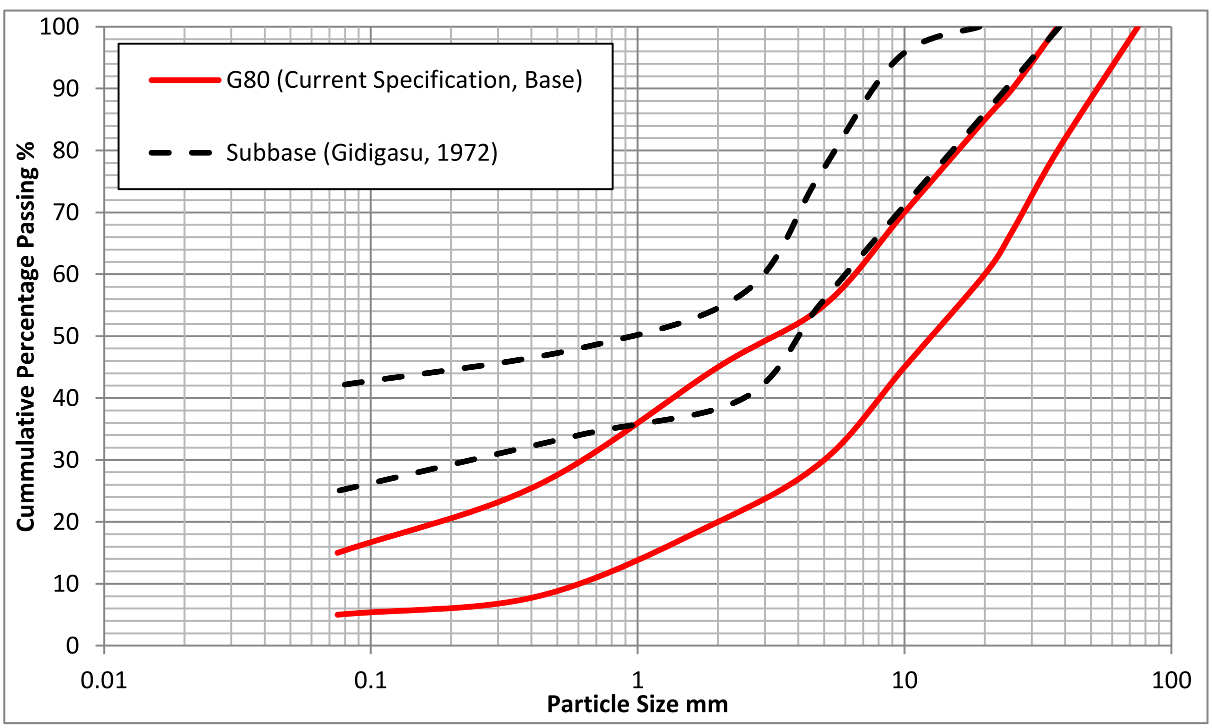
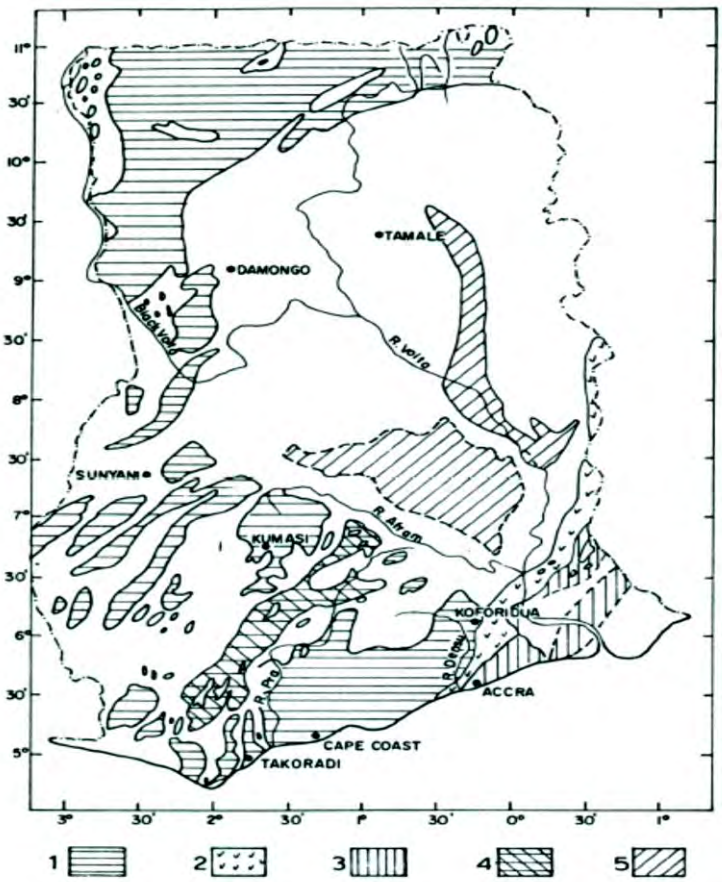



Figure 6.16 – G80 and Subbase Grading Curves

The location of the recommended gravel for road construction is shown in Figure 6.17.




*1= Detrital quartz gravels from decomposed granite pegmatite; 2= Weathered quartzite from Togo series; 3= Isolated outcrops of weathered quartzite and pegmatite in the Dahomeyan series; 4= Isolated outcrops of weathered quartzite in the Tarkwaian series; 5= Drift gravels in occasional pebble beds*

Figure 6.17 - Generalised Distribution of Quartzitic Gravels and Gravelly Soils in Ghana That Are Suitable For Use in Road Construction (Gidigasu et al., 1980)

### 6.2.2 Plasticity

The plastic characteristics of temperate region soils based on Atterberg Limits (liquid and plastic limits) are the established indicators of determining their engineering behaviour. The Atterberg Limits are important parameters for defining the effect of water content on the behaviour of fines in soils.

The plasticity characteristics of soils are governed by the type of clay minerals it contains. The clay mineral, which is by far the most common in West African soils, is kaolin Ahn (1970). Gidigasu (1971) established that the main clay mineral in the siliceous tropical soils in Ghana is kaolinite with some montmorilonite. Kaolinite clay mineral has lesser affinity for water compared to montmorilonite. Kaolin has relatively large flat crystal flakes are often more or less hexagonal consisting of a number of adjacent sheets as shown in Figure 6.18.

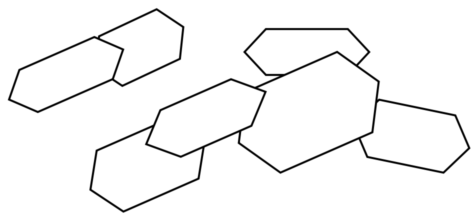
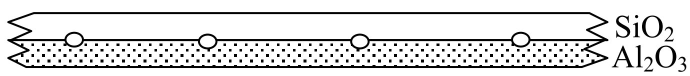
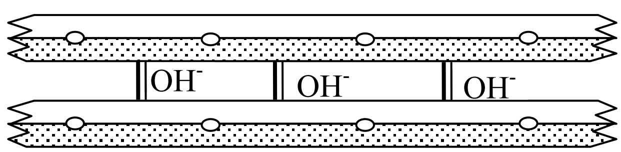


Figure 6.18 – Kaolin Flakes

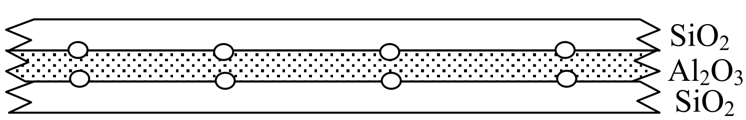
Each of the sheet consist of a double layer, one of a lattice of Silica ($SiO_2$) molecules and the other Alumina lattice ($Al_2O_3$) molecules and bonded together by shared atom as depicted below.




Each of these double layers is held to one another by a hydroxyl ($OH^-$) ion as show below.




In kaolinite, the distance between sheets is small and fairly constant. Kaolinite therefore has what is called 1:1 (one-to-one) layer lattice. The lattice describe as non-expanding. The second important group is clay minerals with 2:1 (two-to-one) layer lattice. Each crystalline unit consist of three layers (two $SiO_2$ molecules and one $Al_2O_3$ molecule).



There are two groups of clay minerals with this 2:1 layer lattice. Illite has a fixed distance between the sheets as has kaolin. The second type of 2:1 lattice clay can separate out a little to make space between them and thus form expanding lattice. Such a clay mineral is called montmorillonite as shown in Figure 6.19.

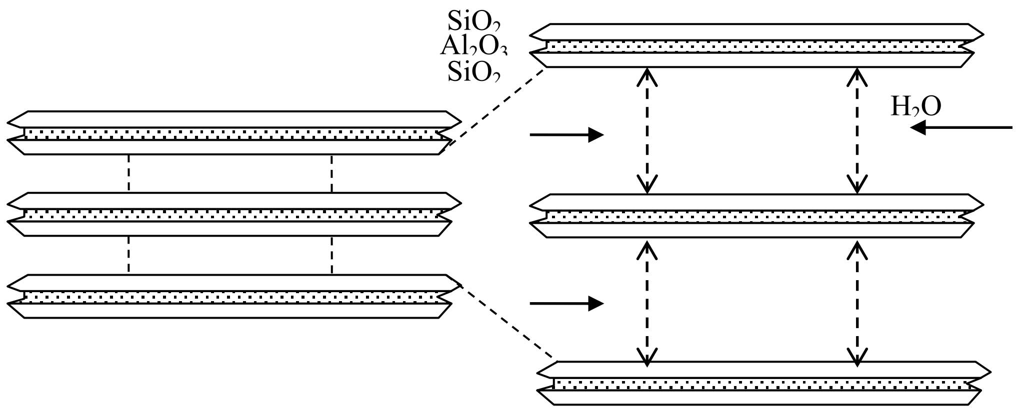



Figure 6.19 – Structure of a Montmorillonite Flake

Clays of the type of montmorillonites swell when wetted, but shrink and may form cracks when dry again. Kaolin in contrast does not expand very much on wetting. Many other types of clay exist, some with properties intermediate between kaolin and montmorillonite. In a given soil you will find more than one type of clay mineral.

In order to have a better appreciation of the clay only, the fraction finer than 425 μm is used for the Atterberg Tests, if this fraction is only small (that is, the soil contains significant amounts of sand or gravel) it might be expected that the soil would have better properties. What is of concern is the constituent in terms of percentage in the soil as well as the type of clay mineral that dominate the fraction.

Townsend et al (1976) established that the aggregation of the clay size particles is due to the electrical bonding between the negatively-charged kaolinite and positively-charged hydrated oxides (sesquioxides) in the fine fraction. This bonding explains the natural low plastic characteristics of some tropical laterite soils as a result of reduction of the ability of clay minerals to absorb water and physical cementation of adjacent particles leading to production of coarser particle sizes. Also dehydrating conditions make soils containing sesquioxide less plastic as it results in a stronger irreversible bond between particles, which makes it resistant to penetration of water. Hydrating conditions thereafter cannot reverse this bonding process. Ferruginous soils in dry tropical areas under savannah vegetation have liquid limits and plasticity indices range between 25 - 50% and 10 - 30% respectively as given in Table 6.7. Evaporation exceeds precipitation during the dry season which lasts more than eight months annually. Ferralitic and Ferrisol soils are found in the humid tropical rainforest areas, with annual duration of the dry season being less than four months. Ferralitic soils have their liquid limits and plasticity index exceeding 50% and 30%. (Bani, 1971) showed that swelling increases as the liquid limit and plasticity index increase and presented the plasticity characteristics of the D‟Hoore soil types.

Research work done to establish the plasticity characteristics of Ghanaian soils are related to the climatic zones, soil types and performance rating under bituminous roads by Gidigasu and Dogbey (1980).

Table 6.7 – Plasticity properties of soils from a range of tropical climatic zones

| **Climatic zones** | | | **Plasticity properties** | | |
|---|---|---|---|---|---|
| **Primary** | **Moisture index** | **Vegetation** | **LL** | **PL** | **PI** |
| Tropical -Continental or Savannah | Dry Sub Humid | Guinea Savannah | 25-53 | 14-24 | 10-30 |
| | Semi - Arid | | 25-40 | 12-22 | 10-20 |
| | | Sudan Savannah | | | |
| Wet Semi-Equatorial | Dry Sub Humid | Moist Semi Deciduous | 25-53 | 14-24 | 10-30 |
| | Moist Sub Humid | | 32-65 | 19-33 | 11-33 |
| South -Western Equatorial | Humid | Rain Forest | 42-72 | 23-34 | 16-36 |
| Dry Equatorial | Dry Sub Humid | Coastal Thicket | 25-53 | 14-24 | 10-30 |
| | Semi-Arid | Coastal Savannah | 25-40 | 12-22 | 10-20 |

With respect to work done on plasticity of the predominant laterite soil types, a follow up study of an earlier investigation by de-Graft Johnson et al (1969) was done by Bhatia and Yeboah (1970) to present the statistical values of the Atterberg‟s limits of the four groups. Bani (1971) also showed that swelling increases as the liquid limit and plasticity index increase and went further to present the plasticity characteristics of the D‛Hoore soil types for Ghana. A summary of findings is presented in Table 6.8.

Table 6.8 - Plasticity Properties of the Laterite Groups

| **Laterite Type** | **Value** | **LL %** | **PL %** | **PI %** | **SL (%)** |
|---|---|---|---|---|---|
| Group 1 *Nodular or Concretionary Laterite* | Range | 14.5 – 58.4 | 8.01 – 31.0 | 1.6 – 36.3 | Not Known |
| | Mean | 34.4 | 19.9 | 14.4 | |
| | Std. Dev. | 10.3 | 4.5 | 7.3 | |
| Group 2 *Iron Stone Hard Pans or Cap Rock* | Range | 16.7 – 53.0 | 1.0 – 40.0 | 4.0 – 28.0 | |
| | Mean | 31.4 | 18.9 | 12.5 | |
| | Std. Dev. | 9.21 | 6.32 | 5.48 | |
| Group 3 *Ground Water Laterite with Detrital Quartz* | Range | 12.9 – 61.8 | 10.0 – 44.4 | 0.9 – 31.3 | |
| | Mean | 37.2 | 24.7 | 15.3 | |
| | Std. Dev. | 9.84 | 7.66 | 5.36 | |
| Group 4 *Colluvial and Terrace Laterites* | Range | 18.4 – 73.0 | 10.2 – 48.0 | 3.2 – 34.9 | |
| | Mean | 43.4 | 23.3 | 20.6 | |
| | Std. Dev. | 13.74 | 8.1 | 7.29 | |
| Ferruginous | Not Known | 25 - 59 | 13 - 31 | 6 – 32 | 2.1 – 6.5 |
| Ferrallitics | | 24 - 54 | 13 - 24 | 9 - 30 | 5.3 – 6.5 |
| Ferrisols | | 26 - 52 | 15 - 25 | 10 - 28 | 0.7 – 6.6 |
| Phyllites | | 37.5 | 27.5 | | Not Known |
| Lateritic Gravels | | 20-38 | 11-25 | 2-16 | |
| Quartzitic Gravels | | 20-30 | 11-21 | 2-12 | |
| Both | | 20-38 | 11-25 | 2-16 | |
| LL – Liquid Limit; PL – Plastic Limit; PI – Plasticity Index and SL – Shrinkage Limit | | | | | |

A generalised plasticity classification of the main Ghanaian soil systems in relation to weathering environment is shown in Figure 6.20 (Gidigasu and Mate-Korley, 1980)

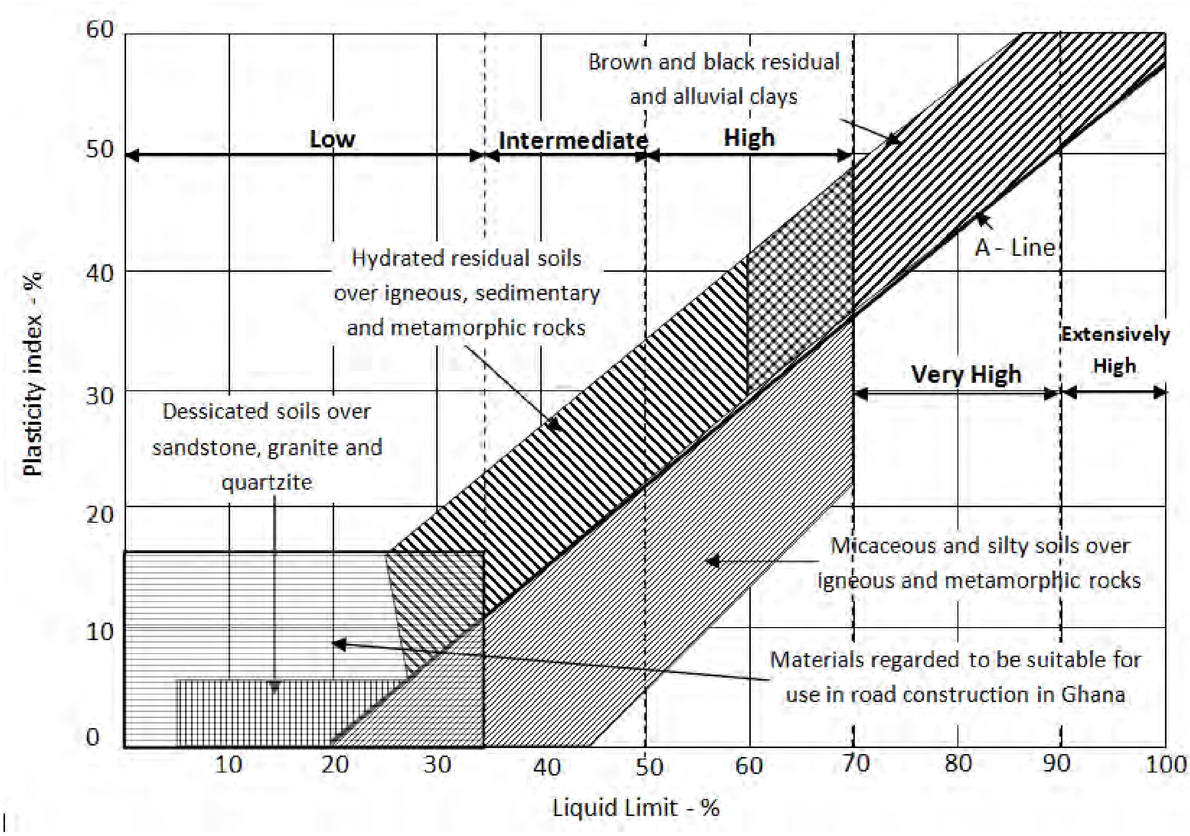



Figure 6.20 - Plasticity Classification of Some Ghanaian Soil Systems (Gidigasu and Martey-Korley, 1980)

#### 6.2.2.1 Selecting Laterites for Road Pavements

In the selection of laterite for use as road pavement material the above data should serve as a guide. The current specification for selection of road pavement materials is given in Tables 6.9 Ghana, to use natural gravel materials for any road works, the LL and PI must not exceed 35% and 16% respectively (as given in Figure 6.20).

Table 6.9 – Current Plasticity Specifications (Specification for Roads and Bridge Works, 2007)

| | Pavement Layer | LL (%) | PI (%) | SL (%) | Plasticity Modulus |
|---|---|---|---|---|---|
| **Current Specifications** | $G80_{Max}$ | 25 | 10 | 5 | 200 |
| | $G60_{Max}$ | 30 | 12 | 6 | 250 |
| | $G40_{Max}$ | 30 | 14 | 7 | 250 |
| | $G30_{Max}$ | 35 | 16 | 8 | 250 |

Only a small fraction of soils in Ghana meet the current limits for suitable material. It can be said that the laterite gravels are borderline or marginal materials while the quartzitic gravels are ideal materials for road base construction. This is also confirmed by work done by Gidigasu (1975) in his rating of laterites based on performance under bituminous surfacing. His conclusions are summarized in Table 6.10.

Table 6.10 – Field Performance Rating (after Gidigasu, 1975)

| Soil Properties | Field Performance Rating | | |
|---|---|---|---|
| | **Excellent** | **Average** | **Poor** |
| Linear Shrinkage, % | 0 - 4 | 4 - 6 | Above 6 |
| Plasticity Index, % | 0 - 6 | 6 - 8 | Above 12 |
| Liquid Limit, % | 14 - 21 | 22 - 30 | Above 30 |
| OMC (%) | - | 8 - 10 | - |

### 6.2.3 Strength of Coarse Particles

From the particle size analysis, the determination of the strength parameters is based on the coarse fraction of the gravel material. The coarse fraction of laterite gravel consists of the concretionary laterite and the quartz fragments. This is true in the case of Ghana, as most of the recommended laterite materials good for road construction are from quartzitic sources. In Ghana, nodular laterite gravels which often contain quartz are seen as excellent material for road base but with relatively weak concretionary nodules. A proportion of quartz and concretionary nodules, between 5% and 20% (slightly to very), must be used to describe a given gravel as either quartzitic or lateritic depending on the relative percentages (Charman, 1988). It is therefore important to measure the quartz content of any given sample of laterite gravel to have a good idea of its strength characteristics.

For road engineering works, the suitability of any gravel, the coarse fraction must possess the following strength qualities;

- Resistance to Abrasion (Hardness);
- Resistance to Fracture and Impact (Toughness); and
- Resistance to Weathering (Soundness).

The various assessments criteria which are used to evaluate the strength characteristics are through either field assessment or laboratory test or both.

**Field Assessment**

Clare (1960), Ackroyd (1971) and de Graft-Johnson et al (1972) have all used colour as basis to assess the relative strength of the laterite gravel from dark brown, reddish brown, yellowish brown to yellow, in that order from hard to soft.

**Laboratory Test**

Bhatia and Hammond (1970) confirmed that iron oxide in aggregates contributes to the strength of the aggregates. According to Gidigasu (1970), for the same soil, gravel fractions were found to have higher specific gravities than fine fractions. This was shown to be as a result of higher concentration of iron oxide in the gravel fraction.

Therefore, as a proxy to measure the iron oxide content, which is the main indicator of strength in laterite; the Aggregate Impact Value (AIV) and Los Angeles Abrasion Value test were used to establish relation with Specific Gravity as an index of iron oxide content and the water absorption test as an index that measure porosity of aggregate (see Figures 6.21 to 6.25). This work was done by Morin and Toder (1969) and De-Graft Johnson et al (1969) respectively for India and Ghana laterite gravels.

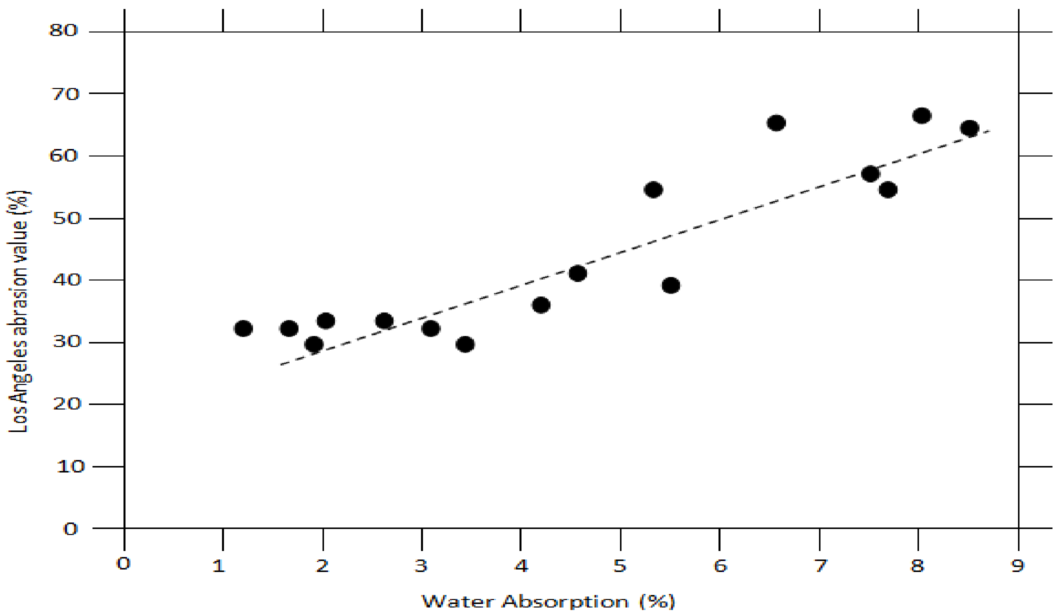



Figure 6.21 – Relation between Water Absorption and Los Angeles Abrasion Value for Typical West African Laterite Rocks (Charman, 1988)

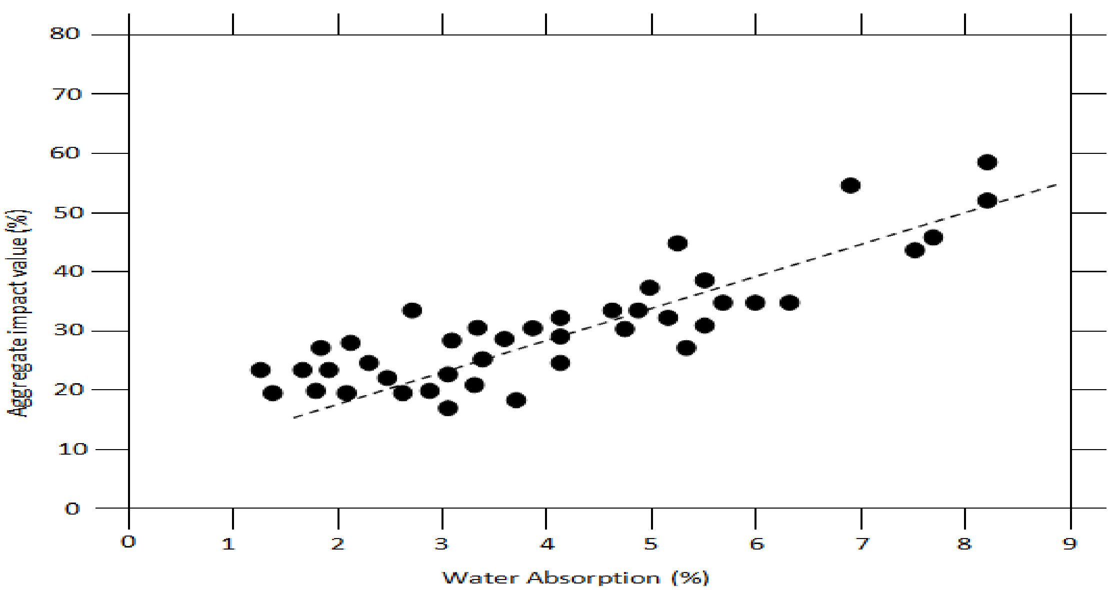



Figure 6.22 – Relation between Water Absorption and Aggregate Impact Value for Typical West African Laterite Rocks (Charman, 1988)

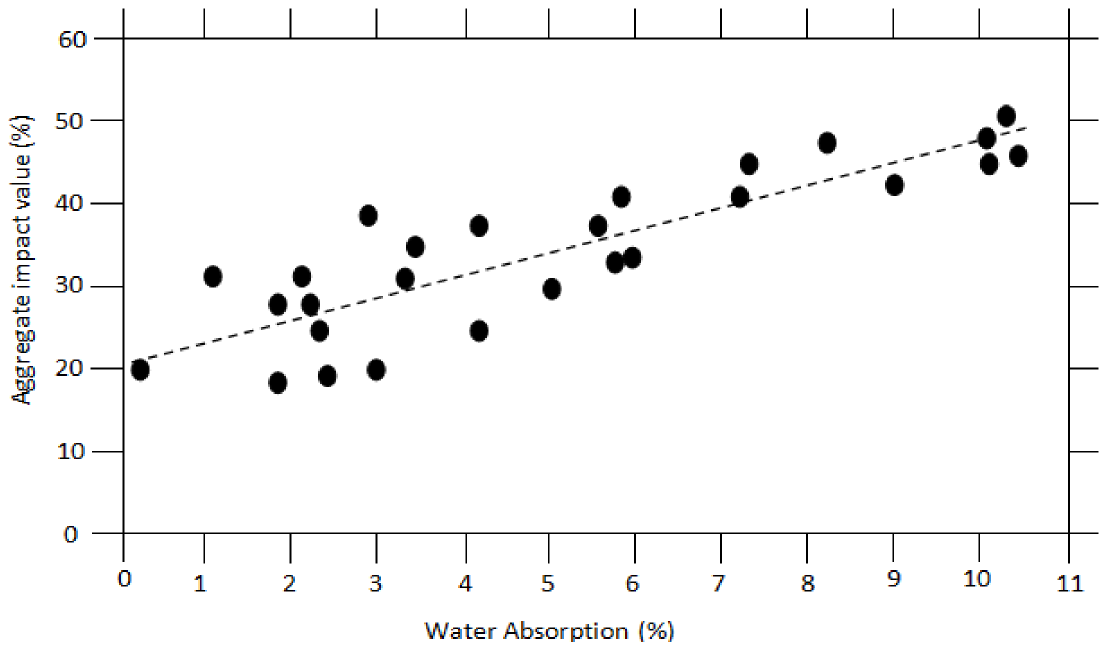



Figure 6.23 – Relation between Water Absorption and Aggregate Impact Value for Typical West African Laterite Pisoliths (Charman, 1988)

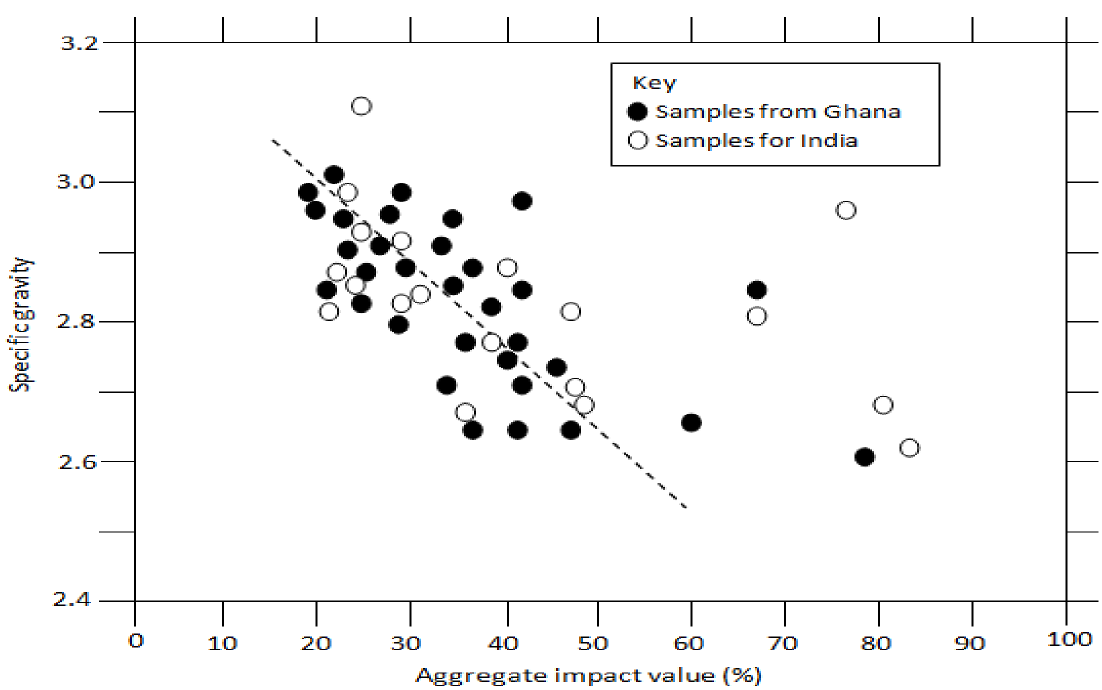



Figure 6.24 – Relation between Specific Gravity and Aggregate Impact Value for Typical Laterite Rocks (Charman, 1988)

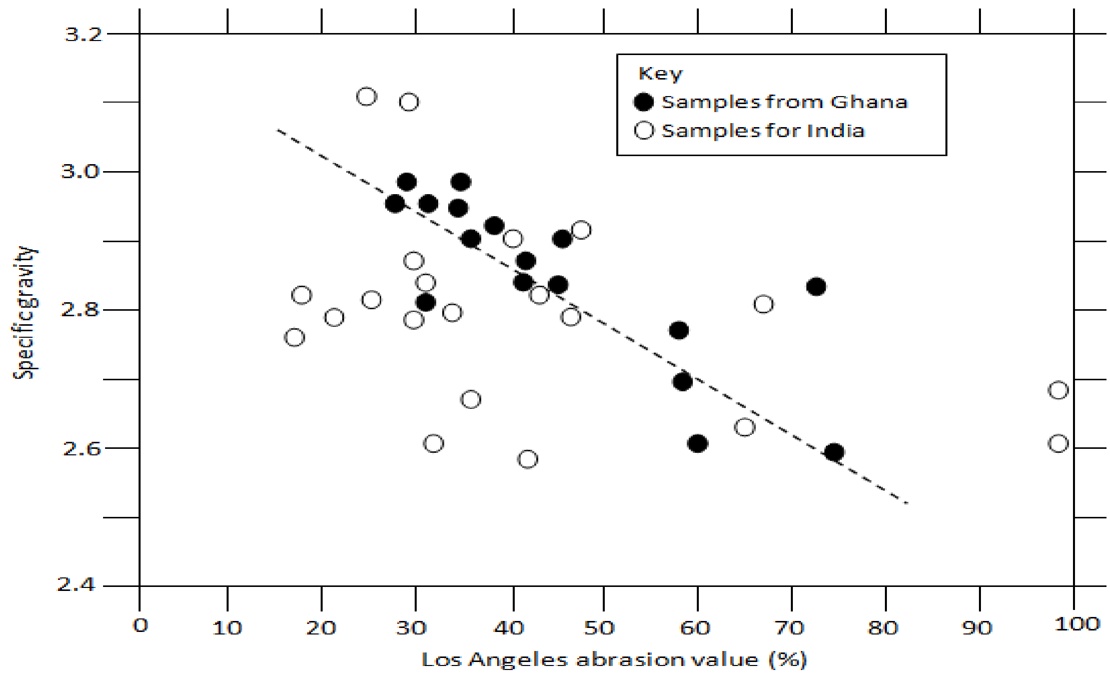



Figure 6.25 – Relation between Specific Gravity and Los Angeles Abrasion Value for Typical Laterite Rocks (Charman, 1988)

The conventional tests used to measure the strength of aggregates and gravel is the Aggregate Crushing Value (ACV), but due to the method"s insensitivity to weak materials (as it produces a compressed lump of fines), 10% Fines Aggregate Crushing Test (FACT) is used to determine the adjusted load required to produce 10% fines, instead of the 400-kN used to determine ACV. Aggregate Impact Value (AIV) is determine by dropping 15 blow from standard falling weight and height to produce fines expressed as a percentage of the total weight of the sample. For weak material there is a high tendency to produce over 20% fines which can lead to misleading results. The Modified AIV (MAIV) test, aims to ensure that the number of blows is only enough to produce under 20% fines. The BS 812, Parts 110 -112, 1990 are the prescribed method for conducting the test. The Los Angeles Abrasion Value (LAAV) Test is conducted using the ASTM

From the point of soundness, De-Graft Johnson et al (1969) suggested that the (MAIV) is a very useful test for rating gravels from the point of weathering and mechanical strength. Data collected during the study revealed that the MAIT, due to its ease of handling and reproducibility of results is very suitable for assessing the durability of the laterite gravels.

The proposed ratings for assessing the weathering characteristics of laterite gravels are summarised in Table 6.11.

Table 6.11 - Ratings for Assessing Weathering Characteristics of Lateritic Gravels (de-Graft Johnson et al., 1969)

| MAIV (%) | Weathering loss, 6 cycles (%) | Ratings |
|---|---|---|
| 30 | 4 | Excellent |
| 30 – 40 | 4 – 8 | Good |
| 40 – 50 | 8 – 13 | Average, generally unsuitable |
| 50 | 13 | Very Poor |

The MAIT is just the same as described in BS 812 except that the fall of hammer is changed from 15 inches to 7.5 inches. A cycle comprises using 500g of sample size ¾ inches to ¼ inches (19.05mm – 6.35mm), drying at 105°C for 24 hours, cooling at room temperature for 3 hours and submerging in water for 5 hours, finally air drying and sieving through ¼ " (6.35mm).

A large number of the laterites of Ghana are mechanically weak and tend to break down as a result of weathering and traffic loading. Based on Studies done by de-Graft Johnson et al (1969), four types of laterite gravels are considered useful engineering materials. These are:

- Nodular or concretionary laterites;
- Iron stone hard pans or cap rock;
- Ground water laterites with detrital quartz; and
- Colluvial and terrace laterites.

The physical features and mechanical characteristics of these groups of laterites are given in Table 6.12. Table 6.13 also summarises the strength characteristics of laterite gravels samples across the country.

Table 6.12 – Mechanical and Physical Characteristics of Laterite Gravels in Ghana (de-Graft Johnson et al., 1969)

| Group | Colour and Description | S.G. | Void Ratio**, e | Absorption 96 hrs, % | MAIV % | ACV % | LAAV % |
|---|---|---|---|---|---|---|---|
| 1-Nodular or Concretionary Laterites | Dark - Brown | 3.582 (6)* | 0.212 (6) | 7.45 (4) | 31.42 (4) | 35.27 (2) | 36.4 (1) |
| | Light - Brown | 3.390 (5) | 0.284 (5) | 10.24 (3) | 37.63 (5) | 41.45 (2) | 40.5 (1) |
| | Yellowish - Brown | 3.182 (6) | 0.334 (6) | 13.10 (4) | 46.50 (4) | 51.20 (2) | - |
| 2-Iron Stone Hard Pans or Cap Rock | Dark - Brown (Honey - Combed) | 2.981 (5) | 0.382 (5) | 16.30 (5) | 41.24 (4) | 38.10 (3) | - |
| | Dark - Brown (Homogenous) | 3.472 (4) | 0.291 (4) | 8.15 (4) | 29.23 (4) | 32.41 (2) | - |
| | Light - Brown or Reddish Brown | 3.041 (4) | 0.311 (4) | 18.42 (4) | 43.60 (4) | 41.67 (3) | - |
| 3-Groundwater Laterite with Detrital Quartz | Hard Pans (Accra Plains) | 3.341 (9) | 0.262 (9) | 11.42 (5) | 34.66 (5) | 32.43 (2) | 39.60 (1) |
| | Hard Pans (Forest Zone) | 3.164 (7) | 0.306 (7) | 8.51 (5) | 42.43 (4) | 45.44 (2) | 51.21 (1) |
| | Cemented Clay or Pseudo boulders | 2.972 (4) | 0.344 (4) | 17.44 (5) | 54.24 (3) | 48.42 (1) | - |
| 4-Colluvial and Terrace Laterites | Detrital Irregular Laterites | 3.120 (3) | 0.340 (3) | 13.49 (3) | 47.62 (3) | 45.48 (2) | - |
| | Concretionary Gravel, Cemented with Clay (Lower Slopes) | 3.260 (4) | 0.306 (4) | 12.65 (3) | 39.62 (2) | 41.61 (1) | - |

* The figures in brackets give the number of tests, for which the average is given in the table.

** Void ratio is for the natural formation.

Table 6.13 – Summary of Strength Characteristics of Laterite Gravels

<table>
<tr><th rowspan="2">Source</th><th rowspan="2" colspan="2">Sample</th><th colspan="4">Strength Characteristics</th><th rowspan="2">SG</th></tr>
<tr><th>LAAV</th><th>AIV</th><th>Absorption</th><th>ACV</th></tr>
<tr><td rowspan="6">Bhatia and Hammond (1970)</td><td colspan="2">Granites</td><td>28.8 – 69.5</td><td>16 – 57.2</td><td>1.7 – 5.8</td><td>-</td><td>2.72 – 3.10</td></tr>
<tr><td colspan="2">Phyllites</td><td>56.5</td><td>29.6 – 47.2</td><td>4.2 – 6.4</td><td>-</td><td>2.73 – 2.88</td></tr>
<tr><td colspan="2">Quartzites</td><td>64.9</td><td>21.4 – 54.9</td><td>2.2 – 8.4</td><td>-</td><td>2.61 – 3.0</td></tr>
<tr><td colspan="2">Limestones</td><td>-</td><td>34.6</td><td>4.6</td><td>-</td><td>2.78</td></tr>
<tr><td colspan="2">Sandstones</td><td>34.5</td><td>23 – 60.5</td><td>2.9 – 5.2</td><td>-</td><td>2.81 – 2.89</td></tr>
<tr><td colspan="2">Gneiss</td><td>-</td><td>21.2 – 34.2</td><td>1.9 – 5.2</td><td>-</td><td>2.7 – 3.28</td></tr>
<tr><td>Hammond (1970)</td><td colspan="2">Kumasi District</td><td>-</td><td>25.4 – 39.9</td><td>3.1 – 5.9</td><td>-</td><td>2.65 – 2.94</td></tr>
<tr><td rowspan="4">De Graft-Johnson et al (1969)</td><td colspan="2">Group 1</td><td>36.4 – 40.5</td><td>31.42 – 46.5*</td><td>7.45 – 13.10</td><td>35.27 – 51.2</td><td>3.18 – 3.58</td></tr>
<tr><td colspan="2">Group 2</td><td>-</td><td>29.23 – 43.6*</td><td>8.15 – 18.42</td><td>32.41 – 41.67</td><td>2.98 – 3.47</td></tr>
<tr><td colspan="2">Group 3</td><td>39.6 – 51.21</td><td>34.66 – 54.24*</td><td>8.51 – 17.44</td><td>32.43 – 48.42</td><td>2.97 – 3.34</td></tr>
<tr><td colspan="2">Group 4</td><td>-</td><td>39.62 – 47.62*</td><td>12.65 – 3.49</td><td>41.61 – 45.48</td><td>3.12 – 3.26</td></tr>
<tr><td rowspan="2">Gidigasu (1991)</td><td rowspan="2">Northern Semi-Arid Zone</td><td>Quartzitic Gravels</td><td>31.4</td><td>13.3</td><td>-</td><td>-</td><td>2.65</td></tr>
<tr><td>Lateritic Gravels</td><td>42.5</td><td>31.4</td><td>-</td><td>-</td><td>2.95</td></tr>
<tr><td colspan="8">Figures with * are Modified Aggregate Impact Values (MAIV)</td></tr>
</table>

Bhatia and Hammond (1970) summarised the ranges of values as the results for the mechanical properties of aggregates studied in Table 6.14.

Table 6.14 – Summary of Results of the Mechanical Properties of Laterite Gravels (after Bhatia and Hammond, 1970)

| Property | Range |
|---|---|
| Specific Gravity | 2.61 – 3.1 |
| Flakiness Index | 2.5% - 9.0% |
| Elongation Index | 2.7% - 16.4% |
| Angularity Number | 5 – 15 |
| AIV | 16.0% - 60.5% |
| LAAV | 28.5% - 69.5% |

The current specification gives minimum requirements for a range of uses of aggregate in pavement construction. These are summarised in Table 6.15.

Table 6.15 – Specification for Aggregates in Ghana (Standard Specification for Roads and Bridge Works, 2007)

| Purpose | Base | Subbase | Chippings for Surface Dressing | Hot Mixed Asphalt | Cold Asphalt |
|---|---|---|---|---|---|
| Heavy vehicles per day in one direction | - | - | 0-25 | - | - |
| LAA (%) max | 45 | 50 | 35 | 40 | 40 |
| SSS (%) max | - | - | 12 | - | - |
| FI max | 30 | 35 | 25 | 25 | 40 |
| 10% Fines Min (dry) kN | 110 | 50 (wet) | 210 | 160 | 160 |
| Wet/Dry % | 75 | 60 | 75 | 75 | 75 |
| Stripping Test (ASTM D 4867) (%) max | - | - | 5 | - | - |
| Water absorption (%) (coarse aggregate) | 2.0 | 2.5 | - | 1.0 | - |

From the above, the general specification is not exclusively meant for laterite aggregate but covers crushed rock and stabilised materials. Works done by De-Graft Johnson et al. (1969 & 1972) and Bhatia and Hammond (1970) are useful for preparing special specification for laterite aggregates whose MAIV limits have been given due to the fact that laterite is a non-standard material (as given in Table 6.16).

Table 6.16 - Recommended specifications physical and mechanical properties of laterite aggregates

| Test | Excellent | Good | Fair | Poor | Source |
|---|---|---|---|---|---|
| MAIV | < 20 | 20 – 30 | 30 – 40 | > 40 | De-Graft Johnson et al (1969) |
| AIV | < 30 | 30 – 40 | 40 – 50 | > 50 | Bhatia and Hammond (1970) & De-Graft Johnson et al (1972) |
| LAAV | < 40 | 40 – 50 | 50 – 60 | > 60 | |
| Water (24 Hrs) Absorption | < 4 | 4 – 6 | 6 – 8 | > 8 | |
| S.G. | > 2.85 | 2.85 – 2.75 | 2.75 – 2.58 | < 2.58 | |

### 6.2.4 Compaction and Bearing Capacity

Compaction is the application of mechanical energy to a soil to rearrange the particles and reduce its void ratio. Compaction is done mainly to increase the density of the soil and hence its bearing capacity. It also reduces the permeability and subsequent settlement under working loads. From the compaction test results, the maximum dry density (MDD) and its corresponding optimum moisture content (OMC) can be determined. Both MDD and OMC form reference values against which field compaction is judged. Invariably, if compaction energy applied during the laboratory test is different to that applied in the field by compaction, then it is necessary to undertake full scale field plant compaction trials. Trial pavements sections are used to select the best method and equipment or machine to achieve the required relative compaction for the material of the respective pavement layers.

The most common Standard Compaction Tests for different material grading are detailed in BS1377:Part4:1990. It is also referred to as the Proctor Test method. The compactive effort is by the use of a rammer of different weights (2.5kg or 4.5kg) and falling at a pre-determined height (300mm or 450mm respectively) depending on the nature of the material determined by percentage amount retained on the 20mm test sieve. With finer material tests, using the 2.5kg rammer is most appropriate. The Modified AASHTO is based on the use of 4.5kg rammer. In all cases the number of blows received per layer of material is 27 blows. (Total layers: 3 and 5 for 2.5kg and 4.5 kg rammer respectively). There is also the West African Compaction Method, which has been adopted by Ghana as the Standard for Compaction Test. The compaction is done in a standard CBR mould in five layers with a 4.54 kg AASHTO rammer, dropping from a height of 457.2 mm and 25 blows per layer.

According to Gidigasu (1970) the factors affecting the density and compaction characteristics of soils can be grouped under the following:

- Genesis of the Soil; and
- Pre-Treatment Procedures before Testing.

#### 6.2.4.1 Genesis of the Soil

Gidigasu (1972) established a significant correlation between the optimum moisture content and the clay content and showed that the maximum dry density decreased with increase in the clay content for both residual and non-residual soil types. Gidigasu (1980) established long term seasonal moisture profiles for typical soil systems in the various climatic zones and presented a summary of the mean compaction values of the subgrade soils.

Further work by Gidigasu and Appeagyei (1982) on subgrade soils and moisture conditions in all the four climatic zones of Ghana, using the standard Proctor Test, established that the results of the compaction characteristics, using the Standard Proctor of subgrade soils show that there is no significant correlation between the clay size content and the index properties and also between the natural moisture content and the air dry and optimum moisture content on the other hand. The correlation between the MDD and OMC was found to be quite good and it was concluded that climatic zones alone cannot form the basis for differentiating the soils in terms of compaction properties.

Analysis of over one thousand samples of laterite gravels of Ghana from all the geomorphologic regions have shown that the MDDs and OMCs for the four groups proposed earlier by de-Graft Johnson et al (1969), range between 1875 – 2323 kg/m$^3$ and OMC between 2 – 16% using the Ghana Method of Compaction (see Table 6.17).

Table 6.17 - Compaction Characteristics of laterite (de-Graft Johnson, 1972)

| **Sample** | **Material Type** | **MDD (kg/m$^3$)** | **OMC (%)** |
|---|---|---|---|
| Group 1 | Nodular or Concretionary Laterite | 1970 – 2270 | 6 - 14 |
| Group 2 | Iron Stone Hard Pans or Cap Rock | 2080 – 2270 | 6 – 14 |
| Group 3 | Ground Water Laterite with Detrital Quartz | 1760 – 2230 | 2 – 16 |
| Group 4 | Colluvial and Terrace Laterites | 1920 – 2150 | 9 – 16 |

#### 6.2.4.2 Pre-Treatment Procedures before Testing

The breakdown of particles during compaction affects the compaction characteristics of the soil as it could lead to better or otherwise in the grading resulting in higher or lower densities. The degree of breakdown can increase or decrease after several reuses of the soil. But since this may be a function of the degree of maturity (laterisation) and the physical characteristics of the particles, if there is any risk of particle breakdown then unused soil (or fresh specimen) is used. It has been shown that oven drying always give the highest MDDs and lowest OMCs while soils at natural moisture contents give the lowest MDDs and highest OMCs.

### 6.2.5 Stabilisation

The purpose of soil stabilization in road construction is to improve the soil and base materials in terms of strength, bearing capacity and durability and to decrease their water sensitivity and volume change during wet/dry cycles. This can be achieved through chemical and mechanical means.

Mechanical stabilization is the improvement of the soil grading, usually by adding material corresponding to the depleted grain sizes in the original soil. The most common chemical stabilizers in Ghana are cement, lime and bitumen. Cement and lime are used to modify clay minerals to some extent and reduce the plasticity. Lime is sometimes used to reduce plasticity and to stabilise highly plastic soils before adding cement. Bitumen is added to stop moisture absorption by the fines fraction by coating them.

#### 6.2.5.1 Mechanical Stabilization

Too much fines content can result in decrease the strength of the mix when wet and will cause the running surface to be slippery when used in unpaved roads. Too few fines, on the other hand, cause lack of binding and difficulty in compaction. The mix will lose its cushioning effect which is provided by the fines and the grain to grain contacts will increase, making weaker aggregates susceptible to fracturing.

The addition of sand to highly plastic soils reduces plasticity and improves the grading of the material. This very much depends on the size fraction added and its quantity. But the rate of reduction varies with laterite soils at different places. The maximum dry densities for soils increase with the addition of sand up to a certain level of and then start decreasing with further addition of sand. So is the effect of sand stabilization on CBR values, which tend to increase up to a particular percentage of sand and with further addition, starts decreasing. With an increase in sand, there was a definite decrease in percentage swell (Castel, 1970).

#### 6.2.5.2 Chemical Stabilisation

Lime is generally considered more appropriate for the stabilisation of more clayey soils, but it is less effective in organic soils. The lime first reduces the plasticity of clays and subsequently develops a cementitious bond. The strength of soil – lime mixtures increases with lime content, to a maximum at about 8% lime. The speed of reaction is slower than cement and therefore compaction delays are less important. This allows for more flexibility in construction.

The properties of compacted cement-stabilized soil are related to the cement content, to the compacted density and to the curing time. In general, the strength increases in direct proportion to the cement content but at different rates for different soils. The tropics have the advantage of high average temperatures for the stabilisation of laterite soils with cement since strength gains are faster.

Bani (1970), Bawa and Gidigasu (1965), Bhatia (1967), Castel (1970), as well as Gidigasu and Appeagyei (1980) studied how the response of the various soil systems to different stabilizers have been evaluated and the inter-relationships between weathering environments, soil texture, strength and bearing properties determined through different procedures. For example, the effect of cement and lime stabilization on the plasticity and linear shrinkage of a typical black clay have been studied in detail (Gidigasu and Appeagyei, 1980).

Cement is more effective than the local lime in reducing the liquid limit and increasing the plastic limit. In Ghana a two-hour period is generally specified for spreading, mixing, watering and compaction during the construction of stabilized pavements. Increasing time during cement stabilization will cause a decrease in strength while increased period of mixing improves the strength of lime stabilized soils. It was, however, concluded by Gidigasu (1980) that the danger of losing strength through prolonged period of mixing or delayed compaction is only applicable to cement stabilised and not to lime stabilised soils. Some of the pre-test preparation and testing factors that have been shown to affect results of various types of stabilisation are the degree of drying and mixing and the elapsed time between mixing and compaction. Studies on different clays stabilized with sand and cement have shown that the mixture containing montmorillonite clay, gave a higher strength than illite and kaolinite clays with the same concentration of cement (Bhatia, 1967).

Cement is used widely in Ghana than lime, due to its availability in adequate amount. Cement may be added to the soil to increase its resistance to the effects of water or to increase its density. The principal chemical factors which decide whether or not a soil will be chemically suitable for cement stabilization are the pH and the organic matter content. The effects of cement on Ghanaian soils have been discussed in detail by Castel (1970). In a review by Gidigasu (1970), it was concluded that almost all Ghanaian soils are suitable for stabilisation with between 4 and 10 % by weight of cement. Results from Studies done on soil-cement stabilisation of about 90 soil types suggest that the unconfined strengths vary not only with the textural groups but also with the climatic-vegetation zones. With between 3 to 7 % cement by weight, the CBR values of most of the soils increased three to five folds.

Gidigasu and Mate-Korley (1980) also investigated the possibility of improving the stability of the micaceous soils through cement stabilisation and results showed that at around 8% cement content, the swell dropped from 2% to 0.5%. It was also reported that the CBR of typical Cape Coast micaceous soils increased 4 to 5 times with the addition of 4% cement and 2 to 4 times with the addition of lime. This gain in strength was largely maintained after a total immersion in water for 7 days. Gidigasu and Appeagyei (1980) also established the effect of cement and locally produced lime on black cotton clays in Ghana.

The significant genetic factors that affect the results of stabilisation include the weathering process, physical, chemical and mineralogical composition of the soil, as well as the depth of the sample, which determine its degree of classification or hydration. According to Gidigasu (1980), soils of the same series, horizon and texture require the same amount of stabilizer for adequate stabilization. Results of lime stabilisation of soils from different horizons of a typical residual profile over phyllite showed that higher strength is developed for soils from the laterite zone than soils from the mottled and pallid zones. It has been observed that soils near the surface usually need a higher percentage of cement for stabilization, due to the presence of certain amount of organic matter. If however the upper 6 – 12in layer of soil is removed, it may require much less cement.

According to Castel (1970), at around 8% cement content, the swell value could drop from about 2% for the untreated soil to about only 0.5% for the stabilized soil.  CBR, after stabilization and curing between 7 – 28 days can considerably increase the stability of the soil to meet the current West African specifications for CBR. Research by Bhatia (1967) on four laterite soils from different locations in Ghana gave compressive strength and CBR characteristics as shown in Figures 6.26 and 6.27 respectively, with increasing amount of cement.

According to Gidigasu (1983), sand stabilisation can also be effective in reducing plasticity and improving the bearing strength of lateritic gravels.

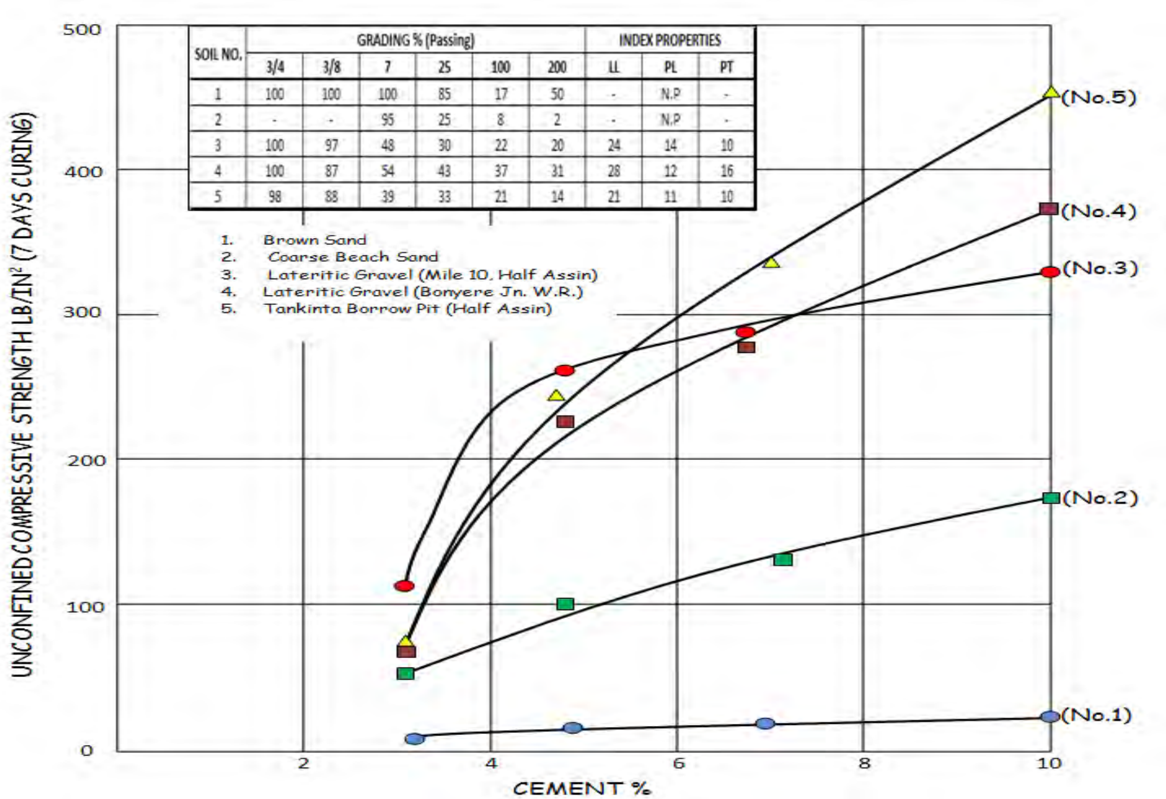



Figure 6.26 - Effect of Cement on the Compressive Strength of Some Ghanaian Soils (Bhatia, 1967)

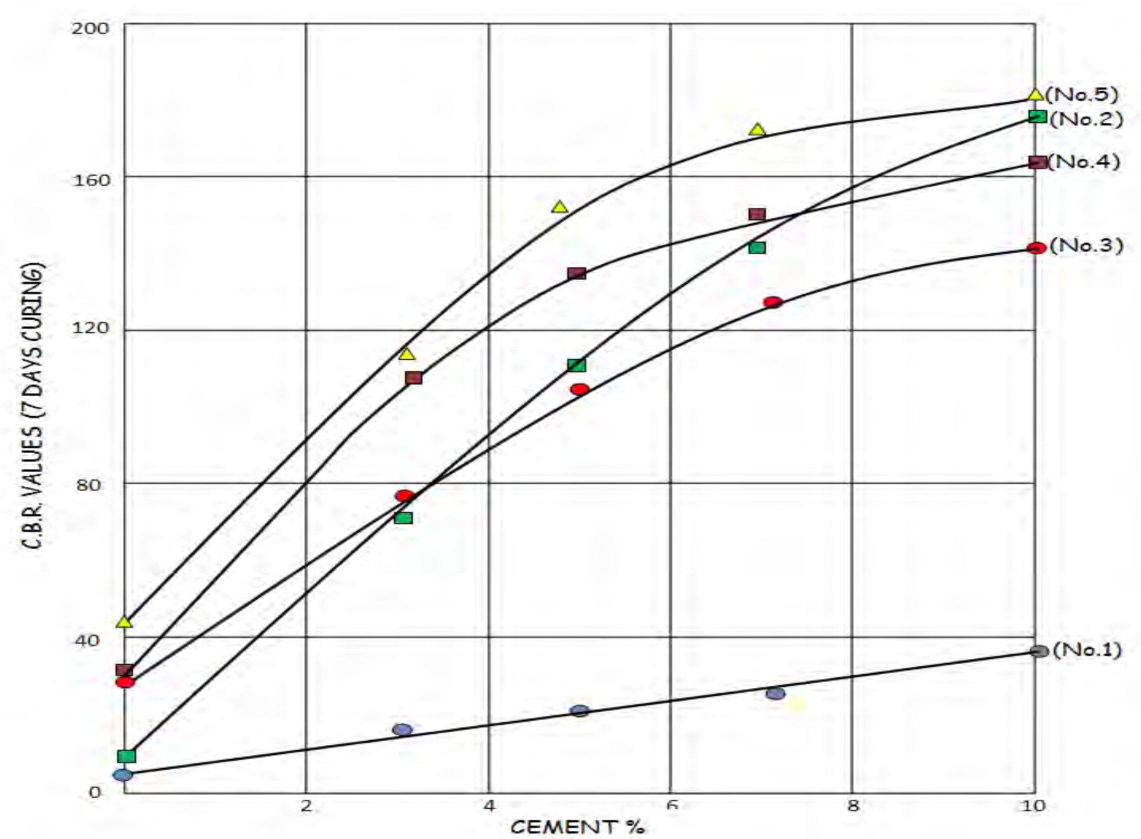



Figure 6.27 - Effects of Cement on CBR Values of Some Ghanaian Soils (Bhatia, 1967)

## 6.3 Summary

The performance of any tropical road built with laterite will depend basically on the particle size distribution of the material, its load bearing capacities and its plasticity behaviour. For unsealed roads like the gravel type, its surface behaviour as well as the overall stability of the material during the changing climatic seasons is of prime concern since only a single layer of the material is placed directly on the subgrade to bear traffic load and also act as the wearing course for driving comfort. The plasticity of such a layer must therefore be balanced between levels where it can hold the particles together during the dry season and also passable during the wet season.

From the grading envelope proposed for surface courses, it can be observed that the maximum particle size has been limited to 20mm which will make the surface fairly smooth for driving comfort and together with the specified amount of fines will keep these coarser sizes together enhancing its stability. Comparing the grading envelopes of materials proposed for the various climatic zones (see Appendix E), the only laterite gravel that falls more within this band is the Decomposed Granites and Pegmatite which can be found in all the climatic zones.

As shown in Table 6.18, the grading requirement proposed for base layer have their maximum particle sizes raised to as high as 37.5 – 75 mm to serve more as a load bearing layer to protect the underlying layers and subgrade from excessive stresses. And as already shown in Figures 6.8 to 6.10, all the selected gravel types are suitable as base materials.

The plasticity requirements of these materials were specified based on their performance and specific uses in the different climatic zones. Based on this, even the same material type is expected to behave differently due to the different climatic conditions they it will be exposed to. It can be observed in Table 6.18 that whiles a maximum PI of 6% has been specified for a dry zone like the Semi-Arid, that specified for a moist zone like the Humid is as high as 10%.

Table 6.18 - Summary of Specifications of Laterite Gravels Presented for Ghana

| | Current Grading Requirement | | | | Proposed Requirement for Base and Subbase | | | | Semi-Arid Zone | | Dry Sub-Humid Zone | | | Moist Sub-Humid Zone | | Humid Zone | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | | | Available Quality Gravel | | | | | | | | |
| **Mat'l Type** | **G80** | **G60** | **G40** | **G30** | **Base 1** | **Base 2** | **Surf. Course** | **Subbase** | **Dec. Granite & Pegt'te** | **Weathered Quartzite in the Dahomeyan Series** | **Dec. Granite & Pegt't** | **Quartz Drift Gravels** | **Weathered Quartzite in the Togo Series** | **Dec. Granite & Pegt't** | **Weathered Quartzite in the Tarkwaian Series** | **Dec. Granite & Pegt't** | **Weathered Quartzite in the Tarkwaian Series** |
| **Size (mm)** | **GRADING ENVELOPES** | | | | | | | | | | | | | | | | |
| **0.075** | 5-15 | 5-22 | - | - | 0 – 15 | 16 – 26 | 16 – 30 | 24 – 42 | 0 – 20 | 9 – 19 | 0 – 20 | 2 – 18 | 9 – 19 | 0 – 20 | 9 – 19 | 0 – 20 | 9 – 19 |
| **0.425** | 8-26 | 8-33 | - | - | 3 – 28 | 18 – 32 | 18 – 42 | 32 – 46 | 2 – 30 | 16 – 32 | 2 – 30 | 6 – 24 | 16 – 32 | 2 – 30 | 16 – 32 | 2 – 30 | 16 – 32 |
| **2** | 20-45 | 20-50 | - | - | 7 – 36 | 20 – 37 | 26 – 52 | 36 – 56 | 12 – 54 | 32 – 50 | 12 – 54 | 14 – 44 | 32 – 50 | 12 – 54 | 32 – 50 | 12 – 54 | 32 – 50 |
| **5** | 30-55 | 30-75 | - | - | 20 – 76 | 36 – 60 | 60 – 86 | 56 – 76 | 28 – 72 | 44 – 64 | 28 – 72 | 33 – 60 | 44 – 64 | 28 – 72 | 44 – 64 | 28 – 72 | 44 – 64 |
| **10** | 45-70 | 45-90 | - | - | 32 – 91 | 48 – 76 | 80 – 100 | 72 – 90 | 52 – 85 | 46 – 72 | 52 – 85 | 52 – 82 | 46 – 72 | 52 – 85 | 46 – 72 | 52 – 85 | 46 – 72 |
| **20** | 60-85 | 75-100 | - | - | 52 – 100 | 70 – 100 | 100 | 86 – 100 | 72 – 100 | 64 – 88 | 72 – 100 | 72 – 95 | 64 – 88 | 72 – 100 | 64 – 88 | 72 – 100 | 64 – 88 |
| **25.4** | 67-90 | 77-100 | - | - | 62 | 78 | | 91 | 74 | 68 – 90 | 74 | 76 – 96 | 68 – 90 | 74 | 68 – 90 | 74 | 68 – 90 |
| **37.5** | 80-100 | 80-100 | - | - | 80 | 90 | | 100 | 79 | 76 – 100 | 79 | 86 – 98 | 76 – 100 | 79 | 76 – 100 | 79 | 76 – 100 |
| **75** | 100 | 100 | - | - | 100 | 100 | | | 100 | 100 | 100 | 100 | 100 | 100 | 100 | 100 | 100 |
| **CBR %** | 80 | 60 | 40 | 30 | | | | | | | | | | | | | |
| **CBR Swell %** | 0.25 | 0.5 | 0.5 | 1.0 | | | | | | | | | | - | | | |
| **COMPACTION REQUIREMENTS** | | | | | | | | | | | | | | | | | |
| **Rel. Density %** | 97 - 98 | | 94 - 95 | | | | | | 100 | 95 | 101 ± 1 | 101 ± 2 | 94 ± 5 | - | | | |
| **MDD kg/m3** | - | | | | | | | | - | | 2130 ± 64 | 2099 ± 64 | 2067 ± 144 | 2160 (Min.) | | | |
| **0MC** | | | | | | | | | | | 8 ± 2 | 9 ± 1 | 9 ± 1 | - | | | |

Table 6.18 - (continued)

| | Current Grading Requirement | | | | Proposed Requirement for Base and Subbase | | | | Semi-Arid Zone | | Dry Sub-Humid Zone | | | Moist Sub-Humid Zone | | Humid Zone | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | | | Available Quality Gravel | | | | | | | | |
| **Mat'l Type** | **G80** | **G60** | **G40** | **G30** | **Base 1** | **Base 2** | **Surf. Course** | **Subbase** | **Dec. Granite & Pegt'te** | **Weathered Quartzite in the Dahomeyan Series** | **Dec. Granite & Pegt't** | **Quartz Drift Gravels** | **Weathered Quartzite in the Togo Series** | **Dec. Granite & Pegt't** | **Weathered Quartzite in the Tarkwaian Series** | **Dec. Granite & Pegt't** | **Weathered Quartzite in the Tarkwaian Series** |
| **PLASTICITY REQUIREMENTS** | | | | | | | | | | | | | | | | | |
| | | | | | | | | | **Base** | **Sub base** | **Base** | **Subbase** | **Subgrade** | **Base** | **Sub base** | **Base** | **Sub base** |
| **% Passing No. 200 (Max)** | - | | | | | | | | 10 | | 9 ± 2 | 10 ± 4 | 36 ± 23 | 12.5 | | 12.5 | |
| **LL % (Max)** | 25 | 30 | 30 | 35 | | | | | 25 | | 24 ± 6 | 37.5 | 34 ± 9 | 37.5 | | 37.5 | |
| **PI % (Max)** | 10 | 12 | 14 | 16 | | | | | 6 | | 7 ± 4 | 10 | 15 ± 7 | 10 | | 10 | |
| **SL % (Max)** | 5 | 6 | 7 | 8 | | | | | - | | - | | | - | | - | |
| **PM** | 200 | 250 | 250 | 250 | | | | | | | | | | 100 | | 100 | |

# CHAPTER 7

# ADAPTATION AND CALIBRATION OF HDM-4 MODEL FOR GHANA

## 7.1 Introduction

The World Bank in 1968 made the first move to produce a road project appraisal model in conjunction with the Transport and Road Research Laboratory (TRRL) and Laboratoire Central des Ponts et Chaussées (LCPC) in response to a Terms of Reference for Highway Design Study. The World Bank in 1972 then commissioned the Massachusetts Institute of Technology (MIT) to produce the Highway Cost Model (Moavenzadeh et al, 1971, 1975). The development of the Highway Cost Model (HCM) highlighted areas where further research is needed, to make the HCM appropriate for use in developing countries with additional relationships specific to those environments.

Kerali (2000) reported on work done in Kenya by TRRL and the World Bank to investigate pavement deterioration (paved and unpaved) and factors affecting vehicle operating cost in developing countries. The result of this study led to the development of Road Transport Investment Model (RTIM) by TRRL for developing countries. The World Bank funded further improvement of the HCM at MIT which led to the production of the first version of Highway Design and Maintenance Standard Model (HDM) (Harral, 1979).

In order to extend the geographic scope of RTIM and HDM, various studies were carried out in the Carribean by TRRL (Morosiuk and Abaynayaka, 1982; Hide, 1982), in Brazil the United Nation Development Programme funded the validity of the model relationships (GEIPOT, 1982). The Central Road Research Institute (CRRI) in 1982 carried out a study on Road User Charges in India. These studies led to development of RTIM2 model by Parsley and Robinson in 1982 and the HDM-III in 1987 by Watanatada et al under the auspices of the TRRL and World Bank respectively. Simplified and user-friendly versions had been developed such as the RTIM3 (Cundill and Withnall, 1993) and the HDM-Q (Hoban, 1987) which incorporate the effect of traffic congestion and the HDM Manager (Archondo-Callao, 1994) which is menu driven.

The use of these models has been very successful in justifying investment at different levels of intervention road project from new construction, through rehabilitation and maintenance activities.

The HDM-4 model is an update of the RTIM3 and HDM-3 models. The vehicle operating cost (VOC) models developed in 1982 were based on then vehicle technology. Over a period of twenty years, the vehicle technology has improved dramatically resulting in less VOC, coupled with extensive research within that span of time have led to better appreciation of environmental factors on road deterioration even though the HDM-3 and RTIM3 deterioration models were still relevant. There was the need to update the technical relationships in the latest models which were in excess of 10 years by 1995 and also to improve the capacity of the RTIM and HDM-2 to deal with diverse pavements under different climatic conditions especially in the industrial countries.

The HDM-4 analytic framework is based on the concept of pavement life cycle analysis. The model has the capacity to predict the life cycle of a road pavement, which typically is between 15 and 40 years. Once a road is constructed and subjected to traffic loading, environmental weathering and effect of inadequate drainage (Kerali, 2000). The rate of deterioration is dependent on the maintenance standards or policies adopted by a Road Agency to preserve the structural integrity of the road in order to carry traffic according to the design. The predicted trend in road deterioration represented by the riding quality and measured by the International Roughness Index (IRI) when subjected to traffic loading. The various maintenance standards are primarily to arrest further deterioration beyond a given IRI and bring the riding quality almost to the design level. The cost of the road over its design life is therefore not only that of new construction but includes the cycles of rehabilitation and maintenance costs incurred by Road Agency over the design life of the pavement. Figure 7.1 depicts the Life Cycle Cost of a given pavement.

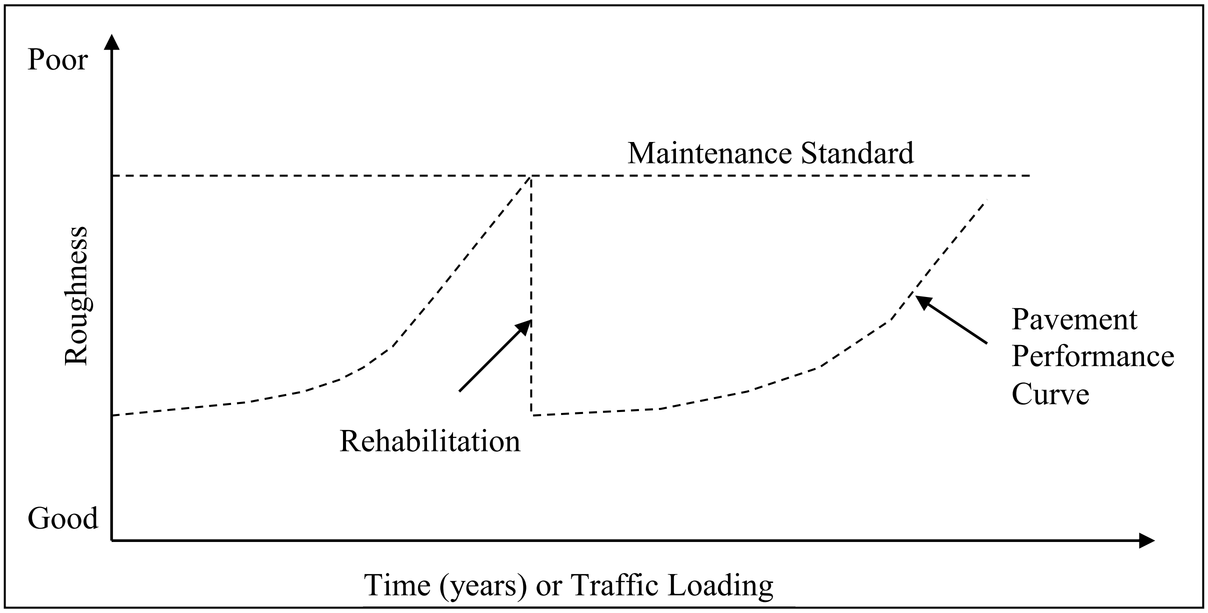



Figure 7.1 – Concept of Life-Cycle Analyses in HDM 4.

The catalogue of pavements to be generated from the M-E design will therefore have to be subjected to whole life cycle analyses to select the ultimate pavement design which is most economic. In order to ensure that the model simulate the condition similar to what is experienced in Ghana, the default variables will have to be calibrated to suit Ghana conditions. The HDM-IV model has three sub-models which will have to be calibrated using data from Ghana environment. These sub models are described by Odoki et al. (2000) as follows:

1. Road deterioration (RD);
2. Road works Effects (WE); and
3. Road user effects (RUE).

Since the HDM-4 model simulates future changes to the road system from current conditions, the reliability of the results is dependent upon two primary considerations:

1. How well the data provided to the model represent the reality of current conditions and influencing factors, in the terms understood by the model; and

2. How well the predictions of the model fit the real behaviour and the interactions between various factors for the variety of conditions to which it is applied

Application of the model thus involves two important steps:

1. **Data input:** a correct interpretation of the data input requirements, and achieving a quality of input data that is appropriate to the desired reliability of the results.

2. **Calibration of outputs:** adjusting the model parameters to enhance how well the forecast and outputs represent the changes and influences over time and under various interventions. Calibration of the HDM model focuses on the components that determine the physical quantities, costs and benefits predicted for the RD, WE, RUE and Socio-Economic Effects (SEE).

In 2007, as part of the policy of the Ministry of Roads and Transport the University of Birmingham was assigned to make the HDM-4 compatible to Ghana‟s road management system and carry field studies to establish local parameters for Ghana through calibration. Odoki, et al. (2007) carried out the HDM-4 model adaptation and calibration for Ghana. The final draft report was prepared by J. B. Koranteng -Yorke as the coordinator for the assignment and was edited by Dr. J. B. Odoki as the team leader.

### 7.2 Adaptation

The primary objective of adaptation is to make the analysis from the model relevant and compatible to Ghana environment in which it is being used. Adaptation of HDM-4 model is therefore basically restructuring of the default configuration data in line with local conditions, standards and practices. It requires the following information from the area of the road network:

- Prevailing climatic condition;
- Road types, respective functional classes, and road condition parameters;
- General traffic pattern per road type (e.g. representative stream per road type, average growth rate, traffic bands etc).

The provision of the above data from Ghana‟s perspective ensured proper adaptation of the HDM-4 to Ghana‟s standards and practices.

### 7.3 Calibration Process

Calibration of HDM-4 is intended to improve the accuracy of predicted pavement performance and vehicle resource consumption. The default equations in HDM-4, if used without calibration, would predict pavement performance that may not accurately match observed conditions on specific road sections. The extent of HDM-4 calibration (Bennett et al., 2000) is defined by as follows:

1. Level 1 (***Application)*:** Determines the values of required input parameters based on a desk study of available data as well as engineering experience of pavement performance and adopts many default values in addition to calibrating the most sensitive parameters with best estimates.

2. Level 2 (***Verification***): Requires measurement of additional inputs and moderate field surveys to calibrate key predictive relationships to local conditions.

3. Level 3 (***Adaptation***): Collects experimental data required to monitor the long-term performance of pavements within the study area and which data should be used to enhance the existing predictive relationship.

The Ghana road agencies have maintained a road database for its road network for some years and stored in their respective databases. For the purpose of calibration work, and the quality of data retrieved from the road database, Level 2 calibration was undertaken. The calibration and adaptation procedures were considered under two aspects:

1. Configuration of HDM-4; and
2. Model calibration.

The process is summarised by the following steps:

(i) Processing of data related to pavement types, pavement condition, traffic, speed, road user effects, etc.;
(ii) Determination of observed trends of the different components surveyed on Ghana‟s roads using Excel Spreadsheet;

(iii) Defining data in HDM-4: calibration of new road sections, representative vehicle fleet characteristics, and project analysis;

(iv) Carrying out HDM-4 runs to determine predicted values and trends using HDM-4 default factors;

(v) Determination of calibration factors for each relationship considered; and

(vi) Comparison of the predicted values obtained from calibrated HDM-4 against those obtained using HDM-4 default data.

These steps are also illustrated in the flow chart shown in Figure 7.2.

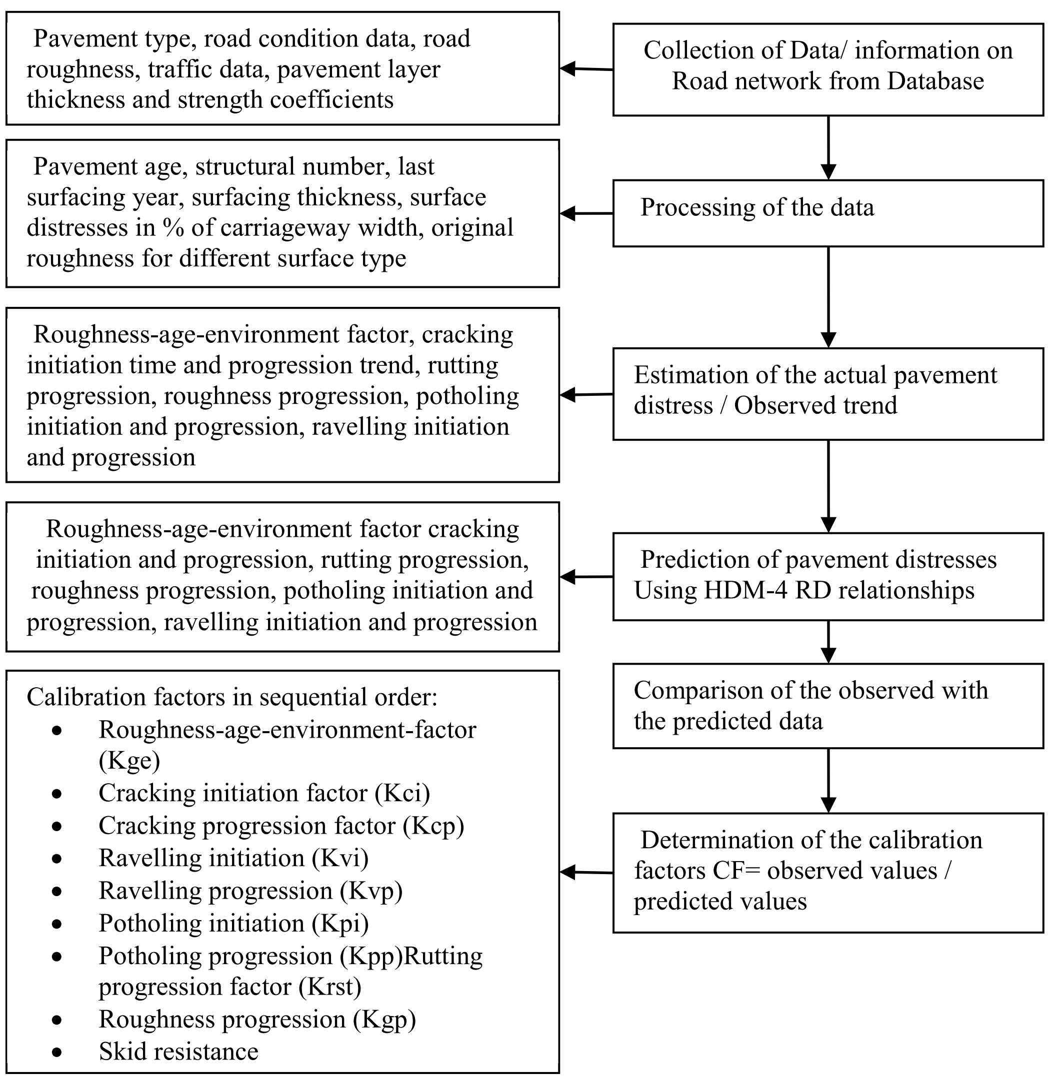



Figure 7.2 - Calibration Methodology Flow-Chart

## 7.4 Types of Calibration Methods

There are two methods of analysis that can be used to calibrate the HDM-4 RD models: time-series analysis and cross-sectional analysis. A review of the existing databases or systems used by Ministry of Roads and Transport (MRT) and its road agencies concluded that reliable time series data were not available for carrying out a time-series based calibration. In the absence of reliable time series data, cross-sectional analysis was carried from sampled road sections in the four climatic zones of the country through field studies.

## 7.5 Field Study

The field studies carried was with the objective of collecting reliable cross-sectional data for calibration of the HDM-4 model. Field studies and data were collected for the calibration of all the three sub models.

### 7.5.1 Road Deterioration (RD)

Road sections for field study were selected based on the following four parameters:

- ***Climatic Zones:*** Four climatic zones were identified in Ghana: Northern (Tropical Continental or Savannah); Middle Belt (Wet-Semi Equatorial); South West (South-Western Equatorial) and Coastal Belt (Dry Equatorial).

- ***Pavement Types:*** The following four bituminous pavement types were identified for the Ghana network according to HDM-4 classification: Asphalt Mix on Granular Base (AMGB); Asphalt Mix on Asphalt Pavement (AMAP); Surface Treatment on Granular Base (STGB); and Surface Treatment on Asphalt Pavement (STAP). Gravel roads are considered separately. Information and data from recent studies and research carried out by Transportation Research Laboratory (TRL) and Ghana Highway Authority (GHA) on unpaved roads were used for the calibration of gravel roads

- ***Traffic Loading:*** Three different traffic loading levels were identified as follows: Heavy Loading > 0.5 million equivalent standard axle (mesa) per year in one direction; Medium Loading 0.1-0.5 mesa/year/direction; and Low Loading < 0.1 mesa/year/direction)

- ***Pavement Age:*** Three ages have been identified: Young Age < 3 years; Middle Age 4 to 8 Years; and Old Age > 8 Years.

The selection criteria gave a matrix with cells representing 36 bituminous pavement calibration sections and a matrix with cells representing 8 gravel road calibration sections for each climatic zone. It should be noted that in some climatic zones, particularly in the Northern and South West Zones, some cells of the matrices are empty (i.e. the representative calibration sections do not exist).

### 7.5.2 Data Collection

Data collected from the selected sites are aimed at satisfying the requirements for initial calibration of HDM-4 models using cross-sectional method of analysis. The amount of data collected was limited to those described below.

For every 50 metres the following parameters were recorded:

- Rut depth, measured using a 2-metre straight edge, in mm;
- All cracking, expressed as a percentage of the total carriageway area;
- Wide cracking (> 3 mm), expressed as a percentage of the total carriageway area;
- Ravelling, expressed as a percentage of the total carriageway area;
- Potholes, in number per km; and
- Sand patch test to determine texture depth, in mm.

The direct profiling technique (Bennett et al, 2000) was used to measure the roughness. A walking profilometer apparatus (MERLIN) was used over the entire length of the selected road section in both directions. This was later expressed in terms of the International Roughness Index (IRI).

In addition to the parameters defined foregoing, the following parameters were measured and recorded at an interval of 50 metres:

- Deflection measurement using the Falling Weight Deflectometer (FWD);
- Coring to ascertain the exact thickness of the asphaltic layers;
- DCP testing to establish the granular layer thicknesses and strengths, and the type and strength of the subgrade;
- Trial pits to confirm DCP data; and
- Laboratory Tests on the recovered cores to establish their Marshal and elastic properties, together with the properties of the recovered bitumen.

### 7.5.3 Road User Effects (RUE)

Road User Effects comprise vehicle operating costs, travel time costs and accident costs. The calibration of the RUE sub-models ensured that the predicted magnitude of each VOC component and the relativity between the different components conform to that observed in Ghana. This entails collection of vehicle fleet data from different regions of Ghana. The RUE model predicts vehicle resource consumption as a function of the characteristics of each vehicle type, and the geometry, surface type and current condition of the road, under both free flow and congested traffic conditions.

The various data collection surveys and desk studies necessary for the calibration of RUE relationships were grouped as follows:

- Traffic characteristics to include traffic growth rates; axle loading survey; desired speed survey; speed-flow survey; and hourly distribution of traffic flow; and
- Vehicle resource consumption survey to include fuel and lubricating oil consumptions; parts consumption and maintenance labour; tyre wear; vehicle utilization and service life.

#### (a) Traffic Volume, Composition And Growth

Traffic volume and composition data were available from existing databases from the Road Agencies. The prediction of future traffic has always been a difficult issue in road investment appraisal yet it is one of the most important data items. The annual traffic growth rates have been estimated for each vehicle type for the road types in Ghana. A desk study was used to review existing procedures for estimating traffic growth in Ghana. A limited field survey and observation were conducted to augment available data.

**(b) Vehicle Mass And Axle Load Survey**

This is a very important relationship. The extent of over-loading and damage to the road was subjected to rigorous analysis in Ghana. The data collected were analysed together with vehicle data supplied by vehicle manufacturers regarding tare and rated gross vehicle weights.

**(c) Free Speed (Desired Speed)**

The HDM-4 speed prediction model is mechanistic, being based on physical and kinematics principles, as well as behavioural constraints (Bennett and Paterson, 2000). Consequently, the basic physical model is highly transferable and the focus of calibration was on the behavioural constraints, defined by:

- ***VDESIR*** - the desired speed of travel. *VDESIR* represents the maximum speed of travel adopted by the driver of a vehicle when no other physical constraints, such as gradient, curvature, roughness or congestion, govern the travel speed. The value of *VDESIR* is influenced by factors such as speed limits and enforcement, road safety, cultural and behavioural attitudes.
- ***Weibull Shape Parameter*** $\boldsymbol{\beta}$ - indicates how far from the constraining speeds the predicted speed will be. This parameter is functionally related to the dispersion of the underlying distribution of the constraining speeds.

The field study carried out was used to estimate the desired speed of travel (VDESIR) for the different categories of vehicles in different environments such as gradient, curvature, roughness or congestion, which govern the travel speed.

**(d) Speed-Flow (Capacity Restraint)**

The speed-flow relationship provides the HDM-4 capability to model the effects of traffic volumes on speed to enable the economics of road capacity improvement to be determined

(Odoki and Kerali, 2000). A number of field surveys were conducted to determine the ultimate capacity ($Q_{ult}$) and the speed at ultimate capacity ($S_{ult}$) also known as jam speed for the various road types (i.e. two-lane road, four-lane road, etc.) encountered in the Ghanaian road network. The surveys were carried out under different conditions of traffic flows. The information thus collected was used for the calibration of the speed-flow model used by HDM-4. The capacity at which free flow ends and the nominal capacity were determined from the information collected from the field survey.

**(e) Hourly Distribution of Traffic Volume**

Over the space of a year most roads undergo different hourly flows. Some roads that are congested during peak hours have reduced flows at night time. Other roads experience major flows at night when trucks prefer to travel. These variations are considered in HDM-4 through the provision of an hourly distribution of traffic volume. This consists of the number of hours per year that the flow is at different levels. They will also vary between road classes.

The derivation of the hourly distribution for the Ghanaian road network was based on an analysis of existing Automatic Traffic Counter data which identified major flow patterns and were used as the basis for deciding the number of flow levels to be adopted.

**(f) Vehicle Resource Consumption And Costs**

A full survey of vehicle resource consumption and costs was conducted in order to collect supplementary data to augment the existing information. The survey carried out was to collect data and information on the basic physical characteristics of the various vehicles types constituting the Ghanaian national vehicle fleet. The information from the survey was used to calibrate model parameters for the following:

- Fuel and lubricating oil consumption rates;
- Tyre type and consumption rates;
- Parts consumption;
- Maintenance labour hours;
- Vehicle utilization and service life;

- Interest rate;
- Crew costs; and
- Overhead costs.

The surveys were conducted among the main transport operators and other companies and organisations operating large vehicle fleets in Ghana. Information was also collected on unit costs (or prices) of vehicles, tyres, fuel, lubricating oil, maintenance labour, etc.

## 7.6 Adaptation

The main objective for the adaptation of HDM-4 is to enable relevant analysis compatible with the current road transport environment in Ghana.

### 7.6.1 Climatic Zones

The Meteorological Services Authority (MSA) of Ghana provided all the climatic information. The climatic averages are based on weather observations between the year 1970 to 2004, over the various climatic zones. The moisture indices for the various climatic zones were obtained from work done by Arulanandan et al. (1963). A summary of the representative zonal climatic averages are shown in Table 7.1.

Table 7.1 - Representative Average Zonal Climatic Attributes (*Source: MSA*)

| **Climatic Elements** | **Unit** | **Zone 1** | **Zone 2** | **Zone 3** | **Zone 4** |
|---|---|---|---|---|---|
| Moisture Index | - | 35 | -30 | 5 | -32 |
| Duration of Dry Season | % of Year | 25 | 50 | 40 | 60 |
| Mean Monthly Precipitation | mm | 161.9 | 61 | 108.4 | 61 |
| Mean Temperature | ºC | 26.5 | 27.3 | 26.4 | 27.5 |
| Days T > 35ºC | No. | 30 | 60 | 30 | 90 |
| Freeze Index | C-Days | n/a | n/a | n/a | n/a |
| % of time driven on snowy roads | % | - | - | - | - |
| % of time driven on wet roads | % | 50 | 30 | 50 | 30 |

### 7.6.2 Traffic Characteristics

The three main road network classifications in Ghana are feeder, urban and trunk roads each with distinctive traffic characteristics. The average percentage composition of vehicle types in a given traffic stream on the road network classes are given in Table 7.2.

Table 7.2 - Representative Average Composition on Ghana Roads

| **Vehicle Type** | **Vehicle No.** | **% Composition** | | |
|---|---|---|---|---|
| | | **Trunk** | **Feeder** | **Urban** |
| Car | 01 | 17.8 | 3.0 | 25.0 |
| Taxi | 02 | 11.8 | 5.0 | 27.0 |
| Pickup | 03 | 19.4 | 20.0 | 6.0 |
| Light Truck | 04 | 3.4 | 15.0 | 5.0 |
| Medium Truck | 05 | 4.3 | 12.0 | 5.0 |
| Heavy Truck | 06 | 4.1 | 2.0 | 5.0 |
| Articulated Truck | 07 | 7.8 | 10.0 | 5.0 |
| Small Bus | 09 | 13.6 | 6.0 | 9.0 |
| Medium Bus | 11 | 10.0 | 6.0 | 8.0 |
| Heavy Bus | 10 | 7.8 | 0.0 | 0.0 |
| Motorcycle | 12 | 0.0 | 2.0 | 5.0 |
| NMT | | | 19.0 | |
| **Total** | | **100.0** | **100** | **100.0** |

The representative Annual Average Daily Traffic (AADT) ranges have been classified into High (H), Medium (M), and Low (L). The representative ranges of AADT for trunk, feeder and urban roads networks are given in Table 7.3.

Table 7.3 - Representative Average Vehicle Composition on Ghana Roads

| **Traffic Level** | **Ranges** | | |
|---|---|---|---|
| | **Trunk** | **Feeder** | **Urban** |
| High | > 3000 | > 400 | 2000 |
| Medium | Between 1200 and 3000 | Between 400 and 150 | Between 1200 and 3000 |
| Low | < 500 | < 75 | < 500 |

The traffic growth factors representative of road and vehicle types are also provided in Table 7.4 based on information obtained from Driver, Vehicle and Licensing Authority (DVLA).

Table 7.4 - Representative Average Vehicle Growth Factors on Ghana Roads

| Road Type | Vehicle Type | Growth Type (%) |
|---|---|---|
| Trunk Road | Cars and Pick-Ups | 6.5 |
| | Others | 5 |
| Urban Road | All | 5 |
| Feeder Road | Cars and Pick-Ups | 3 |
| | Others | 5 |

### 7.6.3 Pavement Classifications

#### 7.6.3.1 Pavement Types

There are eight (8) pavement types in HDM-4, which are used in the modelling process and are listed below:

1. AMGB - Asphalt Mix on Granular Base (crushed rock);
2. AMGB – Asphalt Mix on Granular Base (lateritic sand);
3. AMAP - Asphalt Mix on Asphalt Pavement;
4. AMAB - Asphalt Mix on Asphalt Base;
5. STGB - Surface Treatment on Granular Base;
6. STAP - Surface Treatment on Asphalt Pavement;
7. STSB - Surface Treatment on Stabilised Base; and
8. AMSB - Asphalt Mix on Stabilised Base.

Based on performance and behaviour of these pavement types when in use, some pavement types were treated as composites e.g., AMGB and AMSB since AMSB are improved to behave like AMGB. Similarly STSB was evaluated as STGB. Therefore for the calibration, five of the eight HDM-4 pavement types were adapted for Ghana.

It was noted that all the above pavement types are constructed in Ghana. However, the predominant pavements types in use as depicted in percentages are shown in Table 7.5 are the surface treated types.

Table 7.5 - Distribution of Pavement Types in Ghana

| Pavement Type | Percentage (%) |
|---|---|
| AMGB & AMSB | 11.0 |
| AMAP | 3.0 |
| AMAB | 1.0 |
| STGB | 75.0 |
| STSB | |
| STAP Resealing or Double Surface Dressing | 10.0 |

### 7.6.4 Pavement Age

Pavement age classification was primarily based on the age of the pavement. Material and construction quality of the roads were also considered. Three pavement age groups were generated: (a) *old* pavements are those constructed more than 8 years ago with representative average age of 15 years; (b) *middle age* pavements constructed between 4 and 8 year ago with a representative average age of 6 years and (c) *young* pavements are those which are less or equal to 3 years. Table 7.6 gives a summary of the age profile of the various pavements considered for Ghana.

Table 7.6 - Pavement Age Classification

| Pavement | Age Range | Representative Age (Years) |
|---|---|---|
| Young | 0 - 3 | 2.0 |
| Middle | 4 - 8 | 6.0 |
| Old | > 8 | 15.0 |

### 7.6.5 Functional Classification of Road Types in Ghana

The functional classification of Ghana road network is summarized in Table 7.7are obtained from 2003 MRT Sector Review Report.

Table 7.7 - Functional Classification of Ghana Road Network

| Trunk Road | Feeder Road | Urban Road |
|---|---|---|
| National | Inter District | Major Arterial |
| Inter-Regional | Connector | Minor Arterial |
| Regional | Access | Distributor/Collector<br>Access/Local |
| - | - | |

Trunk roads are classified as follows using socio-economic considerations:

- **National** - roads linking the national capital to regional capitals, important border towns in neighbouring countries, ports and major tourist sites;
- **Inter-Regional** - these are second on the hierarchy of roads serving as important lines of communication between the various regions to ensure regional coherence; and
- **Regional** - link district capitals to their respective regional capitals or to the nearest district capitals and major industrial, trade or tourist centres.

Table 7.8 - Trunk Road Classification Summary

| Road Class | Surface Type | Traffic Range | Road Condition (IRI) |
|---|---|---|---|
| National<br>Regional<br>Inter-regional | Asphaltic concrete<br>or<br>Surface treated | >2,000 High<br>500 – 2000 Medium<br><500 Low | 1 – 4 Good<br>4 – 7 Fair<br>> 7 Poor |
| | Gravel | >500 High<br>250 – 500 Medium<br><250 Low | 1 – 6 Good<br>6 – 9 Fair<br>> 9 Poor |

The feeder road system is to ensure that rural and farming communities‟ transport needs are adequately met. The system provides access to social amenities, farm gates and markets in order to ensure competitive and stable prices for their produce. The functional classes of feeder roads are:

- **Inter-District -** roads that cross more than one district;
- **Connector -** roads that link a trunk road to either another trunk road or feeder road; and

- **Access -** roads that start from either a trunk or higher class feeder road and ends in a community.

The urban road system has been classified into four main classes:

- **Major Arterials -** roadways that serve most of the intercity trips. Principal arterials are further divided into freeways and main arterials;
- **Minor Arterials -** they augment the major arterials in the formation of a network of roads that connect urbanized areas. Travel speeds on the minor arterials can be high as those on the major arterials;
- **Distributor/Collector -** roads which primarily carry traffic within individual urbanized areas and trip distances, are usually shorter than those on the arterial roads; and
- **Access/Local -** these are streets that provide access to residence and to adjacent lands and properties and residential driveways. Through traffic is deliberately discouraged.

Table 7.9 - Urban Road Classification Summary

<table>
<tr><th>Surface Type</th><th>Road Class</th><th>Road Condition (IRI)</th><th>Traffic Range AADT</th></tr>
<tr><td rowspan="3">Asphaltic Concrete<br>Surface Treated</td><td>Arterial</td><td>1 – 5 Good<br>5 – 9 Fair<br>> 9 Poor</td><td rowspan="3">>2,000 High<br>800 – 2000 Medium<br><800 Low</td></tr>
<tr><td>Distributor/ Collector</td><td rowspan="2">1 – 6 Good<br>6 – 10 Fair<br>> 10 Poor</td></tr>
<tr><td>Local/Access</td></tr>
<tr><td rowspan="3">Gravel</td><td>Arterial</td><td rowspan="3">1 – 8 Good<br>8 – 12 Fair<br>> 12 Poor</td><td rowspan="3">> 250 High<br>50 – 250 Medium<br>< 50 Low</td></tr>
<tr><td>Distributor/ Collector</td></tr>
<tr><td>Local/Access</td></tr>
</table>

### 7.6.5 Road Works

Road works are divided under two main categories under HDM-4 namely: preservation and development.

#### 7.6.5.1 Preservation (Maintenance)

Preservation of the existing pavements involves performing maintenance works to offset the deterioration of roads as well as lowering road user cost by providing a smooth running and keeping the road open on a continuous basis.

#### 7.6.5.2 Development

The objective of development works is to expand the capacity of the network through the provision of stronger pavement and the improvement of the geometric characteristics in order to minimize the total cost of road transportation and mitigate environmental impacts.

### 7.6.6 Work classes

Within the above two broad categories, road works are considered in classes. The works classes consider road works in terms of their frequency of application and the budget head used to fund them. The following are the work classes adapted in line with planning and programming practices in Ghana:

(A) **Maintenance Class:** In Ghana there are three work classes under maintenance, which is more in line with the HDM-4 classification; namely:

1. Routine Maintenance;
2. Periodic Maintenance; and
3. Emergency.

(B) **Development Class:** The works in the development class in accordance with practices in Ghana have been grouped into two namely:

1. Improvement of existing roads through major rehabilitation, reconstruction and upgrading

2. New construction, which involves the construction of new roads such as missing links in the existing networks of new alignment

A summarized hierarchical structure of category, class and type of works and activities as adapted for Ghana is given in Table 7.10.

Table 7.10 - Summary of Class and Type of Works (Activities) Adapted for Ghana

| **Work Category** | **Work Class** | **Work Type** | **Work Activity/Operation** |
|---|---|---|---|
| Preservation | Routine Maintenance | Surface Maintenance | Pothole Patching<br>Repair of Depressions<br>Edge Failure repair<br>Grading<br>Reshaping<br>Sectional Patching |
| | | Drainage Maintenance | Ditch Cleaning<br>Re-excavation of Drainage Ditches<br>Cleaning and Repair Culverts<br>Desilting Culverts<br>Repair of Cracks on Drainage Structures<br>Erosion and Scour Repairs |
| | | Road Side Maintenance | Grass Cutting<br>Tree/Bamboo Clearing<br>Repair & Replacement of Guide Post and Guard Rails<br>Road Line Marking |
| | Periodic Maintenance | Regravelling | Placing of adequate subbase gravel on an existing gravel road to strengthen the pavement. |
| | | Resealing | Placing of a fresh seal coat on an existing bituminous surfaced to seal cracks and improve resistance. |
| | | Overlay | Placing of asphaltic concrete on an existing bituminous surfaced or asphaltic concrete road to strengthen the pavement |
| | | Partial Reconstruction | Scarifying of existing bituminous surfaced road, strengthening the base layer with addition of adequate thickness of base material and applying surface treatment. |

Table 7.10 - (Continued)

| Work Category | Work Class | Work Type | Work Activity/Operation |
|---|---|---|---|
| Preservation | | Minor Rehabilitation | Improvement of an unpaved or paved road including widening, earthworks and construction of drainage structures. |
| | Emergency | Special Works | The emergency works has a lumped sum budget. It comprise activities the following activities among others:<br>Clearing debris, Repairing Washouts, Subsidence, Traffic Accidents etc. |
| Development | Improvement | Reconstruction | Full Pavement construction and drainage structures may involve widening and re-alignment |
| | | Major Rehabilitation | Mainly Partial Pavement reconstruction and drainage structures may not involve widening and re-alignment. |
| | | Upgrading | Upgrading the Road Surface Class<br>Gravel to Bituminous Surface Treated (BST)<br>BST to Asphaltic Concrete |
| | New Construction | New Section | Dualization<br>Missing Links |

### 7.6.7 Speed Flow

The average speed of each vehicle type was used to calculate vehicle operating costs, travel time, energy use and emissions. The speeds of motorized transport (MT) vehicles are influenced by a number of factors, which include:

- Vehicle characteristics;
- Road severity characteristics, for example, road alignment, pavement condition, etc;
- The presence of non-motorised transport (NMT);
- Roadside friction, for example, bus stops, roadside stalls, access points to roadside development, etc.; and
- Total MT traffic volume.

The speed-flow model adopted for each motorized transport (MT) is the three-zone model proposed by Hoban et al. (1994). This model is illustrated in Figure 7.3.

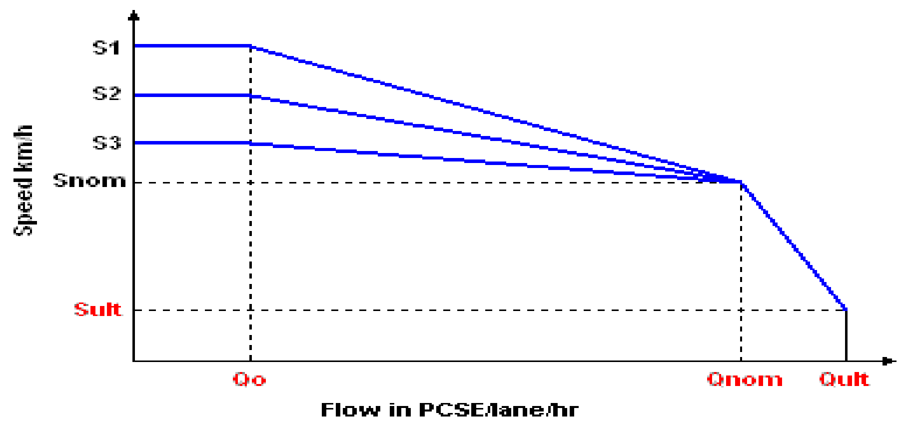



Figure 7.3 - Speed-Flow Model (Hoban et al., 1994)

The following notation applies to Figure 7.3:

| | |
|---|---|
| $Q_o$ | - the flow level below which traffic interactions are negligible in PCSE/h; |
| $Q_{nom}$ | - nominal capacity of the road (PCSE/h); |
| $Q_{ult}$ | - the ultimate capacity of the road for stable flow (PCSE/h); |
| $S_{ult}$ | - speed at the ultimate capacity, also referred to as jam speed (km/h); |
| $S_{nom}$ | - speed at the nominal capacity (km/h); |
| $S_1$ to $S_3$ | - free flow speeds of different vehicle types (km/h); and |
| PCSE | - passenger car space equivalents. |

The model predicts that below a certain volume there are no traffic interactions and all vehicles travel at their free speeds. Once traffic interactions commence the speeds of the individual vehicles decrease until the nominal capacity where all vehicles will be travelling at the same speed, which is estimated as 85% of the free speed of the slowest vehicle type. The speeds can then further decrease towards the ultimate capacity beyond which unstable flow will arise. The parameters for capacity and speed flow for various road types are given in Table 7.11.

Table 7.11 - Capacity and Speed-Flow Model Parameters for Types Of Roads

| Speed Flow Type | Width (m) | XQ1 = ($Q_o/Q_{ult}$) | XQ2 = ($Q_{nom}/Q_{ult}$) | $Q_{ult}$ (PCSE/hour/lane) | $S_{ult}$ (km/hr) |
|---|---|---|---|---|---|
| Feeder 2-Lane Narrow | < | 0.1 | 0.8 | 1350 | 23 |
| Feeder 2-Lane Standard | 7 | 0.1 | 0.9 | 1400 | 25 |
| Urban 2-Lane Narrow | 7 | 0.1 | 0.8 | 1350 | 23 |
| Urban 2-Lane Standard | 7 | 0.1 | 0.9 | 1400 | 25 |
| Urban 2-Lane Wide | >7 | 0.2 | 0.9 | 1600 | 30 |
| Trunk 2-Lane Narrow | 7 | 0.1 | 0.9 | 1400 | 25 |
| Trunk 2-Lane Standard | >7 | 0.1 | 0.9 | 1400 | 25 |

### 7.6.8 Traffic Flow Pattern

The levels of traffic congestion vary with the hour of the day and on different days of the week and year. To take account of this, the number of hours of the year for which different ranges of hourly flows are applicable was considered. Defining the distribution of hourly flows over 8760 hours of the year allows the AADT data to be converted to hourly flows. The flow pattern considered traffic distribution range from the highly varied urban flow to the nearly free flow on inter-regional roads.

## 7.7 Calibration

Calibration of HDM-4 is intended to improve the accuracy of predicted pavement performance and vehicle resource consumption. The pavement deterioration models incorporated in HDM-4 were developed from results of large field experiments conducted in several countries. A fundamental assumption made prior to using HDM is that the pavement performance models will be calibrated to reflect the observed rates of pavement deterioration on the roads where the models are applied

### 7.8 Calibration of Road Deterioration Models

#### 7.8.1 Model Sensitivity

The HDM-4 road deterioration models have a number of adjustment factors (or parameters) and it is important to be aware of the general level of sensitivity of the model to each parameter so that appropriate emphasis can be given to important parameters and less emphasis to second or third order effects. The influences of individual parameters differ according to the particular parameter, the particular result being considered, and the values assigned to other parameters in the particular analysis. The sensitivity of results to variations in a parameter therefore varies somewhat under different circumstances. For a detailed treatment of this issue refer to Volume 5 of HDM-4 documentation series (Bennett and Paterson, 2000).

The HDM-4 road deterioration model parameters to calibrate in sequential order are:

1. Roughness-age-environment coefficient ($K_{ge}$);
2. Cracking initiation ($K_{ci}$);
3. Cracking progression ($K_{cp}$);
4. Ravelling initiation ($K_{vi}$);
5. Ravelling progression ($K_{vp}$);
6. Potholing initiation (due to cracking $K_{pic}$, due to ravelling $K_{pir}$);
7. Potholing progression ($K_{pp}$);
8. Rutting: initial densification ($K_{rid}$, structural deformation ($K_{rst}$), plastic deformation ($K_{rpd}$), wear by studded tyres ($K_{rsw}$);
9. Roughness progression ($K_{yp}$ due the above defects); and
10. Skid resistance and texture depth ($K_{sfc}$).

For convenience, these calibrations (or adjustment) factors can be grouped into just two classes: high impact and low impact. The roughness-age-environment factor is the most important, due to the wider range of its values, followed by the cracking initiation and progression factors. The general roughness progression factor has low priority, despite its moderate sensitivity, because its range is small based on many inter-country validation studies.

### 7.8.2 Roughness – Age – Environment Factor: S-I

This factor, which determines the amount of roughness progression occurring annually on a non-structural time-dependent basis, is related to the pavement environment and is effectively an input data parameter rather than a calibration adjustment. The factor adjusts the environment coefficient, m, which has a base value of 0.023 in the model, representing 2.3 percent annual change independent of traffic (Bennett and Paterson, 2000), that is:

$$\triangle R_{te} = K_{ge} 0.023 R_t \qquad \text{.........Equation 7.1}$$

Where $K_{ge}$ is the roughness age-environment calibration factor (HDM-III), $\triangle R_{te}$ the change in the roughness component due to environment in the 1st-year analysis time increment and $R_t$ the roughness at the beginning of the year. In HDM-4, the calibration factor $K_{gm}$ is equivalent to the HDM-III factor $K_{ge}$. The following discussion uses $K_{ge}$.

**(A) Method 1**

For a Level 1 Calibration, the values are established based on the general environmental conditions and the road construction, drainage standard. This is done as follows:

*Step 1*…………Identify the environment applicable to the immediate vicinity of the road project in terms of the classifications provided in Table 7.12.

*Step 2*…………Select the appropriate value of *m* from Table 7.12 according to the environmental classifications.

*Step 3*…………Determine the effective *m* value, $M_{eff}$, by multiplying m by a factor $k_m$ according to the standard of road construction and drainage Table 7.13, as follows:

$$M_{eff} = mk_m \qquad \text{......Equation 7.2}$$

Where m is the environmental coefficient given in Table 7.12

*Step 4*…………Calculate $K_{ge}$ from $M_{eff}$ as follows:

$$K_{ge} = \frac{M_{eff}}{0.023} \qquad \text{..........Equation 7.3}$$

Table 7.12 - Recommended Values of Environmental Coefficient, m

| Moisture Classification | Temperature Classification | | | |
|---|---|---|---|---|
| | **Tropical** | **Subtropical non-freezing** | **Temperate – shallow freeze** | **Temperate – extended freeze** |
| Arid | 0.005 | 0.010 | 0.025 | 0.040 |
| Semi-arid | 0.010 | 0.016 | 0.035 | 0.060 |
| Sub-humid | 0.020 | 0.025 | 0.060 | 0.100 |
| Humid | 0.025 | 0.030 | 0.100 | 0.200 |
| Per-humid | 0.030 | 0.040 | - | - |

Table 7.13 - Modifying Factor of Environmental Coefficient for Road Construction and Drainage Effects km

| **Construction and Drainage** | **Non-Freezing Environments** | **Freezing Environments** |
|---|---|---|
| High standard materials and drainage, for example, motorways, raised formation, free-draining or non-frost-susceptible materials, special drainage facilities | 0.6 | 0.5 |
| Material quality to normal engineering standards; drainage and formation adequate for local moisture conditions, and moderately maintained | 1.0 | 1.0 |
| Variable material quality in pavement, including moisture or frost-susceptible materials; drainage inadequate or poorly maintained, or formation height near water table. | 1.3 | 1.5 |
| Swelling soil subgrade without remedial treatment | 1.3 – 2.0 | 1.2 – 1.6 |

Although there are four main climatic zones in Ghana, the computed values of $K_{ge}$ range between 0.8 and 1.4, when the factors shown in Tables 7.12 & 7.13 are considered with respect to the zones.

The following sections describe Second Level calibration of these factors for the Ghanaian conditions using available data.

### 7.8.3 Cracking Initiation/Progression

Cracking initiation is predicted in terms of the surfacing age when first visible crack appears on the road surface. Cracking is deemed to have started at the age of the surface. HDM-4 effectively initiates cracking when 0.5% of the surface area is cracked. Therefore sections with crack area less than 0.5% were not used for the 2nd level calibration.

In each of the surfacing-climatic groups, a minimum of 15 pavement sections (of 500m, 2-lane length) was observed for cracking. All sections with total crack area less than 5% from range high, medium and low traffic levels and with surfacing age between 6 to 15 years representing the medium and old pavements respectively were selected for the evaluation of surface age when cracking was initially observed. Therefore, in the selection of the road sections for the observation of crack initiation, all sections were considered including young, middle and old pavement. However, young age sections with 0% cracks were eliminated. Middle and old with 0% crack area were considered to ensure that the defects are adequately measured within the HDM-4 study period especially for the AMGB pavements.

The time predicted for crack initiation by running the HDM with the surface distress default values set to 1.0 was obtained for different pavement types in the four climatic zones.

The cracking initiation factors $K_{ci}$ were calculated for each climatic zone. The summaries of calibrated crack initiation and progression adjustment factors for Ghana are given in Tables 7.14 – 7.16. The reciprocal value of the initiation factor is the progression factor from the HDM-4 manual for calibration.

Table 7.14 - Cracking Initiation for AMGB and AMSB

| **Climate Zone** | **Mean Observed Values (yrs)** | **Mean Predicted Values (Years)** | **$K_{ci}$** | **$K_{cp} = 1/K_{ci}$** |
|---|---|---|---|---|
| Dry Sub Humid | 7.70 | 4.0 | 1.93 | 0.52 |
| Moist Sub Humid | 10.10 | 4.0 | 2.53 | 0.40 |
| Humid | 7.69 | 3.0 | 2.56 | 0.39 |
| Semi-Arid | 7.70 | 4.0 | 1.93 | 0.52 |

Table 7.15 - Cracking Initiation for STGB/STSB

| Climate Zone | Mean Observed Values (yrs) | Mean Predicted Values (Years) | $K_{ci}$ | $K_{cp} = 1/K_{ci}$ |
|---|---|---|---|---|
| Dry Sub Humid | 8.36 | 3.0 | 2.79 | 0.36 |
| Moist Sub Humid | 7.94 | 3.0 | 2.65 | 0.38 |
| Humid | 9.38 | 2.0 | 4.69 | 0.21 |
| Semi-Arid | 8.36 | 3.0 | 2.79 | 0.36 |

Table 7.16 - Cracking Initiation for AMAP

| Climate Zone | Mean Observed Values (yrs) (o) | Mean Predicted Values (Years) (p) | $K_{ci} = o/p$ | $K_{cp} = 1/K_{ci}$ |
|---|---|---|---|---|
| Dry Sub Humid | 8.67 | 5.0 | 1.73 | 0.58 |
| Moist Sub Humid | 7.83 | 5.0 | 1.57 | 0.64 |
| Humid | 7.90 | 4.0 | 1.98 | 0.51 |
| Semi-Arid | 8.67 | 5.0 | 1.73 | 0.58 |

The plots of calibrated versus uncalibrated cracking deterioration trends for selected pavement types for each climate zone are shown in Appendix F.1.

### 7.8.4 Ravelling Initiation/Progression

Ravelling as a surface deterioration phenomenon is prevalent on surface treated roads. In Ghana, 85% of the bituminous roads are surface treated and therefore very relevant. In all, 38 road sections were observed. All these sections have low positive incidence of ravelling (0 < area < 10%).

Table 7.17 - Breakdown of the Sections for Each Climate Zone

| Climate Zone | Number of Sections | Number of Lanes |
|---|---|---|
| Dry Sub Humid/ Semi-Arid | 14 | 2 |
| Humid | 8 | 2 |
| Moist Sub Humid | 16 | 2 |

Based on the persistence incidence of ravelling, the ravelling age was set at 0.9 AGES (AGES = Age of surface). The observed mean ravelling initiation age was evaluated based on the above sections and the values are summarised in Table 7.18.

Table 7.18 - Ravelling initiation and progression for STGB and STSB

| Climate Zone | Mean Observed Values (yrs) | Mean Predicted Values (Years) | Initiation Factor $K_{vi}$ | Progression Factor $K_{vp}$ |
|---|---|---|---|---|
| Dry Sub Humid | 6.30 | 11.0 | 0.57 | 1.75 |
| Moist Sub Humid | 7.14 | 11.0 | 0.65 | 1.54 |
| Humid | 8.44 | 14.0 | 0.60 | 1.67 |
| Semi-Arid | 6.30 | 11.0 | 0.57 | 1.75 |

The plots of calibrated versus uncalibrated ravelling deterioration trends for STGB/STSB pavement type for each climate zone are shown in Appendix F.1.

### 7.8.5 Rutting

There are four (4) components of rutting, namely:

1. Initial densification;
2. Structural deformation;
3. Plastic reformation; and
4. Surface wear due to studded tyres.

Due to the availability of reliable data from GHA"s Pavement Management and Maintenance Programme (PMMP) from 2001 to 2005 for the whole trunk road network, the time series approach was adopted for the calibration of the mean rut depth. Also, the procedure used in collecting structural rut depth data in PMMP conforms to what has been outlined for the second level calibration of HDM-4. The linear regression method was used to determine the rut depth progression as a result of structural deformation. The initial densification, which is a function of degree of relative compaction of the base, subbase and selected subgrade, layers. The effect is predominant during the first three years of the pavement life. The value of $K_{rid}$ was set to 7. The effect of cracking and rainfall (climate) influence rutting attributed to structural deformation.

Plastic deformation occurs in thick bituminous roads (asphalt) under high temperature and heavy traffic loading and stationary loads as experienced at road intersections and on the national trunk roads. In the case of plastic deformation, the calibrated adjustment factors, $K_{rpd}$ were set to zero due to lack of presence of thick bituminous pavements in Ghana.

The details of the associated graphs for each of the pavement type and the climate zones are given in Appendix F.1.The surface types were combined and rut depth adjustment factors determined were used as representatives for the field observations.

Table 7.19 - Summary of the Rutting Initiation and Progression Factors

| Climate Zone | AMAP | | | AMGB | | |
|---|---|---|---|---|---|---|
| | **Slope Observed** | **Slope Predicted** | **$K_{rst}$** | **Slope Observed** | **Slope Predicted** | **$K_{rst}$** |
| Semi - Arid/Dry Sub Humid | 0.3143 | 0.2458 | 1.28 | 0.3143 | 0.9153 | 0.34 |
| Moist Sub-Humid | 0.4221 | 0.2672 | 1.58 | 0.4107 | 0.3475 | 1.182 |
| Humid | 0.53 | 0.282 | 1.88 | 0.53 | 0.2622 | 2.02 |

| Climatic Zone | STGB/STSB | | |
|---|---|---|---|
| | **Slope Observed** | **Slope Predicted** | **$K_{rst}$** |
| Semi – Arid/Dry Sub Humid | 0.5001 | 0.5037 | 0.9929 |
| Moist Sub-Humid | 0.4421 | 0.4547 | 0.9723 |
| Humid | 0.4214 | 0.4443 | 0.9485 |

The plots of calibrated versus uncalibrated rutting deterioration trends for selected pavement types for each climate zone are shown in Appendix F.1.

### 7.8.6 Roughness Progression

This was not calibrated because of its dependence on the various components of defects, which have already been calibrated. The roughness progression factor $k_{yp}$ was therefore set to 1. The plots of calibrated versus uncalibrated roughness deterioration trends for selected pavement types for each climate zone are shown in Appendix F.1.

### 7.8.7 Potholing

In Ghana, much as the occurrence of potholes is usual on the road network, it is not highly prevalent and severe in substantial portions of the road network. However, repairs of potholes are not quickly attended to.

The potholing progression is a function of ravelling, structural cracking and enlargement. The pothole initiation and progression factors $K_{pi}$ and $K_{pp}$ respectively should be calculated if the observed and the predicted varies significantly (i.e. NPT – No. of potholes per kilometre).

### 7.8.8 Edge-break

This is characteristic of narrow roads. In Ghana, most of the rehabilitation and maintenance of the roads are based on carriageway, which is adequate. However, edge-break on the roads are prevalent on roads with unpaved shoulders where human activities are quite high.

### 7.8.9 Texture Depth

There are three types of texture: mega, macro and micro textures which influence from roughness down to skid resistance. The factor $K_{td}$ will be calibrated from the PMMP data.

### 7.8.10 Skid Resistance

Indicative value was calculated using aggregate descriptions of AGES and condition for which default HDM-4 values were found acceptable.

### 7.8.11 Gravel Roads

In the case of gravel/unsealed roads the main areas of concern under road deterioration are material loss and roughness progression.

GHA in conjunction with TRL have undertaken an elaborate study on Ghanaian roads. The data and conclusions drawn after the research have been used to calibrate the HDM-4. For the works effects (WE), the most importance maintenance work is re-gravelling. The gravel loss is predicted using a PI of 14.8, the average value from the study. It was then used to calculate the observed gravel loss where a rotation factor of 0.8 was obtained. The construction defect for the road-base is derived from the defects:

- Poor gradation of materials;
- Poor aggregate chips; and
- Poor compaction.

The summation of these defects gives the value for the cement bound (CBD) base and it is summarised in Table 7.20.

Table 7.20 - Summation of Derived Construction Defects

| Construction Defects | CBD AM | CBD ST | Default Values in HDM4 |
|---|---|---|---|
| Poor Gradation of Materials | 0 | 0.3 | 0.5 |
| Poor Aggregate Shape | 0.1 | 0.2 | 0.5 |
| Poor Construction | 0.1 | 0.2 | 0.5 |
| **Summation** | **0.2** | **0.7** | **1.5** |

Table 7.21 - Effect of Works on Roughness and Rut Depth

| Work Type | Initial Roughness $RI_o$ m/km | | Rut Depth $R_{do}$ | |
|---|---|---|---|---|
| | AM | ST | AM | ST |
| New Construction and Reconstruction | 2 | 2.8 | 0 | 0 |
| Overlay | 2 | - | 15% of RD before Works | |
| Reseal | - | 2.8 | - | - |
| Texture Depth (TD) and Skid Resistance | Use HDM-4 Default Values | | | |
| Cracking, Ravelling etc. | Set All to Zero | | | |

## 7.9 Calibration of Works Effects

### 7.9.1 Material and Construction Quality

The standard of construction is dependent on the materials, degree of compliance with design specifications, construction tolerances and the level of site supervision. The key defect indicators are:

- Construction defect indicator for surfacing (CDS), which influences the initiation of cracking and ravelling, and rutting due to plastic deformation;
- Construction defect indicator for the road-base (CDB), which influences the formation of potholes; and
- Relative compaction of the whole pavement (COMP), which affects rutting.

Materials used for road construction in Ghana are based on specification provided in the "Technical Specification for Road and Bridge Works in Ghana". Even though the technical requirements are clearly spelt out, in practice, there are instances where sub-standard materials have been used.

The following parameters were used to depict the general situation in Ghana. However, in specific instances where these parameters may not reflect the actual situation, the necessary adjusting in terms of material and construction quality should be made.

Table 7.22 - Generalised State of the Pavement in Ghana

| **Parameter** | **AM (Asphalt Mix)** | **ST (Surface Treatment)** |
|---|---|---|
| *1. Construction Quality Defect Indicators* | | |
| CDS (Construction Defects for Surfaces) | 1.2 | 1.1 |
| CBD (Construction Defects for Sub base | 0.55 | 0.55 |
| *2. Compaction* | 95% | 90% |
| *3. Previous Surface Condition* | | |
| PACA (Previous Areas of Cracks) | 20% only on AMAP | - |
| PACW (Previous Area of Wide Cracks) | 5% only on AMAP | - |

## 7.10 Calibration of Road User Effects

### 7.10.1 Representative Vehicles

There are ten (10) representative motorised vehicles and three (3) non-motorized vehicles. The representative vehicle, the class and other characteristics are given in Table 7.23.

It was assumed that a common National Vehicle Fleet is used on the entire road network as there are few restrictions on vehicles moving on the National Road Networks. Working hours for vehicles were assumed to be 10 hours for 300 days in a year. The Constant Life approach was used for depreciation prediction for private cars as the operating cost is not taken into account. For all the other types of vehicles, the Optimal Life approach was used.

### 7.10.2 Vehicle Mass

The vehicle mass influences the vehicles, fuel and tyre consumption and through the associated heavy vehicle damage factor, has a major impact on the rate of pavement deterioration.

The effect of vehicle mass on pavement in flat terrain may not be serious. However, in gradients fuel consumption is proportional to the vehicle mass.

In Ghana, axle load enforcement is now being taken as a major issue. The Axle Load Policy has been prepared. Vehicles in Ghana are generally overloaded based on data from axle load weigh stations and studies carried out so far indicate that over 31% of heavy trucks and buses operating in Ghana are overloaded. Sometimes the excess loadings are as much as 7 and 10 tonnes for heavy buses and goods trucks respectively. Overloading is a major contributing factor to pavement damage in Ghana. Loading practices in Ghana for commercial vehicles are that they are always fully loaded from destination before start of journeys. In instances where they are “chartered”, they may be half-full. For private cars 50% are mostly full during the working days of the week while the rest are half empty.

Based on the formula and observations of the loading practices in Ghana, the vehicle masses for the various classes of vehicles (m) were calculated.

Table 7.23 - Representative Vehicle Loading Proportions

| **Vehicle Type** | **Class** | **% of Vehicles** | | | **Overload per Axle (tonnes)** |
|---|---|---|---|---|---|
| | | **Full = $P_f$** | **Half Full = $P_n$** | **Empty = $P_e$** | |
| Bicycle | Bicycle | 100 | - | - | - |
| Car | Passenger | 50 | 30 | 20 | - |
| Light Truck | Truck | 70 | 20 | 10 | 7 |
| Medium Truck | Truck | 70 | 20 | 10 | 10 |
| Heavy Truck | Truck | 70 | 20 | 10 | 10 |
| Medium Bus | Bus | 90 | 10 | 0 | 7 |
| Heavy Bus | Bus | 90 | 10 | 0 | 7 |
| Articulated Truck | Truck | 90 | 0 | 10 | 7 |
| Motorcycle | Motorcycle | 10 | 90 | - | - |

Table 7.24 - Representative Vehicle Masses

| **Vehicle Type** | $m = \frac{PeTARE + Ph(0.5TARE + 0.5GVW) + Pf(GVW) + Po(zoGVW)}{100}$ .............*Equation 7.4* Where *Pi* is the percentage of vehicle empty, half full, full or overloaded; GVW the manufacturer‟s gross weight, TARE the empty weight and zo the overloaded weight relative to the GVW in decimal |
|---|---|
| Bicycle | 100 |
| Car | 1600 |
| Light Truck | 6250 |
| Medium Truck | 11,620 |
| Heavy Truck | 27,300 |
| Small Bus | 3,720 |
| Medium/Heavy Bus | 9,000 |
| Articulated Truck | 38,000 |
| Motorcycle | 200 |

### 7.10.3 Vehicle damage factor

In Ghana, the measure of damage of roadway is based on Equivalent Standard Axle Load (ESAL) of 8.2 tonnes for a dual wheel on a single axle.

### 7.10.4 Desired speed of travel (VDESIR)

VDESIR represents the maximum speed of travel adopted by a driver when there are no other physical constraints such as gradient, curvature, roughness etc. The desired speed of travel for vehicle types were observed on a number of road sections and established the observed mean operating speeds for the vehicle types. The predicted Average Operating Speed was obtained by running HDM-4 with the default values.

Table 7.25 - Predicted Average Operating Speeds

| Vehicle Type | Mean Operating Speed km/h (observed) (o) | Predicted Average Operating Speed km/h (p) | Ratio = (o)/(p) |
|---|---|---|---|
| Car | 74 | 98.33 | 0.753 |
| Taxi | 74 | 98.33 | 0.753 |
| Pickup | 80 | 95.26 | 0.840 |
| Light Truck | 67 | 76.65 | 0.874 |
| Medium Truck | 65 | 85.58 | 0.760 |
| Heavy Truck | 62 | 89.38 | 0.694 |
| Articulated Truck | 62 | 83.69 | 0.741 |
| Small Bus | 72 | 88.95 | 0.809 |
| Medium Bus | 74 | 83.93 | 0.882 |
| Heavy Bus | 75 | 88.77 | 0.845 |
| Motorcycle | 65 | 81.40 | 0.799 |
| | | | **Avg. = 0.794** |

The ratios established were used to adjust the VDESIR used in the modelling. The HDM-4 was run interactively using the same parameters as prevailed in the speed survey and varying the "ß" parameter until the predicted and the field observed speeds agreed. The final ß values which give same speeds for the observed and the predicted are shown in Table 7.28.

Table 7.26 - Desired Speed Adjustment Parameters

| Vehicle Type | "ß" value |
|---|---|
| Car | 0.467 |
| Taxi | 0.467 |
| Pickup | 0.361 |
| Light Truck | 0.357 |
| Medium Truck | 0.338 |
| Heavy Truck | 0.524 |
| Articulated Truck | 0.441 |
| Small Bus | 0.406 |
| Medium Bus | 0.338 |
| Heavy Bus | 0.333 |
| Motorcycle | 0.433 |

The average of the ratios of the speeds (predicted/observed) was used to determine VDESIRMULT (VDES multiplying factor). The role of ß is important in understanding the calibration for the speed model. The "ß" (draw down measure) indicates how far from the constraining speeds the predicted speeds will be. The predicted speed is the probabilistic minimum of the constraining speed.

The greater the value of ß, the further away the predicted mean speed will be for the constraining speeds. Therefore when ß equals to zero (0) it means there are no constraints.

### 7.10.5 Fuel Consumption

This was based on raw fuel consumption from on-road measurement travelling at different speeds in different parts of the country for the representative vehicle types on the three main road types (Gravel, Surface Treated and Asphalt Concrete). Consumption for different road conditions was observed (free flow and congested – medium and high).

The various average speeds under these conditions were observed. The observed average fuel consumption for the various vehicle types was computed for the country. HDM-4 default fuel consumption values were used for new and similar vehicles. HDM-4 default values for idle fuel consumption were also used. Calibrated and default values are shown in Table 7.27.

Table 7.27 - Calibrated and HDM-4 Default Parts Consumption Values

| Type of Vehicle | Year | IRI = 3.5 (Default) | IRI = 3.5 | IRI = 5.5 (Road Condition) | IRI = 7.5 | IRI = 9.5 | IRI = 11.5 | IRI = 16 |
|---|---|---|---|---|---|---|---|---|
| Articulated Truck | 2005 | 0.356 | 0.245 | 0.273 | 0.293 | 0.312 | 0.333 | 0.385 |
| | Total | 0.356 | | | | | | |
| Car | 2005 | 0.217 | 0.282 | 0.362 | 0.442 | 0.523 | 0.603 | 0.776 |
| | Total | 0.217 | | | | | | |
| Heavy Bus | 2005 | 0.119 | 0.138 | 0.195 | 0.240 | 0.281 | 0.322 | 0.413 |
| | Total | 0.119 | | | | | | |
| Heavy Truck | 2005 | 0.270 | 0.203 | 0.226 | 0.242 | 0.258 | 0.276 | 0.318 |
| | Total | 0.270 | | | | | | |
| Light Truck | 2005 | 0.159 | 0.147 | 0.187 | 0.218 | 0.248 | 0.279 | 0.348 |
| | Total | 0.159 | | | | | | |
| Medium Bus | 2005 | 0.114 | 0.131 | 0.184 | 0.226 | 0.265 | 0.303 | 0.390 |
| | Total | 0.114 | | | | | | |
| Medium Truck | 2005 | 0.255 | 0.297 | 0.359 | 0.408 | 0.454 | 0.502 | 0.612 |
| | Total | 0.255 | | | | | | |
| Motorcycles | 2005 | 0.088 | 0.088 | 0.120 | 0.147 | 0.173 | 0.199 | 0.257 |
| | Total | 0.088 | | | | | | |
| Pickups | 2005 | 0.153 | 0.142 | 0.180 | 0.210 | 0.239 | 0.268 | 0.335 |
| | Total | 0.153 | | | | | | |
| Small Bus and Vans | 2005 | 0.269 | 0.165 | 0.210 | 0.245 | 0.279 | 0.313 | 0.391 |
| | Total | 0.269 | | | | | | |
| Taxi | 2005 | 0.327 | 0.424 | 0.532 | 0.621 | 0.706 | 0.794 | 0.989 |
| | Total | 0.327 | | | | | | |

Vehicle fuel consumption values for bituminous surface in good condition (IRI = 2.5) are given in the Table 7.28. Fuel consumption for bituminous roads in fair and poor condition can also be computed.

Table 7.28 - Vehicle Fuel Consumption for Good Bituminous Road

| Vehicle Type | Fuel Consumption (FC) (L/1000 km) | | =+ FC/Power Total |
|---|---|---|---|
| | Observed | Predicted | |
| Car | 109.65 | 85.45 | 0.0870 |
| Taxi | 109.65 | 85.45 | 0.0870 |
| Pickup | 82.02 | 83.31 | 0.0561 |
| Small Bus | 107.05 | 106.41 | 0.0675 |
| Medium Bus | 163.00 | 159.00 | 0.0583 |
| Heavy Bus | 286.90 | 229.86 | 0.0730 |
| Light Truck | 170.30 | 130.00 | 0.0761 |
| Medium Truck | 205.60 | 170.50 | 0.0695 |
| Heavy Truck | 418.00 | 385.91 | 0.0611 |
| Articulated Truck | 477.40 | 477.30 | 0.0550 |
| Motorcycle | 30.00 | 26.65 | 0.7650 |

### 7.10.6 Parts Consumption

The calibration was done against a background of majority of the vehicles used which were predominantly second hand imports mainly from Europe. The road network character is mainly categorised as fair to poor with lower vehicle utilisation.

The age of the traffic vehicle meant higher parts consumption but it was counteracted by the following factors:

1. Lower standard of vehicle maintenance and service. This implies lower parts consumption;
2. Lower vehicle utilisation hence less parts consumption;
3. Availability of cheaper locally manufactured vehicle parts; and
4. The existence of cannibalism practices i.e., removing spare parts from one vehicle and fitting them on another vehicle.

A recent research work done on vehicles in Ghana as part of fulfilment for an MSc in Road Management and Engineering was modified to supplement the survey done for the calibration.

### 7.10.7 Tyre Wear

The objective of calibration of tyres is to determine the tyre wear coefficient and the wearable rubber volume. It was based on the fact that most of the tyres are "second-hand" imported from Europe. Tyres with thinner groove are considered to last longer. Legal limits in terms of depth of the grooves on the tyres were not fully followed. The quality of the rubber used in manufacturing the car tyres has an effect on the rate of wear and the volume of wearable rubber.

It was estimated that between 20-30% of vehicles in Ghana use high grade rubber car tyres (e.g., Pirelli, Michelin, Continental, Goodyear, and Firestone) while the lower grade tyres (e.g., Hankook, Yokohama etc.) comprises the remaining 70%.

Table 7.29 - Tyre Grade and Utilisation

| **Tyre Type** | **Duration** | |
|---|---|---|
| | **Time of Usage** | **Tyre Utilisation (Km)** |
| Lower Grade | 6 months | 35,000 |
| (70% of Distribution) | * 1.5 years | 30,000 |
| High Grade | 2 years | 150,000 |
| (20% of Distribution) | * 4 years | 80,000 |

* Car, Taxi

Therefore, weighted average tyre life in km in Ghana is illustrated as follows:

$$
\begin{aligned}
\text{Tyre Life (TL)} &= (0.7 \times 35{,}000) + (0.3 \times 150{,}000) \\
&= 24{,}500 + 45{,}000 \\
&= 69{,}500 \qquad \text{......Equation 7.5} \\
&\pm 70{,}000
\end{aligned}
$$

The HDM-4 model for tyre wear is based on 100% high tyre quality and by running the HDM-4, the wear coefficient for a car was 0.00204.

Using a lower quality new tyre the wear is about 50% more than the high quality tyre. Therefore, the coefficient for the car will be 0.00306.

The weighted or adjusted coefficient of wear for a car = (0.3 x 0.00204) + (0.7 x 0.00306) = 0.002754.

After running HDM-4 with the default values of 0.04, an equivalent number of new tyres will make a journey of 1000 km. Therefore, one (1) tyre equates to 25,000 km.

Running the HDM-4 with the new observed coefficient for Ghana which is 0.002754, the equivalent number of tyres per 10000 km is 0.05. Therefore, one (1) tyre will cover a journey of 20,000 km.

The adjusted coefficient formula to be used for vehicle in Ghana is derived as follows:

$$AC_k = (0.3 \times Q_k) + (0.7 \times 1.5\ Q_k) \qquad \text{.......Equation 7.6}$$

Where $Q_k$ is the default HDM-4 coefficient for 100% quality for a given vehicle type k. Therefore,

$$AC_k = 1.35\ Q_k \qquad \text{.......Equation 7.7}$$

Hence for Taxi using equation 9 above $AC_k$ = 1.35 x 0.00204 = 0.002754. The justification for this approach in calibrating the tyre parameters was based on the evaluation standards and practices in Ghana and Europe (it was used to derive the HDM-4 default values). The allowable volume of wear of tyre for vehicle in Europe is strictly enforced and the vehicle owners do comply with the standards. Therefore, the basis of calibrating the tyre-wear using the European enforcement standards for default compensate for the observed practices in Ghana (using of second hand tyres from Europe).

## 7.11 Conclusions

This chapter has described the approach, methodology and procedures used to calibrate HDM-4 models for road deterioration, works effects, and road user effects for full application on Ghana road networks. The calibration was conducted at level 1, level 2 and level 3. The availability of real (field) cross sectional and time series data ensured that the calibration process was achievable to defined HDM-4 specifics for Ghana road networks. The calibrated results compare well with HDM-4 default results realistic to the existing Ghana road network characteristics.

# CHAPTER 8

# ADAPTATION AND CALIBRATION OF MECHANISTIC-EMPIRICAL PAVEMENT DESIGN METHOD FOR GHANA

## 8.1 Introduction

After the AASHTO road test, a number of mechanistic design methodologies have been developed. The fundamental principles underlying these models are traffic loading, material and structural system response and environmental interaction (Bhutta, 1988). The two oldest mechanistic programmes in use are the SHELL method (1963, 1977) and the Asphalt Institute Method (1981, 1991).

The Shell Method is based on a layered linear-elastic system, with viscoelastic asphalt layers treated in a step-wise incremental linear fashion to simplify the nonlinear viscous effect.

The Asphalt Institute method is similar in principle to the SHELL method. Major differences are:

- The definition of traffic (ESAL), using a simplified AASHTO equivalency relationship;
- Witczak equations is used in evaluating the Hot Mix Asphalt (HMA) elastic stiffness; and
- The failure criteria are based upon the Asphalt Institute equations.

VESYS mechanistic programme relies on a full viscoelastic characterization of the HMA layers and similar in outline to both the Asphalt Institute and SHELL methods. It analyses the problem as a probabilistic rather than a deterministic hence the input data are mean values and standard deviation. It is basically a research tool and not appropriate for practical use for pavement design.

The KENLAYER was developed by Huang at the University of Kentucky in 1993. It is similar in outline to all of the mechanistic methods previously described. However the KENLAYER is significantly different as more material models are available for linear-elastic, non-linear elastic, viscoelastic and combinations thereof. Different material parameters may be entered for each season variations; there is more detailed characterization of traffic loading with respect number and speed; up to 19 material layers can be explicitly examined; the user can specify the parameters of the critical failure criteria. The programme can also be easily calibrated using the observed field failure parameter to set the platform for a given environmental condition. Considering the design parameter and composition of the pavement layers the KENLAYER is suitable for design of road pavement in Ghana tropical condition

The KENLAYER mechanistic computer program can therefore be used to design flexible pavements with no joints or rigid layers. The backbone assumption of the KENLAYER Analysis is a solution for an elastic multilayer system under a circular loaded area. The solutions are superimposed for multiple wheels, applied iteratively for non-linear layers, and iterated at various times for viscoelastic layers. As a result, KENLAYER can be applied to layered systems under different wheel load configurations viz; from single, dual, dual-tandem, or dual-tridem wheels with each layer behaving differently; linear elastic, nonlinear elastic, or viscoelastic. (Huang, 2004)

The input requirements of the KENLAYER are based on general and specific parameters. The parameters are for general analysis and some for peculiar analysis of pavement structures. The structure of the KENLAYER programme showing the various requirements and data for each type of analysis based on the material type chosen for the respective pavement layers is shown as Figure 8.1.

The Programme provides default values that have proven to give desirable results and also suggests values of material properties presented in literature based on works done on the particular subject by others researchers.  Some of the default values suggested help in the iterative processes and the general mechanism of the Programme and not necessarily material properties. Some of the input parameters require field data and laboratory testing of samples to attain more accurate results.

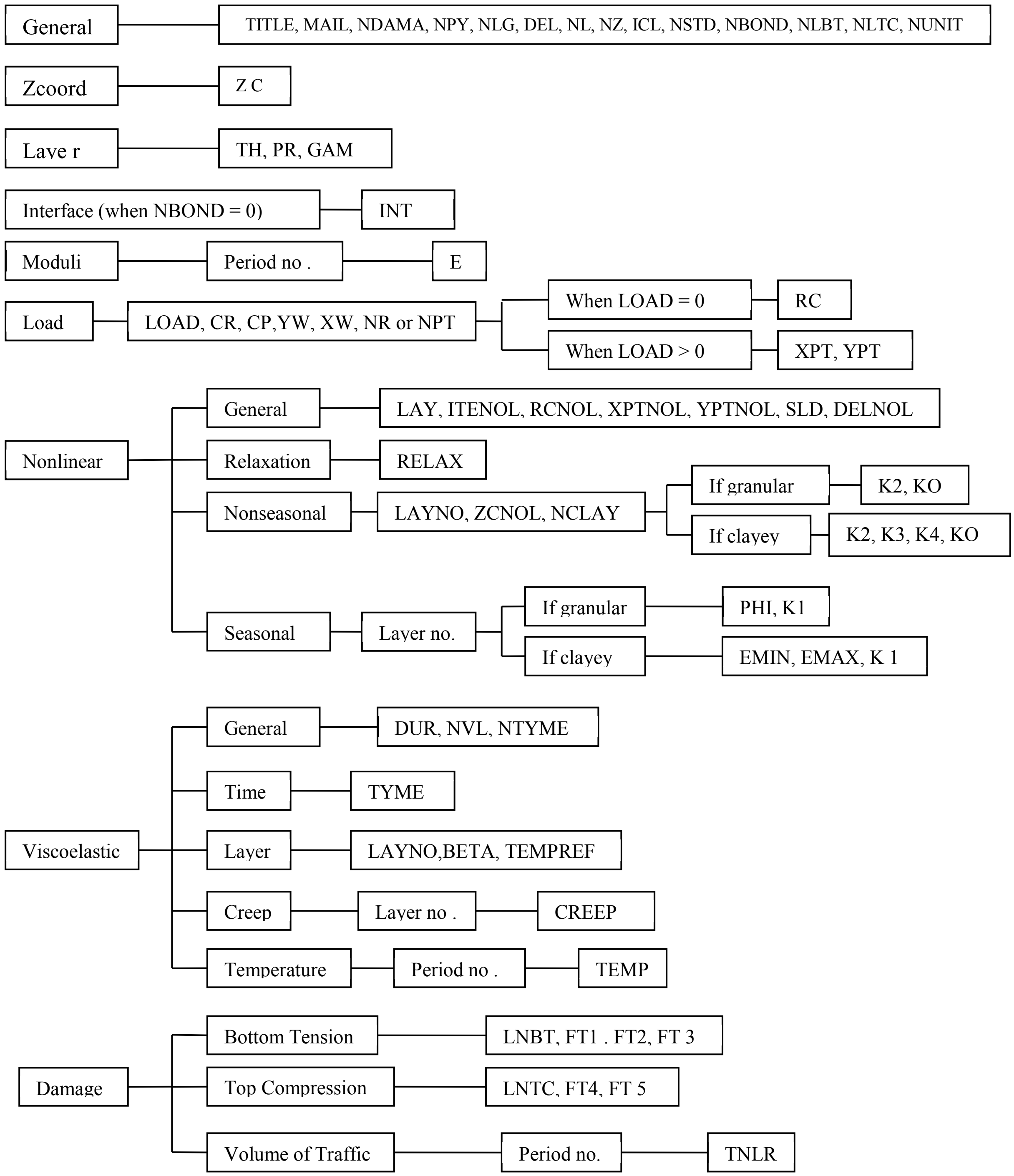



Figure 8.1 – Structure of the KENLAYER (Huang, 2004)

## 8.2 Adaptation of KENLAYER for Ghana

In order to adapt the KENLAYER for Mechanistic-Empirical Design for Ghana, Table 8.1 presents the General parameters and Specific Data requirement (Table 8.2) needed to ensure a reliable output.

Table 8.1, General Input Parameters for the KENLAYER

| | | | |
|---|---|---|---|
| **MATL** | Material Type | **NBOND** | Interface bonding |
| **NDAMA** | Damage Analysis | **NLBT** | No. of layers for bottom tension |
| **NPY** | No. of periods per year | **NLTC** | No. of layers for top compression |
| **DEL** | Tolerance for numerical integration | **NUNIT** | System of units |
| **NL** | No. of layers | **CP** | Contact pressure |
| **NZ** | No. of Z coordinates for analysis | **XPT** | X coordinates of point to be analysed |
| **ICL** | Max. cycle of numerical integration | **YPT** | X coordinates of point to be analysed |
| **NSTD** | Type of responses | **ZC** | Z or vertical coordinates |
| **RC** | Radial coordinates | **LAYNO** | Layer number of the nonlinear layer at which elastic modulus is stress dependent |
| **LOAD** | Type of loading | **ZCNOL** | Z coordinate of points for computing elastic modulus of nonlinear layer |
| **NR** | No. of radial coordinates to be analyzed under a single wheel | **NCLAY** | Type of nonlinear layer |
| **NOLAY** | No. of layers | **DUR** | Load duration |
| **ITENOL** | Max. no. of iterations | **NVL** | No. of viscoelastic layers |
| **RCNOL** | Radial coordinate for nonlinear analysis | **NTYME** | No. of time durations for creep compliances |
| **XPTNOL** | X coordinate for nonlinear analysis | **SLD** | Slope of load distribution |
| **YPTNOL** | Y coordinate for nonlinear analysis | **DELNOL** | Tolerance for nonlinear analysis |

Table 8.2 – Parameter and Source of Data for M-E Analysis with KENLAYER

| Parameter | Properties | | Source |
|---|---|---|---|
| Layer Properties | Thickness (TH) | | Measured in the Field during construction |
| | Poisson‟s Ratio (P) | | Estimated from Default Values |
| | Unit weight (GAM) | | |
| Moduli | Elastic/Resilient modulus (E) | | Back-calculated from FWD field data |
| Loading | Contact radius (CR) | | Calculated |
| | Distance between two dual tires (YW) | | Measured in the field from Axle Load Survey |
| | Tandem distance between two axles (XW) | | |
| | No. of Load repetition (NR or NPT) | | Counted in the Field |
| Nonlinear | Granular | Angle of Internal Friction (PHI) | Estimated from Default Values |
| | | Coefficient of granular layers ($K_1$) | Selected from Work done on Laterite Soils in by Nigerian Researchers. |
| | Clayey | Minimum Elastic Modulus (EMIN) | Estimated from Charts |
| | | Maximum Elastic Modulus (EMAX) | |
| | | Break Point Elastic Modulus ($K_1$) | |
| Viscoelastic | Times at which Creep Compliances are to be Specified (TYME) | | Specified from Laboratory Test |
| | Temperature Shift Coefficient (BETA) | | Estimated from Plot |
| | Reference Temperature of each Viscoelastic Layer at which Creep Compliances are measured (TEMPREF) | | Measured from Sample |
| | Creep compliances of the viscoelastic layer at reference temperature (CREEP) | | Determined from Laboratory Test |
| | Pavement temperature of each viscoelastic layer (TEMP) | | Measured from Field Studied |
| Damage | Fatigue Coefficients ($FT_1$, $FT_2$, $FT_3$) | | Estimated from Default values |
| | Permanent deformation coefficients ($FT_4$, $FT_5$) | | |
| | Total number of load repetitions for each load group during each period (TNLR) | | Estimated from Field Survey |

## 8.3 Loading Requirements

The KENLAYER takes into account the loading magnitude and configuration and the number of Load Repetitions for a given configuration. In considering the effects of vehicular and traffic in pavement design, three different procedures are considered in the KENLAYER Software.

### 8.3.1 Fixed Traffic

All wheel loads must be converted to an equivalent single-wheel load (ESWL) and usually the heaviest wheel load anticipated is used for the design. The ESWL is estimated from the following factors;

- Equal vertical stress criterion;
- Equal vertical deflection criterion;
- Equal tensile strain criterion;
- Equal contact pressure criterion; and
- Equivalent contact radius criterion.

### 8.3.2 Fixed Vehicle

With the Fixed Vehicle approach, the number of repetitions of a standard vehicle or axle load, usually 80kN single-axle load serves as the standard to which all axle loads are converted by equivalent standard axle load (ESAL) to be used for design.

### 8.3.3 Variable Traffic and Vehicle:

This procedure best suits the mechanistic methods of design, as both traffic and vehicle are considered individually by dividing the loads into a number of groups, and the stresses, strains, and deflections under each load group determined separately and used for design purposes.

The KENLAYER employs the Variable Traffic and Vehicle procedure of estimating load due to the inconsistency in equivalency for the various criteria used in determining the ESLF and ESWL, making predictions quite difficult.

## 8.4 Nonlinear Materials

If the layer is nonlinear elastic, the modulus varies with the state of stresses and a method of successive approximations is then applied until it converges. According to Hung (2004), first, the system is considered to be linear and the stresses due to multiple-wheel loads are superimposed. From the stresses, thus computed a new set of moduli for each nonlinear layer is then determined. The system is considered linear again, and the process is repeated until the moduli converge to a specified tolerance. The resilient modulus is the elastic modulus to be used with the elastic theory since most paving materials experience some permanent deformation after each load application and therefore not elastic. The resilient modulus test for granular materials and fine-grained soils is specified by AASHTO (1989) in "T274-82 Resilient Modulus of Subgrade Soils".

For the granular materials, the nonlinear coefficient $k_1$ and exponent $k_2$ must be determined.

$$E = k_1 \, \theta^{\,k2} \qquad \text{..........Equation 8.1}$$

Where; E is the resilient modulus, $k_1$and $k_2$ are experimentally derived constants from statistical analysis, and $\theta$ is the stress variant.

For fine-grained soils;

$$E = k_1 + k_3 \,(k_2 - \zeta_d), \text{ when } \zeta_d < k_2 \qquad \text{..........Equation 8.2}$$

$$E = k_1 - k_4 \,(\zeta_d - k_2), \text{ when } \zeta_d > k_2 \qquad \text{..........Equation 8.3}$$

Where; $k_1$, $k_2$, $k_3$ and $k_4$ are material constants, and $\zeta_d$ is the deviator stress.

For nonlinear analysis, Huang (2004) prescribes three methods for analyzing granular materials based on the PHI value selection (see Table 8.3). These methods indicate the point in the granular layer where the stresses are to be calculated and the nature of modulus to be expected and selected.

Table 8.3 – Selection of PHI for the Granular Materials

| Method | PHI | Position of Stress Point |
|---|---|---|
| 1 | 0 | The granular layer is subdivided into a number of layers and the modulus is calculated at the mid-point of each layer. |
| 2 | Minimum Modulus, $K_1$ | The granular layer is considered as a single layer, and an appropriate stress point is selected to compute the modulus. |
| 3 | Angle of Internal Friction | The layer is considered as a single layer with the stress point at the mid height of the layer. |

## 8.5 Viscoelastic Materials

A viscoelastic material possesses both the elastic property of a solid and the viscous behaviour of a liquid. Hot Mix Asphalt (HMA) is analyzed as a viscoelastic material. Viscoelastic materials are characterized from two methods:

1. Mechanical Model; and
2. Creep-Compliance Curve.

The KENLAYER uses the latter due to its simplicity. For viscoelastic layers, elastic solutions under static loads are obtained at a specified number of time durations, usually 11, at reference temperatures and then fitted with a Dirichlet Series so that the compliances at any other temperature can be obtained by the time-temperature superposition principle (Huang, 2004) The elastic solutions obtained at these durations are fitted with a Dirichlet series to be used for analyzing moving loads.

A direct method for analyzing viscoelastic layer systems under static loads is to assume the viscoelastic layer to be elastic with a modulus varying with the loading time and the elastic modulus is the reciprocal of the creep compliance at that loading time.

## 8.6 Damage Analyses

The damage caused by fatigue cracking and permanent deformation in each period over all load groups is summed up to evaluate the design life. The damage analysis is based on the horizontal tensile strain at the bottom of a specified asphalt layer and the vertical compressive strain on the surface of a specified layer, usually subgrade. The damage ratios for fatigue cracking and permanent deformation are evaluated and the design life of the pavement estimated as the reciprocal of the damage ratio.

Failure criterion for fatigue cracking is expressed as:

$$N_f = f_1 (\epsilon t)^{-f2} (E_1)^{-f3} \qquad \text{..........Equation 8.4}$$

In which $N_f$ = allowable number of load repetitions to prevent fatigue cracking, $\epsilon t$ = tensile strain at the bottom of asphalt layer, $E_1$ = elastic modulus of asphalt layer, and $f1, f2, f3$ = constants.

Failure criterion for permanent deformation is expressed as;

$$N_d = f4 (\epsilon c)^{-f5} \qquad \text{..........Equation 8.5}$$

Where $N_d$ = allowable number of load repetitions to limit permanent deformation, $\epsilon c$ = compressive strain on the top of subgrade, and $f4, f5$ = constants.

## 8.7 Adaptation for Use in Ghana

### 8.7.1 Calibration

As shown in Figure 8.2, in theory, there is always some accumulated strain which is not recovered when load is applied to paving materials over time. This unrecovered strain or accumulated strain is the permanent deformation as a result of the loading applied over the period. (Huang, 1993)

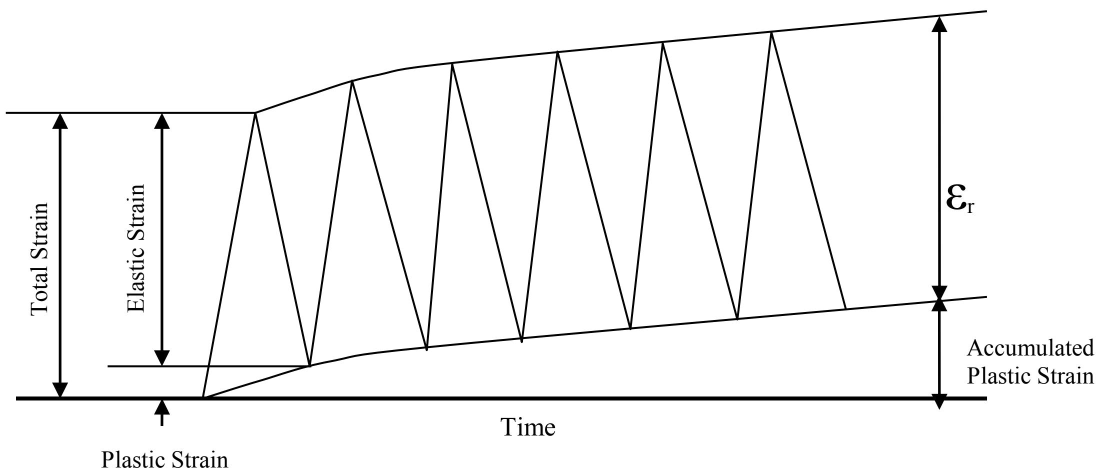



Figure 8.2 - Strain Under Repeated Loading (Huang, 1994)

Figure 8.3 is a typical deflection curve plotted from values measured from the field. These values were measured with the Falling Weight Deflectometer (FWD) and were collected for 91 hours in 7 consecutive days after each hourly passage of traffic load. The data were collected for only 13 hours in the day i.e.: (6am – 6pm), accounting for the breaks in the curve after each thirteenth hour. Although deflection values could not be collected after 6pm, an installed traffic counting equipment (Marksman) collected the volume of traffic during this period. The deflection measured at the beginning of each day is assumed to be attributed to the load from the traffic for this period (6pm - 6am).

As seen from Figure 8.2, and as can also be seen from Figure 8.3, at the start of each day, the deflections are lower as compared to the last value attained the previous day. This indicates the recovery attained by the pavement due to the reduced load during the night. The unrecovered or plastic accumulated strain for the whole period of testing is taken as the difference between Hour 2 (7am, Day 1) and Hour 80 (7am, Day 7) as indicated on the plot by $\Delta\delta$. The cumulated traffic volume for this period was estimated as the load to have caused this cumulative plastic strain over the period.

The temperature of the asphalt layer was taken at each time deflection was measured. Installed sensors also automatically recorded the temperatures in all the layers at 30 minutes intervals throughout the period. The moduli of all the pavement layers were back-calculated from the deflections measured at the time (see Table 8.4).

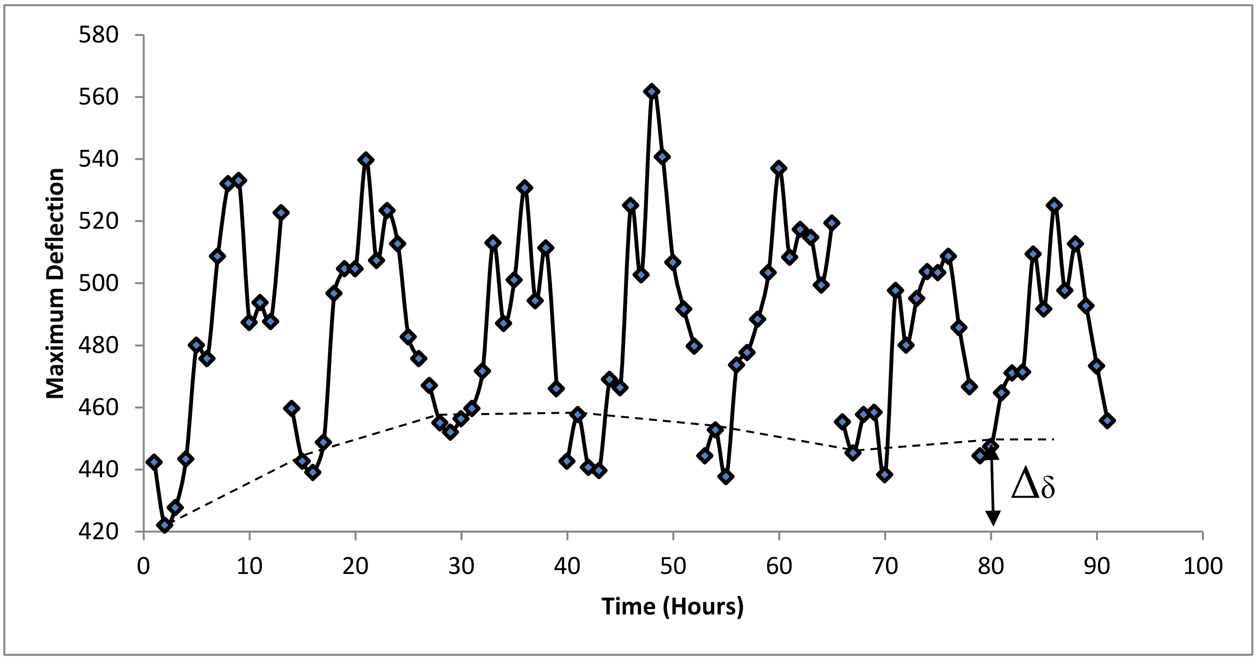



Figure 8.3 – Field Deflection Plots over Time (Sogakope Site)

Table 8.4 – Summary of Parameters Collected From Field Studies

| Pavement Layer | Temperature | Moduli (kPa) | Deflection Δδ (cm) Measured (FWD) |
|---|---|---|---|
| HMA | 43 | 2055000 | 0.0025 |
| Base | 43 | 511000 | |
| Subbase | 42 | 282000 | |
| Subgrade | - | 115000 | |

### 8.7.2 Loading

Twelve loading groups, based on the Ghana Highway Authority (GHA) Classification, have been considered for the load estimation. From traffic count and axle load survey data collected from the two sites (Sogakope and Akumadan), values for the following were estimated to be used in the analysis (see Table 8.5):

- Contact Radius (CR);
- Contact Pressure (CP);
- Distance between tandem axles (XW);
- Centre to centre distance between two dual tires (YW); and
- Number of Load Repetitions (TNLR).

Table 8.5 - Loading Groups, Contact Pressure and Axle Information

| No. | Group | No. of Axles | Contact Pressure (CP) (kPa) | Contact Radius (CR) (cm) | Dual Wheel Spacing (YW) (cm) | Axle -Axle Dist. (XW) (cm) |
|---|---|---|---|---|---|---|
| 1 | Cars | 2 | 241 | 9.8 | 0 | 0 |
| | Taxi | 2 | 241 | 9.4 | 0 | 0 |
| 2 | Pick-up | 2 | 345 | 10.6 | 0 | 0 |
| 3 | Small Bus | 2 | 241 | 23.8 | 0 | 0 |
| 4 | Medium Bus | 2 | 345 | 30.6 | 0 | 0 |
| 5 | Large Bus | 2 | 827 | 17.5 | 0 | 0 |
| 6 | Light Truck | 2 | 827 | 20.7 | 0 | 0 |
| 7 | Medium Truck | 3 | 827 | 17.5 | 33 | 0 |
| 8 | Heavy Truck | 3 | 827 | 16.5 | 33 | 130 |
| 9 | Articulated Truck | 4 | 827 | 25.3 | 33 | 130 |

### 8.7.3 Material Characterization

The pavement is basically divided into HMA layers and granular layers. The Wearing and Binder Courses were considered as a single layer during analysis since their properties were almost the same. Other properties of the all the layers measured, estimated or selected have been summarized in Table 8.6.

Table 8.6 – Summary of Material Properties

| No. | Layer | Thickness (TH) (cm) | Material Type | Elasticity | Poisson Ratio (PR) | Unit Weight (GAM) (kN/m3) |
|---|---|---|---|---|---|---|
| 1 | HMA Layer | 16.5 | Hot Mixed Asphalt (HMA) | Viscoelastic | 0.35 | 26.3 |
| 3 | Base | 20 | Graded Crushed Stone (GCS) | Nonlinear | 0.38 | 22.64 |
| 4 | Subbase | 20 | Granular | Nonlinear | 0.42 | 21.88 |
| 5 | Subgrade | ∞ | Granular | Nonlinear | 0.43 | 20.75 |

### 8.7.3.1 Viscoelastic Layers

The top 2 layers of the pavements are considered viscoelastic materials and their creep compliances are needed to estimate their moduli. The creep compliances were measured at 9 time periods at a reference temperature presented in Table 8.6. Figure 8.4 shows a typical result of the static load creep test done on samples of the HMA layers. From the plot, the creep compliances for the various times were calculated as the time change of permanent strain with stress. Thus;

$$D\ (t) = \frac{\varepsilon(t)}{\sigma\ (t)} \qquad \text{..........Equation 8.6}$$

In which D (t) is the creep compliance, ε (t) is the permanent strain and ζ (t) is the stress.

Average values of the creep compliance of the samples used in the analysis have been given in Table 8.7.

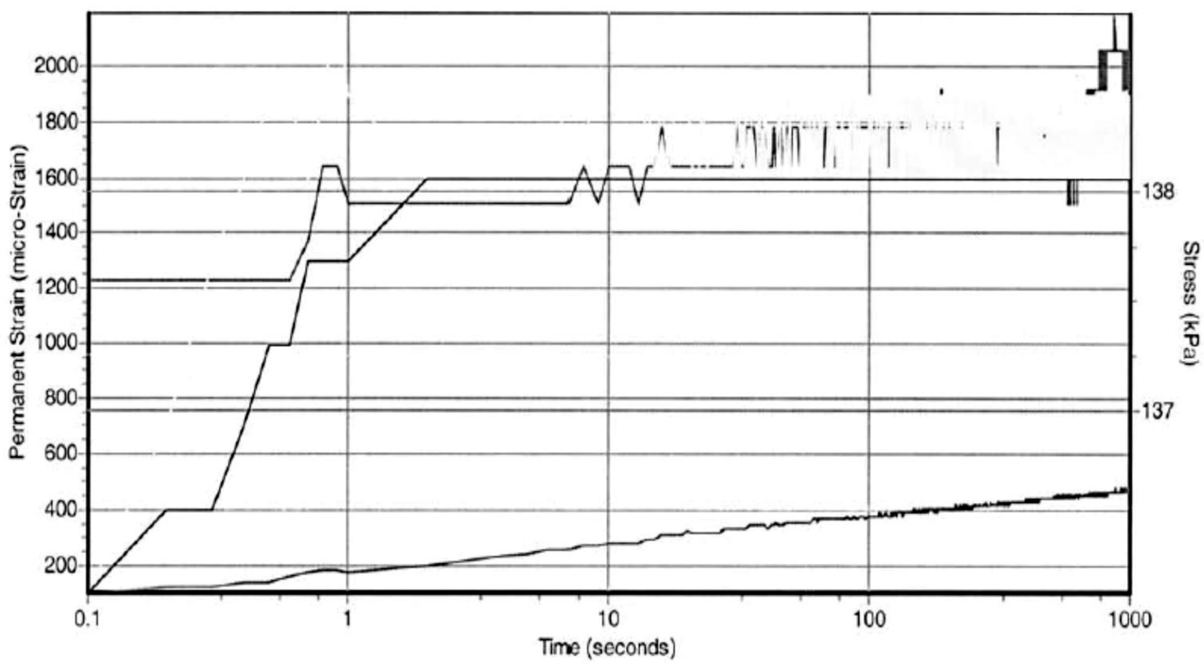



Figure 8.4 – Typical plot from the Static Load Creep Test

Table 8.7 – Creep Compliance Values

| **Ref. Temp. (°C)** | **Creep Compliances (per kPa)** | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| | **Time (s)** | | | | | | | | |
| | **0.1** | **0.5** | **1** | **5** | **10** | **50** | **100** | **500** | **1000** |
| 36.0 | 0.0004588 | 0.00080716 | 0.00100869 | 0.00183324 | 0.00203430 | 0.00259359 | 0.00269160 | 0.00322100 | 0.003382 |

Source: Estimated from the Static Load Creep Plot

### 8.7.3.2 Granular Layers

The bottom 3 layers (i.e. Base, Subbase and Subgrade) are all considered to be granular in nature and therefore the clay parameters were ignored. KENLAYER requires Seasonal and Non-seasonal parameters of the materials considered to be nonlinear elastic. Parameters required depends on whether the material is granular or clayey. All the Nonlinear materials under this analysis are considered to be granular and hence only granular parameters (PHI, $k_1$, $k_2$ and $K_0$) were selected for the KENLAYER.

Method 3 was selected for the base layer and hence considered as a single layer with its values shown in Table 8.8. Huang (2004) reported typical nonlinear constants $k_1$ and $k_2$ for several granular layers presented by Rada and Witczak (1981). Method 1 was selected for the subbase and subgrade layers and hence considered as subdivided into several layers. Jimoh and Akinyemi presented typical values of $k_1$ and $k_2$ for Nigerian laterites which have been adapted for Ghanaian Laterites. Nigeria and Ghana share the same climatic and environmental conditions and most of the research conducted on laterite soils in colonial English West Africa was based on materials from these two countries.

### 8.7.4 Analysis of Results

The KENLAYER was used to simulate the field behaviour of the pavement materials by inputting the load and other parameters estimated from field data and known to have caused the amount of unrecovered strain at the end of the test period. The programme was ran at 60% accuracy (DEL of 0.6) since at this level, the calculated unrecovered strain ($\Delta\delta$) presented as the vertical displacement shifted more towards the field measurements. Also at this level, the model became more sensitive to changes in other parameters and after several adjustments of the parameters; a more reasonable result was obtained.

From the result of the KENLAYER analysis, the HMA layer‟s vertical displacement is seen to be the maximum deflection estimated for all the layers. This calculated displacement is assumed to be equivalent to the unrecovered strain measured from the field deflection plot.

From Figure 8.5, it can be seen that when load is applied on the pavement for some time, due to its flexible nature, the layers are caused to deflect. Stresses and strains are also induced in all the layers; the most important of these is the horizontal tensile strain ($\varepsilon_t$) at the bottom of the HMA layer and the vertical compressive strain ($\varepsilon_c$) on top of the subgrade. From the fatigue and rutting models discussed earlier in this chapter, when these strains exceed some limits, the pavement will fail.

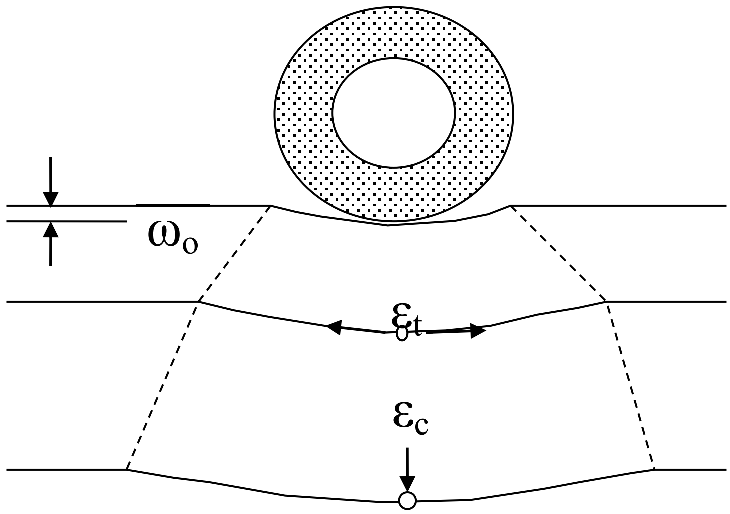



Figure 8.5 – Mechanistic Design Theory (Failure Modes and Critical Strains in Flexible Pavement)

Results from the KENLAYER gave the stresses and strains induced in various vertical coordinates across the pavement layers, inputted into the programme for analyses. Damage analysis gave the critical strains indicated in Table 8.8 and also the allowable load repetitions and damage ratios for the various load groups at various time periods at which the creep compliances were measured. At the end of the results, the damage ratios are summed for all the critical strains; both the horizontal tensile strain at the bottom of the HMA layer and the vertical compressive strain on top of the subgrade. The maximum of the two summed damage ratios is considered the most critical and the design life of the pavement is estimated as the inverse of this critical damage ratio.

Table 8.8 – Summary of Results from the Damage Analysis

| No. | Group | Input Load Repetitions (TNLR) | Permissible Load Repetition (Np) | Tensile Strain At Bottom Of HMA | | | Compressive Strain on Top Of Subgrade | | |
|---|---|---|---|---|---|---|---|---|---|
| | | | | Value (Et) | Damage Ratio (DRt) = Np ÷ Nf | Allowable Load Repetitions (Nf) Eqn. 8.4 | Value (Ec) | Damage Ratio (DRc) = Np ÷ Nd | Allowable Load Repetitions (Nd) Eqn. 8.5 |
| 1 | Cars/Taxis | 4 | - | - | - | - | - | - | - |
| 2 | Vans, Pick-ups | 3 | - | - | - | - | - | - | - |
| 3 | Small Buses | 0 | - | - | - | - | - | - | - |
| 4 | Mammy Wagon/Medium Buses | 52 | 2.036E+03 | 3.325E-05 | 3.508E-06 | 5.804E+08 | 1.652E-04 | 1.74E-05 | 1.168E+08 |
| 5 | Large Buses | 1949 | | | | | | | |
| 6 | Light Trucks | 25841 | 2.584E+04 | 4.160E-05 | 9.307E-05 | 2.776E+08 | 1.955E-04 | 4.71E-04 | 5.490E+07 |
| 7 | Medium Truck | 88027 | 8.804E+04 | 5.797E-05 | 9.450E-04 | 9.316E+07 | 2.835E-04 | 8.47E-03 | 1.040E+07 |
| 8 | Heavy Truck | 393229 | 4.251E+05 | 5.632E-05 | 4.150E-03 | 1.024E+08 | 2.799E-04 | 3.86E-02 | 1.102E+07 |
| 9 | Semi-Trailers (Light) | 32142 | 4.197E+04 | 5.580E-05 | 3.974E-04 | 1.056E+08 | 2.792E-04 | 3.77E-03 | 1.113E+07 |
| 10 | Semi-Trailers (Heavy) | 23531 | 1.330E+05 | 5.499E-05 | 1.200E-03 | 1.108E+08 | 2.780E-04 | 1.17E-02 | 1.135E+07 |
| 11 | Truck-Trailers | 17358 | 1.715E+04 | 5.600E-05 | 1.642E-04 | 1.044E+08 | 2.786E-04 | 1.52E-03 | 1.125E+07 |
| 12 | Large Truck and Others | 1743 | | | | | | | |
| **Sum of Damage Ratio (= ∑DRi)** | | | | | **6.953E-03** | | | **6.454E-02** | |
| **Maximum Damage Ratio (= Max: ∑DRi)** | | | | **6.454E-02** | | | | | |
| **Design Life of Pavement (=1/ Max: ∑DRi)** | | | | **15.49** | | | | | |

# CHAPTER 9

# PILOT STUDY FOR NEW FRAMEWORK OF PAVEMENT DESIGN FOR GHANA

## 9.0 Introduction

From the findings of this research, a pilot study has been undertaken to enable recommendations and conclusions to be drawn with respect to a new pavement design in tropical Ghana. The field work established parameters for the Semi-Arid coastal climatic zone.

The scope of the pilot study was to carry out a mechanistic-empirical design, using the calibrated mechanistic model, KENLAYER, with input from field study and information obtained from the literature review and other secondary data; and to use the calibrated HDM-4 economic analysis model to design economic road pavement for the climatic zones of Ghana.

The approach of the pilot study was to carry out mechanistic-empirical design to produce catalogue of candidate pavements for the Semi-Arid climatic zone and then subject these catalogue of pavements to a Life Cycle Analysis in order to establish the most economic pavement design.

## 9.1 Mechanistic Pavement Design for Climate Zones of Ghana

A typical cross-section of asphaltic concrete pavements in Ghana consists of viscoelastic and nonlinear layers and therefore, the combined mechanistic-empirical analysis which is an option in the KENLAYER was adopted for Ghana. Most of the inputs have been established through field studies (empirical) under this research and work done elsewhere under similar environmental and climatic conditions. The inputs required for carrying out a mechanistic pavement design are given in chapter 8. There are sets of input parameters that must be met to enable a successful pavement analysis to be carried out. The calibrated KENLAYER mechanistic model was used to design a catalogue of pavements. The accuracy level of the

KENLAYER model was set at 0.985 as was used to calibrate the model to simulate observed deflection on the field. In order to establish the deflection at various levels, the z- coordinates were measured from the top of the asphalt layer as shown in Fig 8.4

Determination of key parameters for mechanistic-pavement design from field studies is discussed below in line with the aims and objectives of the research.

### 9.1.1 Loading Analysis

Inaccuracy in traffic estimation which is very important to design has led to earlier failure of the road in Ghana as the projected design life traffic are attained within few years when road is opened to traffic. In order to prevent any underestimation overestimation, actual traffic data was collected from site through the installation of permanent traffic counter in the test section as described in Chapter 4 of this thesis was disaggregated into the vehicle classifications used in Ghana. Table 4.2 gives the equivalence classification of vehicles between Ghana and the European systems. In order to apportion the right percentage of vehicles to their respective class as per the Ghana"s system, Table 7.2 was used to establish the various ratios for a given vehicle fleet. Table 9.1 below presents the process in arriving at the Annual Load Repetitions for the Semi-Arid Zone. The annual traffic volumes and the growth rates for the various traffic classes were then used to estimate the design life traffic volume of the respective climatic zones.

A total of 9 Load Groups, (same as Ghana Vehicle Classification), given in Table 9.1 is used for the pavement design. Since vehicle classifications are different, the assumption used may either lead to underestimate or overestimate of the traffic in some cases.

Table 9.1 - Annual Traffic Volume Conversions for Semi-Arid Climate

| European Vehicle Class. | Ghana Equivalent Vehicle Class | Ghana Vehicle Class. | Ratios from Composition | Annual Traffic Volume per European Class | Annual Traffic Volume per Ghana Class. |
|---|---|---|---|---|---|
| CS2 | Cars | 1 | 0.363 | 482628 | 175322 |
| | Taxi | | 0.241 | | 116725 |
| | Pick-up | 2 | 0.396 | | 191081 |
| CS6 | Small Bus | 3 | 0.434 | 3654 | 1583 |
| | Medium Bus | 4 | 0.318 | | 1164 |
| | Large Bus | 5 | 0.248 | | 908 |
| CS4 | Light Truck | 6 | 0.288 | 83323 | 24008 |
| | Medium Truck | 7 | 0.364 | | 30363 |
| | Heavy Truck | 8 | 0.348 | | 28951 |
| CS5 | Articulated Truck | 9 | 1 | 20578 | 20578 |
| CS1 | Motorcycle | NA | | 403 | 403 |
| CS3 | Car+ Trailer | NA | | 474 | 474 |

#### 9.1.1.1 Projected Traffic Volumes and Cumulative Standard Axle Loads

Table 9.2 gives the computed project design life traffic and the cumulative equivalent standard axle load values for the Semi-Arid test section (Sogakope) based on the field monitored actual annual traffic volume and the growth rates for the various vehicle classes.

Table 9.2 – Projected Traffic Volumes and Load

| Ghana Equivalent Vehicle Class | Annual Traffic | Projected Traffic Volume (15 Years) | Cumm. ESAL |
|---|---|---|---|
| Cars | 175322 | 450900 | 30 |
| Taxis | 116725 | 300198 | 50 |
| Pick-up | 191081 | 491430 | 85 |
| Small Bus | 1583 | 3291 | 90 |
| Medium Bus | 1164 | 2420 | 769 |
| Large Bus | 908 | 1888 | 26076 |
| Light Truck | 24008 | 49911 | 361674 |
| Medium Truck | 30363 | 63122 | 1504882 |
| Heavy Truck | 28951 | 60187 | 6611757 |
| Articulated Truck | 20578 | 42780 | 7560208 |
| Motorcycle | 403 | 838 | 7560208 |
| Car+ Trailer | 474 | 985 | 7582838 |
| Equivalent Standard Axle ($x10^6$) | | | **7.58 mesa** |

### 9.1.1.2 Tyre Pressure of Load Groups

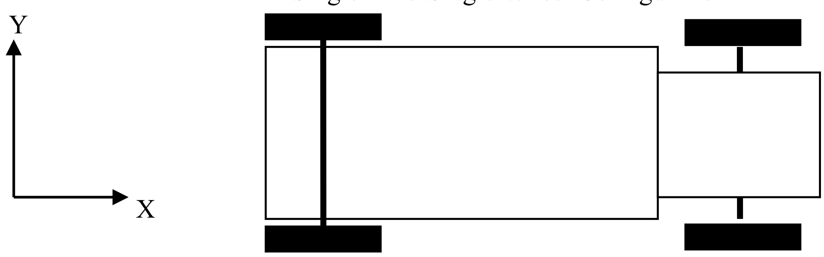
In order to establish the actual operational tyre pressures used for various load groups, a tyre pressure survey was conducted among ten vulcanizing operators in Accra the capital of Ghana, to obtain the mean tyre pressures for the respective load groups. In Ghana, there is no restriction with respect to movement of traffic on the road network and therefore the survey was not restrictive to the test section area as most of the commercial vehicles emanates or end in Accra.

In order to establish the axle configuration information required for the KENLAYER input, the following explanation is needed:

The distance between axles is measured in the x-direction and is depicted as XW. The distance between dual wheels is measured in the y-direction and depicted as YW. These have been illustrated as follows:

(a)

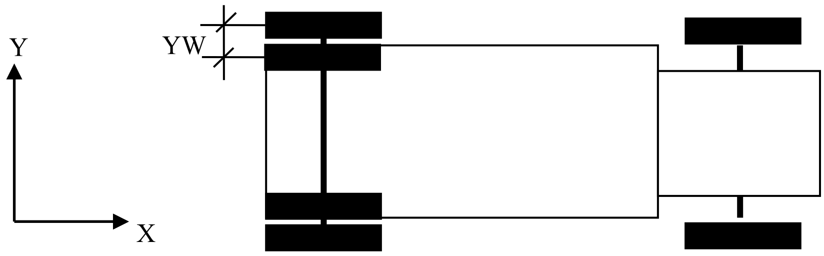
Single Axle-Single Wheel Configuration




XW = 0 cm, YW = 0 cm

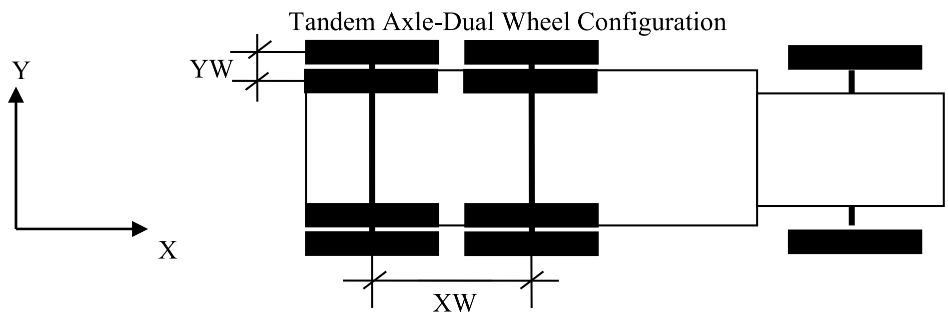
(b)

Single Axle-Dual Wheel Configuration



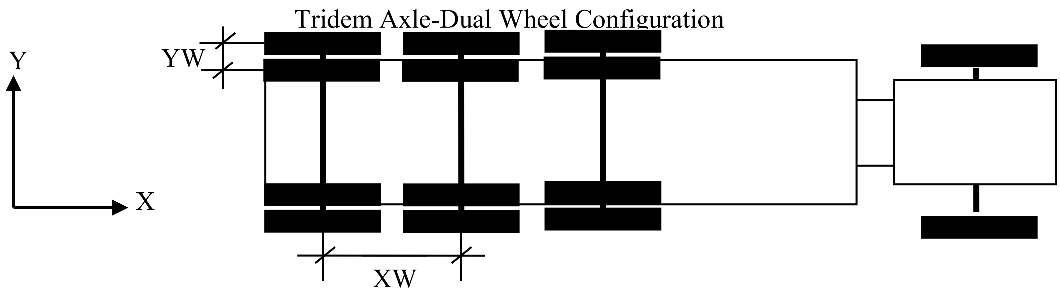

XW = 0 cm, YW = 33 cm

(c)




XW = 130 cm, YW = 33 cm

(d)




XW = 130 cm, YW = 33 cm

The points to be analysed by the KENLAYER was determined from the YW and XW values. YW = 33 and XW=130. The midpoints of these values were used for the stress and strain analysis.

Information on the Load Groups and their related tyre pressure values, dual wheel spacing and axle-axle distance for each load group have been given in Table 8.5 in the previous chapter.

### 9.1.2 Strength Properties

The asphaltic concrete layers comprising the binder/dense bituminous macadam and the wearing courses were considered as a composite asphaltic concrete (AC) layer with viscoelastic properties. The crushed rock base, lateritic sub-base and the subgrade were taken as independent granular layers with nonlinear elastic properties. In accordance with the basic Burmister"s multilayer system theory and enhanced by Huang (1994), the theory can now be applied to a multilayer system of any number. Each layer is assumed to be homogenous, isotropic and linearly elastic with resilient modulus and Poisson ratio. Using the range of values given for the various types of pavement materials (Huang, 2004), the exact Poisson ratio for each layer was selected for the KENLAYER in order to achieve the design life required. The thicknesses of the respective layers where determined at the point in the pavement where the temperatures of the pavement layers were measured.

The geometric and material properties of the pavement material properties used for the analysis are given in Tables 9.3. For the Poisson Ratios, Huang (2004) recommends a range of 0.3 – 0.4 for HMA and untreated granular base layers, and for fine-grained soils a range of 0.3 – 0.5.

Table 9.3 - Geometric and Material Properties of Pavement Structure for Design Semi-Arid Climate Zone

| **Layer** | **Material** | **Resilient Modulus (KPa)** | **Poisson Ratio** | **Thickness (mm)** |
|---|---|---|---|---|
| 1 | AC Surface | 2055000 | 0.35 | 120-180 (165) |
| 2 | Crushed Rock Base | 511000 | 0.38 | 150-300 (200) |
| 3 | Lateritic Subbase | 282000 | 0.42 | 150-300 (200) |
| 4 | Subgrade | 115,000 | 0.43 | ∞ |

*Figures in bracket were actual field measurement*

The respective unit weights (GAM) of the layer materials were estimated from the specific gravity and densities of the pavement material properties given in Appendix A. Table 9.4 below gives figures used in the study.

Table 9.4 - Estimation of Unit Weight of Layer Materials

| **Layer Material** | **Specific Gravity** | **Density (kg/m$^3$)** | **Unit Weight (KN/m$^3$)** |
|---|---|---|---|
| AC Surface | 2.68 | 2680 | 26.30 |
| Crushed Rock Base | 2.31 | 2310 | 22.65 |
| Latertic Subbase | - | 2230 | 21.88 |
| Subgrade | - | 2117 | 20.75 |

*Kg x Acceleration due to gravity gives Newton*

### 9.1.3 Nonlinear Material Analysis Parameters

In order to evaluate the stresses and strains in the nonlinear layers, the k-values were obtained from work done in Nigeria for Lateritic Granular materials (Jimoh and Akinyemi, no date) and the field work under this study were used as the basis to establish the parameters for

Ghana. Nigeria and Ghana is only 15° Longitude apart and therefore the climatic factors are the same leading to same soil formation factors. For the crushed rock layer, values given by Huang (2004) were used. The coefficient of the earth pressure $K_O$ is estimated from Equation 9.1;

$$Ko = 1 - \sin(PHI) \text{.................................... Equation 9.1}$$

The PHI values for the granular layers were selected based on the methods recommended by Huang (2004) for the selection of PHI for the nonlinear analysis of the granular layers. There are three methods of analysis summarized in Chapter 8. The granular layers (crushed rock base, lateritic subbase and the subgrade) are treated as three divided independent layers which is consistent with assumptions required to use Method 1. Table 9.5 summarizes the K-values and PHI used for the structural analysis.

Table 9.5 – K -Values for Nonlinear Granular Materials

| **Granular Layer** | **$K_O$** | **$K_1$ (KPa)** | **$K_2$** | **PHI (ø)** |
|---|---|---|---|---|
| Crushed Rock Base | 1 | 51100 | 0.6 | 0° |
| Lateritic Sub Base | 1 | 282000 | 0.6 | 0° |
| Subgrade | 1 | 115000 | 0.51 | 0° |

### 9.1.4 Viscoelastic Analysis Parameters

Creep is the slow plastic movement of the material in a surface layer in the line and direction of traffic flow or gradient (Khanna, 2005). One of the important requirements for mechanistic analysis for viscoelastic material is the Creep Compliance. This was obtained from laboratory test of core samples taken from test sections. The reference temperature used for the test was 36°C. The Static Load Creep Test was carried out for 1000 seconds and 9 creep compliance values were obtained during the period. The creep compliance values were obtained from the graph generated during test.

The BETA (β) for the Temperature Shift Coefficient is needed to adjust the temperature if the creep test was not carried out at the same temperature as the asphalt pavement. The average pavement temperature (T) of 36.9°C was used for the test. Therefore BETA (β) value was calculated from Equation 9.2:

$$\beta = \frac{\log(aT)}{T - T0} \text{........................................ Equation 9.2}$$

| | | | |
|---|---|---|---|
| Where | $T_o$ | = | Reference Temperature |
| | $T$ | = | Average Pavement Temperature |
| | $a_T$ | = | time-temperature shift factor |
| | $a_T$ | = | $t_T / t_{To}$ |
| | $t_T$ | = | time to obtain a creep compliance at temperature T |
| | $t_{To}$ | = | time to obtain a creep compliance at reference temperature $T_o$ |

Figure 2.38 as presented in Huang (2004) relates the reference temperature ($T_o$) to the shift factor log ($a_T$) in a chart, and from the log ($a_T$) selected, the value of $\beta$ calculated was between 3 and 4.

### 9.1.5 Coefficient of Damage

In all cases, the bottom of the Asphaltic Concrete (Layer 1) was used for damage analysis in the form of tensile strain for fatigue cracking. The top of the Subgrade (Layer 4) was used to determine permanent deformation due to compressive strains. The coefficients for the respective damages are those obtained from figures provided by the Asphalt Institute.

### 9.1.6 Stress Location in Nonlinear Layers

The stress analysis and the central locations of the wheel were taken for nonlinear layers. The following locations were used for the pavement analysis:

For Single Axle (wheel), the stress analysis location was taken to be at the centre of the wheel where the radial coordinate for nonlinear (RCNOL) = 0, the Slope of Load Distribution (SLD) = 0. For Dual Axle (wheel), the x-coordinate (XPT) is zero and the y-coordinate (YPTNOL) = YW/2.

The default factor for Relaxation of 0.5 is used. General layout of pavement used for the structural analysis is shown in Fig 9.1

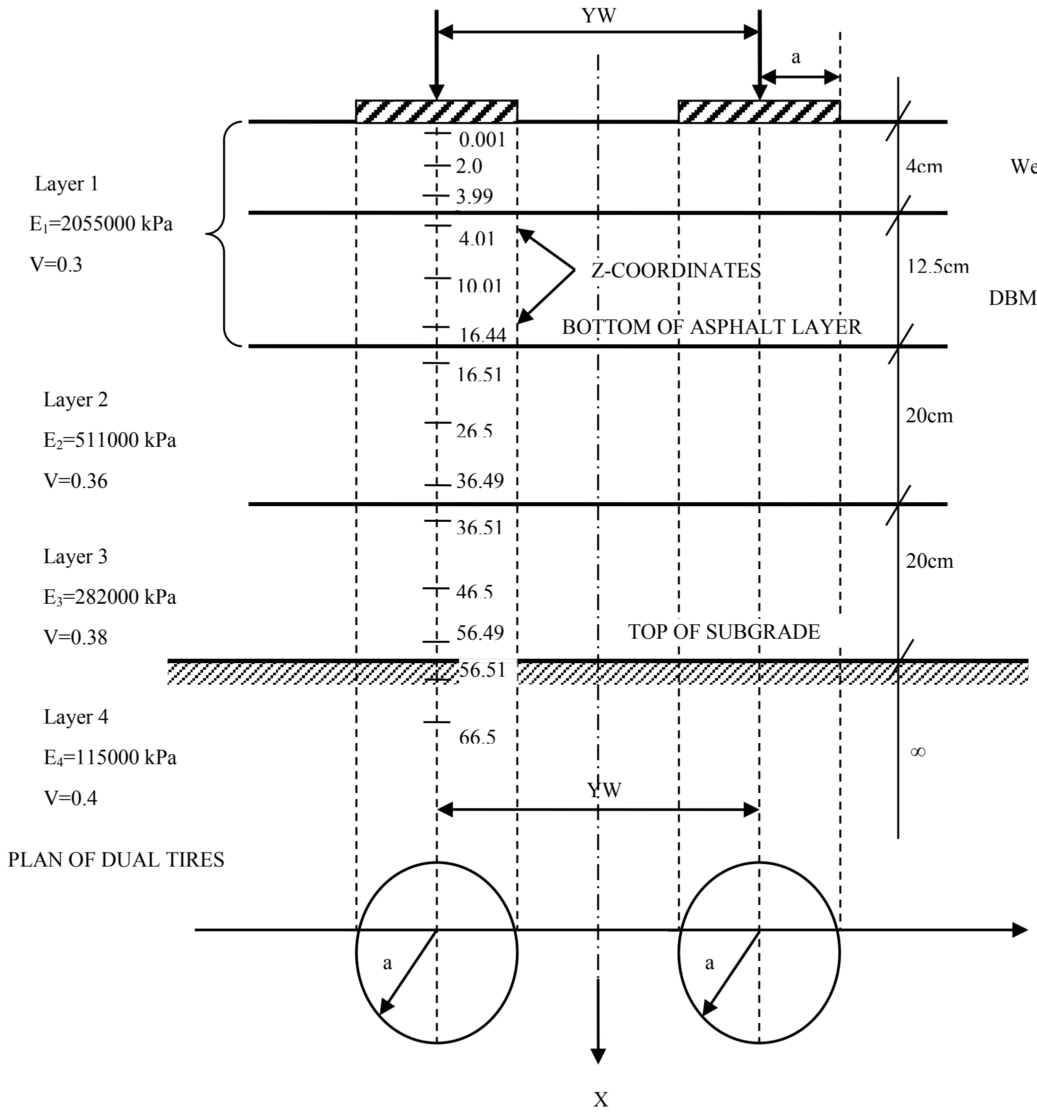



Figure 9.1 - Structural Presentation of Pavement for Analysis for Dry Sub-Humid Zone

### 9.1.7 Result of Pavement Analysis

From the thicknesses of the various pavement layers considered a total of sixty (60) possible candidate pavements could be evolved for the structural analysis to determine the design life of the respective candidate pavements. Sixty (60) candidate pavements were prepared for KENLAYER analysis for Semi-Arid climatic zone. A summary of the outputs are presented in Tables 9.6.

Table 9.6 - Output of Candidate Pavement Design of Dry Sub-Humid Climate Zone

| No. | Layer Thickness (mm) | | | Design Life | Bottom of AC Tensile Strain | Top of Subgrade Compressive Strain |
|---|---|---|---|---|---|---|
| | HMA | Base | Subbase | | | |
| 1 | 180 | 300 | 300 | 25.86 | 0.000692 | 0.001950 |
| 2 | 165 | 300 | 300 | 25.42 | 0.000694 | 0.001956 |
| 3 | 140 | 300 | 300 | 24.72 | 0.000697 | 0.001966 |
| 4 | 120 | 300 | 300 | 24.20 | 0.000699 | 0.001972 |
| 5 | 180 | 250 | 300 | 20.60 | 0.000699 | 0.001988 |
| 6 | 165 | 250 | 300 | 20.24 | 0.000701 | 0.001994 |
| 7 | 140 | 250 | 300 | 19.67 | 0.000704 | 0.002004 |
| 8 | 120 | 250 | 300 | 19.25 | 0.000707 | 0.002011 |
| 9 | 180 | 200 | 300 | 16.97 | 0.000700 | 0.002010 |
| 10 | 165 | 200 | 300 | 16.67 | 0.000702 | 0.002016 |
| 11 | 140 | 200 | 300 | 15.93 | 0.000708 | 0.002034 |
| 12 | 120 | 200 | 300 | 15.59 | 0.000709 | 0.002041 |
| 13 | 180 | 150 | 300 | 16.1 | 0.000680 | 0.001988 |
| 14 | 165 | 150 | 300 | 15.73 | 0.000681 | 0.001994 |
| 15 | 140 | 150 | 300 | 15.37 | 0.000683 | 0.002001 |
| 16 | 120 | 150 | 300 | 14.91 | 0.000688 | 0.002015 |
| 17 | 180 | 300 | 250 | 23.10 | 0.000682 | 0.001954 |
| 18 | 165 | 300 | 250 | 22.71 | 0.000684 | 0.001960 |
| 19 | 140 | 300 | 250 | 22.10 | 0.000686 | 0.001969 |
| 20 | 120 | 300 | 250 | 21.65 | 0.000688 | 0.001976 |
| 21 | 180 | 250 | 250 | 18.54 | 0.000600 | 0.001750 |
| 22 | 165 | 250 | 250 | 18.22 | 0.000610 | 0.001780 |
| 23 | 140 | 250 | 250 | 17.72 | 0.000590 | 0.001720 |
| 24 | 120 | 250 | 250 | 17.35 | 0.000693 | 0.002010 |
| 25 | 180 | 200 | 250 | 15.49 | 0.000680 | 0.002010 |
| 26 | 165 | 200 | 250 | 15.31 | 0.000680 | 0.002000 |
| 27 | 140 | 200 | 250 | 15.05 | 0.000690 | 0.002020 |
| 28 | 120 | 200 | 250 | 14.73 | 0.000688 | 0.002023 |
| 29 | 180 | 150 | 250 | 14.70 | 0.000660 | 0.001980 |
| 30 | 165 | 150 | 250 | 14.46 | 0.000440 | 0.001350 |
| 31 | 140 | 150 | 250 | 14.09 | 0.000475 | 0.001485 |
| 32 | 120 | 150 | 250 | 13.88 | 0.000664 | 0.001997 |
| 33 | 180 | 300 | 200 | 21.13 | 0.000665 | 0.001946 |
| 34 | 165 | 300 | 200 | 20.79 | 0.000667 | 0.001951 |
| 35 | 140 | 300 | 200 | 20.25 | 0.000669 | 0.001960 |
| 36 | 120 | 300 | 200 | 19.86 | 0.000670 | 0.001967 |
| 37 | 180 | 250 | 200 | 17.19 | 0.000660 | 0.001970 |
| 38 | 165 | 250 | 200 | 16.91 | 0.000670 | 0.001980 |
| 39 | 140 | 250 | 200 | 16.47 | 0.000670 | 0.001990 |
| 40 | 120 | 250 | 200 | 16.14 | 0.000671 | 0.001995 |
| 41 | 180 | 200 | 200 | 15.91 | 0.000463 | 0.001460 |
| 42 | 165 | 200 | 200 | 15.64 | 0.000650 | 0.001961 |
| 43 | 140 | 200 | 200 | 15.25 | 0.000650 | 0.001970 |

Table 9.6 - (continued...)

| No. | Layer Thickness (mm) | | | Design Life | Bottom of AC Tensile Strain | Top of Subgrade Compressive Strain |
|---|---|---|---|---|---|---|
| | HMA | Base | Subbase | | | |
| 44 | 120 | 200 | 200 | 14.94 | 0.000650 | 0.001977 |
| 45 | 180 | 150 | 200 | 14.18 | 0.000620 | 0.001950 |
| 46 | 165 | 150 | 200 | 13.99 | 0.000630 | 0.001950 |
| 47 | 140 | 150 | 200 | 13.71 | 0.000620 | 0.001951 |
| 48 | 120 | 150 | 200 | 13.29 | 0.000627 | 0.001962 |
| 49 | 180 | 300 | 150 | 20.10 | 0.000637 | 0.001921 |
| 50 | 165 | 300 | 150 | 19.80 | 0.000640 | 0.001926 |
| 51 | 140 | 300 | 150 | 19.32 | 0.000641 | 0.001934 |
| 52 | 120 | 300 | 150 | 18.97 | 0.000642 | 0.001941 |
| 53 | 180 | 250 | 150 | 17.65 | 0.000620 | 0.001920 |
| 54 | 165 | 250 | 150 | 17.31 | 0.000630 | 0.001930 |
| 55 | 140 | 250 | 150 | 16.75 | 0.000630 | 0.001930 |
| 56 | 120 | 250 | 150 | 16.92 | 0.000628 | 0.001936 |
| 57 | 180 | 200 | 150 | 15.77 | 0.000610 | 0.001920 |
| 58 | 165 | 200 | 150 | 15.55 | 0.000610 | 0.001920 |
| 59 | 140 | 200 | 150 | 15.18 | 0.000610 | 0.001930 |
| 60 | 120 | 200 | 150 | 14.68 | 0.000611 | 0.001940 |

However, thirty-seven (37) candidate pavements were considered for further analysis based on the design life. Sensitivity analysis was carried to establish the effect due to changes in the design parameters such the pavement layer thickness and the layer modulus.

## 9.2 Sensitivity Analysis on Effect of Layer Thicknesses on Strains

Sensitivity analysis was carried on the failure parameters with respect to changes in the tensile and compressive strains at the bottom of the Asphaltic Concrete and at the top of the subgrade respectively.

This was done by increasing and decreasing the thicknesses of the pavement layers and the resilient modulus.

For the 3 layers under consideration, two of the pavement layers thicknesses were kept fixed at each time and the layer of interest varied. The strains induced in pavement layers by the respective loading groups were computed from KENLAYER runs. Buses and 4-axle vehicles were the loading groups of interest for the sensitivity analyses. This because buses are typical for carrying passengers while the 4-axle is dominant for causing the worst damage to the pavement with respect to cargo haulage.

The pavement thicknesses were either increased or reduced by 2.5% to 15%, to see how the pavement will respond to a constant load applied to all cases. Table 9.7 show the results obtained from KENLAYER runs.

Table 9.7 – Results of Sensitivity Analyses on Pavement Thicknesses (Semi-Arid -Sogakope Site)

| **Thickness (mm)** | | | **Design Life (Yrs)** | **Buses** | | **4 – Axle** | |
|---|---|---|---|---|---|---|---|
| **HMA (h1)** | **Base (h2)** | **Subbase (h3)** | | **HMA Bottom Tensile Strain** | **Subgrade Top Compressive Strain** | **HMA Bottom Tensile Strain** | **Subgrade Top Compressive Strain** |
| **189.75** | 200 | 200 | 16.08 | 3.236E-05 | 1.641E-04 | 5.531E-05 | 2.092E-04 |
| **181.5** | 200 | 200 | 15.93 | 3.262E-05 | 1.644E-04 | 5.569E-05 | 2.098E-04 |
| **173.25** | 200 | 200 | 15.79 | 3.290E-05 | 1.646E-04 | 5.620E-05 | 2.104E-04 |
| **165** | 200 | 200 | 15.64 | 3.319E-05 | 1.649E-04 | 5.665E-05 | 2.110E-04 |
| **156.75** | 200 | 200 | 15.51 | 3.348E-05 | 1.652E-04 | 5.700E-05 | 2.116E-04 |
| **148.5** | 200 | 200 | 15.38 | 3.387E-05 | 1.661E-04 | 5.746E-05 | 2.122E-04 |
| **140.25** | 202 | 200 | 15.26 | 3.417E-05 | 1.663E-04 | 5.911E-05 | 2.128E-04 |
| 165 | **230** | 200 | 16.55 | 3.512E-05 | 1.643E-04 | 5.917E-05 | 2.806E-04 |
| 165 | **220** | 200 | 16.5 | 3.420E-05 | 1.639E-04 | 5.774E-05 | 2.062E-04 |
| 165 | **205** | 200 | 16.02 | 3.345E-05 | 1.647E-04 | 5.691E-05 | 2.097E-04 |
| 165 | **200** | 200 | 15.64 | 3.319E-05 | 1.649E-04 | 5.665E-05 | 2.110E-04 |
| 165 | **195** | 200 | 15.4 | 3.284E-05 | 1.649E-04 | 5.615E-05 | 2.123E-04 |
| 165 | **180** | 200 | 14.81 | 3.184E-05 | 1.636E-04 | 5.499E-05 | 2.160E-04 |
| 165 | **170** | 200 | 14.48 | 3.119E-05 | 1.639E-04 | 5.409E-05 | 2.183E-04 |
| 165 | 200 | **230** | 15.48 | 3.321E-05 | 1.655E-04 | 5.629E-05 | 2.089E-04 |
| 165 | 200 | **220** | 15.74 | 3.320E-05 | 1.654E-04 | 5.626E-05 | 2.084E-04 |
| 165 | 200 | **205** | 15.68 | 3.320E-05 | 1.651E-04 | 5.654E-05 | 2.102E-04 |
| 165 | 200 | **200** | 15.64 | 3.319E-05 | 1.649E-04 | 5.665E-05 | 2.110E-04 |
| 165 | 200 | **195** | 15.57 | 3.310E-05 | 1.644E-04 | 5.669E-05 | 2.117E-04 |
| 165 | 200 | **180** | 15.45 | 3.294E-05 | 1.622E-04 | 5.690E-05 | 2.137E-04 |
| 165 | 200 | **170** | 15.44 | 3.290E-05 | 1.617E-04 | 5.694E-05 | 2.148E-04 |

From Table 9.8, the following observations can be made;

**1. HMA Thickness:**

- Increasing the HMA thickness increases the design lives, reduces the strains and vice versa as expected.
- Changes in the values of the tensile strains were relatively higher than the compressive strain values with each change in HMA thickness Changes in the HMA thickness affect all layers but the effect is more prominent on the tensile strains under the bottom of the HMA layer than the compressive strains on the subgrade.

**2. Base Thickness:**

- Increasing the base thickness leads to increases the design life and vice versa as expected.
- With the exception of the compressive strains, variations in thicknesses of the base layer do not give any reasonable trend in effects on the tensile strains.
- The effect is more prominent on the compressive strains than the tensile strains.

**3. Subbase Thickness:**

- Increasing the Sub-base thickness also increases the design lives and vice versa like all the top layers.
- It can be observed that the strains under the 4-axle load gives a trend as expected, i.e., increasing thickness decreasing strains and vice versa, unlike the strains under the Buses load which gives unreasonable trends with respect to thickness variations.

It can be concluded that the impact due to variation in the subbase thickness is more significant under heavy traffic loads compared to light loads and this affects the compressive strains in the subgrade more than the tensile strains under the HMA leading to permanent damage of the pavement.

Table 9.8 - Summary of Sensitivity Analysis for Pavement Thicknesses

| Site | Layer | Change in Thickness (%) | Change in Design Life (%) | Buses | | 4 - Axle | |
|---|---|---|---|---|---|---|---|
| | | | | Change in HMA bottom tensile strain (%) | Change in Subgrade top compressive strain (%) | Change in HMA bottom tensile strain (%) | Change in Subgrade top compressive strain (%) |
| Sogakope | HMA | +15.0 | +2.8 | -2.5 | -0.5 | -2.4 | -0.9 |
| | | +10.0 | +1.9 | -1.7 | -0.3 | -1.7 | -0.6 |
| | | +5.0 | +1.0 | -0.9 | -0.2 | -0.8 | -0.3 |
| | | -5.0 | -0.8 | +0.9 | +0.2 | +0.6 | +0.3 |
| | | -10.0 | -1.7 | +2.0 | +0.7 | +1.4 | +0.6 |
| | | -15.0 | -2.4 | +3.0 | +0.8 | +4.3 | +0.9 |
| | Base | +15.0 | +2.4 | +4.8 | -0.7 | +4.3 | -1.8 |
| | | +10.0 | +5.5 | +3.0 | -0.6 | +1.9 | -2.3 |
| | | +2.5 | +2.4 | +0.8 | -0.1 | +0.5 | -0.6 |
| | | -2.5 | -1.5 | -1.1 | 0.0 | -0.9 | +0.6 |
| | | -10.0 | -5.3 | -4.1 | -0.8 | -2.9 | +2.4 |
| | | -15.0 | -7.4 | -6.0 | -0.6 | -4.5 | +3.5 |
| | Subbase | +15.0 | -1.0 | +0.1 | +0.4 | -0.6 | -1.0 |
| | | +10.0 | +0.6 | +0.0 | +0.3 | -0.7 | -1.2 |
| | | +2.5 | +0.3 | +0.0 | +0.1 | -0.2 | -0.4 |
| | | -2.5 | -0.4 | -0.3 | -0.3 | +0.1 | +0.3 |
| | | -10.0 | -1.2 | -0.8 | -1.6 | +0.4 | +1.3 |
| | | -15.0 | -1.3 | -0.9 | -1.9 | +0.5 | +1.8 |
| Decrease (-) and Increase (+) | | | | | | | |

Plots of the various sensitivities due to changes in the pavement layer thicknesses under the two load groups being considered are shown in Figures 9.2, 9.3 and 9.4 for HMA, Base and Sub-base respectively.

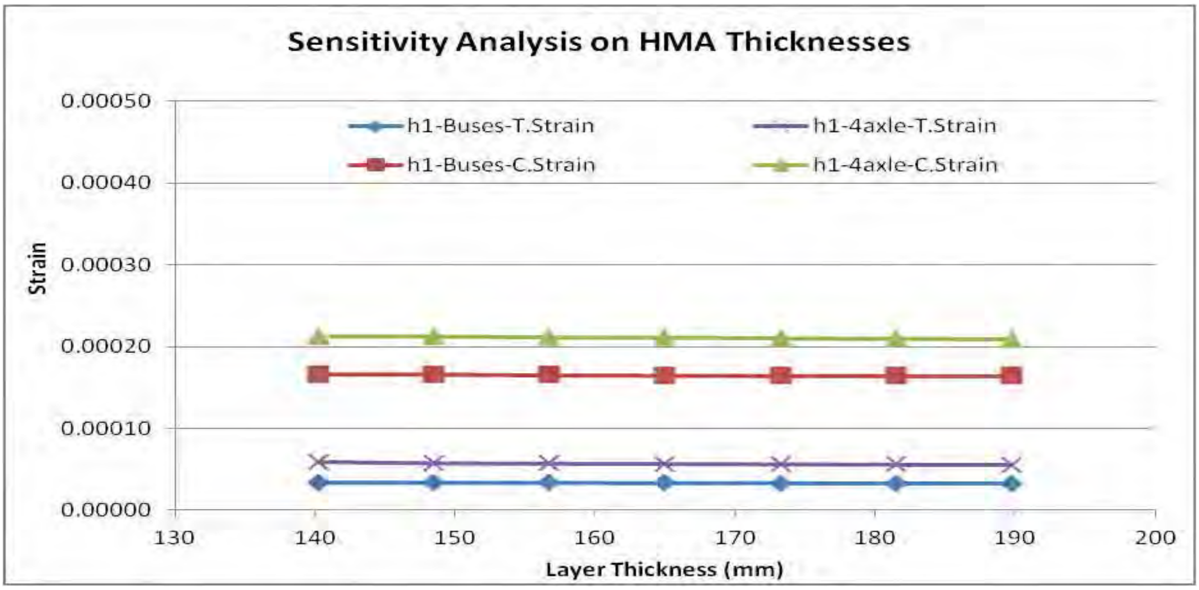



Figure 9.2 – Sensitivity of the HMA Thickness

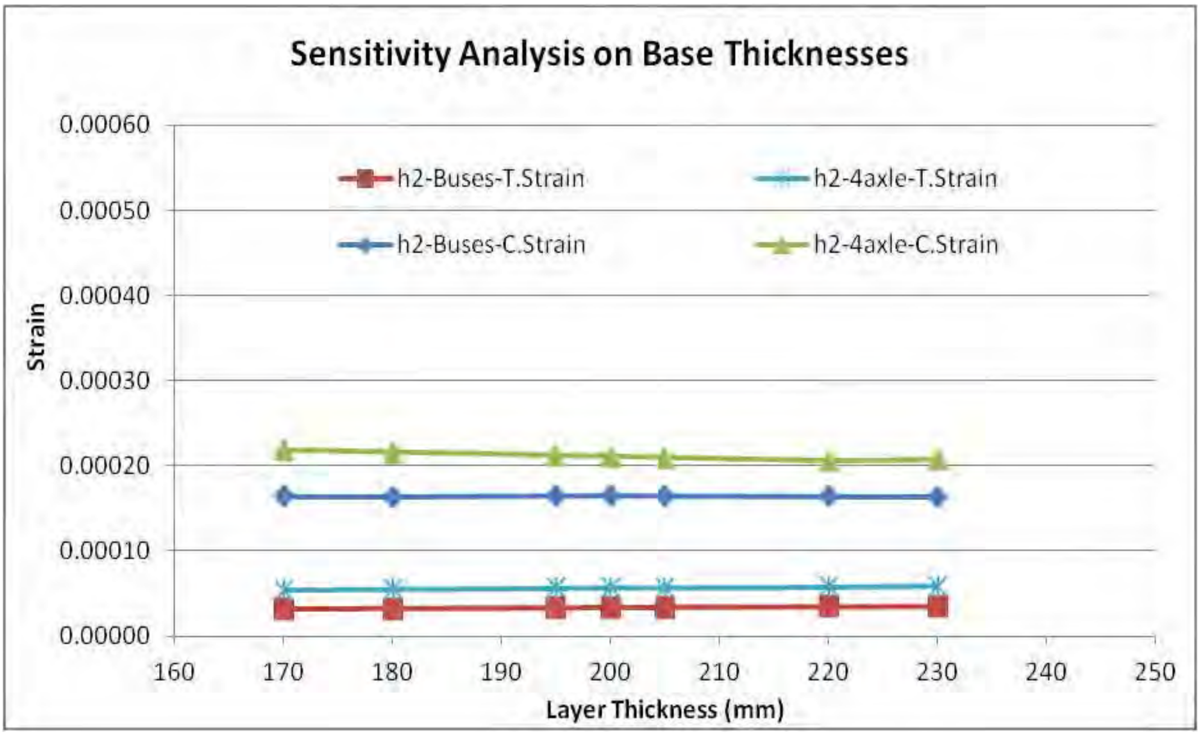



Figure 9.3 – Sensitivity of the Base Thickness

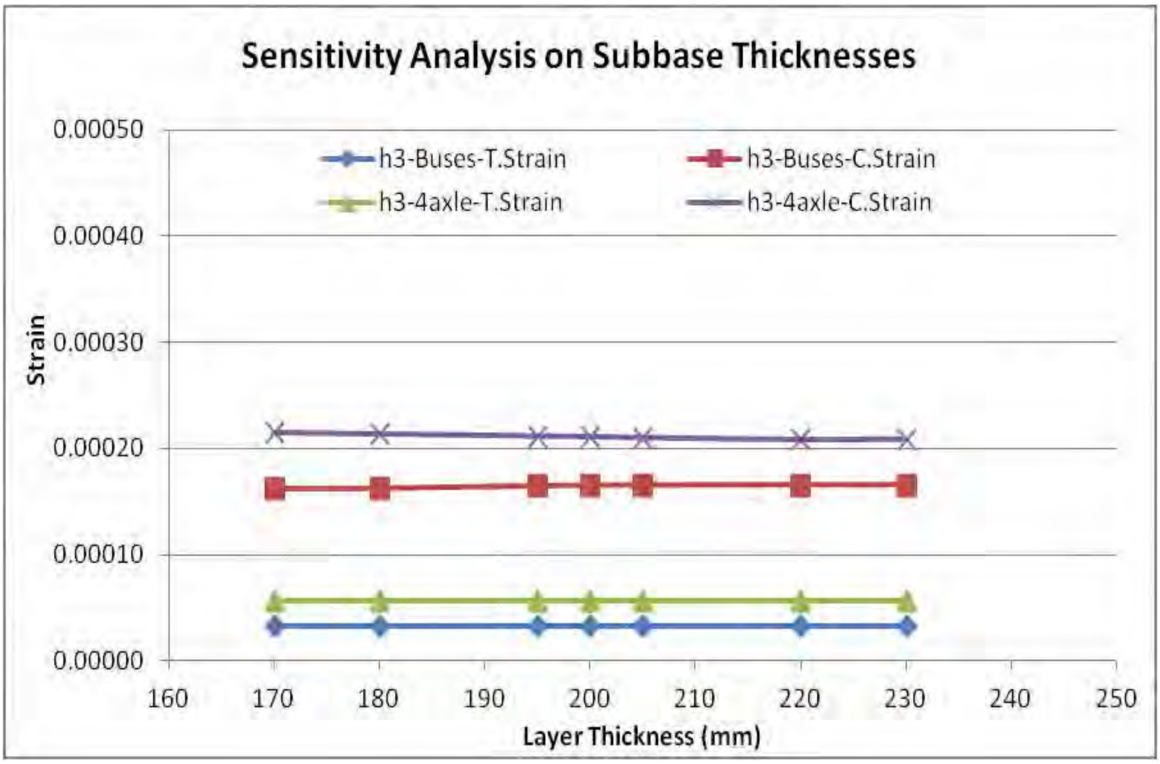



Figure 9.4 – Sensitivity of the Subbase Thickness

- As can be seen from the three Figures, 9.2, 9.3 and 9.4, the strains induced by the 4-axle vehicles are always higher than those induced by the buses irrespective of the layer under consideration. This is due to the fact that the loading configurations for the two types of vehicle are different and is the axle loads. Buses are usually used to carry passengers and the 4-axle vehicles are loaded with cargo and construction materials.

### 9.3 Sensitivity Analysis of the Effects of Layer Moduli on Pavement Strains

The magnitude of deformation (strain) of the road pavement associated with a given deviator stress at a point in the pavement structure due to traffic loading is dependent on the soil properties as soft soils may experience significant deformation while stiff soils deformation may be slight (Buchanan, 2007). A portion of the deformation may be recoverable or resilient (elastic) while the unrecoverable is plastic. The ratio of the deviator stress to the recoverable strain is the resilient modulus ($M_R$) of the material assuming that it is homogeneous and isotropic material.

The $M_R$ is therefore an indicator of how a pavement layer will resist (stiffness) deformation under traffic loading. It is therefore not a measure of strength rather the character of the pavement layer material under loading that prevents failure.

The pavement layers were kept at constant thicknesses while their moduli were varied to see the effects of these variations on the design lives as well as on the tensile and compressive strains. The strains under the two load groups were computed from the KENLAYER for each variation in modulus of the respective pavement layer, each period keeping three of the moduli constant, and then varying the other moduli of interest. The results are as shown in Table 9.9.

Table 9.9 – Results of Layer Moduli Variations on Strains

| Layer of Variation | Modulus ( $M_R$) - MPa | | | | Design Life (Yrs) | Buses | | 4 - Axle | |
|---|---|---|---|---|---|---|---|---|---|
| | HMA ($M_1$) | Base ($M_2$) | Subbase ($M_3$) | Subgrade ($M_4$) | | HMA Bottom Tensile Strain | Subgrade Top Compressive Strain | HMA Bottom Tensile Strain | Subgrade Top Compressive Strain |
| HMA | **2500** | 511 | 282 | 115 | 16.46 | 4.29E-06 | 9.54E-05 | 1.29E-05 | 2.14E-04 |
| | **2055** | 511 | 282 | 115 | 15.65 | 7.95E-06 | 1.20E-04 | 8.73E-06 | 2.18E-04 |
| | **1500** | 511 | 282 | 115 | 14.48 | 3.95E-07 | 9.77E-05 | 1.41E-06 | 2.23E-04 |
| Base | 2055 | **766.5** | 282 | 115 | 15.7 | 3.32E-05 | 1.65E-04 | 5.65E-05 | 2.11E-04 |
| | 2055 | **511** | 282 | 115 | 15.6 | 3.32E-05 | 1.65E-04 | 5.67E-05 | 2.11E-04 |
| | 2055 | **255.5** | 282 | 115 | 15.3 | 3.60E-05 | 1.59E-04 | 6.28E-05 | 2.17E-04 |
| Subbase | 2055 | 511 | **423** | 115 | 19.8 | 3.28E-05 | 1.56E-04 | 5.57E-05 | 1.97E-04 |
| | 2055 | 511 | **282** | 115 | 15.6 | 3.32E-05 | 1.65E-04 | 5.67E-05 | 2.11E-04 |
| | 2055 | 511 | **141** | 115 | 11.59 | 3.33E-05 | 1.71E-04 | 5.79E-05 | 2.31E-04 |
| Subgrade | 2055 | 511 | 282 | **172.5** | 76.87 | 2.33E-05 | 1.12E-04 | 4.05E-05 | 1.51E-04 |
| | 2055 | 511 | 282 | **115** | 15.64 | 3.32E-05 | 1.65E-04 | 5.67E-05 | 2.11E-04 |
| | 2055 | 511 | 282 | **57.5** | 1.1 | 5.87E-05 | 3.05E-04 | 9.62E-05 | 3.52E-04 |

From Table 9.9, the following observations may be made;

1. **HMA Modulus ($M_1$):**

- The variation in the modulus has a positive impact on the design life as $M_1$ is increased while the design life is decreased when $M_1$ is lowered.

2. **Base Modulus ($M_2$):**

- Increasing $M_2$ increases the design life and vice versa
- It is clear from the results that decreasing the $M_2$ generally causes an increase in both strains but significantly it can be observed that the tensile strains under the HMA layer are far higher than their corresponding compressive strains in the subgrade.
- It can therefore be concluded that variation of the Base modulus has more impact on the HMA tensile strains than the compressive strains on the subgrade.

3. **Subbase Modulus ($M_3$):**

- The design life varies proportionally with changes in value of $M_3$. Changes in the $M_3$ cause higher changes in the compressive strains than the tensile strains.
- Changes in the sub-base modulus affect both types of strain significantly but affects the subgrade compressive strains the most.

- High values of the compressive strains indicate failure is likely to occur as a result of a weak subgrade and in this regard, the subbase is the most critical layer in the pavement design.

4. **Subgrade Modulus ($M_4$):**

- Increasing $M_4$ has positive impact on the design while a decrease or lowering $M_4$ has a significant reduction on the pavement life.
- The magnitude of the strains s induced by vehicle load on top of the subgrade is reduced when the subgrade modulus was increased and vice versa when the modulus was reduced.
- The behaviour exhibited by the subgrade is similar to that of the subbase as a result of variation of their moduli. However, layer materials above the subbase show comparatively higher resistance to deformation as a result in the material modulus.

Table 9.10 summaries the effect of the change of the moduli of the respective pavement layers on the tensile and compressive strains due to buses and 4-axle trucks.

Table 9.10 – Summary of Effects of Moduli Variations on Strains

| **Layer** | **Change In Modulus (%)** | **Change In Design Life (%)** | **Buses** | | **4-Axle** | |
|---|---|---|---|---|---|---|
| | | | **Change In HMA Bottom Tensile Strain (%)** | **Change In Subgrade Top Compressive Strain (%)** | **Change In HMA Bottom Tensile Strain (%)** | **Change In Subgrade Top Compressive Strain (%)** |
| HMA (M1) | +21.7 | 5.2 | -46.0 | -20.5 | 47.8 | -1.8 |
| | -27.0 | -7.5 | -95.0 | -18.6 | -83.8 | 2.3 |
| Base (M2) | +50.0 | +0.1 | 0.0 | 0.0 | -0.2 | 0.0 |
| | -50.0 | -2.4 | +8.6 | -3.8 | +10.8 | +2.6 |
| Subbase (M3) | +50.0 | +26.4 | -1.1 | -5.6 | -1.7 | -6.6 |
| | -50.0 | -25.9 | +0.4 | +3.6 | +2.2 | +9.3 |
| Subgrade (M4) | +50.0 | +391.5 | -29.9 | -32.1 | -28.5 | -28.3 |
| | -50.0 | -93.0 | +77.0 | +85.0 | +69.8 | +66.6 |
| (-) – Decrease by (+) – Increase by | | | | | | |

Plots of the various sensitivities due to changes in the pavement layers moduli under the two load groups being considered are shown in Figures 9.5, 9.6 and 9.7 for HMA, Base and Sub-base respectively.

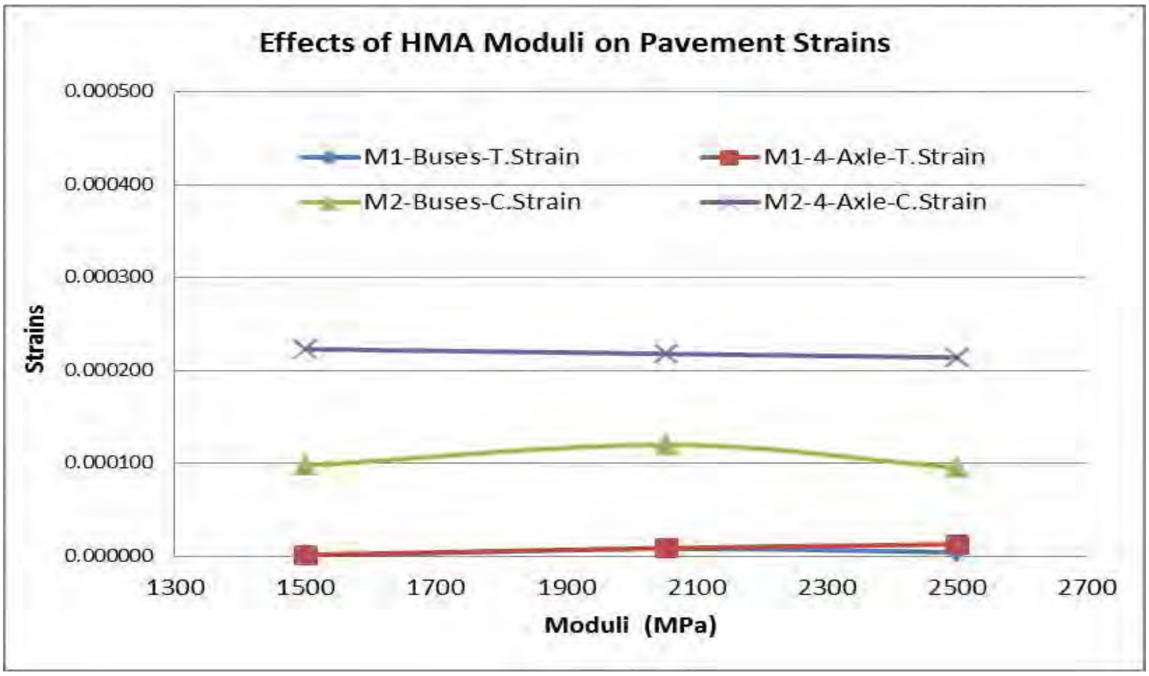



Figure 9.5 – Effects of M1 on Pavement Strains

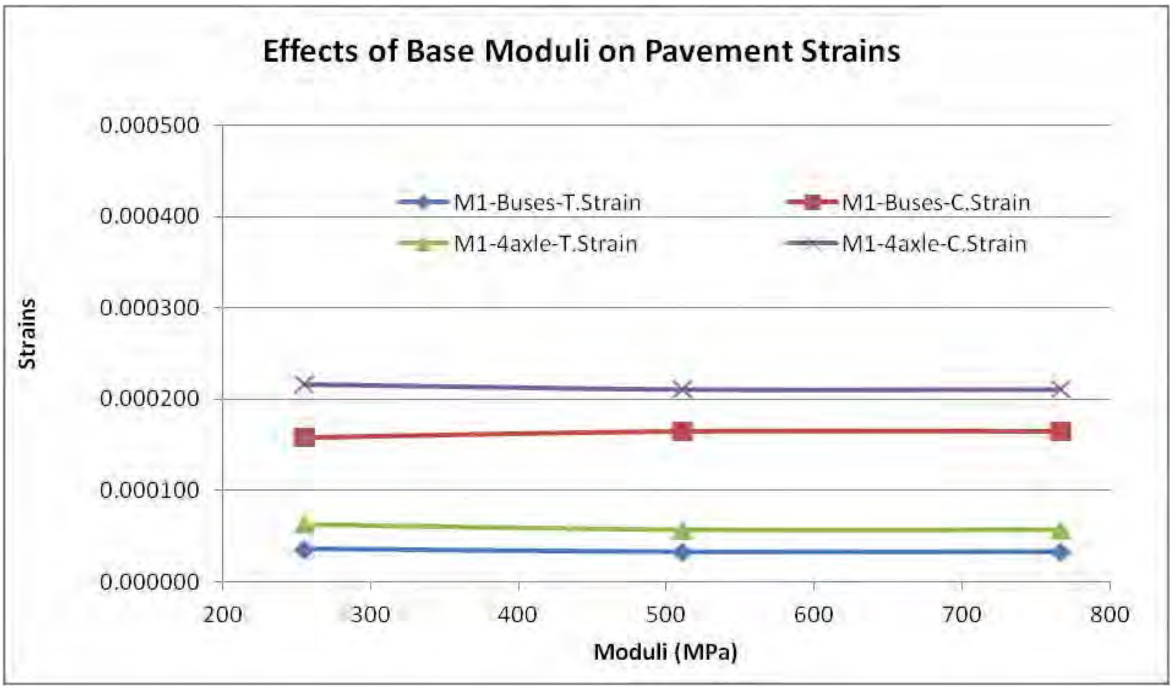



Figure 9.6 – Effects of M2 on Pavement Strains

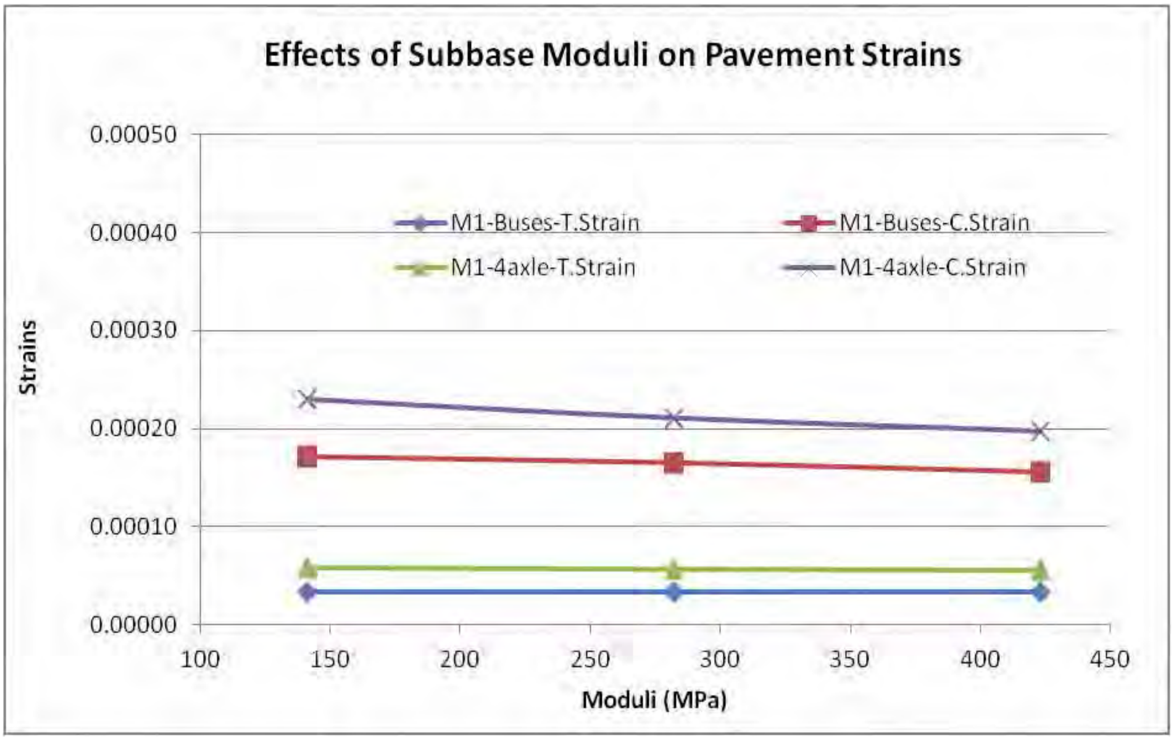



Figure 9.7 – Effects of M3 on Pavement Strains

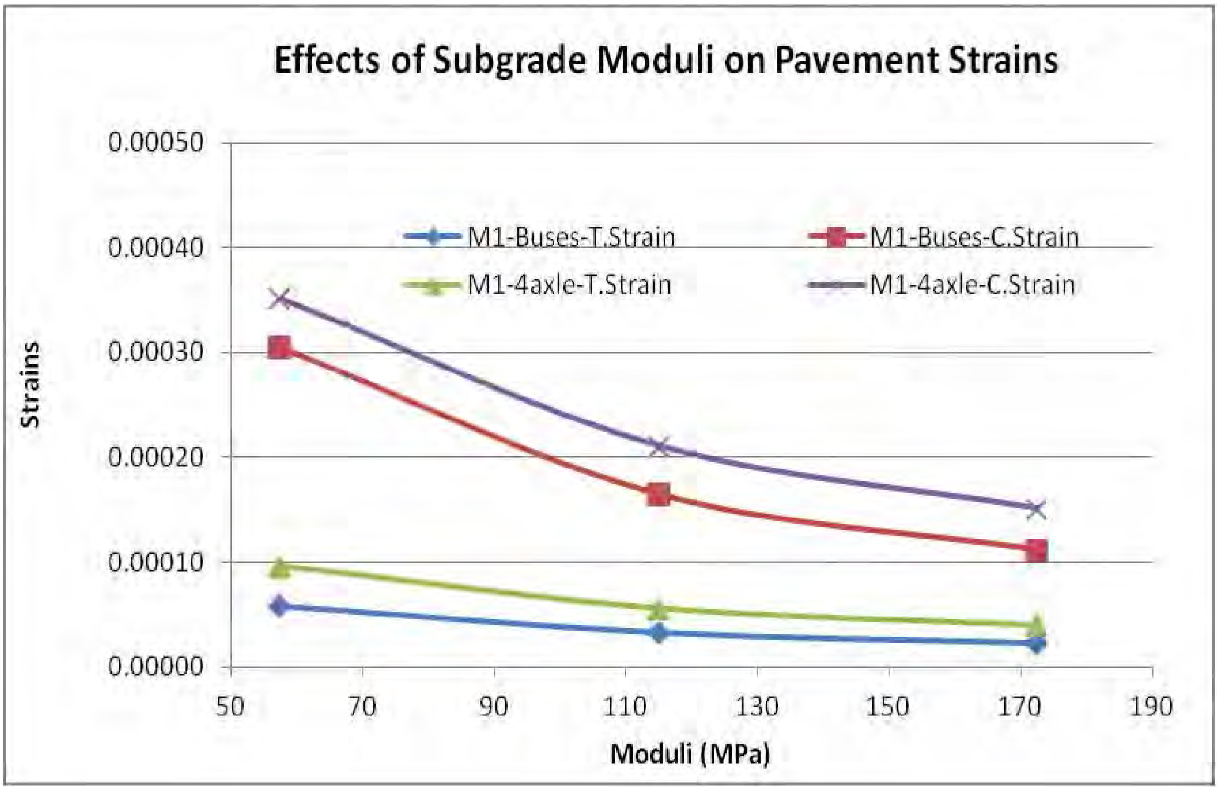



Figure 9.8 – Effects of M4 on Pavement Strains

As can be seen in Figures 9.5, 9.6 and 9.7, the strains under the 4-Axle loading are always higher than those of the buses (see above). Also it can be seen that the difference between the compressive strain at the top of the Subgrade and the tensile strain under the HMA layer is significantly smaller under the 4-Axle loading group compared to that of the strains under buses.

## 9.4 Selection of Candidate Pavement Design for Economic Evaluation

From the result of the KENLAYER, a total of thirty-seven (37) candidate pavement designs were selected for economic evaluation. Table 9.11 gives the profiles of the selected pavements designs.. The financial cost used in the whole life cycle analysis was computed from departmental rates obtained from the Department of Urban Roads of Ghana‟s Ministry of Roads and Highways. The details are given in Appendix K.

Table 9.11 - Selected Pavement Designs Layer Thicknesses for Dry Sub-Humid Zone

| No. | Asphalt Concrete Layer (HMA) (mm) | Base Layer (mm) | Sub-base Layer (mm) | Design Life (years) | Financial Unit Cost per km (Ghana Cedis) | Financial Unit Cost per km (USD) Rate @ 1.5 | Economic Cost (85% of Financial Cost) (USD) |
|---|---|---|---|---|---|---|---|
| 1 | 165 | 200 | 200 | 15.64 | 1,341,130.40 | 894,086.94 | 759,973.90 |
| 2 | 180 | 175 | 200 | 14.87 | 1,415,849.01 | 943,899.34 | 802,314.44 |
| 3 | 180 | 145 | 200 | 14.14 | 1,395,774.51 | 930,516.34 | 790,938.89 |
| 4 | 180 | 145 | 180 | 14.11 | 1,392,471.51 | 928,314.34 | 789,067.19 |
| 5 | 180 | 175 | 180 | 14.82 | 1,412,546.01 | 941,697.34 | 800,442.74 |
| 6 | 120 | 250 | 300 | 19.25 | 1,118,320.84 | 745,547.23 | 633,715.14 |
| 7 | 180 | 250 | 300 | 20.60 | 1,477,750.26 | 985,166.84 | 837,391.81 |
| 8 | 165 | 250 | 300 | 20.24 | 1,392,662.90 | 928,441.94 | 789,175.65 |
| 9 | 140 | 250 | 300 | 19.67 | 1,240,250.65 | 826,833.76 | 702,808.70 |
| 10 | 180 | 200 | 300 | 16.97 | 1,450,652.76 | 967,101.84 | 822,036.56 |
| 11 | 165 | 200 | 300 | 16.67 | 1,359,205.40 | 906,136.94 | 770,216.40 |
| 12 | 140 | 200 | 300 | 15.93 | 1,206,793.15 | 804,528.76 | 683,849.45 |
| 13 | 120 | 200 | 300 | 15.59 | 1,084,863.34 | 723,242.23 | 614,755.89 |
| 14 | 180 | 150 | 300 | 16.10 | 1,415,635.26 | 943,756.84 | 802,193.31 |
| 15 | 165 | 150 | 300 | 15.73 | 1,319,387.90 | 879,591.94 | 747,653.15 |
| 16 | 140 | 150 | 300 | 15.37 | 1,171,775.65 | 781,183.76 | 664,006.20 |
| 17 | 120 | 150 | 300 | 14.91 | 1,049,845.84 | 699,897.23 | 594,912.64 |
| 18 | 180 | 300 | 250 | 23.10 | 1,507,750.26 | 1,005,166.84 | 854,391.81 |
| 19 | 165 | 300 | 250 | 22.71 | 1,416,302.90 | 944,201.94 | 802,571.65 |
| 20 | 140 | 300 | 250 | 22.10 | 1,263,890.65 | 842,593.76 | 716,204.70 |
| 21 | 120 | 300 | 250 | 21.65 | 1,141,960.84 | 761,307.23 | 647,111.14 |
| 22 | 180 | 250 | 250 | 18.54 | 1,474,292.76 | 982,861.84 | 835,432.56 |
| 23 | 165 | 250 | 250 | 18.22 | 1,382,845.40 | 921,896.94 | 783,612.40 |
| 24 | 140 | 250 | 250 | 17.72 | 1,230,433.15 | 820,288.76 | 697,245.45 |
| 25 | 120 | 250 | 250 | 17.35 | 1,108,503.34 | 739,002.23 | 628,151.89 |
| 26 | 180 | 200 | 250 | 15.49 | 1,440,835.26 | 960,556.84 | 816,473.31 |
| 27 | 165 | 200 | 250 | 15.31 | 1,349,387.90 | 899,591.94 | 764,653.15 |
| 28 | 140 | 200 | 250 | 15.05 | 1,196,975.65 | 797,983.76 | 678,286.20 |
| 29 | 120 | 200 | 250 | 14.73 | 1,075,045.84 | 716,697.23 | 609,192.64 |
| 30 | 180 | 150 | 250 | 14.70 | 1,407,377.76 | 938,251.84 | 797,514.06 |
| 31 | 165 | 150 | 250 | 14.46 | 1,315,930.40 | 877,286.94 | 745,693.90 |
| 32 | 140 | 150 | 250 | 14.09 | 1,163,518.15 | 775,678.76 | 659,326.95 |
| 33 | 120 | 150 | 250 | 13.88 | 1,041,588.34 | 694,392.23 | 590,233.39 |
| 34 | 180 | 300 | 200 | 21.13 | 1,499,492.76 | 999,661.84 | 849,712.56 |
| 35 | 165 | 300 | 200 | 20.79 | 1,408,045.40 | 938,696.94 | 797,892.40 |
| 36 | 140 | 300 | 200 | 20.25 | 1,255,633.15 | 837,088.76 | 711,525.45 |
| 37 | 120 | 300 | 200 | 19.86 | 1,133,703.34 | 755,802.23 | 642,431.89 |

### 9.5 Strategic Selection of Economic Pavement

The design process does not end with the selection of the pavement type that just meet the design life obtained from the KENLAYER. During the life of the pavement, rehabilitation and maintenance activities are carried out to ensure that the condition of the road does not deteriorate to an extent that will need total reconstruction. The main aim of the economic analysis is to look at the cost and benefit profiles of a given pavement design in terms of its construction, maintenance and rehabilitation costs and road user benefits for different scenarios and identify the most advantageous construction, maintenance and rehabilitation (M-R) strategies that will maximize road user benefit, given budget and other constraints.

A Life Cycle Cost analysis is conducted to compare alternate pavement designs which will assist in the selection of the final economic pavement design. In using the KENLAYER a catalogue of pavement designs is created, which is fed into the calibrated HDM-IV, to carry out economic evaluation of these candidate pavements using life cycle strategy..

#### 9.5.1 The HDM-IV Model Approach

The model permits the evaluation of several maintenance and rehabilitation alternatives for given pavement designs. The model computes the aggregate costs of carrying out specified maintenance, rehabilitation and construction policies; the associated vehicle operating costs and the time streams of the total life cycle costs discounted at a given rate to find the Net Present Values. Using these criteria, optimal pavement designs under a given budget constraints are obtained.

#### 9.5.2 Application of the HDM-4 Model

In order to determine whether an adequate return in terms of benefits results from making an investment, cost-benefit analysis must be carried out. This is done through net present value (NPV) decision rule. The NPV is used to determine which investment option gives the highest return of those considered (Robinson et al., 1998). The HDM-4 application assumed an analysis period of 25 years. This is based on economic life of the road asset which extends beyond the design life. Normally, in Ghana, most of asphaltic concrete roads have a design life of 15 to 20 years. Most of the donor funded highway investment in Ghana had used a discount rate of 12% notably among these investments are Highway Sector Investment Project and Road Sector Development Program. For this research, the NPV value was used as measure of effectiveness of a given pavement design. The optimum pavement design is therefore the one that maximizes the net present value for a given discount rate. The NPV analysis result provides the basis for the selection of the optimum pavement design appropriate for the Semi-Arid climatic zone. The HDM-IV output gives a scientific platform for further analysis to obtain a pavement which is appropriate for given a given traffic and climatic condition. Thus the model can be said to be instructive as an example for selection of economic pavement design.

## 9.6 HDM-4 Approach to Life Cycle Analysis

This section of the pilot study describes the methodology used to achieve the objective of the Life Cycle Analysis.

### 9.6.1 Data for HDM-IV

The data used for the study were mostly obtained from Ghana Highway Authority Road Database, Meteorological Services Agency and fieldwork taken for the calibration of the HDM-IV model for Ghana. The data collection process and the associated calibration of the models in HDM-4 were discussed in Chapter 7.

### 9.6.2 Construction Standard

On the average new construction is executed over 36 calendar months with a corresponding cost stream of three years. The first year of construction is used for the mobilization of the required resources and accommodation. Construction during the first year is about 25% of the total, in the second year a further 35-50% is achieved and the remaining 25-65% is achieved in the last 12 months of the construction period. Under this pilot study the physical completion over the 36 months was 25% in the first year, 50% in the second year and the remaining 25% in the third year. Types of upgrading construction considered under the pilot study was upgrading from gravel to asphaltic concrete. The bills of quantities for these two interventions used in derive the cost per kilometre given in Appendix K.

### 9.6.3 Unit Cost of Maintenance Alternative

The primary use of the unit cost is to convert an assessment of physical and operational needs on the network for project cost estimation. All unit costs for the financial and economic evaluation are given in US dollar. The unit cost is calculated based on the four main components; labour, equipment, materials and overheads.

### 9.6.4 Traffic Volume and Forecasting

For evaluation of the economic benefits, traffic volume estimation and forecasting were the main factors used. Actual traffic volumes have been obtained through the installation of permanent traffic counters at the test sections. Traffic Characterization is based on Section 9.1.1.

### 9.6.5 Maintenance Standards and Policies

Each of the candidate pavements was subjected to number probable maintenance policy alternatives, with the intention to obtain a life cycle cost which is the most economic. The policies consisted of different maintenance strategies, with specification of the deterioration levels at which it will be triggered. The pavement maintenance options are restricted to only asphaltic concrete and ranged from pothole patching, crack sealing and overlay. Rehabilitation and reconstruction are categorized as improvement works. As a benchmark against which to compare differences in NPVs of candidate pavement designs, a “Do minimum” case was defined as that including only the basic routine maintenance activities, (e.g. drainage cleaning, bush clearing, minimal vegetation control, shoulder repairs and miscellaneous activities). These are also included in the other maintenance alternatives. The choice of maintenance operations is strongly dependent on the differences in the deterioration characteristics. The deterioration characteristics are also dependent on climatic conditions under which the road pavement is built and used. Unlike unpaved roads where the deterioration is linear, it is nonlinear for paved roads, thus offering more option for choice and timing of maintenance. In order to have a fair basis for selection of optimum pavement design, the NPV, Agency Cost with respect to construction and maintenance cost and the Vehicle Operating Cost (VOC) for the candidate pavement designs were examined at a unit rate (i.e. cost or benefit per kilometre). Table 9.12 gives the summary of the maintenance standards and alternatives used.

Table 9.12 - Maintenance Policy Alternatives for Asphaltic Concrete

<table>
<tr><th rowspan="2">Policy Code</th><th rowspan="2">Pothole Patching (%)</th><th rowspan="2">Crack Sealing</th><th colspan="4">Overlay Thickness</th><th rowspan="2">Trigger (yrs)</th></tr>
<tr><th>30mm</th><th>50mm</th><th>70mm</th><th>100mm</th></tr>
<tr><td rowspan="6">Used to Differentiate Any Two Maintenance Strategies</td><td>100</td><td>100</td><td colspan="4">@4.0 IRI</td><td rowspan="6">5-8</td></tr>
<tr><td>100</td><td>100</td><td colspan="4">@5.0 IRI</td></tr>
<tr><td>100</td><td>100</td><td colspan="4">@6.0 IRI</td></tr>
<tr><td>100</td><td>100</td><td colspan="4">@7.0 IRI</td></tr>
<tr><td>100</td><td>100</td><td colspan="4">@ 8.0 IRI</td></tr>
<tr><td>100</td><td>100</td><td colspan="4">@9.0 IRI</td></tr>
<tr><td colspan="8">Example Code: 5088I depicts<br>A maintenance Policy of Overlay 50mm AC @ 8.0 IRI – 8yrs Intervals</td></tr>
</table>

From Table 9.12 over 65 maintenance policy alternatives can be evolved.. All the alternatives include basic Routine Maintenance, such as drainage cleaning, vegetation clearing, repair of shoulder and miscellaneous activities. Under this Pilot study a total of 33 maintenance policy alternatives were used. The maintenance intervention used under this pilot study was scheduled intervention in line with maintenance practices in Ghana.

### 9.6.6 Analysis of HDM-IV Output

In determining which pavement type is the best in terms of Economic Analyses, the Least Life Cycle Cost for all all the pavement design types under consideration were obtained. From Table 9.13, the Life Cycle Alternatives of the 37pavements types under consideration are provided. The Construction and Maintenance Policy Alternative which gives the Least Life Cycle Cost is selected from each pavement type for further analyses.

The Net Economic Benefit or Net Present Value (NPV) and the Economic Internal Rate of Return (EIRR) were determined from those selected (see Table 9.13). Pavements that give high NPV and EIRR are then again selected and their total Road Agency Costs (RAC) found.

Table 9.13 - HDM-IV Output for Life Cycle Analysis for Semi-Arid Climate Zone

| Pavement No. | Pavement | Total Life Cycle Alternatives | Least Life Cycle Cost (million US$) | Net Present Value, NPV | Economic Internal Rate of Return, EIRR (%) |
|---|---|---|---|---|---|
| 1 | 165-200-200 | 6 IRI, 70mm - 7yrs | 1556.217 | 84.04 | 21 |
| 2 | 180-175-200 | 6 IRI, 70mm - 7yrs | 1559.816 | 80.29 | 20.3 |
| 3 | 180-145-200 | 6 IRI, 70mm - 7yrs | 1534.952 | -14.9 | 10.5 |
| 4 | 180-145-180 | 6 IRI, 70mm - 7yrs | 1534.793 | -14.74 | 10.5 |
| 5 | 180-175-180 | 6 IRI, 70mm - 7yrs | 1559.672 | 81.01 | 20.4 |
| 6 | 120-250-300 | 6 IRI, 70mm - 7yrs | 1545.485 | 95.22 | 23.6 |
| 7 | 180-250-300 | 6 IRI, 70mm - 7yrs | 1562.798 | 77.18 | 19.7 |
| 8 | 165-250-300 | 7 IRI, 70mm - 8yrs | 1606.361 | 67.89 | 19 |
| 9 | 140-250-300 | 4 IRI, 70mm - 8yrs | 1599.749 | 78.02 | 21 |
| 10 | 180-200-300 | 6 IRI, 70mm - 7yrs | 1561.493 | 78.54 | 20 |
| 11 | 165-200-300 | 6 IRI, 70mm - 7yrs | 1562.798 | 77.18 | 19.7 |
| 12 | 140-200-300 | 4 IRI, 70mm - 8yrs | 1598.138 | 79.7 | 21.4 |
| 13 | 120-200-300 | 7 IRI, 70mm - 7yrs | 1592.265 | 85.82 | 22.9 |
| 14 | 180-150-300 | 7 IRI, 70mm - 7yrs | 1608.197 | 69.21 | 19.3 |
| 15 | 165-150-300 | 7 IRI, 70mm - 8yrs | 1603.561 | 74.05 | 20.2 |
| 16 | 140-150-300 | 7 IRI, 70mm - 8yrs | 1596.451 | 81.46 | 21.8 |
| 17 | 120-150-300 | 7 IRI, 70mm - 8yrs | 1590.578 | 87.58 | 23.4 |
| 18 | 180-300-250 | 7 IRI, 70mm - 8yrs | 1612.634 | 64.59 | 18.5 |
| 19 | 165-300-250 | 7 IRI, 70mm - 8yrs | 1608.229 | 69.18 | 19.3 |
| 20 | 140-300-250 | 7 IRI, 70mm - 8yrs | 1600.888 | 76.83 | 20.8 |
| 21 | 120-300-250 | 7 IRI, 70mm - 8yrs | 1595.015 | 82.95 | 22.2 |
| 22 | 180-250-250 | 7 IRI, 70mm - 8yrs | 1611.022 | 66.27 | 18.8 |
| 23 | 165-250-250 | 7 IRI, 70mm - 8yrs | 1606.618 | 70.86 | 19.6 |
| 24 | 140-250-250 | 7 IRI, 70mm - 8yrs | 1599.276 | 78.51 | 21.2 |
| 25 | 120-250-250 | 7 IRI, 70mm - 6yrs | 1593.404 | 84.63 | 22.6 |
| 26 | 180-200-250 | 7 IRI, 70mm - 8yrs | 1609.411 | 67.95 | 19.1 |
| 27 | 165-200-250 | 7 IRI, 70mm - 8yrs | 1605.006 | 72.54 | 19.9 |
| 28 | 140-200-250 | 7 IRI, 70mm - 8yrs | 1597.665 | 80.19 | 21.5 |
| 29 | 120-200-250 | 7 IRI, 70mm - 8yrs | 1591.792 | 86.31 | 23.1 |
| 30 | 180-150-250 | 7 IRI, 70mm - 8yrs | 1607.799 | 69.63 | 19.4 |
| 31 | 165-150-250 | 7 IRI, 70mm - 8yrs | 1603.395 | 74.22 | 20.3 |
| 32 | 140-150-250 | 7 IRI, 70mm - 8yrs | 1596.053 | 81.87 | 21.9 |
| 33 | 120-150-250 | 7 IRI, 70mm - 8yrs | 1587.218 | 89.27 | 23.7 |
| 34 | 180-300-200 | 7 IRI, 70mm - 8yrs | 1612.236 | 65.01 | 18.6 |
| 35 | 165-300-200 | 7 IRI, 70mm - 8yrs | 1607.831 | 69.6 | 19.4 |
| 36 | 140-300-200 | 7 IRI, 70mm - 8yrs | 1600.49 | 77.25 | 20.9 |
| 37 | 120-300-200 | 7 IRI, 70mm - 8yrs | 1594.617 | 83.37 | 22.3 |

Pavements from Table 9.13 that have their EIRR of 15% and above were selected for further analyses. The choice of 15% EIRR threshold is in line with road investment justification used in Ghana by development partners such the World Bank, European Union, African Development Bank etc. The NPV/RAC ratios were calculated using HDM-4 for the selected pavements (Table 9.14) and accordingly those pavements with the highest ratios can be considered to be the most economic..

Table 9.14 - NPV-RAC Ratios for Pavement Types

| Pavement No. | Pavement | Total Life Cycle Alternatives | Net Present Value( NPV) | Road Agency Cost ( RAC) | NPV/RAC |
|---|---|---|---|---|---|
| 33 | 120-150-250 | 7 IRI, 70mm - 8yrs | 89.27 | 60.788 | 1.469 |
| 17 | 120-150-300 | 7 IRI, 70mm - 8yrs | 87.58 | 62.893 | 1.393 |
| 29 | 120-200-250 | 7 IRI, 70mm - 8yrs | 86.31 | 64.158 | 1.345 |
| 13 | 120-200-300 | 7 IRI, 70mm - 7yrs | 85.82 | 64.651 | 1.327 |
| 25 | 120-250-250 | 7 IRI, 70mm - 6yrs | 84.63 | 65.837 | 1.285 |
| 1 | 165-200-200 | 6 IRI, 70mm - 7yrs | 84.04 | 67.41 | 1.247 |
| 37 | 120-300-200 | 7 IRI, 70mm - 8yrs | 83.37 | 67.102 | 1.242 |
| 21 | 120-300-250 | 7 IRI, 70mm - 8yrs | 82.95 | 67.517 | 1.229 |
| 32 | 140-150-250 | 7 IRI, 70mm - 8yrs | 81.87 | 68.599 | 1.193 |
| 16 | 140-150-300 | 7 IRI, 70mm - 8yrs | 81.46 | 69.013 | 1.180 |
| 5 | 180-175-180 | 6 IRI, 70mm - 7yrs | 81.01 | 70.995 | 1.141 |
| 28 | 140-200-250 | 7 IRI, 70mm - 8yrs | 80.19 | 70.278 | 1.141 |
| 2 | 180-175-200 | 6 IRI, 70mm - 7yrs | 80.29 | 71.161 | 1.128 |
| 12 | 140-200-300 | 4 IRI, 70mm - 8yrs | 79.7 | 70.771 | 1.126 |
| 24 | 140-250-250 | 7 IRI, 70mm - 8yrs | 78.51 | 71.958 | 1.091 |
| 10 | 180-200-300 | 6 IRI, 70mm - 7yrs | 78.54 | 72.908 | 1.077 |
| 9 | 140-250-300 | 4 IRI, 70mm - 8yrs | 78.02 | 72.451 | 1.077 |
| 36 | 140-300-200 | 7 IRI, 70mm - 8yrs | 77.25 | 73.223 | 1.055 |
| 20 | 140-300-250 | 7 IRI, 70mm - 8yrs | 76.83 | 73.637 | 1.043 |
| 7 | 180-250-300 | 6 IRI, 70mm - 7yrs | 77.18 | 74.268 | 1.039 |
| 11 | 165-200-300 | 6 IRI, 70mm - 7yrs | 77.18 | 74.268 | 1.039 |
| 6 | 120-250-300 | 6 IRI, 70mm - 7yrs | 95.22 | 96.225 | 0.990 |
| 31 | 165-150-250 | 7 IRI, 70mm - 8yrs | 74.22 | 76.25 | 0.973 |
| 15 | 165-150-300 | 7 IRI, 70mm - 8yrs | 74.05 | 76.423 | 0.969 |
| 27 | 165-200-250 | 7 IRI, 70mm - 8yrs | 72.54 | 77.929 | 0.931 |
| 23 | 165-250-250 | 7 IRI, 70mm - 8yrs | 70.86 | 79.609 | 0.890 |
| 30 | 180-150-250 | 7 IRI, 70mm - 8yrs | 69.63 | 80.84 | 0.861 |
| 35 | 165-300-200 | 7 IRI, 70mm - 8yrs | 69.6 | 80.874 | 0.861 |
| 14 | 180-150-300 | 7 IRI, 70mm - 7yrs | 69.21 | 81.255 | 0.852 |
| 19 | 165-300-250 | 7 IRI, 70mm - 8yrs | 69.18 | 81.288 | 0.851 |
| 26 | 180-200-250 | 7 IRI, 70mm - 8yrs | 67.95 | 82.52 | 0.823 |
| 8 | 165-250-300 | 7 IRI, 70mm - 8yrs | 67.89 | 82.683 | 0.821 |
| 22 | 180-250-250 | 7 IRI, 70mm - 8yrs | 66.27 | 84.199 | 0.787 |
| 34 | 180-300-200 | 7 IRI, 70mm - 8yrs | 65.01 | 85.464 | 0.761 |
| 18 | 180-300-250 | 7 IRI, 70mm - 8yrs | 64.59 | 85.879 | 0.752 |

From the study, pavements selected for construction was based on those with design life within the range of 15 years and 20 years were considered. From these, other factors such as availability and cost of pavement materials under variation and finally select one that best suits all factors considered. Table 9.15 gives the final selected pavements ranked in terms of the most economic.

Table 9.15 - Final Selection of Economic Pavement Design for Semi-Arid

| **Rank** | **Pavement** | **Total Life Cycle Alternatives** | **Road Agency Cost, RAC** | **NPV/RAC** | **Obtained Design Life (years)** | **Required Design Life (years)** |
|---|---|---|---|---|---|---|
| 1 | 120-300-250 | 7 IRI, 70mm - 8yrs | 67.517 | 1.229 | 21.65 | 20 |
| 2 | 140-300-200 | 7 IRI, 70mm - 8yrs | 73.223 | 1.055 | 20.25 | |
| 3 | 140-300-250 | 7 IRI, 70mm - 8yrs | 73.637 | 1.043 | 22.10 | |
| 4 | 180-250-300 | 6 IRI, 70mm - 7yrs | 74.268 | 1.039 | 20.60 | |
| | | | | | | |
| 1 | 120-250-250 | 7 IRI, 70mm - 6yrs | 65.837 | 1.285 | 17.35 | 15 |
| 2 | 165-200-200 | 6 IRI, 70mm - 7yrs | 67.41 | 1.247 | 15.64 | |
| 3 | 140-250-250 | 7 IRI, 70mm - 8yrs | 71.958 | 1.091 | 17.72 | |
| 4 | 180-200-300 | 6 IRI, 70mm - 7yrs | 72.908 | 1.077 | 16.97 | |
| 5 | 165-200-300 | 6 IRI, 70mm - 7yrs | 74.268 | 1.039 | 16.67 | |

# CHAPTER 10

# FINDINGS, CONCLUSIONS AND RECOMMENDATIONS

## 10.1 Introduction

The main objective of this study was to evolve a new framework for asphaltic concrete pavement designs for tropical countries using Ghana as a case study. This chapter summarises the findings and conclusions of the study and provides recommendations for further studies to extend the frontiers of knowledge in tropical pavement design.

The preceding chapters of this research outlined the study background as well as the relevant literature review on the subject matter. These chapters highlighted the study approach and methodology, presented and described the data collected, and used statistical tools to establish relationships between the design parameters such as the resilience modulus, actual traffic loadings, pavement layer temperature, and the seasonal effect of moisture content on the various granular layers as a result of seasonal rainfall. From the previous studies on engineering properties of Ghana laterite soils and the current technical specification, a comparative analysis was carried out to establish the structural integrity of laterite for use as road pavement material. The technical specification compiled in the study will serve as a guide in the selection and improvement of local laterite materials for use as a pavement layer material. The calibration of the KENLAYER and the HDM-4 models to improve on the reliability of outputs were also carried out.

This final chapter is in three main sections. The first section presents the findings in relation to the study objectives, while the second section provides the study conclusions. The final section is devoted to the study"s recommendations and areas for further research.

## 10.2 Findings

### 10.2.1 Summary of General Findings

The new framework developed in this study will provide a rational and scientific basis to design road pavements for tropical countries using mechanistic-empirical pavement design principles which is a clear departure from the use of design manuals from other countries whose climatic conditions are completely different from Ghana"s tropical conditions. The findings of the study are summarised as follows:

1. The differences in behaviour of soils formed under tropical and temperate climates have been well established from the literature review in Chapter 2. Their engineering properties therefore will have to be determined using different standards for testing. Evaluation of tropical soils for road engineering using temperate soil standards as reference will lead to difficulties in design. In Chapter 6, comparative analysis was carried out based on earlier studies on Ghana"s tropical laterite soils and Ghana"s technical specification. Most of the quartzitic laterite soils were found to meet the accepted standards for either base or subbase materials. However, majority of the soils formed must be improved through stabilization before they can meet the required specification for road pavement construction.

2. In view of current scientific knowledge on the subject matter and improvements in technology, field studies for the current study were carried out in a cost effective manner to establish basis for comparison of parameters established from earlier studies.

3. The tropical design framework developed in this study has shown a more rational method which gives the pavement engineer control over the design parameters and a better method in the derivation of the candidate roads. This pavement design framework incorporates life cycle cost analysis which enables the determination of the appropriate maintenance standards with respect to routine and periodic maintenance. This framework, therefore, bridges the gap between technical design of pavement and its economic evaluation before selection of a given candidate pavement for construction. Based on the pilot study in Chapter 9, the framework has demonstrated that pavement design using mechanistic-empirical principles with input from a given

local condition gives a better result than the current approach where empirical studies are used to develop general manuals and monographs for use without serious consideration to the basis used to evolve these design parameters and their relevance to the environment in which they are going to be used; especially in tropical countries.

The advantage of using the proposed framework to select the optimum pavement design is that one can generate a host of candidate pavements using the KENLAYER and select a group from the sensitivity analysis which is based on a set of robust parameters and assumptions.

### 10.2.2 Preparation of Technical Specification for Tropical Laterite of Ghana

From the literature review of earlier studies, a clear difference between temperate and tropical soil formation has been established and, hence, difference in their respective engineering properties are well known. Consequently, the need to have distinct technical specifications to guide their use for road construction is therefore justified. Any attempt to develop specifications for tropical soils from temperate soil characteristics and engineering properties as the reference, will not give a true reflection of the capacity of tropical laterite soils and will lead to under estimation leading to ineffective or costly pavement design which is essentially the result of a high factor of safety adopted to avoid early pavement failure.

Extensive work has already been carried out on the general specifications for laterite materials in Ghana. Most significant among these are the various grading specifications proposed; some based on the mode of formation and physical properties of the soils, and some, on environmental factors like geology, rainfall, topography and drainage.

This work has come out with a proposed specification to support the use of group of laterite s which previous research had established their suitability for use as road construction material in Ghana. These proposed specifications have also been compared with the current specification and one advantage of the proposed specification is that of its specificity to particular types of laterite gravels that can be selected for use in the various climatic zones.

Further the proposed specifications require consideration of the properties of the materials, including plasticity, strength of coarse particles and compaction characteristics; as well as a method for rating them based on field performance. The suitability of each material can therefore be evaluated according to objective criteria.

### 10.2.3 Determination of Key Pavement Design Parameters for Ghana

The current research has been able to establish a methodology for tropical pavement design through the use of existing data from experimental test sections in two of the four climatic zones in Ghana to establish key design parameters.

Pioneer work under this research, is the field experimentation to monitor the pavement layer temperature which is a key parameter for the design of bituminous layers. This is the first time in Ghana where through establishment of experimental test sections to consistently monitor the temperature of pavement layers.

Field monitoring of bituminous layer of pavement established and average of 36°C as against assumed average of 25-30°C used as basis for mix designs of the asphaltic concrete pavement layer.

The maximum and minimum monthly ambient and pavement temperatures of the four (4) layers were established for the Moist Sub-humid and the Sub-Dry Humid Zone climatic zones. The study revealed that a linear relationship exists between temperature and pavement surface deflection. The pavement layer modulus varies inversely with cumulative traffic.

Actual elastic recovery of the pavement was also established to validate the use of viscoelastic and nonlinear theories for mechanistic road pavement design. The elastic recovery pattern was used as the basis to calibrate the KENLAYER for the pavement design analysis.

Finally, clear trends were established when the pavement temperature and moisture regime in the granular layers of the pavement monitored at the test sections were compared with rainfall, sunshine duration and evapotranspiration data obtained from the Meteorological Services Authority.

The temperature levels of all the pavement layers were found to be high during the wet periods as a result of high sunshine durations experienced during these periods.

Moisture content levels of the granular layers are generally low during periods of high rainfall as a result of the high levels of evapotranspiration and temperature during these periods which leads to high loss of moisture.

During periods of low rainfall, temperature and evapotranspiration are equally low leading to less loss of moisture. The effect of this is increase in seepage of moisture into the lower layers of the pavement.

### 10.2.4 Development of Pavement Design Framework for Ghana

The main objective of this study was to develop a rational basis for tropical pavement designs using mechanistic-empirical design principles. This was achieved through the following:

(i) Empirical and other studies on tropical laterite soils have established almost all the input parameters needed to undertake a mechanistic-empirical pavement design. However, the proposed framework arising from this study also eliminates the use of design manuals and monographs which are not suitable for tropical climatic environments. With respect to proper characterization of strength pavement material, this was achieved through the back-calculation of the resilient modulus from the Falling Weight Deflectometer (FWD), deflection field measurements.

(ii) In the calibration of the pavement design tools, this study has also showed that the proposed framework is systematic and reliable as very few assumptions are made in the pavement design process. All the important key parameters necessary to embark on mechanistic-empirical designs were established through actual field studies and data from Ghana. The KENLAYER was also calibrated using field data from the test sections. As part of the mechanistic-empirical design process, economic analysis of the candidate pavement design is required to select the most economic and cost effective pavement design by carrying out whole life cycle analysis. This was done using the HDM-4. This elaborate and objective process established from the research will be useful to the practising highway engineer working in Ghana. The gap between technical design and economic analysis has been bridged through this research.

Hitherto, most pavement designs used in Ghana did not take account of future maintenance regime and the cost thereof. Through the use of the HDM-4, the maintenance standard which gives the least whole life cycle cost has been established. This framework also gives a better appreciation of the total cost in the choice of a particular design. This study also reveals that most of the advocates of the mechanistic-empirical design do not go through the full process of both the engineering design and the economic analysis of the pavement for a given situation. Huang‟s (2004) effort ended with only the engineering design aspect without considering the economics of the candidate pavements generated by the KENLAYER mechanistic tool. Kerali et al., (1998); Odoki and Kerali, (2000) and Khan and Odoki‟s, (2010) established the capabilities of the HDM-4 to carry out life cycle analyses and economic evaluation of road pavement but not the engineering design. Under the aegis of this research, the full process of mechanistic-empirical design process has been achieved.

### 10.2.5 Application of the Proposed Pavement Design Framework to Select Optimum Design

In Chapter 9, a pilot study for the selection of economic pavement designs for the Dry Sub-Humid Zone for Ghana was carried out to demonstrate the processes involved. This pilot study has shown that pavement designs based on the conventional method and the new framework produce different results under an unconstrained budget. Given the same design life, the new approach is expected to give a better performance when constructed.

## 10.3 Conclusion

The objective of this study was to develop a pavement design framework for tropical countries using Ghana as a Case Study. Following an extensive analysis and discussion of gathered data, recommendations to improve the road pavement design process in tropical countries, based on empirical studies were made. In terms of contribution to knowledge, the study made the following specific findings:

1. The study has succeeded in using the empirical approach to establish the key parameters for road pavement design for Ghana. Gidigasu (1970, 1971, 1972 and 1980) and de Graft Johnson et al. (1969 and 1972) have done a lot of work by outlining challenges in tropical pavement design. Some of these challenges, particularly the over reliance on manuals developed elsewhere have been addressed through this research;
2. This study has also established a rational method of tropical pavement design using mechanistic-empirical design principles and, in addition, key design parameters have also been evolved; and

3. This Study has also enhanced the use of the mechanistic-empirical design method in Ghana;

## 10.4 Advantages and limitations of the New Pavement Design Framework

The advantages of the proposed framework as outlined as follows:

1. The input requirements are not complex and can be obtained locally;
2. The stresses and strains within the pavement layers can be generated for various classes of traffic loading and sensitivity analysis can be carried out at these critical locations;
3. The method does not require graphs or reliance of thresholds from empirical studies; and
4. Optimum pavement can only be selected after the economic analysis to establish life cycle cost.

The following are some of the limitations of the proposed framework which should be addressed through further research:

1. The mechanical application of the default values could give misleading results;
2. There is the need for periodic calibration of the tools used in the design to simulate local conditions; and
3. The non-linear layers response to loadings uses elastic principles.

## 10.5 Recommendations (Areas for Future Research)

This study has revealed other research needs and priorities that require attention in order to complement what has already been established. The following areas are recommended for further research:

- Further works similar to the one undertaken in this research, are needed in the two remaining climatic zones of Ghana;

- Studies to establish the K-values for non-linear materials for Ghana will have to be undertaken for all the geological zones in Ghana;
- Actual control test sections will have to be constructed to evaluate the field performance of pavement designs, based on the proposed approach developed under this Study;
- Studies to evolve a database for road pavement designs, by setting up Long Term Pavement Performance monitoring sites within the four climatic zones in Ghana
- Further studies into various design methods adopted for Ghana and their associated maintenance costs; and
- The assumptions used in the KENLAYER analysis must be localised for a more reliable and environmentally specific outputs.

This study has made a modest contribution to the selection of appropriate laterite materials for use as pavement materials in the various climatic zones of Ghana. In addition, adoption of the proposed framework will result in the development of technically feasible and economical and cost effective pavement designs for Ghana and other tropical countries.

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# APPENDIX A

## Pavement Material Properties

### SUMMARY OF RESULT

TEST PIT LOGS OF ALIGNMENT SOILS

| SITE | DEPTH (M) | MATERIAL DESCRIPTION | REMARKS |
|---|---|---|---|
| AKOMADAN | 0.00 – 0.03 | Bituminous surface dressing | Coarse |
| | 0.03 – 0.23 | GRAVEL, Sandy, Silty, Dense, Grey | |
| | 0.23 – 1.00 | GRAVEL, Sandy, Clayey, Medium Dense, Red Brown | |
| SOGAKOPE | 0 – 0.03 | Surface Dressing | Base |
| | 0.03 – 0.36 | Brownish gravelly clay | Subbase |
| | 0.36 – 0.38 | Humus Layer | Subgrade |
| | 0.38 – 0.68 | Light grey silty sand | |
| | 0.68 – 1.86 | Brownish sand (fine grains) | |

SUB GRADE MATERIAL

| SITE | DEPTH (M) | MATERIAL DESCRIPTION | NMC % | % PASSING (MM) | | | | | | | PLASTIC PROPERTIES | | | | AASHTO GROUP | MODIFIED COMPACTION | | CBR (96 HRS SOAKING) COMPACTIVE EFFORT | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | 0.075 | 0.425 | 2.3 | 5 | 10 | 20 | 37.5 | LL % | PL % | PI % | PM % | | MDD mg/m3 | OMC % | 95 % | 98 % | 100 % |
| AKUMADAN | 0.03 – 0.23 | SAND, Gravelly, Clayey | 5.0 | 16 | 27 | 58 | - | - | 94 | - | 21 | 9 | 12 | 324 | A-2-6 | 2.12 | 4.9 | 34 | 52 | - |
| | 0.30 – 0.45 | GRAVEL, Sandy, Clayey | 8.0 | 17 | 28 | 40 | - | - | 99 | - | 25 | 14 | 11 | 296 | A-2-6 | - | - | - | - | - |
| SOGAKOPE | 0.25 – 0.55 | - | - | 22 | 33 | 44 | 59 | 89 | 99 | 100 | 38 | 14 | 24 | 792 | A-2-6 | 2.19 | 7.9 | 10 | 20 | 20 |

LABORATORY PROPERTIES OF BASE AND SUBBASE MATERIALS

| SITE | MATERIAL DESCRIPTION | % PASSING (MM) | | | | | | | | PLASTIC PROPERTIES | | | | | MODIFIED COMPACTION | | CBR (96 HRS SOAKING) % |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | 0.075 | 0.425 | 2.0 | 5.0 | 10.0 | 20.0 | 37.5 | 50.0 | LL % | PL % | PI % | PM % | SWELL AT 95 % | MDD Kg/m3 | OMC % | |
| AKOMADAN | SUBBASE | 12.5 | 20.3 | 22.7 | 31.5 | 62.6 | 93.4 | 100 | 100 | 36.5 | 22.4 | 14.1 | 286 | 0.05 | 2230 | 6.7 | 49 |
| | FILL | 31.4 | 49.5 | 52.3 | 65.9 | 85.1 | 96.5 | 100 | 100 | 42.0 | 22.9 | 19.1 | 945 | 0.04 | 2117 | 7.3 | 33 |
| | BASE | 6.1 | 15.8 | 32.0 | 50.0 | 66.6 | 81.8 | 100 | 100 | - | - | - | - | - | 2310 | 7.8 | - |
| SOGAKOPE | SUBBASE | 11.0 | 26.0 | 39.0 | 48.0 | 89.0 | 97.0 | 100 | 100 | 24.0 | 13.0 | 11.0 | 301 | - | - | - | - |
| | BASE | 10.0 | 18.0 | 34.0 | 49.0 | 64.0 | 80.0 | 100 | 100 | 26.0 | 23.0 | 23.0 | 235 | - | 2300 | 6.4 | 101 |

FIELD PROPERTIES FOR SUBBASE AND BASE COURSES

| SITE | MATERIAL DESCRIPTION | AVERAGE THICKNESS MM | FIELD DRY DENSITY (g/cm3) | | | COMPACTION (%) | | | MOISTURE CONTENT (%) | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | MAX | MIN | AVG | MAX | MIN | AVG | MAX | MIN | AVG |
| AKOMADAN | SUBBASE | 150 | 7.3 | 8.2 | 7.8 | 98.2 | | | 6.9 | 7.8 | 7.4 |
| | BASE | 200 | 3.9 | 3.7 | 3.8 | 100 | | | 4.7 | 6.8 | 5.8 |
| SOGAKOPE | SUBBASE | 200 | 2.17 | 2.09 | 2.12 | 99.4 | 95.9 | 97.5 | 7.1 | 6.5 | 6.8 |
| | BASE | 200 | 2.361 | 2.278 | 2.305 | 88.3 | 85.2 | 86.2 | 7.2 | 4.8 | 6.0 |

PROPERTIES OF ASPHALT CONCRETE LAYERS

| SITE | LAYER | MIX TEMP °C | COMPACTION TEMP °C | | | THICKNESS (MM) | | | % PASSING (MM) | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | MAX | MIN | AVG | MAX | MIN | AVG | 50 | 37.5 | 28 | 20 | 14 | 10 | 6 | 4 | 2 | 1 | 0.43 | 0.3 | 0.15 | 0.075 |
| AKOMADAN | BINDER COURSE | 164 | 140 | 139 | 140 | 70 | 55 | 63.1 | | | | 100 | 84 | 72 | 57 | 48 | 33 | 24 | 18 | 15 | 9 | 6 |
| | WEARING COURSE | 164 | 140 | 139 | 140 | 50 | 46 | 48 | | | | 100 | 99 | 81 | 66 | 52 | 40 | 29 | 20 | 17 | 12 | 8 |
| SOGAKOPE | DBM | 160 | 130 | | | 125 | | | 100 | 98.1 | 86.3 | | 65.6 | | 56.9 | | 29.6 | 21.6 | | 9.5 | 5.5 | 3 |
| | WEARING COURSE | 162 | 130 | | | 40 | | | | | | | 98.9 | 84 | 64.6 | 50.6 | 38.8 | 29.1 | 20.4 | 17.1 | 11.1 | 6.8 |

RESULTS OF MARSHAL TEST

| SITE | LAYER | BULK S.G. | BITUMEN CONTENT (%) | VMA (%) | VOID (%) | VFB (%) | STABILITY (N) | FLOW (MM) |
|---|---|---|---|---|---|---|---|---|
| AKOMADAN | BINDER COURSE | 2.399 | 4.6 | 14.7 | 4.6 | 69.3 | 11.990 | 2.8 |
| | WEARING COURSE | 2.474 | 5.1 | 14.6 | 5.0 | 74.9 | 12.654 | 3.0 |
| SOGAKOPE | DBM | 2.655 | 4.3 | | 4.0 | 71.0 | 14.700 | 2.8 |
| | WEARING COURSE | 2.665 | 5.3 | | 4.2 | 72 | 15.6 | 2.7 |

AGGREGATE PROPERTIES OF BINDER AND WEARING COURSES (AKOMADAN)

| SITE | PROPERTY | CEMENT | FILLER | 0/4 | 4/10 | 10/14 | 14/20 |
|---|---|---|---|---|---|---|---|
| AKOMADAN (BINDER COURSE AND WEARING COURSE ) | S.G. | 3.120 | 2.456 | | | | |
| | REL. DENSITY (SSD) | | | 2.665 | 2.671 | 2.677 | 2.681 |
| | WATER ABSORPTION % | | | 2.700 | 2.698 | 2.698 | 2.695 |
| | LAAV % | | | 0.8 | 0.1 | 0.45 | 0.3 |
| | ACV % | | | | 29 | 26.9 | 22.7 |
| | AIV % | | | | 16.9 | 18.7 | 20.5 |
| | FLAKINESS % | | | | 15.7 | 17.2 | 18.7 |
| | SAND EQUIVALENT | | | 65 | | 13.7 | |
| | STRIPPING | < 5 | | | | | |
| | CHLORIDE CONTENT % | 0.01 | | | | | |
| | SULFATE CONTENT % | 0.13 | | | | | |
| | | | | | | | |
| | SODIUM SULPHATE SOUNDNESS (FINE AGG.) % M/M | < 600 µm | 3.5 | | | | |
| | | < 1.18 µm | 2.3 | | | | |
| | | < 2.36 µm | 4.6 | | | | |
| | | < 4.75 µm | 2.8 | | | | |
| | | < 9.5 µm | 4.1 | | | | |

AGGREGATE PROPERTIES OF DBM LAYER (SOGAKOPE)

| SITE | PROPERTIES | RESULT | | SPEC. LIMITS |
|---|---|---|---|---|
| | | **40 – 6 MM** | **6 – 0 MM** | |
| SOGAKOPE (DBM) | S.G. | 2.68 | 2.63 | |
| | ABSORPTION % | 0.40 | 1.30 | |
| | SAND EQUIVALENT % | 84.5 | | |
| | LAAV % | 24.0 | | 35 MAX |
| | ACV % | 20.5 | | 28 MAX |
| | 10 % FINES VALUE (DRY) KN | 22.2 | | |
| | 10 % FINES VALUE (WET/DRY) KN | 89.5 | | 75 MIN |
| | FLAKINESS INDEX % | 12.4 | | 25 MAX |

AGGREGATE PROPERTIES OF WEARING COURSE (SOGAKOPE)

| SITE | PROPERTIES | RESULT | | | | SPEC. LIMITS |
|---|---|---|---|---|---|---|
| | | **14 – 10 MM** | **10 – 6 MM** | **6 – 0 MM** | **CEMENT** | |
| SOGAKOPE (WEARING COURSE) | S.G. | 2.68 | 2.67 | 2.67 | 3.12 | |
| | ABSORPTION % | 0.5 | 0.8 | 1.1 | | |
| | SAND EQUIVALENT % | | | | | |
| | LAAV % | 23.5 | | | | 30 |
| | ACV % | 20.6 | | | | 28 |
| | 10 % FINES VALUE (DRY) KN | 12.51 | | | | |
| | 10 % FINES VALUE (WET/DRY) KN | 90.7 | | | | 75 MIN |
| | FLAKINESS INDEX % | 14.9 | | | | 20 |

# APPENDIX B

## Sample of Climatic Data

| TABLE B.10: Monthly Rainfall Total (mm) (Wenchi Met. St.) | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | | | | | | |
| **Year** | **Jan** | **Feb** | **March** | **April** | **May** | **June** | **July** | **Aug** | **Sept** | **Oct** | **Nov** | **Dec** |
| | | | | | | | | | | | | |
| **1971** | 0.5 | 101.1 | 102.6 | 209.0 | 162.1 | 210.6 | 109.0 | 124.0 | 241.6 | 102.6 | 0.3 | 63.2 |
| **1972** | 0.0 | 23.9 | 101.6 | 130.6 | 302.0 | 227.6 | 158.0 | 35.3 | 92.7 | 252.7 | 0.0 | 19.6 |
| **1973** | 0.0 | 15.7 | 136.7 | 97.0 | 231.9 | 91.2 | 153.2 | 124.2 | 180.6 | 140.0 | 15.7 | 0.0 |
| **1974** | 0.0 | 25.9 | 103.9 | 74.4 | 126.5 | 115.8 | 85.1 | 76.7 | 151.6 | 239.3 | 11.9 | 5.1 |
| **1975** | 0.0 | 35.3 | 162.6 | 100.8 | 162.6 | 93.0 | 260.4 | 61.0 | 84.1 | 186.7 | 31.0 | 1.8 |
| **1995** | 0.0 | 1.9 | 76.8 | 232.3 | 163.5 | 177.9 | 115.8 | 117.6 | 290.6 | 145.6 | 16.9 | 33.7 |
| **1996** | 0.0 | 94.4 | 66.8 | 202.4 | 198.1 | 177.8 | 90.8 | 138.6 | 82.5 | 74.5 | 1.8 | 28.0 |
| **1997** | 0.0 | 0.0 | 121.8 | 93.2 | 183.7 | 153.0 | 74.2 | 78.0 | 99.7 | 197.6 | 18.3 | 0.0 |
| **1998** | 2.1 | 10.4 | 2.0 | 207.5 | 156.5 | 205.8 | 25.7 | 57.7 | 174.4 | 157.0 | 40.1 | 13.5 |
| **1999** | 2.4 | 37.2 | 193.7 | 192.5 | 160.5 | 111.0 | 113.0 | 105.8 | 101.5 | 188.3 | 82.9 | TR |
| **2000** | 49.2 | 0.0 | 31.8 | 153.2 | 135.9 | 294.0 | 170.4 | 73.6 | 90.4 | 138.6 | 49.7 | 0.0 |

| TABLE B.11: Mean Daily Maximum Temperature (°C) (Wenchi Met. St.) | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | | | | | | |
| **Year** | **Jan** | **Feb** | **March** | **April** | **May** | **June** | **July** | **Aug** | **Sept** | **Oct** | **Nov** | **Dec** |
| | | | | | | | | | | | | |
| **1971** | 33.3 | 33.9 | 34.2 | 32.5 | 30.9 | 29.8 | 28.1 | 28.1 | 28.9 | 29.6 | 29.4 | 30.7 |
| **1972** | 33.0 | 34.8 | 34.4 | 32.9 | 31.8 | 30.0 | 28.3 | 28.3 | 29.1 | 29.9 | 31.0 | 31.3 |
| **1973** | 33.2 | 34.1 | 33.9 | 32.6 | 31.9 | 28.9 | 27.5 | 27.9 | 28.7 | 28.9 | 30.9 | 30.8 |
| **1974** | 32.5 | 33.9 | 33.0 | 33.0 | 32.1 | 29.1 | 28.5 | 28.6 | 29.0 | 29.3 | 30.7 | 31.5 |
| **1975** | 32.9 | 32.8 | 32.9 | 31.5 | 30.8 | 28.9 | 27.9 | 28.5 | 28.7 | 29.4 | 29.9 | 31.3 |
| **1995** | 33.8 | 36.1 | 35.3 | 33.1 | 31.8 | 30.3 | 28.8 | 28.7 | 29.4 | 30.0 | 31.6 | 30.9 |
| **1996** | 33.4 | 33.9 | 34.1 | 32.9 | 32.3 | 30.0 | 28.7 | 28.3 | 28.9 | 30.0 | 32.4 | 31.3 |
| **1997** | 33.4 | 35.6 | 35.4 | 32.4 | 31.1 | 29.3 | 28.0 | 28.3 | 29.0 | 30.9 | 31.9 | 32.6 |
| **1998** | 34.7 | 36.1 | 38.0 | 34.6 | 32.6 | 30.6 | 28.4 | 28.9 | 29.4 | 30.0 | 32.0 | 32.3 |
| **1999** | 34.5 | 34.1 | 34.3 | 33.1 | 31.6 | 29.6 | 28.3 | 28.3 | 29.1 | 30.0 | 31.4 | 31.4 |
| **2000** | 33.4 | 35.1 | 35.1 | 33.3 | 32.2 | 29.7 | 28.2 | 27.9 | 29.2 | 30.1 | 31.2 | 31.3 |

| TABLE B.12: Mean Daily Minimum Temperature (°C) (Wenchi Met. St.) | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | | | | | | |
| **Year** | **Jan** | **Feb** | **March** | **April** | **May** | **June** | **July** | **Aug** | **Sept** | **Oct** | **Nov** | **Dec** |
| | | | | | | | | | | | | |
| **1971** | 19.7 | 21.9 | 22.5 | 22.8 | 21.8 | 21.9 | 20.8 | 21.6 | 21.4 | 21.6 | 21.7 | 19.7 |
| **1972** | 20.2 | 22.0 | 22.7 | 22.5 | 22.2 | 21.6 | 21.2 | 21.0 | 21.2 | 21.1 | 21.2 | 20.1 |
| **1973** | 19.4 | 22.6 | 21.9 | 22.5 | 22.1 | 21.0 | 21.7 | 20.8 | 20.7 | 21.2 | 20.9 | 19.6 |
| **1974** | 20.3 | 21.8 | 22.0 | 22.7 | 21.9 | 22.0 | 21.5 | 20.7 | 20.5 | 21.1 | 21.6 | 20.1 |
| **1975** | 20.5 | 22.2 | 22.5 | 22.2 | 22.0 | 21.7 | 21.3 | 21.2 | 20.8 | 20.8 | 20.9 | 19.9 |
| **1995** | 19.0 | 21.7 | 22.8 | 22.7 | 22.2 | 21.9 | 21.5 | 21.8 | 21.5 | 21.3 | 21.1 | 20.6 |
| **1996** | 21.5 | 22.2 | 22.9 | 22.8 | 22.5 | 21.8 | 21.1 | 21.4 | 21.5 | 20.9 | 20.7 | 21.7 |
| **1997** | 21.6 | 21.5 | 23.5 | 22.4 | 22.2 | 21.7 | 21.2 | 21.2 | 22.0 | 22.0 | 22.3 | 21.8 |
| **1998** | 19.9 | 23.0 | 24.9 | 23.7 | 23.1 | 22.0 | 21.8 | 21.3 | 21.6 | 21.6 | 22.2 | 21.2 |
| **1999** | 21.5 | 21.7 | 22.6 | 22.1 | 21.7 | 21.7 | 21.1 | 20.7 | 20.9 | 20.4 | 20.9 | 20.3 |
| **2000** | 21.9 | 20.8 | 23.3 | 22.5 | 22.4 | 21.6 | 21.0 | 20.2 | 21.1 | 21.3 | 21.7 | 20.2 |

| TABLE B.13: Mean Daily Relative Humidity (%) at 0600 hours (Wenchi Met. St.) | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | | | | | | |
| **Year** | **Jan** | **Feb** | **March** | **April** | **May** | **June** | **July** | **Aug** | **Sept** | **Oct** | **Nov** | **Dec** |
| | | | | | | | | | | | | |
| **1971** | 82 | 77 | 81 | 89 | 90 | 92 | 95 | 93 | 92 | 95 | 91 | 88 |
| **1972** | 68 | 75 | 86 | 92 | 94 | 96 | 96 | 95 | 95 | 97 | 94 | 83 |
| **1973** | 72 | 88 | 91 | 91 | 91 | 91 | 92 | 94 | 96 | 91 | 95 | 82 |
| **1974** | 83 | 85 | 89 | 96 | 89 | 92 | 93 | 96 | 98 | 96 | 92 | 84 |
| **1975** | 81 | 86 | 83 | 90 | 92 | 94 | 96 | 93 | 91 | 95 | 94 | 86 |
| **1995** | 37 | 66 | 86 | 92 | 96 | 96 | 96 | 96 | 97 | 96 | 92 | 87 |
| **1996** | 94 | 87 | 90 | 93 | 95 | 97 | 97 | 97 | 97 | 96 | 91 | 96 |
| **1997** | 86 | 47 | 77 | 92 | 94 | 96 | 95 | 96 | 96 | 95 | 95 | 82 |
| **1998** | 56 | 67 | 73 | 90 | 93 | 96 | 95 | 94 | 96 | 96 | 94 | 85 |
| **1999** | 81 | 71 | 90 | 92 | 95 | 95 | 96 | 94 | 96 | 96 | 95 | 77 |
| **2000** | 83 | 47 | 81 | 91 | 94 | 95 | 96 | 95 | 96 | 95 | 94 | 80 |

| TABLE B.14: Mean Daily Relative Humidity (%) at 1500 hours (Wenchi Met. St.) | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | | | | | | |
| **Year** | **Jan** | **Feb** | **March** | **April** | **May** | **June** | **July** | **Aug** | **Sept** | **Oct** | **Nov** | **Dec** |
| **1971** | 48 | 42 | 39 | 52 | 63 | 77 | 71 | 75 | 75 | 72 | 66 | 48 |
| **1972** | 31 | 31 | 43 | 59 | 65 | 70 | 72 | 72 | 72 | 70 | 62 | 46 |
| **1973** | 44 | 35 | 48 | 62 | 67 | 71 | 69 | 71 | 69 | 68 | 58 | 44 |
| **1974** | 36 | 42 | 51 | 63 | 64 | 66 | 66 | 69 | 71 | 73 | 61 | 49 |
| **1975** | 32 | 39 | 41 | 55 | 62 | 67 | 73 | 70 | 68 | 74 | 59 | 43 |
| **1995** | 14 | 20 | 44 | 59 | 66 | 69 | 73 | 76 | 72 | 72 | 57 | 54 |
| **1996** | 42 | 45 | 49 | 58 | 63 | 71 | 73 | 74 | 73 | 70 | 48 | 56 |
| **1997** | 40 | 17 | 31 | 56 | 65 | 73 | 72 | 73 | 75 | 72 | 62 | 45 |
| **1998** | 27 | 31 | 28 | 57 | 64 | 70 | 72 | 68 | 69 | 71 | 62 | 47 |
| **1999** | 30 | 33 | 50 | 60 | 64 | 68 | 71 | 70 | 71 | 73 | 69 | 37 |
| **2000** | 41 | 24 | 34 | 55 | 59 | 69 | 71 | 74 | 72 | 69 | 63 | 38 |

| TABLE B.15: Mean Daily Duration of Bright Sunshine (hours) (Wenchi Met. St.) | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | | | | | | |
| **Year** | **Jan** | **Feb** | **March** | **April** | **May** | **June** | **July** | **Aug** | **Sept** | **Oct** | **Nov** | **Dec** |
| | | | | | | | | | | | | |
| **1971** | 7.5 | 7.5 | 7.3 | 6.9 | 6.8 | 7.1 | 4.8 | 2.9 | 4.2 | 6.1 | 7.3 | 6.5 |
| **1972** | 7.7 | 7.6 | 6.8 | 6.2 | 7.3 | 7.5 | 2.6 | 2.7 | 4.1 | 5.8 | 8.0 | 6.8 |
| **1973** | 6.6 | 7.4 | 6.5 | 7.1 | 7.7 | 6.0 | 3.4 | 3.4 | 4.3 | 6.1 | 7.4 | 7.2 |
| **1974** | 7.6 | 8.0 | 7.1 | 7.4 | 6.9 | 5.3 | 4.3 | 3.1 | 3.8 | 5.2 | 7.4 | 6.0 |
| **1975** | 5.3 | 6.3 | 6.4 | 6.7 | 6.4 | 5.5 | 4.3 | 3.3 | 4.3 | 5.4 | 7.8 | 6.2 |
| **1996** | 7.4 | 7.6 | 7.7 | 7.6 | 7.3 | 6.2 | 4.9 | 3.7 | 4.1 | 5.9 | 7.4 | 6.0 |
| **1997** | 6.6 | 6.0 | 6.2 | 7.5 | 7.1 | 6.5 | 4.8 | 3.3 | 4.3 | 5.5 | 7.8 | 5.6 |
| **1998** | 5.6 | 7.1 | 7.9 | 7.8 | 6.9 | 5.6 | 4.5 | 3.5 | 4.8 | 5.9 | 7.7 | 6.1 |
| **1999** | 7.2 | 7.6 | 7.7 | 6.9 | 6.4 | 5.8 | 4.4 | 3.7 | 4.4 | 4.7 | 7.3 | 7.5 |
| **2000** | 7.0 | 8.4 | 6.5 | 7.6 | 7.4 | 6.0 | 4.5 | 3.5 | 3.8 | 6.4 | 6.7 | 7.1 |

| TABLE B.16: Mean Daily Potential Evapotranspiration (ins) (Wenchi Met. St.) | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | | | | | | |
| **Year** | **Jan** | **Feb** | **March** | **April** | **May** | **June** | **July** | **Aug** | **Sept** | **Oct** | **Nov** | **Dec** |
| | | | | | | | | | | | | |
| **1971** | 170.2 | 144.8 | 119.4 | 111.8 | 104.1 | 78.7 | 68.6 | 68.6 | 68.6 | 83.8 | 106.7 | 119.4 |
| **1972** | 142.2 | 157.5 | 147.3 | 106.7 | 88.9 | 81.3 | 68.6 | 63.5 | 71.1 | 78.7 | 109.2 | 127.0 |
| **1973** | 182.9 | 182.9 | 177.8 | 121.9 | 116.8 | 81.3 | 76.2 | 66.0 | 71.3 | 91.4 | 111.8 | 119.4 |
| **1974** | 185.4 | 190.5 | 137.2 | 134.6 | 111.8 | 88.9 | 76.2 | 83.8 | 71.1 | 78.7 | 106.7 | 132.1 |
| **1975** | 226.1 | 165.1 | 147.3 | 109.2 | 94.0 | 94.0 | 71.1 | 63.5 | 83.8 | 86.4 | 91.4 | 101.6 |
| **1995** | 246.4 | 245.3 | 185.1 | 128.8 | 104.1 | 89.4 | 73.7 | 70.3 | 78.2 | 84.4 | 115.3 | 119.3 |
| **1996** | 157.5 | 163.1 | 157.5 | 125.4 | 115.3 | 84.9 | 75.4 | 69.8 | 75.4 | 87.2 | 140.1 | 113.1 |
| **1997** | 167.1 | 258.2 | 210.9 | 127.7 | 100.1 | 75.9 | 69.2 | 71.4 | 70.3 | 88.9 | 106.9 | 125.4 |
| **1998** | 227.3 | 223.3 | 253.1 | 147.4 | 112.5 | 86.6 | 69.2 | 83.3 | 79.3 | 81.0 | 110.8 | 143.4 |
| **1999** | 199.1 | 192.9 | 157.5 | 126.4 | 123.8 | 81.9 | 66.9 | 74.6 | 74.9 | 82.3 | 112.3 | 125.6 |
| **2000** | 197.1 | 206.2 | 179.1 | 129.3 | 105.2 | 82.1 | 70.2 | 72.6 | 77.0 | 84.7 | 110.6 | 135.4 |

# APPENDIX C

## Monthly Temperature Values

**TABLE C.1: MONTHLY TEMPERATURE VALUES - (Mean, Standard Deviation and Coefficient of Variation) - JANUARY**

| | SOGAKOPE SITE | | | AKUMADAN SITE | | |
|---|---|---|---|---|---|---|
| **Temperature** | **Mean** | **Standard Deviation** | **Coefficient of Variation** | **Mean** | **Standard Deviation** | **Coefficient of Variation** |
| **Ambient** | 32.44 | 0.31 | 0.97 | 34.06 | 5.52 | 16.21 |
| **Subbase** | 35.54 | 0.29 | 0.81 | 37.75 | 0.52 | 1.39 |
| **Base** | 35.97 | 0.91 | 2.52 | 38.83 | 3.37 | 8.68 |
| **DBM/Binder** | 37.21 | 3.62 | 9.74 | 39.17 | 4.92 | 12.55 |
| **WC** | 37.75 | 6.68 | 17.71 | 39.33 | 7.76 | 19.72 |

Source: Field Data

**TABLE C.2: MONTHLY TEMPERATURE VALUES - (Mean, Standard Deviation and Coefficient of Variation) - FEBRUARY**

| | SOGAKOPE SITE | | | AKUMADAN SITE | | |
|---|---|---|---|---|---|---|
| **Temperature** | **Mean** | **Standard Deviation** | **Coefficient of Variation** | **Mean** | **Standard Deviation** | **Coefficient of Variation** |
| **Ambient** | 33.04 | 0.44 | 1.32 | 32.77 | 4.83 | 14.75 |
| **Subbase** | 37.02 | 0.68 | 1.85 | 37.32 | 1.38 | 3.71 |
| **Base** | 37.00 | 1.31 | 3.53 | 38.07 | 3.53 | 9.26 |
| **DBM/Binder** | 38.37 | 4.16 | 10.85 | 38.42 | 5.01 | 13.05 |
| **WC** | 38.85 | 7.12 | 18.34 | 38.68 | 7.71 | 19.94 |

Source: Field Data

**TABLE C.3: MONTHLY TEMPERATURE VALUES - (Mean, Standard Deviation and Coefficient of Variation) - MARCH**

| | SOGAKOPE SITE | | | AKUMADAN SITE | | |
|---|---|---|---|---|---|---|
| **Temperature** | **Mean** | **Standard Deviation** | **Coefficient of Variation** | **Mean** | **Standard Deviation** | **Coefficient of Variation** |
| **Ambient** | 32.55 | 0.35 | 1.06 | 30.18 | 4.10 | 13.60 |
| **Subbase** | 37.88 | 0.74 | 1.95 | 35.16 | 0.88 | 2.50 |
| **Base** | 36.89 | 1.40 | 3.79 | 34.62 | 3.16 | 9.12 |
| **DBM/Binder** | 38.11 | 4.29 | 11.25 | 34.61 | 4.33 | 12.51 |
| **WC** | 38.27 | 7.07 | 18.47 | 34.42 | 6.43 | 18.67 |

Source: Field Data

**TABLE C.4: MONTHLY TEMPERATURE VALUES - (Mean, Standard Deviation and Coefficient of Variation) - APRIL**

| | SOGAKOPE SITE | | | AKUMADAN SITE | | |
|---|---|---|---|---|---|---|
| **Temperature** | **Mean** | **Standard Deviation** | **Coefficient of Variation** | **Mean** | **Standard Deviation** | **Coefficient of Variation** |
| **Ambient** | 32.49 | 0.54 | 1.67 | 29.29 | 3.57 | 12.20 |
| **Subbase** | 38.34 | 1.51 | 3.94 | 33.33 | 0.58 | 1.75 |
| **Base** | 37.12 | 1.86 | 5.01 | 33.33 | 2.60 | 7.81 |
| **DBM/Binder** | 37.68 | 3.63 | 9.64 | 33.52 | 3.72 | 11.11 |
| **WC** | 37.67 | 6.31 | 16.76 | 33.64 | 5.69 | 16.91 |

Source: Field Data

**TABLE C.5: MONTHLY TEMPERATURE VALUES - (Mean, Standard Deviation and Coefficient of Variation) - MAY**

| | SOGAKOPE SITE | | | AKUMADAN SITE | | |
|---|---|---|---|---|---|---|
| **Temperature** | **Mean** | **Standard Deviation** | **Coefficient of Variation** | **Mean** | **Standard Deviation** | **Coefficient of Variation** |
| **Ambient** | 28.63 | 2.93 | 10.23 | | | |
| **Subbase** | 32.16 | 4.04 | 12.57 | | | |
| **Base** | 31.82 | 3.77 | 11.86 | | | |
| **DBM/Binder** | 32.11 | 4.77 | 14.84 | | | |
| **WC** | 32.45 | 6.39 | 19.68 | | | |

Source: Field Data

**TABLE C.6: MONTHLY TEMPERATURE VALUES - (Mean, Standard Deviation and Coefficient of Variation) - JUNE**

| | SOGAKOPE SITE | | | AKUMADAN SITE | | |
|---|---|---|---|---|---|---|
| **Temperature** | **Mean** | **Standard Deviation** | **Coefficient of Variation** | **Mean** | **Standard Deviation** | **Coefficient of Variation** |
| **Ambient** | 29.29 | 1.04 | 3.56 | 28.26 | 3.47 | 12.27 |
| **Subbase** | 33.56 | 1.83 | 5.44 | 32.74 | 0.70 | 2.13 |
| **Base** | 33.52 | 2.25 | 6.70 | 32.13 | 2.59 | 8.06 |
| **DBM/Binder** | 33.94 | 3.82 | 11.24 | 32.30 | 3.75 | 11.60 |
| **WC** | 34.34 | 5.76 | 16.77 | 32.28 | 5.82 | 18.05 |

Source: Field Data

**TABLE C.7: MONTHLY TEMPERATURE VALUES - (Mean, Standard Deviation and Coefficient of Variation) - JULY**

| | SOGAKOPE SITE | | | AKUMADAN SITE | | |
|---|---|---|---|---|---|---|
| **Temperature** | **Mean** | **Standard Deviation** | **Coefficient of Variation** | **Mean** | **Standard Deviation** | **Coefficient of Variation** |
| **Ambient** | 29.17 | 1.16 | 3.97 | 28.39 | 3.23 | 11.38 |
| **Subbase** | 34.10 | 0.93 | 2.73 | 33.24 | 0.62 | 1.87 |
| **Base** | 34.04 | 1.64 | 4.83 | 33.50 | 2.72 | 8.11 |
| **DBM/Binder** | 34.52 | 4.08 | 11.83 | 33.65 | 3.95 | 11.75 |
| **WC** | 34.84 | 6.54 | 18.77 | 33.68 | 6.02 | 17.89 |

Source: Field Data

**TABLE C.8: MONTHLY TEMPERATURE VALUES - (Mean, Standard Deviation and Coefficient of Variation) - AUGUST**

| | SOGAKOPE SITE | | | AKUMADAN SITE | | |
|---|---|---|---|---|---|---|
| **Temperature** | **Mean** | **Standard Deviation** | **Coefficient of Variation** | **Mean** | **Standard Deviation** | **Coefficient of Variation** |
| **Ambient** | 29.04 | 1.13 | 3.90 | 28.20 | 3.35 | 11.89 |
| **Subbase** | 33.66 | 1.41 | 4.20 | 32.65 | 0.22 | 0.67 |
| **Base** | 33.44 | 2.05 | 6.12 | 32.26 | 2.25 | 6.97 |
| **DBM/Binder** | 33.92 | 3.96 | 11.68 | 32.30 | 3.38 | 10.46 |
| **WC** | 34.18 | 6.25 | 18.30 | 32.29 | 5.26 | 16.27 |

Source: Field Data

**TABLE C.9: MONTHLY TEMPERATURE VALUES - (Mean, Standard Deviation and Coefficient of Variation) - SEPTEMBER**

| | SOGAKOPE SITE | | | AKUMADAN SITE | | |
|---|---|---|---|---|---|---|
| **Temperature** | **Mean** | **Standard Deviation** | **Coefficient of Variation** | **Mean** | **Standard Deviation** | **Coefficient of Variation** |
| **Ambient** | 32.81 | 1.27 | 3.87 | 30.13 | 3.90 | 12.95 |
| **Subbase** | 36.87 | 0.48 | 1.30 | 34.19 | 0.24 | 0.71 |
| **Base** | 37.20 | 1.08 | 2.89 | 34.97 | 2.61 | 7.46 |
| **DBM/Binder** | 38.01 | 3.83 | 10.07 | 35.04 | 3.93 | 11.22 |
| **WC** | 38.40 | 6.73 | 17.52 | 34.82 | 6.42 | 18.44 |

Source: Field Data

**TABLE C.11: MONTHLY TEMPERATURE VALUES - (Mean, Standard Deviation and Coefficient of Variation) - OCTOBER**

| | SOGAKOPE SITE | | | AKUMADAN SITE | | |
|---|---|---|---|---|---|---|
| **Temperature** | **Mean** | **Standard Deviation** | **Coefficient of Variation** | **Mean** | **Standard Deviation** | **Coefficient of Variation** |
| **Ambient** | 32.92 | 1.22 | 3.70 | 31.74 | 4.74 | 14.92 |
| **Subbase** | 36.90 | 0.74 | 2.00 | 35.68 | 0.85 | 2.37 |
| **Base** | 37.19 | 1.17 | 3.14 | 36.82 | 3.17 | 8.60 |
| **DBM/Binder** | 37.79 | 3.69 | 9.75 | 37.19 | 4.64 | 12.47 |
| **WC** | 38.04 | 6.49 | 17.07 | 37.46 | 7.44 | 19.86 |

Source: Field Data

**TABLE C.12: MONTHLY TEMPERATURE VALUES - (Mean, Standard Deviation and Coefficient of Variation) - NOVEMBER**

| | SOGAKOPE SITE | | | AKUMADAN SITE | | |
|---|---|---|---|---|---|---|
| **Temperature** | **Mean** | **Standard Deviation** | **Coefficient of Variation** | **Mean** | **Standard Deviation** | **Coefficient of Variation** |
| **Ambient** | 32.83 | 1.12 | 3.42 | 32.76 | 5.15 | 15.71 |
| **Subbase** | 36.67 | 0.38 | 1.04 | 37.22 | 0.49 | 1.32 |
| **Base** | 36.89 | 0.98 | 2.66 | 38.41 | 3.17 | 8.26 |
| **DBM/Binder** | 37.57 | 3.70 | 9.86 | 38.66 | 4.70 | 12.16 |
| **WC** | 37.98 | 6.53 | 17.19 | 38.72 | 7.59 | 19.59 |

Source: Field Data

**TABLE C.13: MONTHLY TEMPERATURE VALUES - (Mean, Standard Deviation and Coefficient of Variation) - DECEMBER**

| | SOGAKOPE SITE | | | AKUMADAN SITE | | |
|---|---|---|---|---|---|---|
| **Temperature** | **Mean** | **Standard Deviation** | **Coefficient of Variation** | **Mean** | **Standard Deviation** | **Coefficient of Variation** |
| **Ambient** | | | | 33.09 | 4.93 | 14.89 |
| **Subbase** | | | | 37.41 | 0.37 | 1.00 |
| **Base** | | | | 38.16 | 2.85 | 7.46 |
| **DBM/Binder** | | | | 38.45 | 4.24 | 11.04 |
| **WC** | | | | 38.66 | 6.83 | 17.68 |

Source: Field Data

# APPENDIX D

## Field Data 1 – Sample of FWD Test Data

| TABLE D.1: Maximum Deflections Measured (Akumadan Site) | | | | | | | |
|---|---|---|---|---|---|---|---|
| DAY | TIME | MAX. DEFLECTION D1 (µm) | | | | | |
| | | Kumasi Bound | | | Techiman Bound | | |
| | | CH 0 | CH 28 | CH 57 | CH 0 | CH 28 | CH 57 |
| 20-02-11 | 6 | 314 | 305 | 296 | 365 | 423 | 361 |
| 20-02-11 | 7 | 337 | 322 | 308 | 376 | 455 | 346 |
| 20-02-11 | 8 | 301 | 325 | 304 | 379 | 418 | 336 |
| 20-02-11 | 9 | 298 | 323 | 304 | 373 | 388 | 344 |
| 20-02-11 | 10 | 313 | 331 | 328 | 377 | 432 | 344 |
| 20-02-11 | 11 | 325 | 352 | 322 | 378 | 459 | 341 |
| 20-02-11 | 12 | 343 | 344 | 329 | 413 | 465 | 345 |
| 20-02-11 | 13 | 343 | 370 | 351 | 431 | 453 | 364 |
| 20-02-11 | 14 | 362 | 368 | 343 | 301 | 478 | 364 |
| 20-02-11 | 15 | 351 | 374 | 339 | 292 | 481 | 398 |
| 20-02-11 | 16 | 347 | 357 | 347 | 435 | 482 | 368 |
| 20-02-11 | 17 | 338 | 363 | 342 | 417 | 456 | 364 |
| 20-02-11 | 18 | 348 | 335 | 361 | 404 | 495 | 355 |

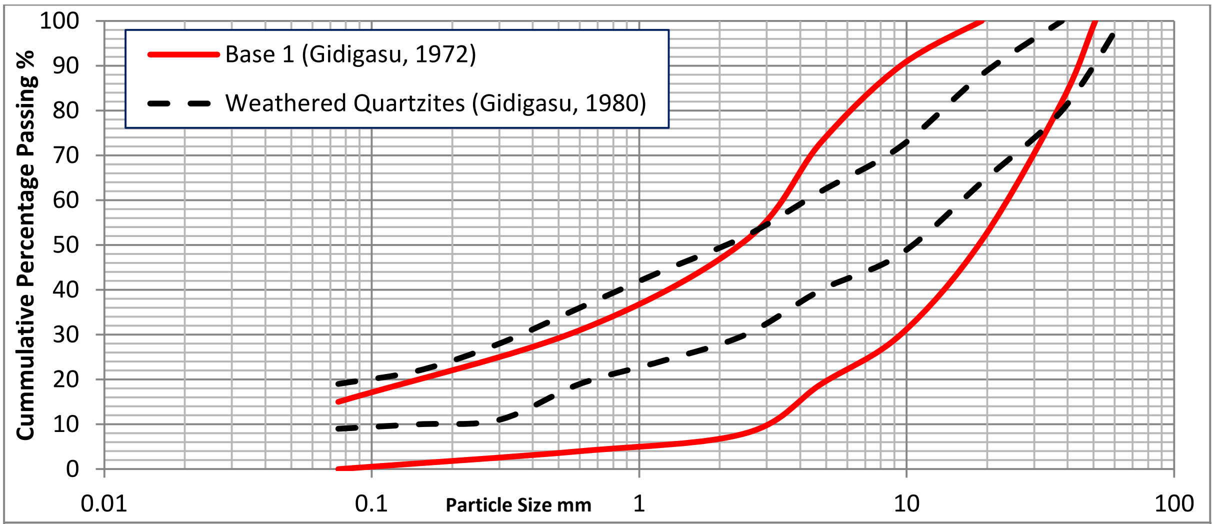
| TABLE D.3: Temperature Readings Collected (Akumadan Site) | | | | | | | |
|---|---|---|---|---|---|---|---|
| Date and time | Manual Measurement | | Equipment Readings | | | | |
| | Asphalt temp (°c) | Surface temp (°c) | Ambient Temp (°C) | Thermocouple 1 (°C) | Thermocouple 2 (°C) | Thermocouple 3 (°C) | Thermocouple 4 (°C) |
| 20/02/11 6:00 | 30 | 26 | 27.5 | 36.62 | 34.28 | 32.54 | 30.2 |
| 20/02/11 7:00 | 30 | 27 | 27.6 | 36.72 | 34.09 | 32.34 | 30.25 |
| 20/02/11 8:00 | 30 | 28 | 28.3 | 36.53 | 33.68 | 32.34 | 31.12 |
| 20/02/11 9:00 | 31 | 31 | 29.4 | 36.34 | 33.55 | 32.67 | 32.61 |
| 20/02/11 10:00 | 31 | 35 | 33.1 | 36.13 | 33.86 | 34.32 | 37.18 |
| 20/02/11 11:00 | 39 | 41 | 36.9 | 36.08 | 34.8 | 37.07 | 42.26 |
| 20/02/11 12:00 | 44 | 47 | 40.1 | 35.91 | 36.54 | 40.39 | 47.41 |
| 20/02/11 13:00 | 48 | 50 | 41.6 | 35.95 | 38.58 | 43.34 | 50.19 |
| 20/02/11 14:00 | 50 | 52 | 42.1 | 35.99 | 40.36 | 45.41 | 51.9 |
| 20/02/11 15:00 | 51 | 50 | 41 | 35.87 | 41.63 | 46.22 | 51.78 |
| 20/02/11 16:00 | 51 | 47 | 39.5 | 36 | 42.58 | 46.35 | 50.06 |
| 20/02/11 17:00 | 45 | 41 | 37.1 | 36.05 | 42.86 | 45.18 | 46.1 |
| 20/02/11 18:00 | 41 | 38 | 35 | 36.16 | 42.57 | 43.55 | 42.91 |

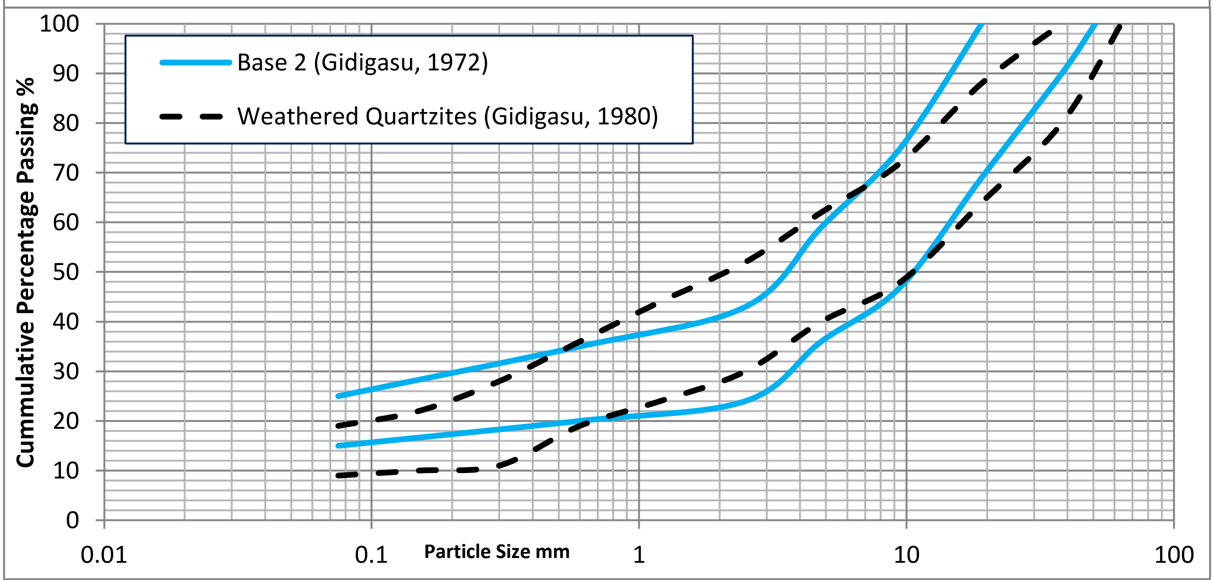
| TABLE D.2: Resilient Moduli Calculated (kPa) (Akumadan Site) | | | | | | | | | | | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| DAY | TIME | KUMASI BOUND | | | | | | | | | | | | TECHIMAN BOUND | | | | | | | | | | | |
| | | E1 | | | E2 | | | E3 | | | E4 | | | E1 | | | E2 | | | E3 | | | E4 | | |
| | | CH 0 | CH 28 | CH 57 | CH 0 | CH 28 | CH 57 | CH 0 | CH 28 | CH 57 | CH 0 | CH 28 | CH 57 | CH 0 | CH 28 | CH 57 | CH 0 | CH 28 | CH 57 | CH 0 | CH 28 | CH 57 | CH 0 | CH 28 | CH 57 |
| 20-02-11 | 6 | 3838 | 3921 | 3851 | 1002 | 980 | 962 | 530 | 519 | 509 | 180 | 184 | 237 | 2993 | 3398 | 2890 | 869 | 833 | 1204 | 460 | 441 | 637 | 148 | 85 | 87 |
| | 7 | 3627 | 3499 | 3733 | 964 | 902 | 915 | 510 | 477 | 484 | 152 | 187 | 226 | 3093 | 1788 | 3135 | 804 | 902 | 1244 | 425 | 477 | 658 | 145 | 89 | 90 |
| | 8 | 3874 | 3294 | 3817 | 1081 | 880 | 962 | 572 | 466 | 509 | 180 | 209 | 211 | 3723 | 3234 | 2334 | 655 | 858 | 1180 | 346 | 454 | 624 | 188 | 89 | 147 |
| | 9 | 4172 | 4063 | 3781 | 1118 | 829 | 944 | 591 | 439 | 499 | 154 | 180 | 216 | 4370 | 4134 | 2066 | 640 | 810 | 1152 | 339 | 429 | 609 | 174 | 97 | 136 |
| | 10 | 3356 | 2984 | 3064 | 1073 | 849 | 770 | 568 | 449 | 407 | 169 | 212 | 282 | 2816 | 3005 | 2523 | 768 | 781 | 1011 | 406 | 413 | 535 | 178 | 91 | 162 |
| | 11 | 2893 | 2663 | 3003 | 924 | 802 | 802 | 489 | 424 | 425 | 222 | 203 | 287 | 2247 | 2013 | 2328 | 853 | 773 | 1043 | 451 | 409 | 552 | 173 | 102 | 170 |
| | 12 | 2785 | 2318 | 3116 | 878 | 792 | 700 | 465 | 419 | 371 | 192 | 260 | 374 | 2168 | 2050 | 2425 | 668 | 698 | 977 | 354 | 370 | 517 | 197 | 110 | 175 |
| | 13 | 2195 | 2171 | 2242 | 870 | 702 | 756 | 460 | 372 | 400 | 247 | 254 | 322 | 1943 | 1792 | 1993 | 622 | 824 | 982 | 329 | 436 | 520 | 215 | 102 | 165 |
| | 14 | 2162 | 2002 | 2344 | 856 | 767 | 761 | 453 | 406 | 403 | 204 | 241 | 340 | 1600 | 1624 | 1842 | 486 | 779 | 985 | 257 | 412 | 521 | 234 | 97 | 176 |
| | 15 | 2335 | 1998 | 2257 | 784 | 713 | 773 | 415 | 377 | 409 | 258 | 252 | 322 | 1615 | 1543 | 1529 | 530 | 779 | 836 | 280 | 412 | 442 | 177 | 96 | 203 |
| | 16 | 2219 | 1933 | 2131 | 905 | 766 | 773 | 479 | 406 | 409 | 213 | 281 | 322 | 1641 | 1476 | 1876 | 689 | 813 | 991 | 365 | 430 | 524 | 194 | 97 | 165 |
| | 17 | 2105 | 1956 | 2076 | 1042 | 787 | 831 | 551 | 417 | 440 | 195 | 246 | 295 | 1770 | 1826 | 2178 | 751 | 805 | 937 | 397 | 426 | 496 | 184 | 105 | 165 |
| | 18 | 2832 | 2508 | 2144 | 895 | 837 | 731 | 474 | 443 | 387 | 171 | 242 | 301 | 2491 | 1226 | 2035 | 710 | 843 | 994 | 376 | 446 | 526 | 169 | 96 | 166 |

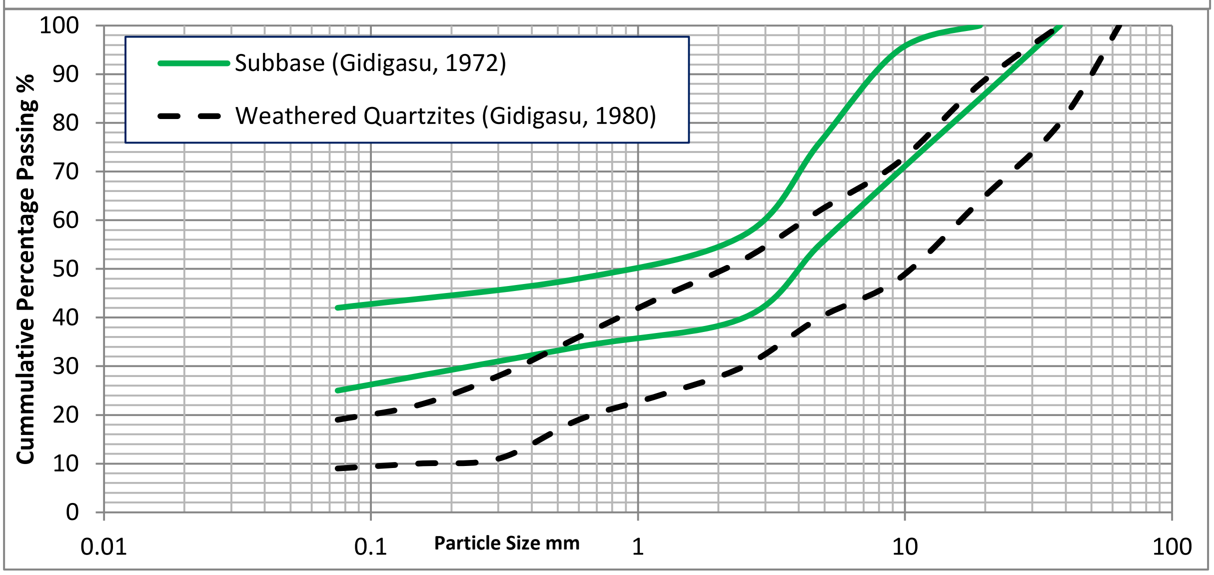
| TABLE D.4: Traffic Data Collected (Akumadan Site) | | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Date | TECHIMAN BOUND | | | | | | | | | | | | | | | |
| | From | To | cars | taxis | pick up/van/4wd veh | small bus | med. Bus/ mammy wagon | large bus | light truck | Med. truck | heavy truck | semi-trailer (light) | semi-trailer (heavy) | truck trailer | extra large truck & others | Total ESAs | Cumm. ESAs |
| 2/20/2011 | 6:00 AM | 7:00 AM | 4 | 2 | 4 | 7 | 0 | 5 | 2 | 1 | 3 | 0 | 3 | 3 | 7 | 121 | 121 |
| 2/20/2011 | 7:00 AM | 8:00 AM | 11 | 6 | 9 | 20 | 0 | 4 | 4 | 2 | 0 | 5 | 2 | 5 | 6 | 101 | 222 |
| 2/20/2011 | 8:00 AM | 9:00 AM | 5 | 7 | 12 | 29 | 0 | 4 | 6 | 3 | 0 | 5 | 3 | 1 | 3 | 86 | 307 |
| 2/20/2011 | 9:00 AM | 10:00 AM | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 307 |
| 2/20/2011 | 10:00 AM | 11:00 AM | 11 | 15 | 17 | 23 | 0 | 3 | 6 | 1 | 0 | 2 | 1 | 1 | 2 | 46 | 354 |
| 2/20/2011 | 11:00 AM | 12:00 PM | 19 | 15 | 44 | 42 | 1 | 6 | 8 | 6 | 0 | 7 | 0 | 4 | 9 | 139 | 493 |
| 2/20/2011 | 12:00 PM | 1:00 PM | 9 | 6 | 15 | 23 | 0 | 5 | 4 | 2 | 0 | 0 | 0 | 3 | 9 | 74 | 567 |
| 2/20/2011 | 1:00 PM | 2:00 PM | 16 | 14 | 23 | 27 | 1 | 2 | 6 | 1 | 0 | 2 | 0 | 3 | 5 | 59 | 626 |
| 2/20/2011 | 2:00 PM | 3:00 PM | 12 | 10 | 11 | 22 | 0 | 1 | 8 | 1 | 0 | 2 | 3 | 6 | 6 | 85 | 711 |
| 2/20/2011 | 3:00 PM | 4:00 PM | 11 | 18 | 23 | 33 | 1 | 8 | 10 | 6 | 0 | 1 | 2 | 6 | 7 | 118 | 829 |
| 2/20/2011 | 4:00 PM | 5:00 PM | 15 | 19 | 18 | 28 | 3 | 2 | 5 | 0 | 4 | 2 | 1 | 1 | 3 | 109 | 938 |
| 2/20/2011 | 5:00 PM | 6:00 PM | 11 | 14 | 10 | 29 | 1 | 7 | 10 | 2 | 1 | 3 | 0 | 4 | 5 | 106 | 1044 |
| 2/20/2011 | 6:00 PM | 6:00 AM | 30 | 30 | 45 | 68 | 2 | 11 | 17 | 6 | 2 | 7 | 4 | 9 | 15 | 250 | 1294 |

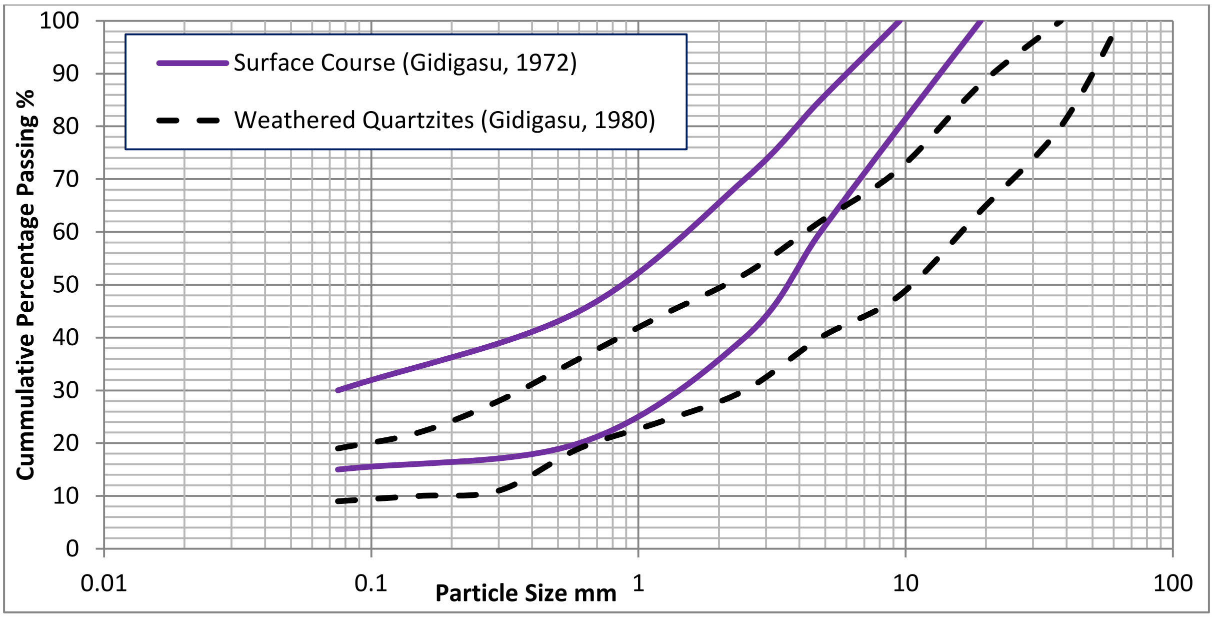
# APPENDIX E

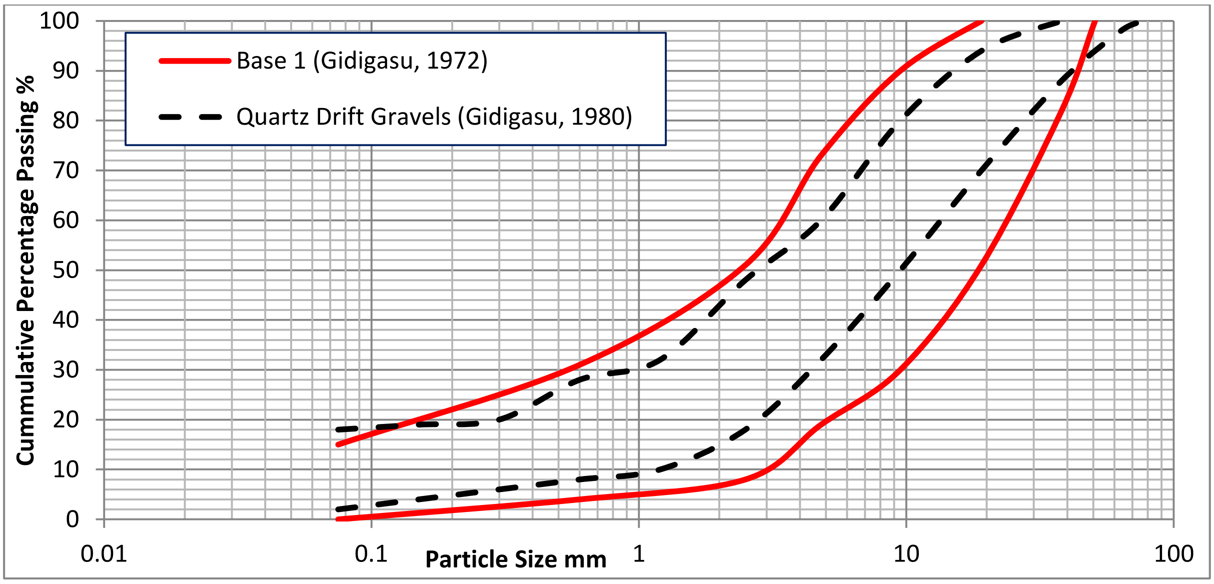
## Grading Curves

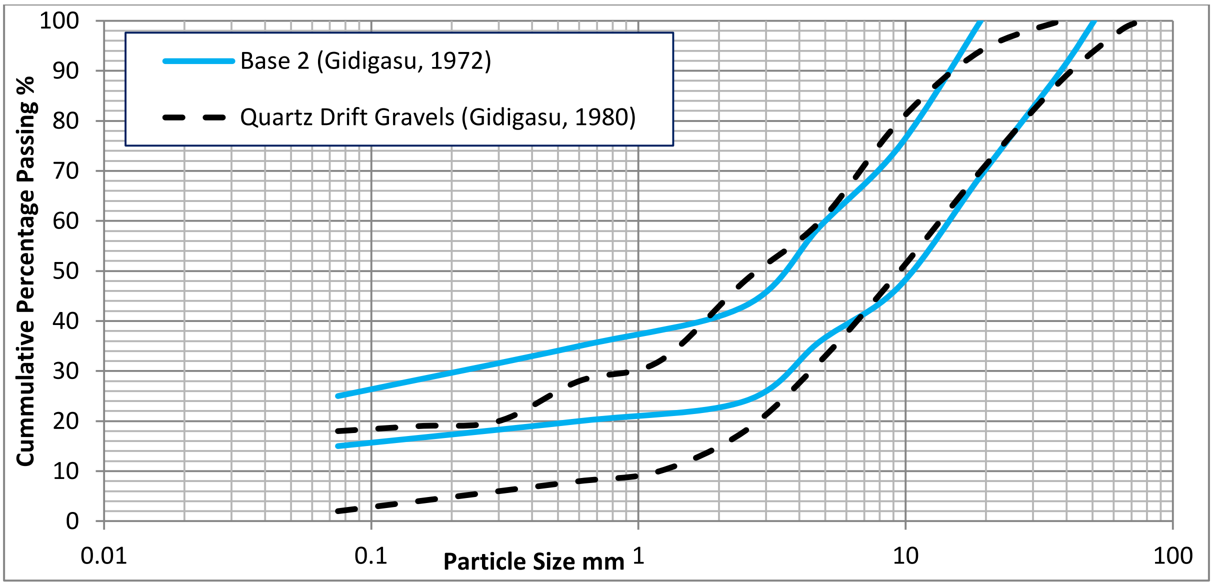


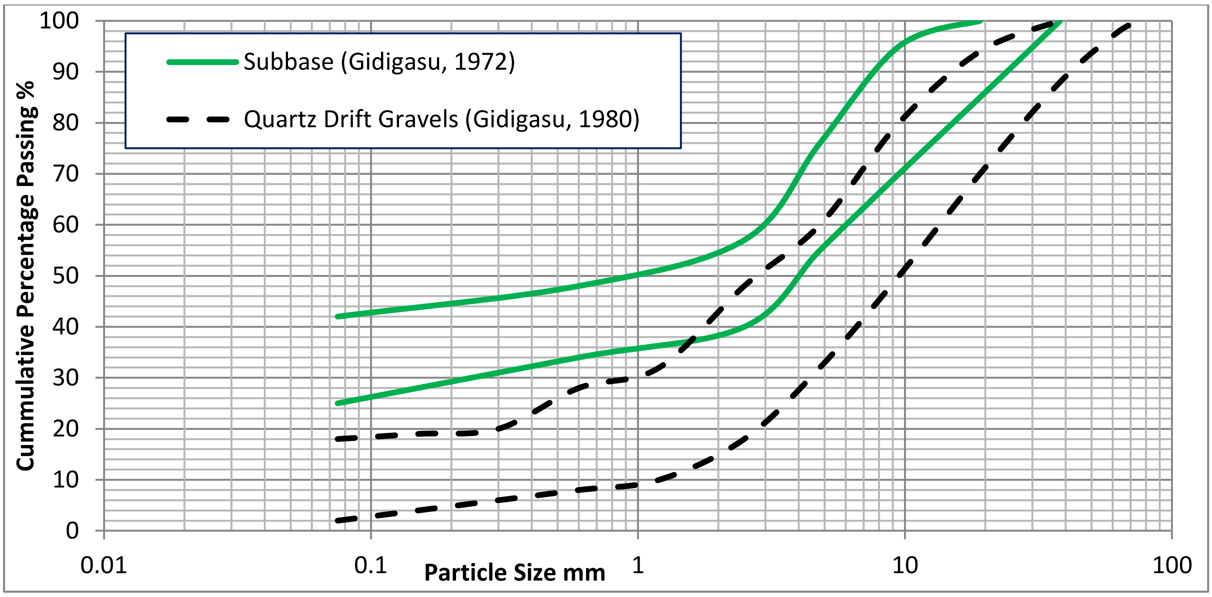



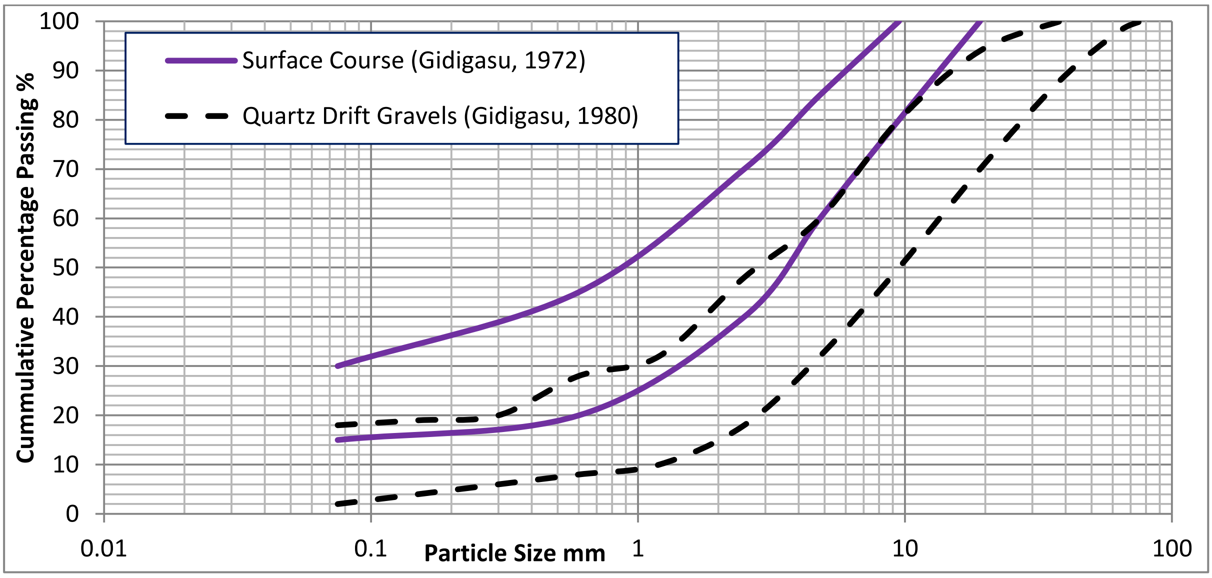

Surface Course (Gidigasu, 1972)
Weathered Quartzites (Gidigasu, 1980)
Cummulative Percentage Passing %
Particle Size mm
0.01
0.1
1
10
100


Base 1 (Gidigasu, 1972)
Quartz Drift Gravels (Gidigasu, 1980)
Cummulative Percentage Passing %
Particle Size mm
0.01
0.1
1
10
100

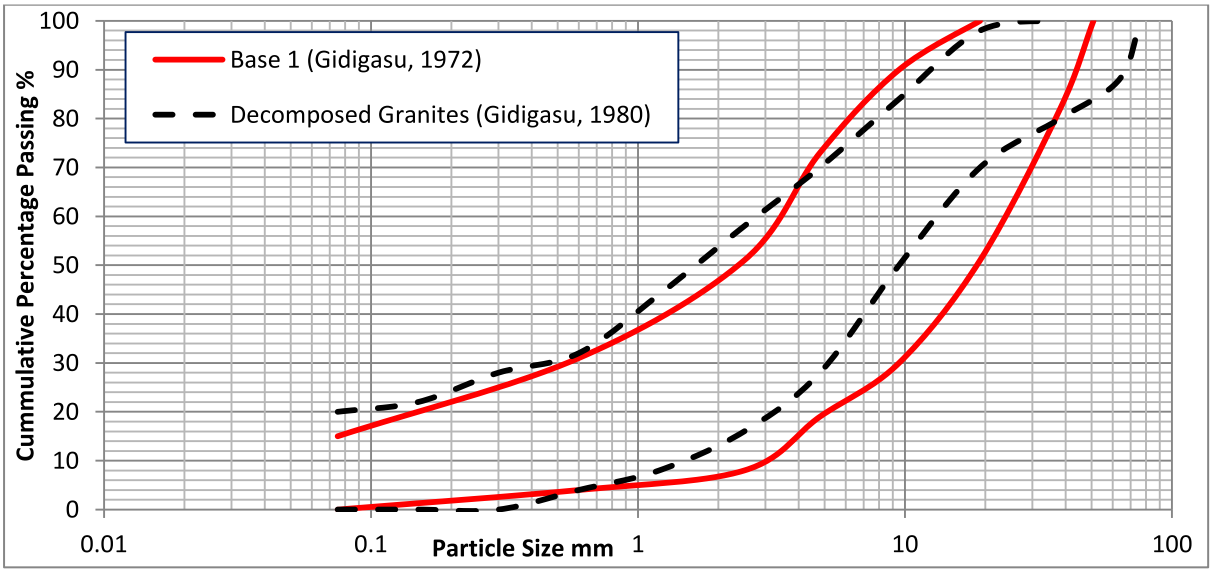

Base 2 (Gidigasu, 1972)
Quartz Drift Gravels (Gidigasu, 1980)
Cummulative Percentage Passing %
Particle Size mm
0.01
0.1
1
10
100



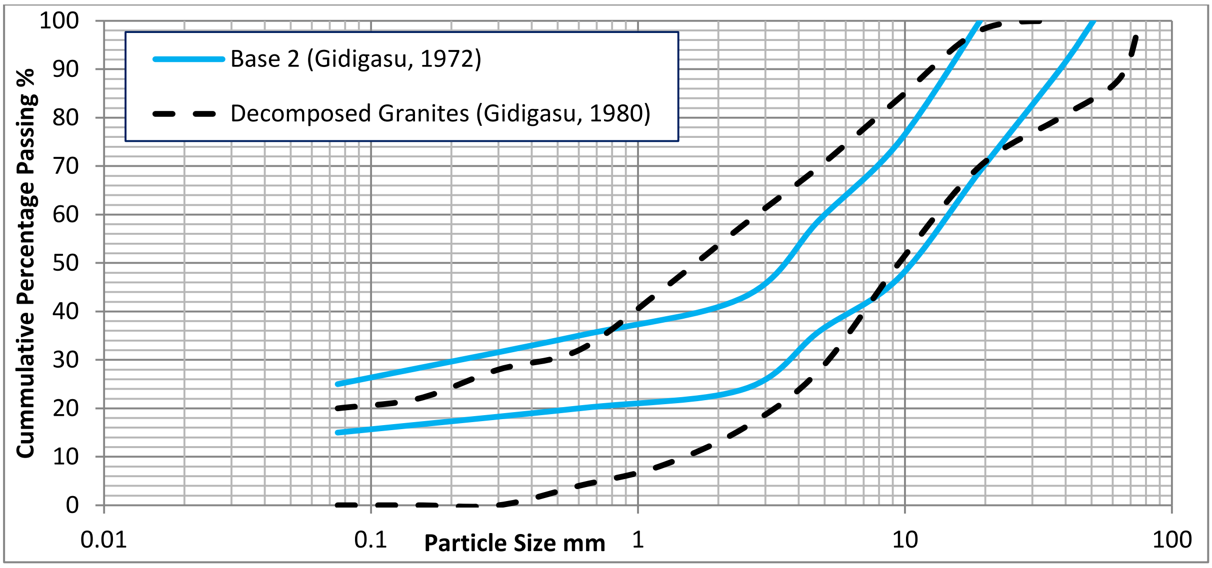
Subbase (Gidigasu, 1972)
Quartz Drift Gravels (Gidigasu, 1980)
Cummulative Percentage Passing %
Particle Size mm
0.01
0.1
1
10
100

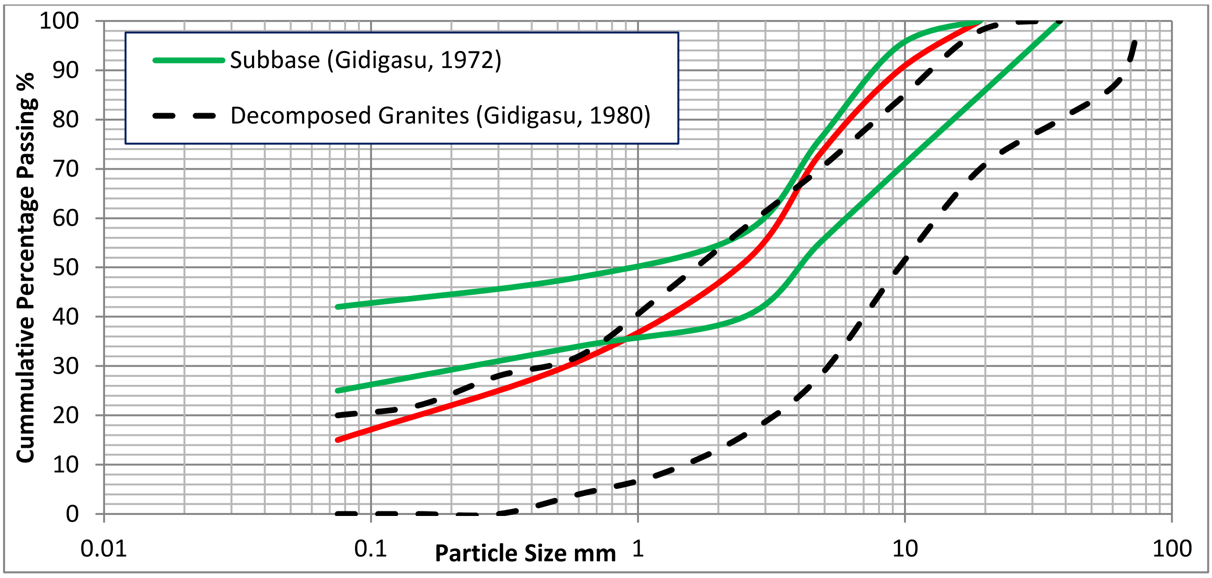



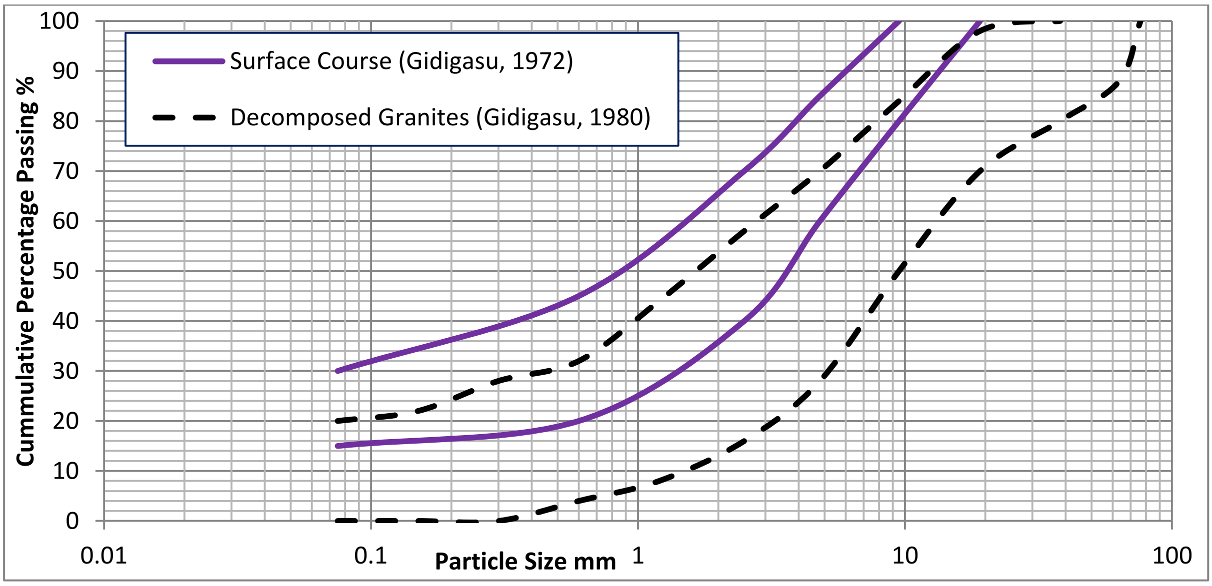
Surface Course (Gidigasu, 1972)
Quartz Drift Gravels (Gidigasu, 1980)
Cummulative Percentage Passing %
Particle Size mm

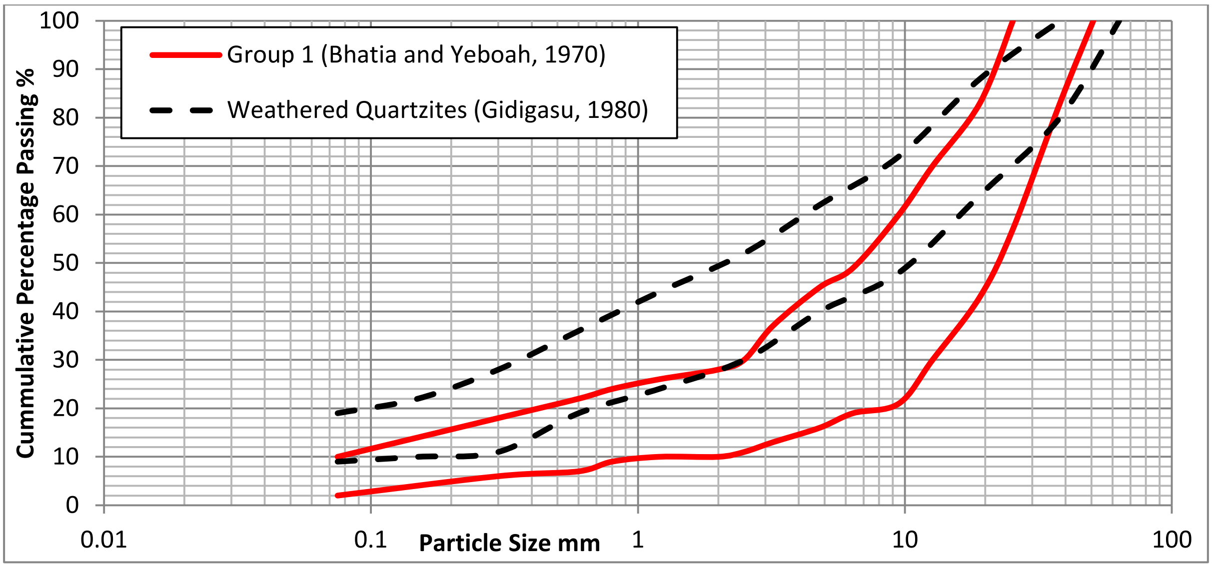



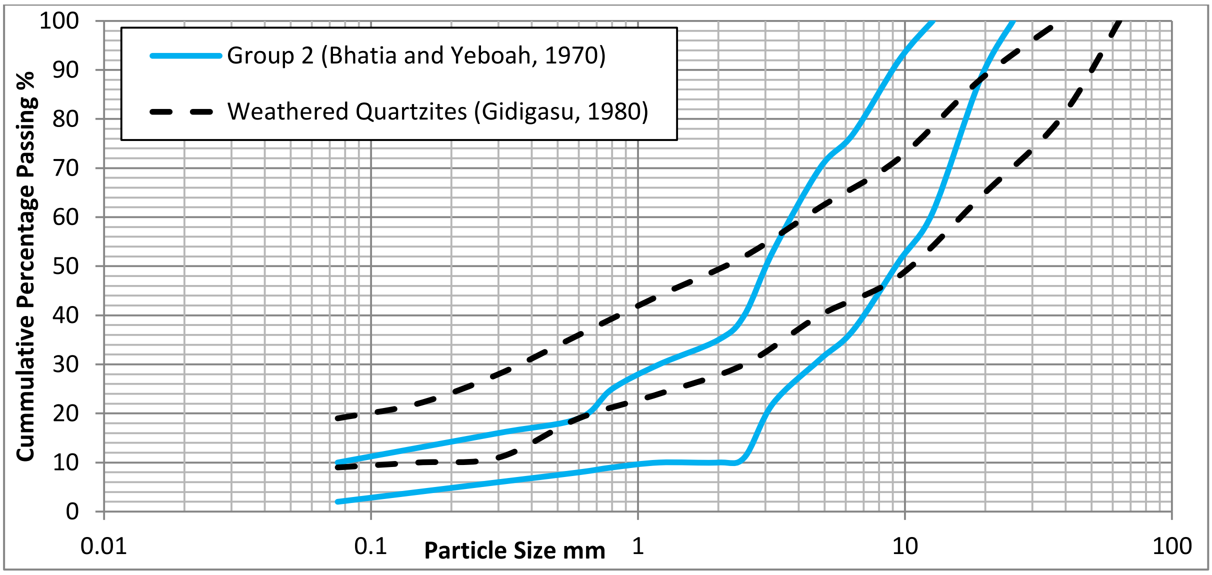
Base 1 (Gidigasu, 1972)
Decomposed Granites (Gidigasu, 1980)
Cummulative Percentage Passing %
Particle Size mm

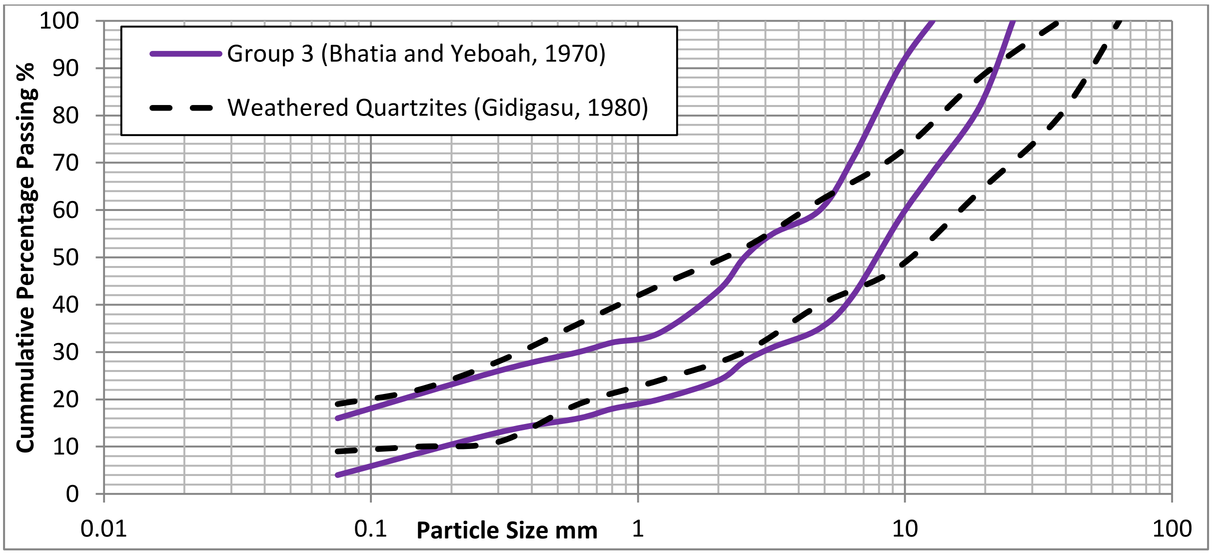

Base 2 (Gidigasu, 1972)
Decomposed Granites (Gidigasu, 1980)
Cummulative Percentage Passing %
Particle Size mm
0.01
0.1
1
10
100
0
10
20
30
40
50
60
70
80
90
100

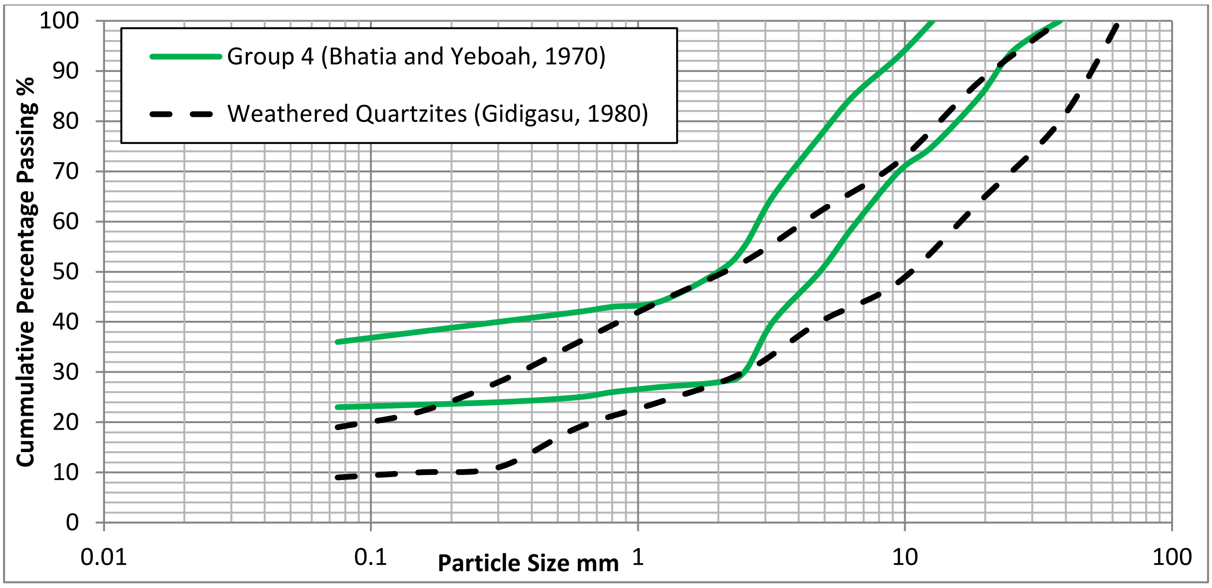

Subbase (Gidigasu, 1972)
Decomposed Granites (Gidigasu, 1980)
Cummulative Percentage Passing %
Particle Size mm
0.01
0.1
1
10
100
0
10
20
30
40
50
60
70
80
90
100

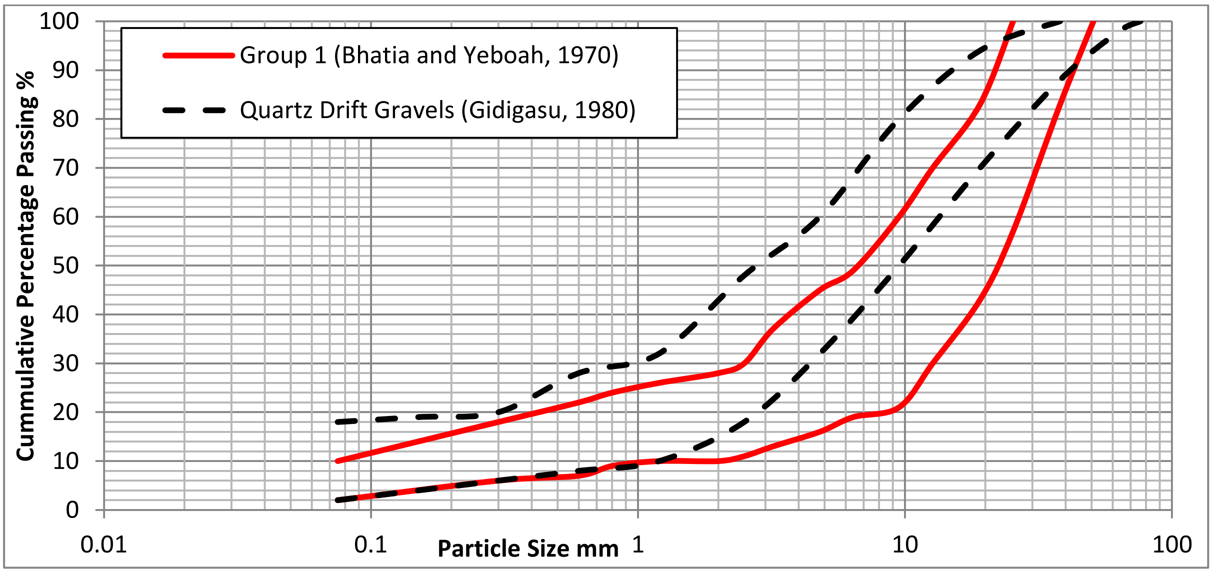

Surface Course (Gidigasu, 1972)
Decomposed Granites (Gidigasu, 1980)
Cummulative Percentage Passing %
Particle Size mm
0.01
0.1
1
10
100
0
10
20
30
40
50
60
70
80
90
100

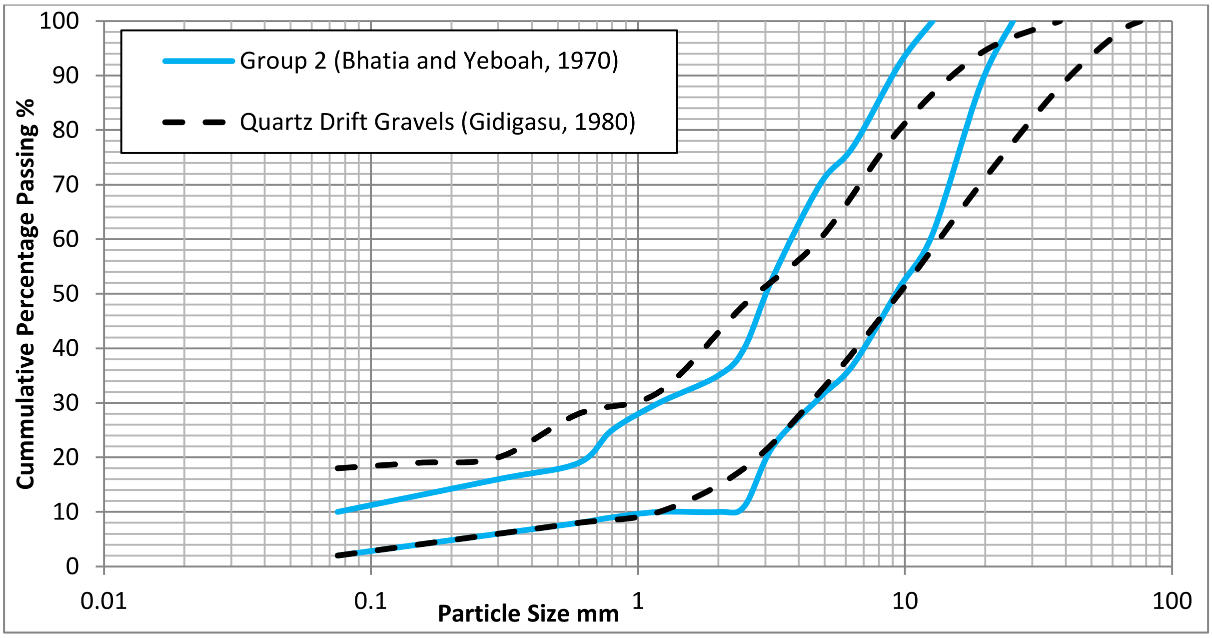

Group 1 (Bhatia and Yeboah, 1970)
Weathered Quartzites (Gidigasu, 1980)
Cummulative Percentage Passing %
Particle Size mm
0
10
20
30
40
50
60
70
80
90
100
0.01
0.1
1
10
100

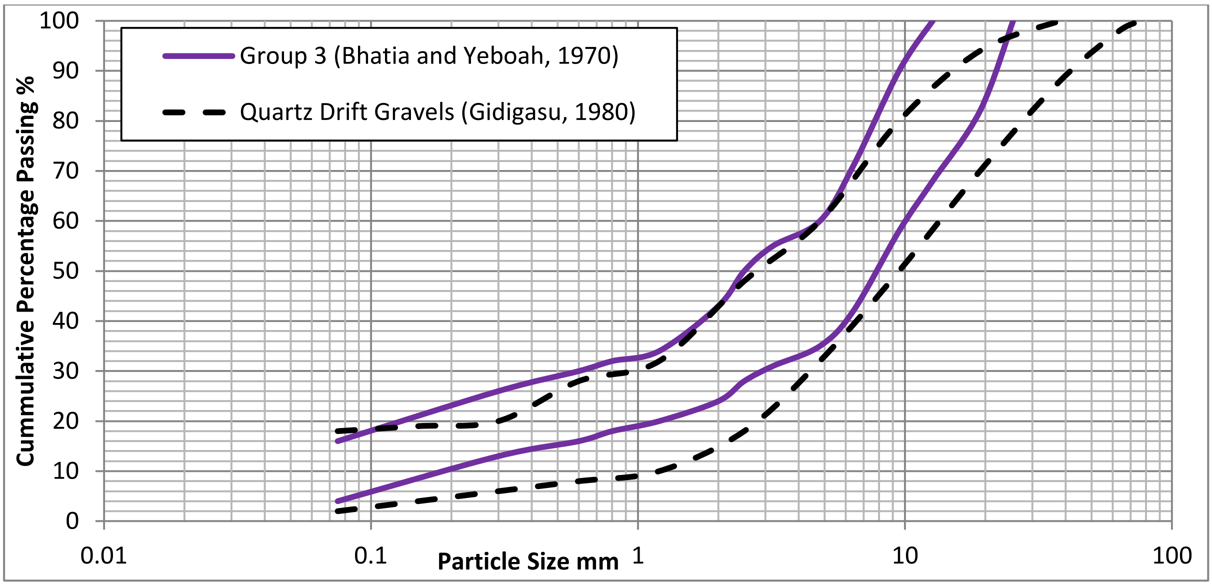

Group 2 (Bhatia and Yeboah, 1970)
Weathered Quartzites (Gidigasu, 1980)
Cummulative Percentage Passing %
Particle Size mm
0
10
20
30
40
50
60
70
80
90
100
0.01
0.1
1
10
100

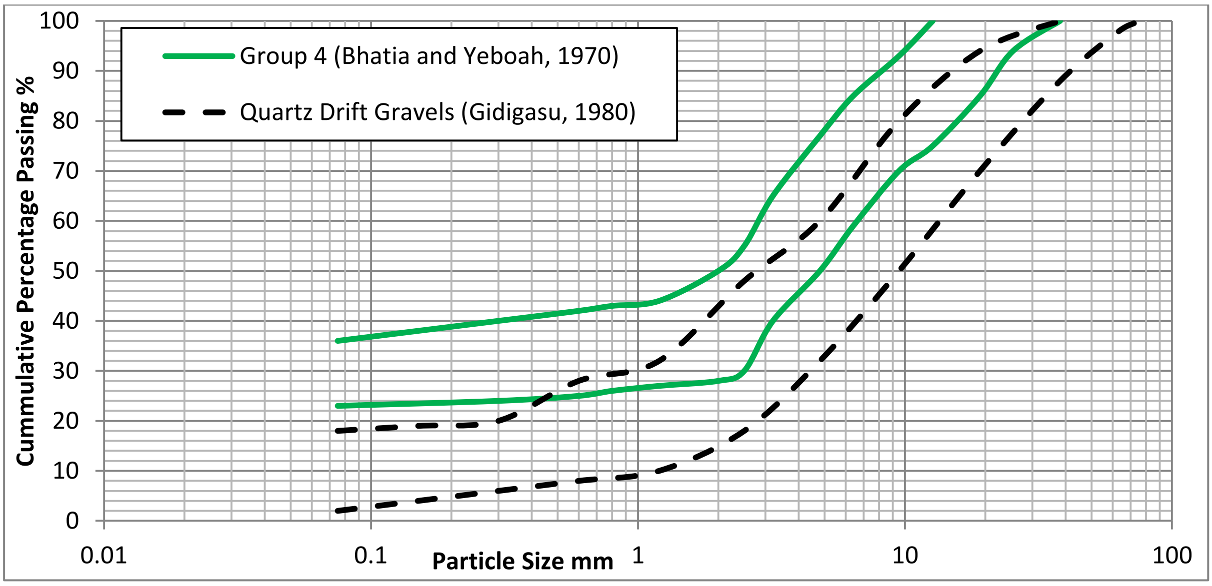

Group 3 (Bhatia and Yeboah, 1970)
Weathered Quartzites (Gidigasu, 1980)
Cummulative Percentage Passing %
Particle Size mm
0
10
20
30
40
50
60
70
80
90
100
0.01
0.1
1
10
100

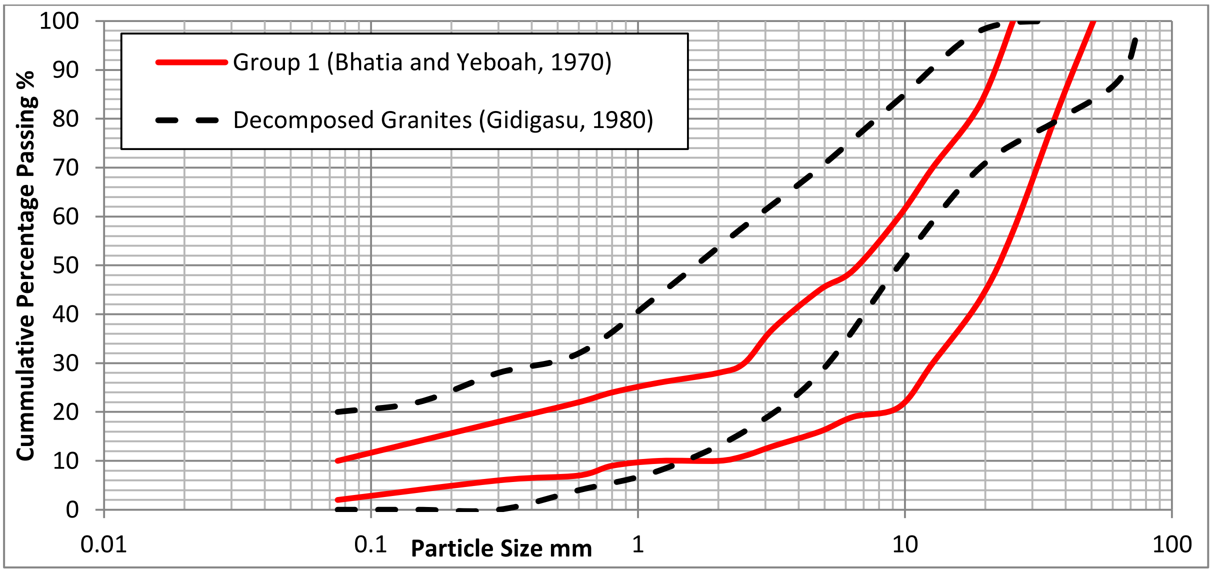

Group 4 (Bhatia and Yeboah, 1970)
Weathered Quartzites (Gidigasu, 1980)
Cummulative Percentage Passing %
Particle Size mm
100
90
80
70
60
50
40
30
20
10
0
0.01
0.1
1
10
100

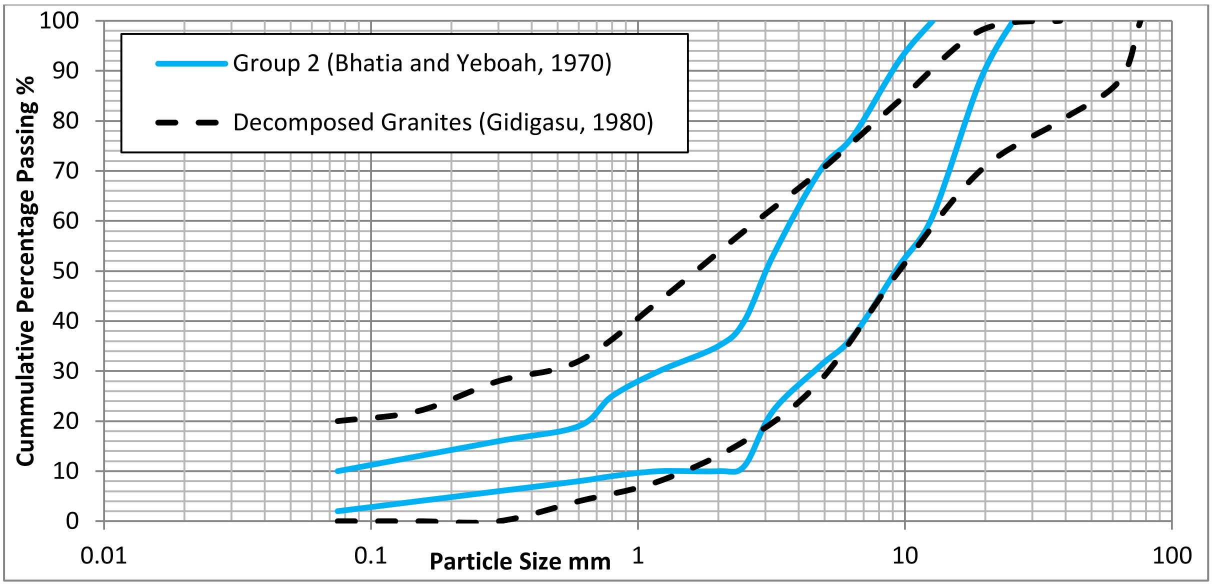

Group 1 (Bhatia and Yeboah, 1970)
Quartz Drift Gravels (Gidigasu, 1980)
Cummulative Percentage Passing %
Particle Size mm
100
90
80
70
60
50
40
30
20
10
0
0.01
0.1
1
10
100

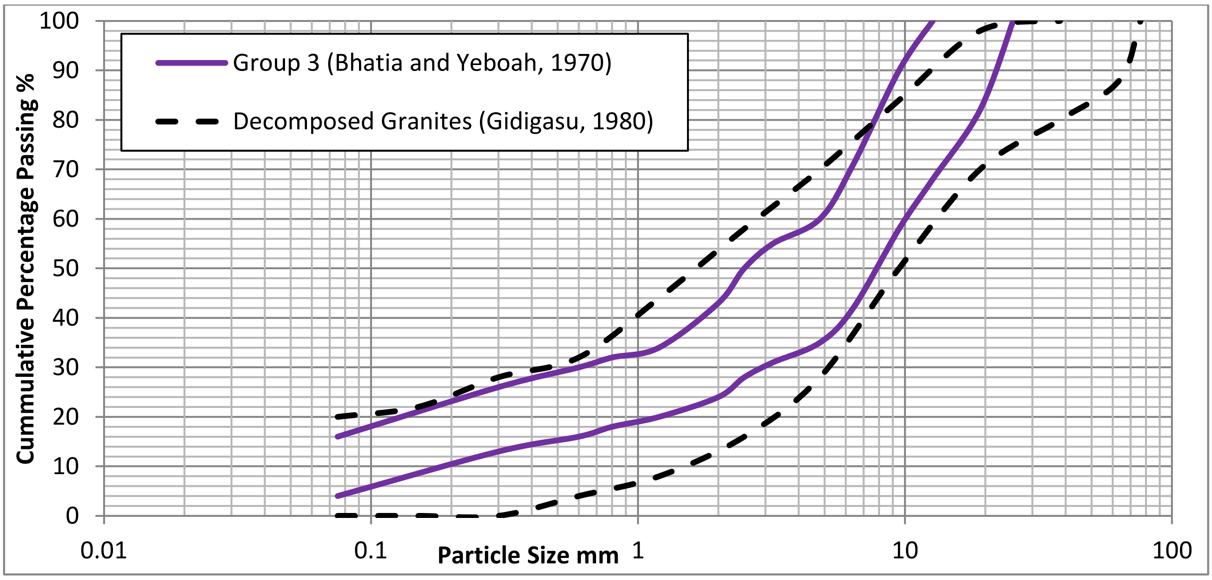

Group 2 (Bhatia and Yeboah, 1970)
Quartz Drift Gravels (Gidigasu, 1980)
Cummulative Percentage Passing %
Particle Size mm
100
90
80
70
60
50
40
30
20
10
0
0.01
0.1
1
10
100

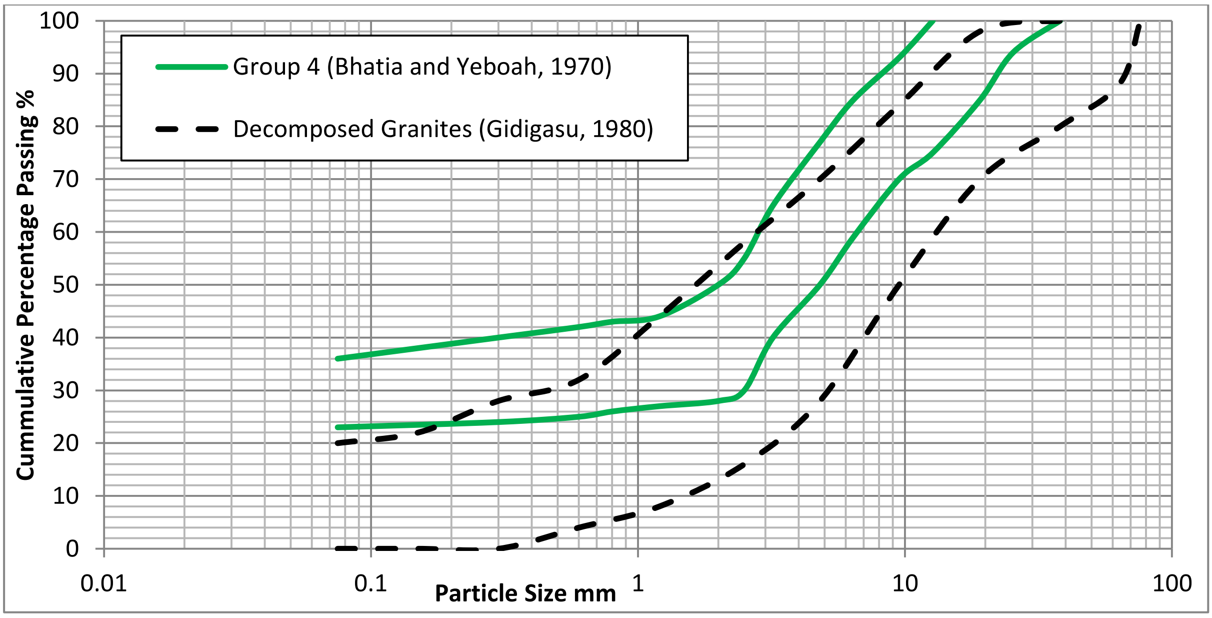

Group 3 (Bhatia and Yeboah, 1970)
Quartz Drift Gravels (Gidigasu, 1980)
Cummulative Percentage Passing %
Particle Size mm
100
90
80
70
60
50
40
30
20
10
0
0.01
0.1
1
10
100

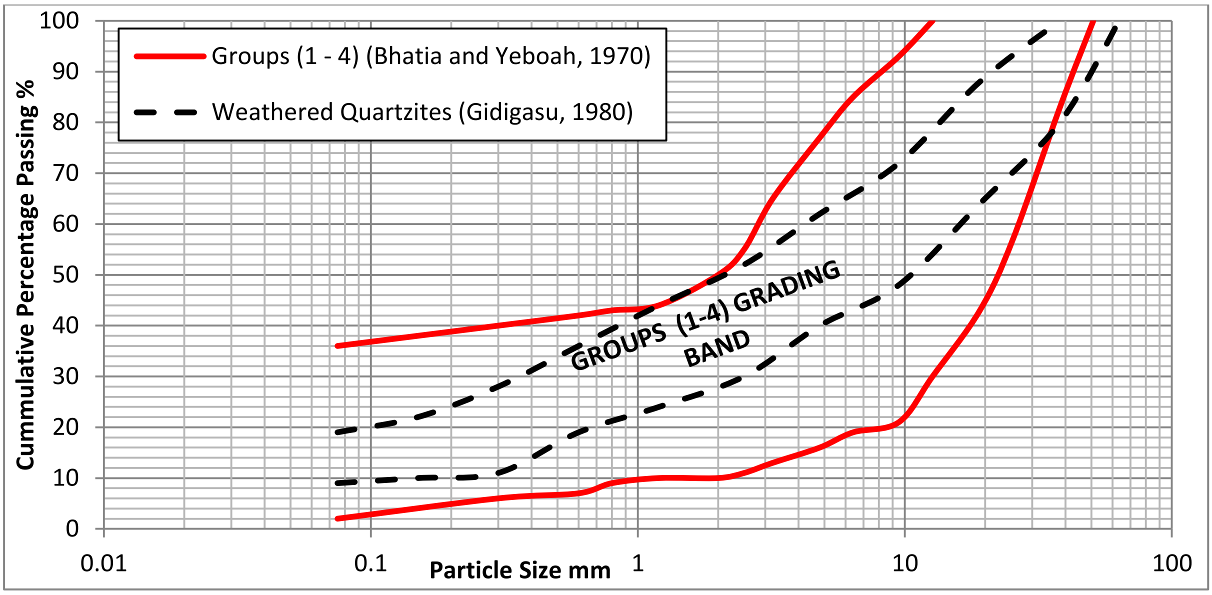

Group 4 (Bhatia and Yeboah, 1970)
Quartz Drift Gravels (Gidigasu, 1980)
Cummulative Percentage Passing %
Particle Size mm
100
90
80
70
60
50
40
30
20
10
0
0.01
0.1
1
10
100

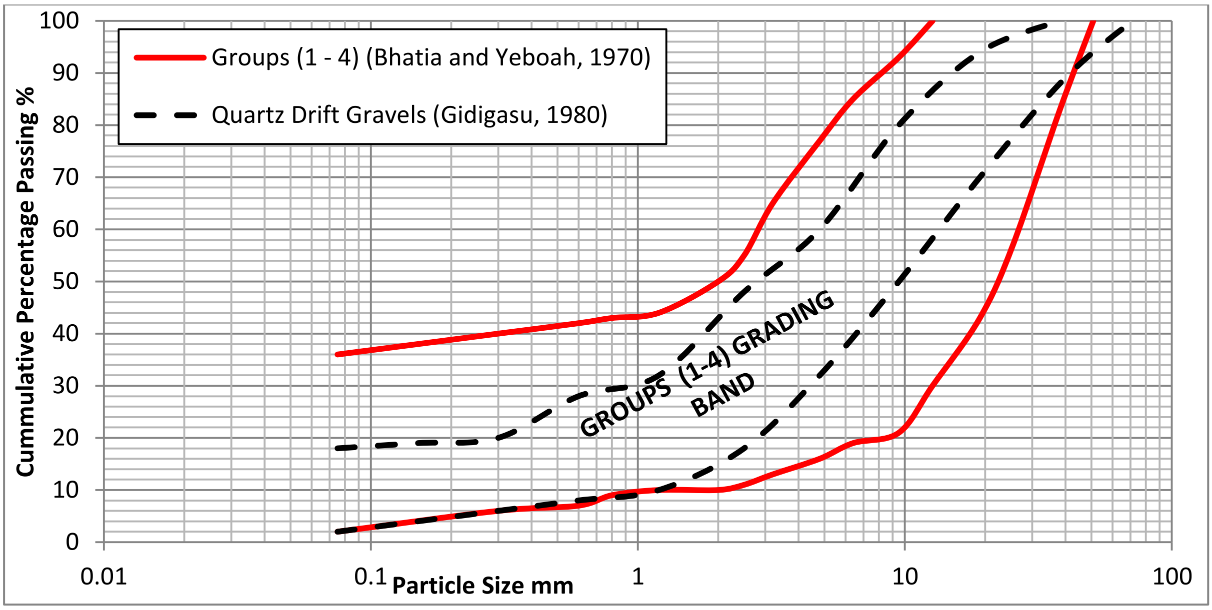

Group 1 (Bhatia and Yeboah, 1970)
Decomposed Granites (Gidigasu, 1980)
Cummulative Percentage Passing %
Particle Size mm
100
90
80
70
60
50
40
30
20
10
0
0.01
0.1
1
10
100

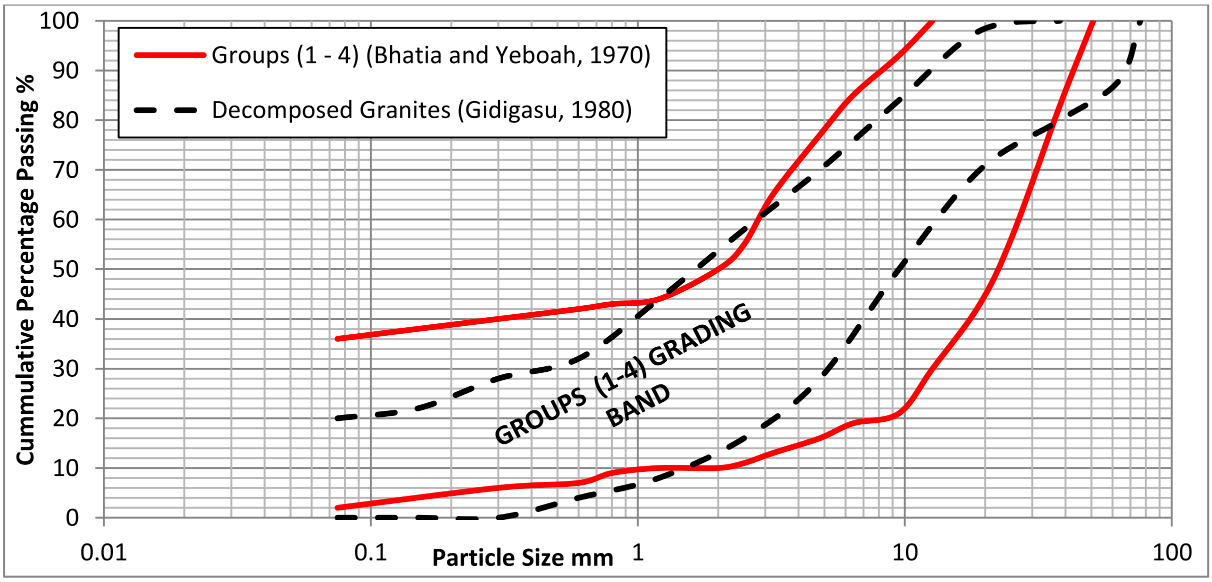

Group 2 (Bhatia and Yeboah, 1970)
Decomposed Granites (Gidigasu, 1980)
Cummulative Percentage Passing %
Particle Size mm
0 10 20 30 40 50 60 70 80 90 100
0.01 0.1 1 10 100

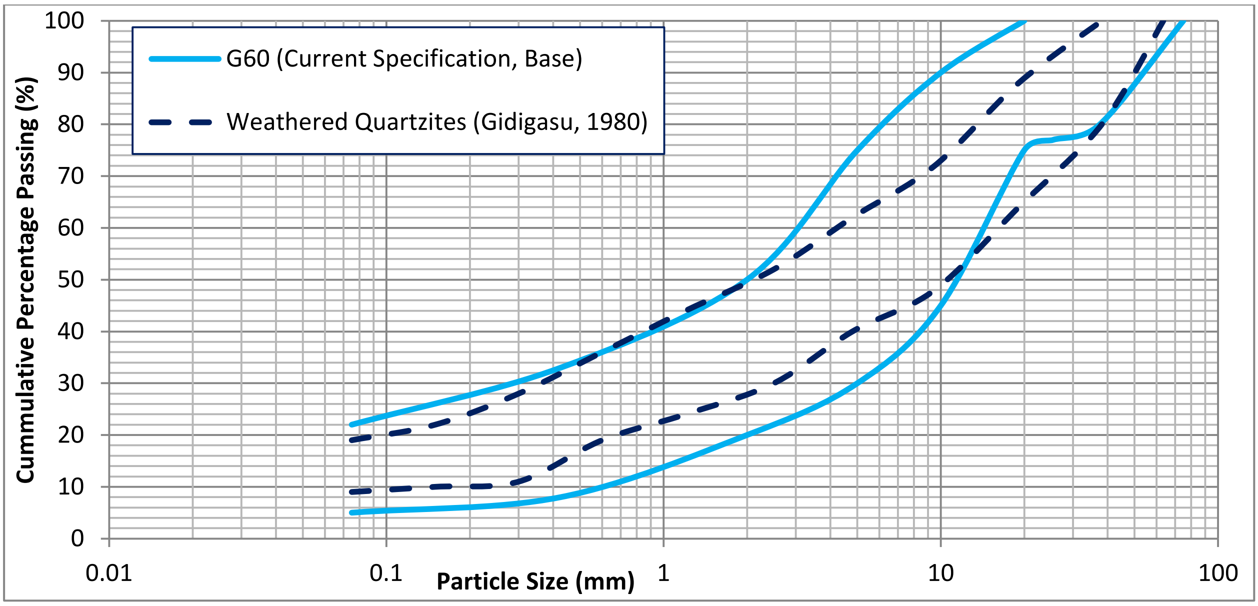

Group 3 (Bhatia and Yeboah, 1970)
Decomposed Granites (Gidigasu, 1980)
Cummulative Percentage Passing %
Particle Size mm
0 10 20 30 40 50 60 70 80 90 100
0.01 0.1 1 10 100


Group 4 (Bhatia and Yeboah, 1970)
Decomposed Granites (Gidigasu, 1980)
Cummulative Percentage Passing %
Particle Size mm
0 10 20 30 40 50 60 70 80 90 100
0.01 0.1 1 10 100

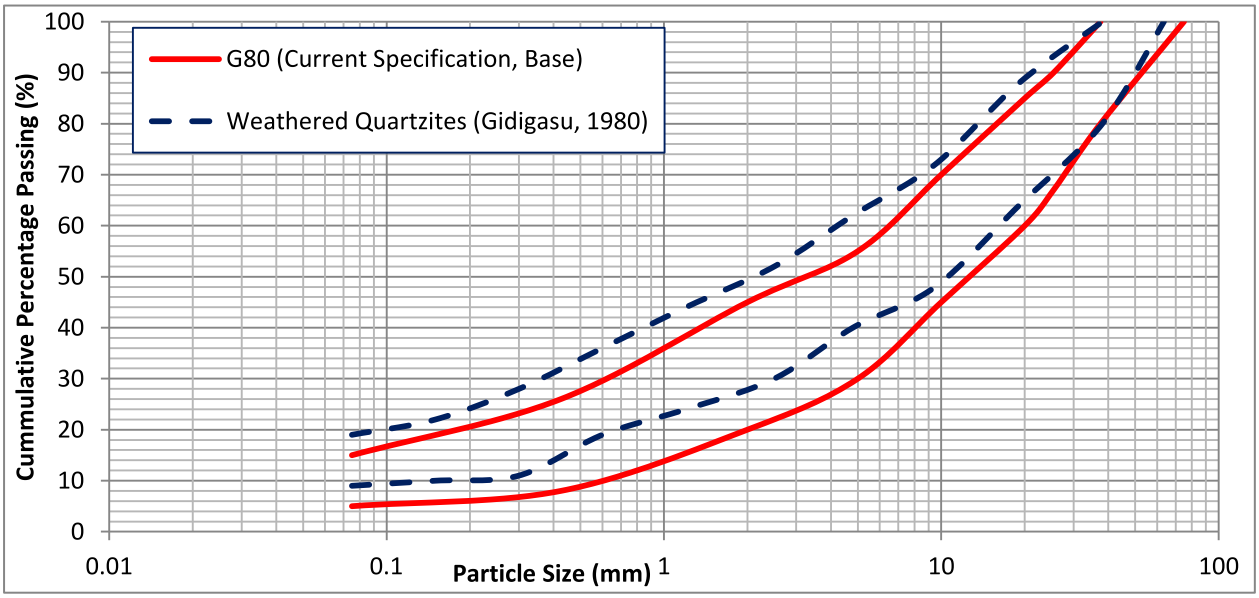

Groups (1 - 4) (Bhatia and Yeboah, 1970)
Weathered Quartzites (Gidigasu, 1980)
GROUPS (1-4) GRADING BAND
Cummulative Percentage Passing %
Particle Size mm
100
90
80
70
60
50
40
30
20
10
0
0.01
0.1
1
10
100


Groups (1 - 4) (Bhatia and Yeboah, 1970)
Quartz Drift Gravels (Gidigasu, 1980)
GROUPS (1-4) GRADING BAND
Cummulative Percentage Passing %
Particle Size mm
100
90
80
70
60
50
40
30
20
10
0
0.01
0.1
1
10
100

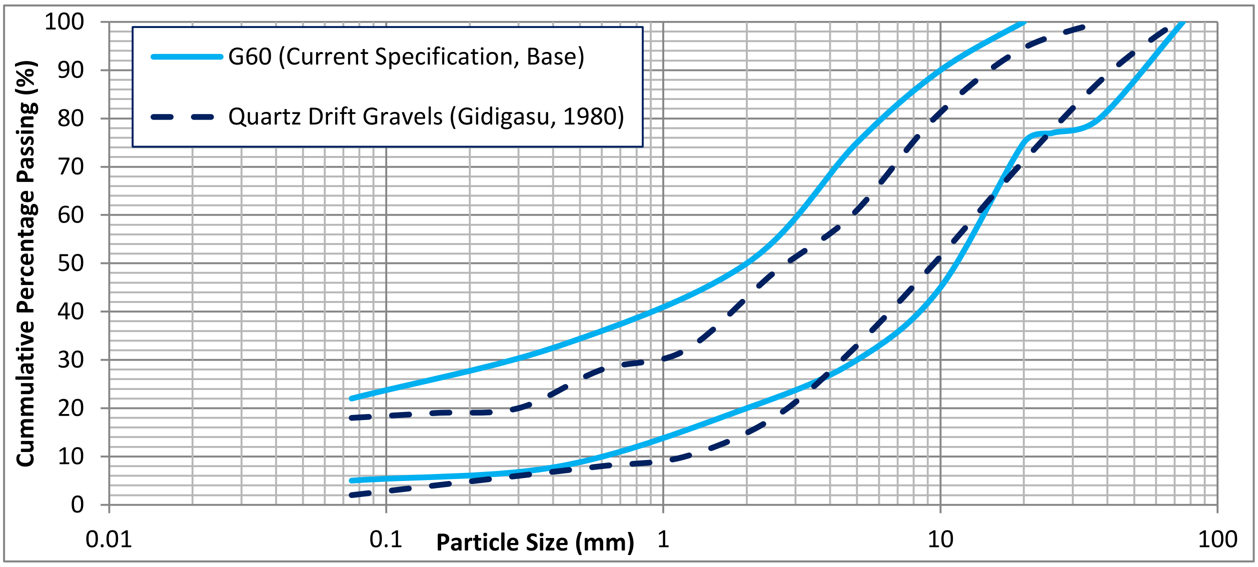

Groups (1 - 4) (Bhatia and Yeboah, 1970)
Decomposed Granites (Gidigasu, 1980)
GROUPS (1-4) GRADING BAND
Cummulative Percentage Passing %
Particle Size mm
100
90
80
70
60
50
40
30
20
10
0
0.01
0.1
1
10
100



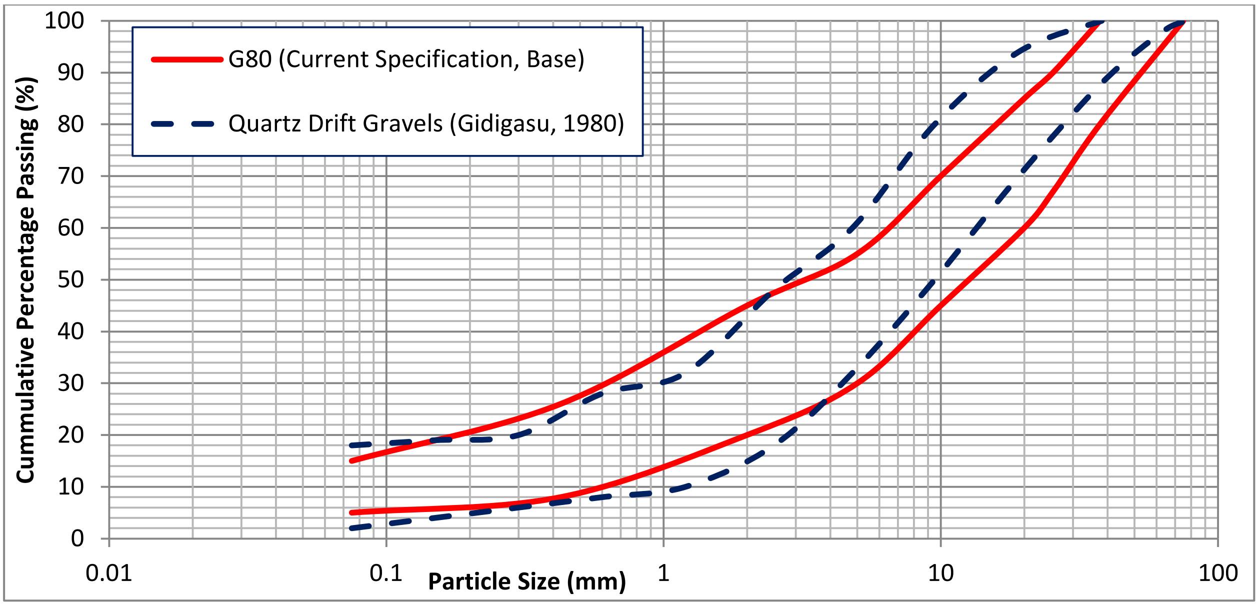
G60 (Current Specification, Base)
Weathered Quartzites (Gidigasu, 1980)
Cummulative Percentage Passing (%)
Particle Size (mm)
100
90
80
70
60
50
40
30
20
10
0
0.01
0.1
1
10
100

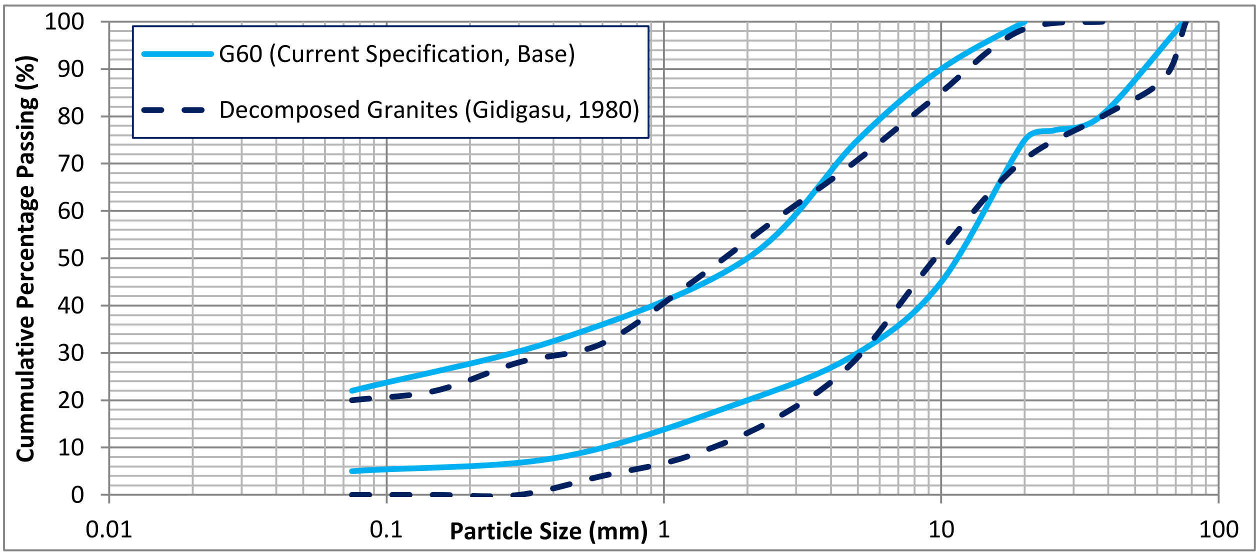



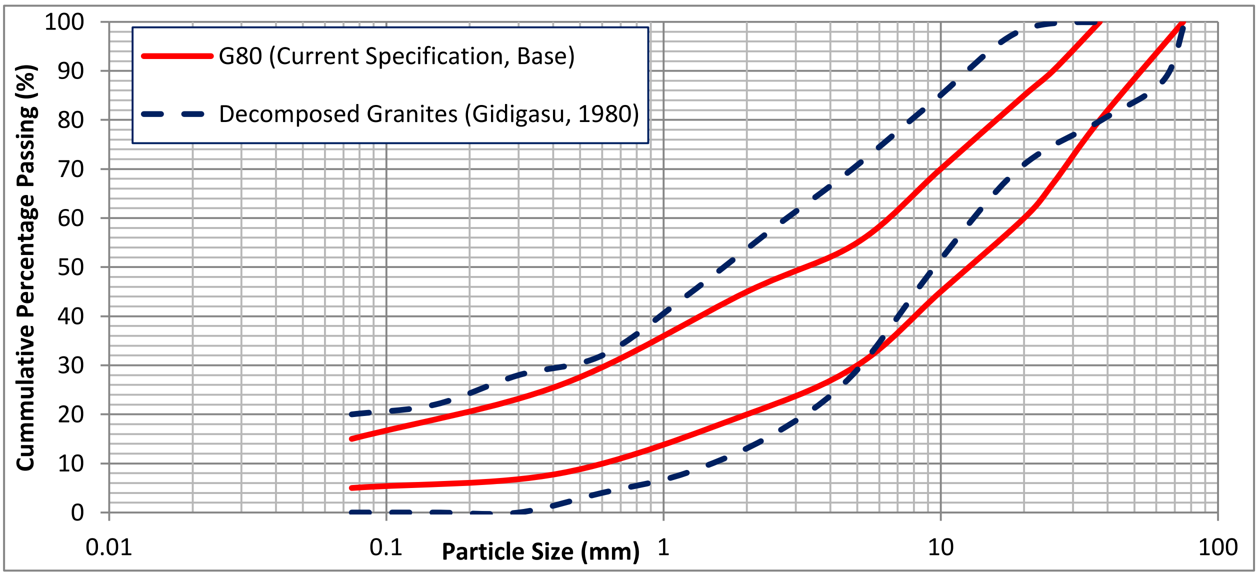
G80 (Current Specification, Base)
Weathered Quartzites (Gidigasu, 1980)
Cummulative Percentage Passing (%)
Particle Size (mm)
100
90
80
70
60
50
40
30
20
10
0
0.01
0.1
1
10
100

G60 (Current Specification, Base)
Quartz Drift Gravels (Gidigasu, 1980)
Cummulative Percentage Passing (%)
Particle Size (mm)
100
90
80
70
60
50
40
30
20
10
0
0.01
0.1
1
10
100


G80 (Current Specification, Base)
Quartz Drift Gravels (Gidigasu, 1980)
Cummulative Percentage Passing (%)
Particle Size (mm)
0
10
20
30
40
50
60
70
80
90
100
0.01
0.1
1
10
100


G60 (Current Specification, Base)
Decomposed Granites (Gidigasu, 1980)
Cummulative Percentage Passing (%)
Particle Size (mm)
0
10
20
30
40
50
60
70
80
90
100
0.01
0.1
1
10
100


G80 (Current Specification, Base)
Decomposed Granites (Gidigasu, 1980)
Cummulative Percentage Passing (%)
Particle Size (mm)
0
10
20
30
40
50
60
70
80
90
100
0.01
0.1
1
10
100

# APPENDIX F

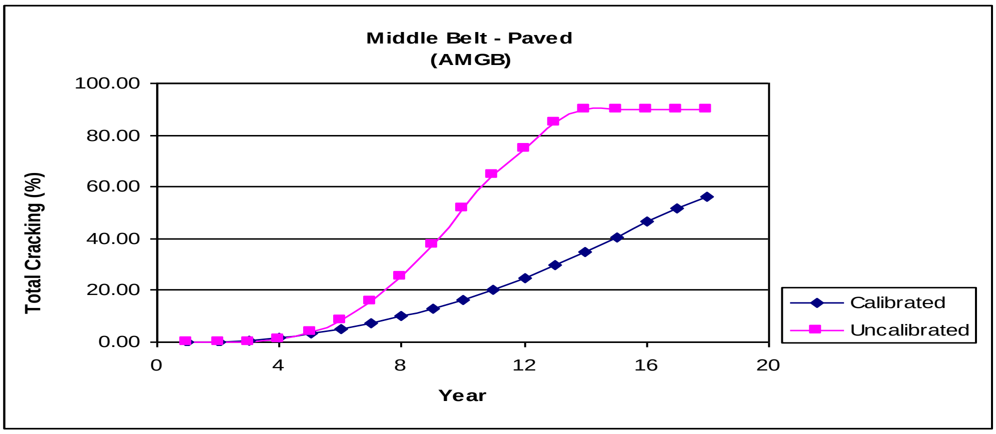
## Summary of Calibration Results Demonstrations

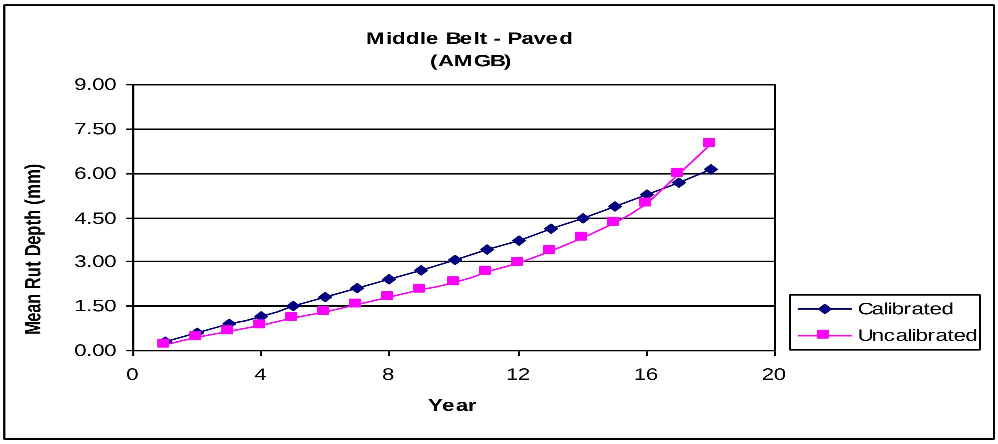
### F.1 Road Deterioration Calibration Results (Graphs)

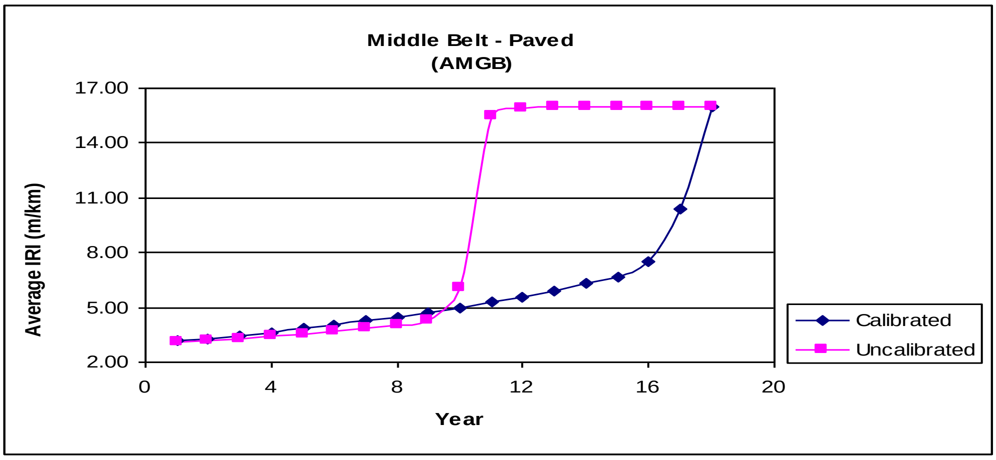
#### F.1.1 AMGB

***Middle Belt***

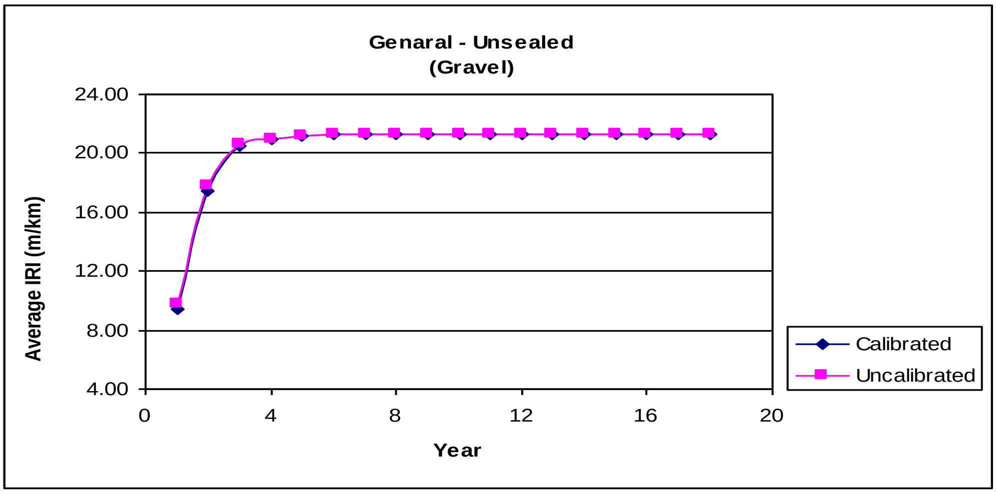
Paved



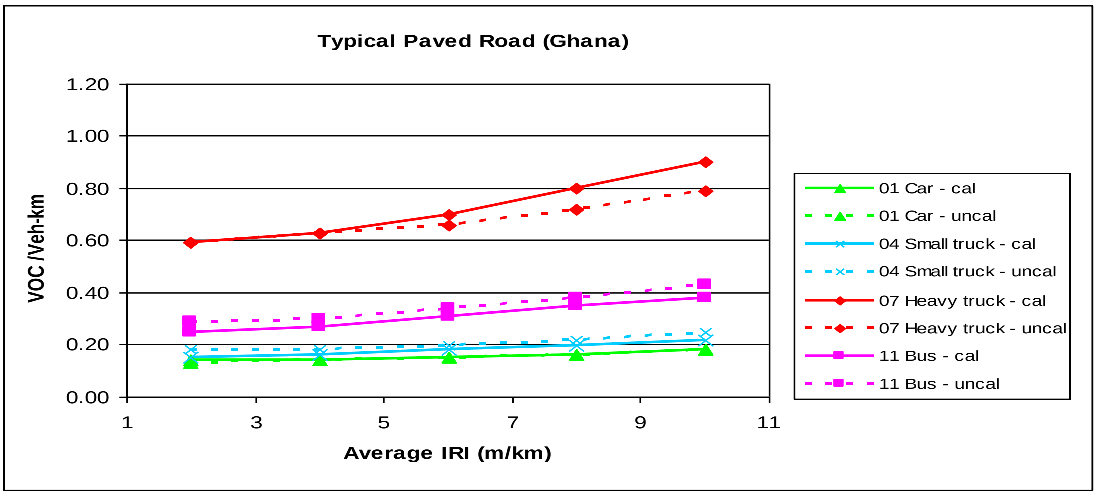
Unsealed (referred to for all other zones)




## F.2 Road User Costs (VOC) Calibration Results (Graphs)

### F.2.1 VOC

**Paved Road**

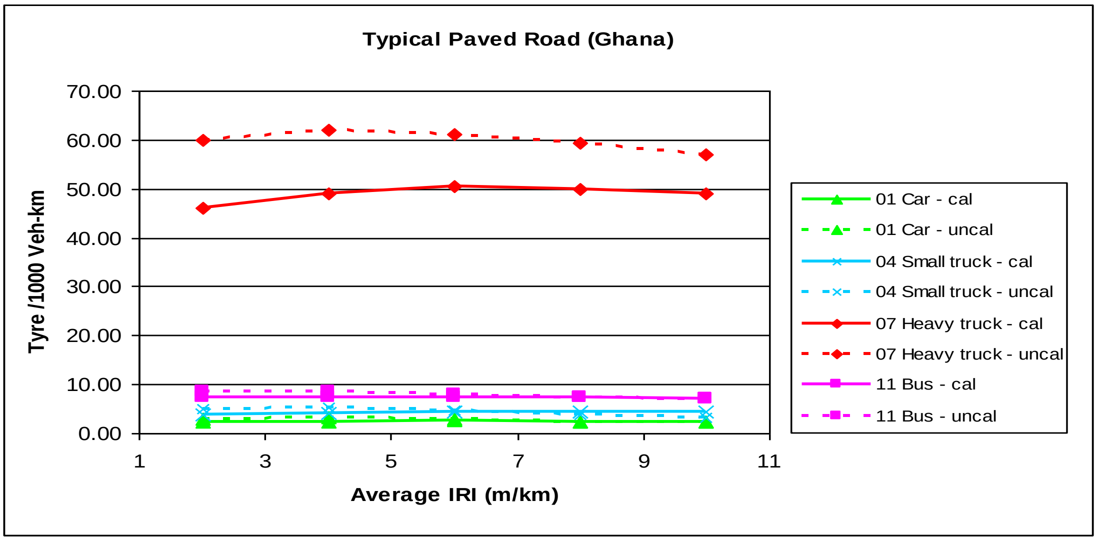


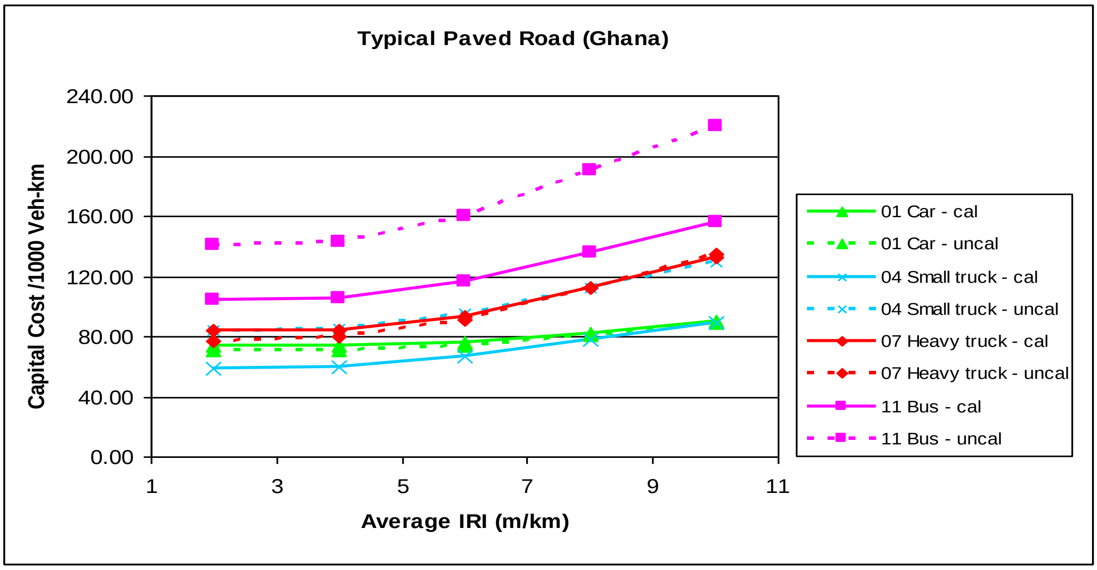

| Vehicle type | Good (IRI = 2) | | Fair (IRI = 4.5) | | Poor (IRI = 7) | |
|---|---|---|---|---|---|---|
| | Calibrated | Un-calibrated | Calibrated | Un-calibrated | Calibrated | Un-calibrated |
| 01 Car | 0.14 | 0.13 | 0.15 | 0.15 | 0.16 | 0.18 |
| 04 Small truck | 0.15 | 0.18 | 0.17 | 0.19 | 0.20 | 0.21 |
| 07 Heavy truck | 0.59 | 0.59 | 0.65 | 0.65 | 0.80 | 0.70 |
| 11 Bus | 0.25 | 0.29 | 0.30 | 0.32 | 0.35 | 0.36 |

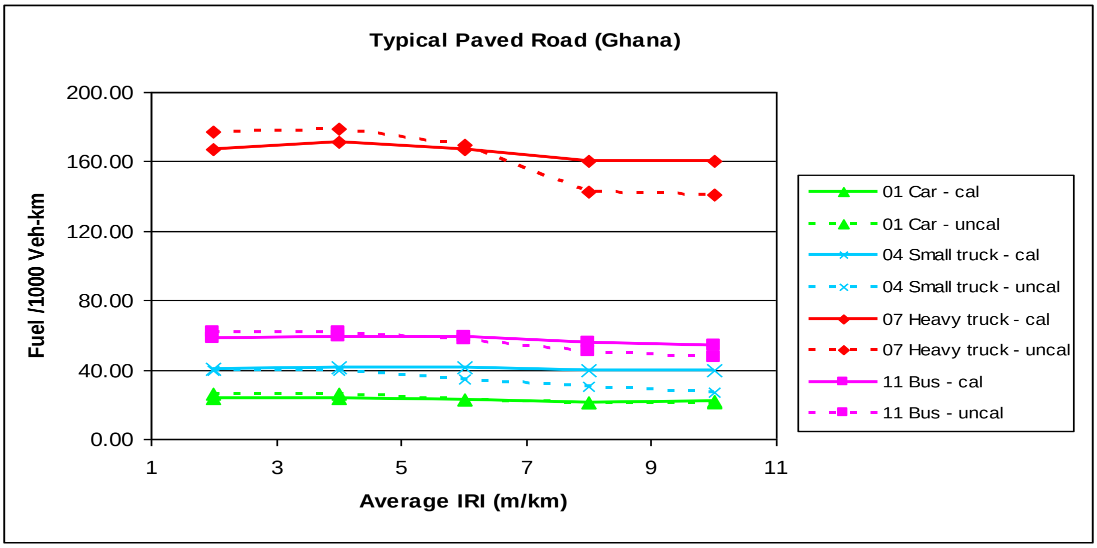
**Unsealed Road**

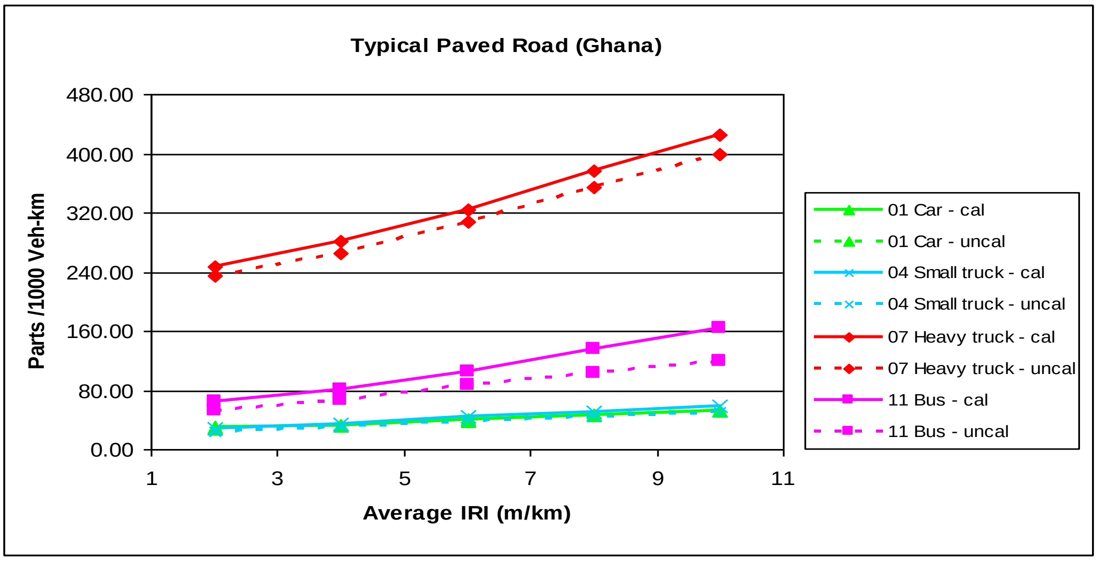
| Vehicle Type | Good (IRI = 4.5) | | Poor (IRI = 20) | |
|---|---|---|---|---|
| | Calibrated | Un-calibrated | Calibrated | Un-calibrated |
| 01 Car | 0.18 | 0.17 | 0.27 | 0.26 |
| 04 Small truck | 0.23 | 0.26 | 0.34 | 0.37 |
| 07 Heavy truck | 0.87 | 0.83 | 1.32 | 1.25 |
| 11 Bus | 0.39 | 0.44 | 0.61 | 0.66 |

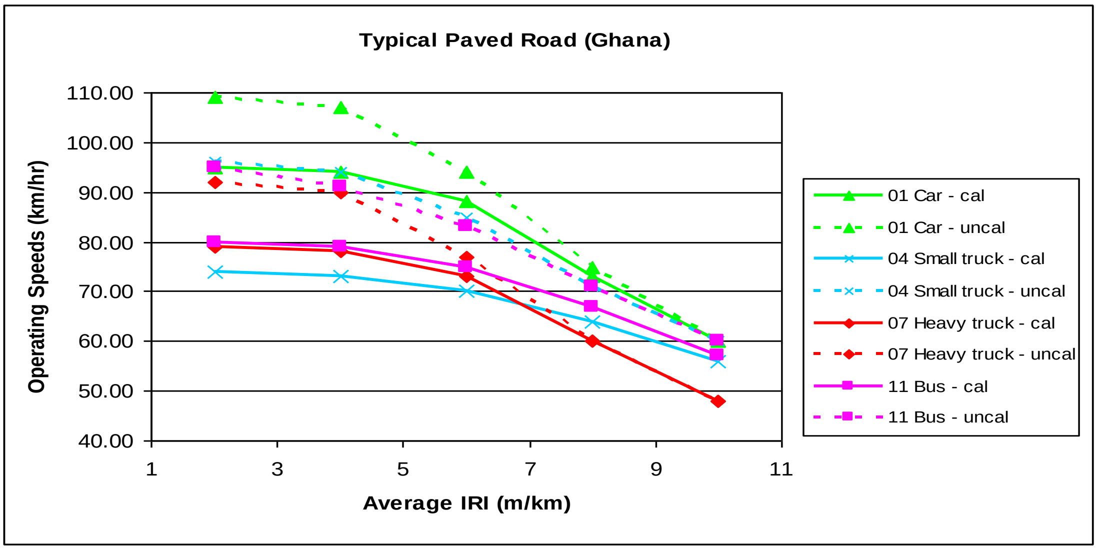
### F.2.2 VOC Components (Selected)

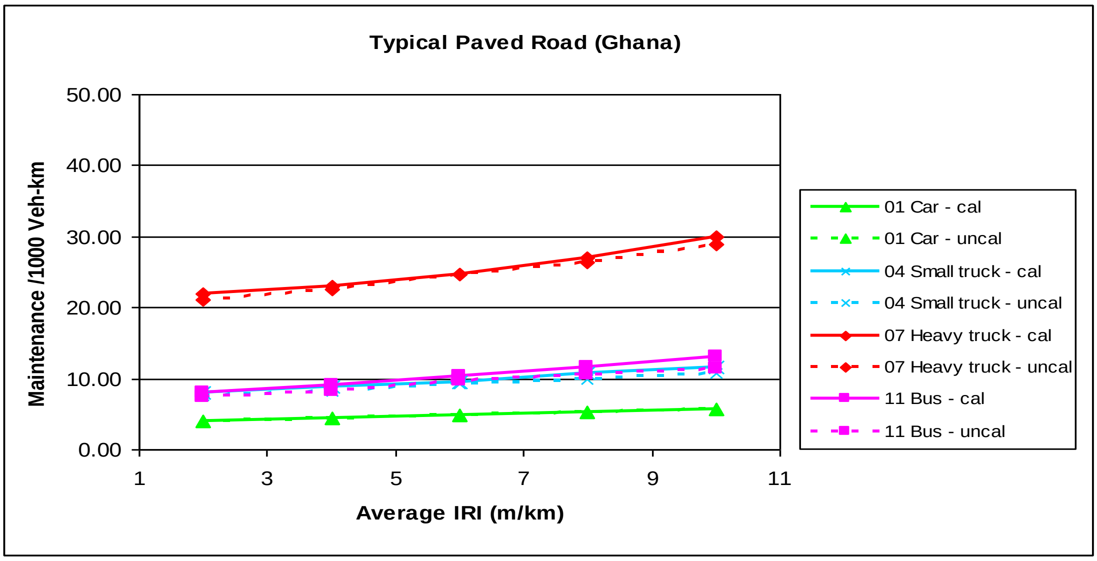

Typical Paved Road (Ghana)
Tyre /1000 Veh-km
70.00
60.00
50.00
40.00
30.00
20.00
10.00
0.00
1
3
5
7
9
11
Average IRI (m/km)
01 Car - cal
01 Car - uncal
04 Small truck - cal
04 Small truck - uncal
07 Heavy truck - cal
07 Heavy truck - uncal
11 Bus - cal
11 Bus - uncal


Typical Paved Road (Ghana)
Capital Cost /1000 Veh-km
240.00
200.00
160.00
120.00
80.00
40.00
0.00
1
3
5
7
9
11
Average IRI (m/km)
01 Car - cal
01 Car - uncal
04 Small truck - cal
04 Small truck - uncal
07 Heavy truck - cal
07 Heavy truck - uncal
11 Bus - cal
11 Bus - uncal


Typical Paved Road (Ghana)
Fuel /1000 Veh-km
200.00
160.00
120.00
80.00
40.00
0.00
1
3
5
7
9
11
Average IRI (m/km)
01 Car - cal
01 Car - uncal
04 Small truck - cal
04 Small truck - uncal
07 Heavy truck - cal
07 Heavy truck - uncal
11 Bus - cal
11 Bus - uncal


Typical Paved Road (Ghana)
Parts /1000 Veh-km
480.00
400.00
320.00
240.00
160.00
80.00
0.00
1
3
5
7
9
11
Average IRI (m/km)
01 Car - cal
01 Car - uncal
04 Small truck - cal
04 Small truck - uncal
07 Heavy truck - cal
07 Heavy truck - uncal
11 Bus - cal
11 Bus - uncal


Typical Paved Road (Ghana)
Operating Speeds (km/hr)
110.00
100.00
90.00
80.00
70.00
60.00
50.00
40.00
1
3
5
7
9
11
Average IRI (m/km)
01 Car - cal
01 Car - uncal
04 Small truck - cal
04 Small truck - uncal
07 Heavy truck - cal
07 Heavy truck - uncal
11 Bus - cal
11 Bus - uncal


Typical Paved Road (Ghana)
Maintenance /1000 Veh-km
50.00
40.00
30.00
20.00
10.00
0.00
1
3
5
7
9
11
Average IRI (m/km)
01 Car - cal
01 Car - uncal
04 Small truck - cal
04 Small truck - uncal
07 Heavy truck - cal
07 Heavy truck - uncal
11 Bus - cal
11 Bus - uncal

# APPENDIX G

## Field Data 2 – Sample of Traffic Data

| AKUMADAN TRAFFIC READINGS | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| DATE | TIME | LANE 1 | | | | | | LANE 2 | | | | | |
| | | CS1 | CS2 | CS3 | CS4 | CS5 | CS6 | CS1 | CS2 | CS3 | CS4 | CS5 | CS6 |
| 290909 | 0 | 0 | 5 | 0 | 5 | 0 | 4 | 0 | 9 | 4 | 5 | 2 | 12 |
| 290909 | 100 | 0 | 4 | 2 | 7 | 1 | 7 | 0 | 4 | 8 | 4 | 3 | 8 |
| 290909 | 200 | 0 | 7 | 1 | 3 | 1 | 2 | 0 | 2 | 2 | 2 | 1 | 6 |
| 290909 | 300 | 0 | 2 | 1 | 4 | 1 | 4 | 0 | 4 | 1 | 3 | 0 | 5 |
| 290909 | 400 | 0 | 4 | 0 | 6 | 0 | 3 | 0 | 5 | 4 | 11 | 0 | 3 |
| 290909 | 500 | 0 | 4 | 0 | 2 | 0 | 2 | 0 | 5 | 3 | 3 | 0 | 7 |
| 290909 | 600 | 0 | 10 | 2 | 1 | 1 | 3 | 0 | 19 | 2 | 5 | 0 | 5 |
| 290909 | 700 | 2 | 51 | 1 | 7 | 2 | 2 | 0 | 65 | 0 | 8 | 0 | 8 |
| 290909 | 800 | 1 | 83 | 3 | 13 | 3 | 7 | 1 | 86 | 1 | 9 | 4 | 5 |
| 290909 | 900 | 3 | 85 | 4 | 14 | 2 | 9 | 4 | 86 | 3 | 11 | 2 | 5 |
| 290909 | 1000 | 1 | 77 | 4 | 13 | 0 | 3 | 6 | 91 | 0 | 16 | 6 | 6 |
| 290909 | 1100 | 0 | 79 | 0 | 12 | 1 | 4 | 3 | 73 | 2 | 9 | 3 | 6 |
| 290909 | 1200 | 1 | 80 | 0 | 10 | 0 | 7 | 2 | 71 | 3 | 16 | 1 | 4 |
| 290909 | 1300 | 0 | 79 | 2 | 17 | 1 | 3 | 2 | 68 | 1 | 8 | 11 | 1 |
| 290909 | 1400 | 0 | 91 | 5 | 13 | 3 | 5 | 1 | 62 | 1 | 13 | 4 | 8 |
| 290909 | 1500 | 1 | 63 | 1 | 8 | 0 | 4 | 3 | 72 | 0 | 9 | 1 | 4 |
| 290909 | 1600 | 0 | 84 | 5 | 18 | 1 | 5 | 0 | 78 | 4 | 16 | 5 | 4 |
| 290909 | 1700 | 1 | 92 | 3 | 20 | 1 | 6 | 3 | 66 | 3 | 9 | 3 | 7 |
| 290909 | 1800 | 2 | 74 | 1 | 12 | 5 | 9 | 2 | 85 | 4 | 11 | 2 | 4 |
| 290909 | 1900 | 3 | 100 | 3 | 17 | 4 | 9 | 2 | 60 | 5 | 17 | 2 | 5 |
| 290909 | 2000 | 3 | 58 | 3 | 15 | 3 | 7 | 0 | 56 | 4 | 12 | 0 | 6 |
| 290909 | 2100 | 0 | 29 | 1 | 11 | 5 | 8 | 0 | 21 | 3 | 8 | 0 | 7 |
| 290909 | 2200 | 1 | 36 | 3 | 11 | 0 | 9 | 0 | 21 | 3 | 3 | 2 | 10 |
| 290909 | 2300 | 0 | 14 | 4 | 11 | 2 | 7 | 0 | 3 | 5 | 6 | 4 | 3 |

| SOGAKOPE TRAFFIC READINGS | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| **DATE** | **TIME** | **LANE 1** | | | | | | **LANE 2** | | | | | |
| | | **CS1** | **CS2** | **CS3** | **CS4** | **CS5** | **CS6** | **CS1** | **CS2** | **CS3** | **CS4** | **CS5** | **CS6** |
| 120509 | 0 | 0 | 6 | 0 | 4 | 6 | 0 | 0 | 13 | 0 | 10 | 2 | 1 |
| 120509 | 100 | 0 | 3 | 0 | 17 | 2 | 0 | 0 | 12 | 0 | 14 | 4 | 1 |
| 120509 | 200 | 0 | 4 | 0 | 8 | 2 | 0 | 0 | 9 | 0 | 10 | 2 | 0 |
| 120509 | 300 | 0 | 5 | 0 | 6 | 1 | 0 | 0 | 3 | 0 | 3 | 0 | 0 |
| 120509 | 400 | 0 | 8 | 0 | 4 | 1 | 2 | 0 | 10 | 0 | 3 | 1 | 0 |
| 120509 | 500 | 0 | 9 | 0 | 6 | 3 | 2 | 0 | 32 | 0 | 7 | 2 | 0 |
| 120509 | 600 | 0 | 33 | 0 | 17 | 3 | 0 | 0 | 61 | 0 | 8 | 0 | 0 |
| 120509 | 700 | 0 | 97 | 0 | 34 | 8 | 0 | 1 | 113 | 0 | 24 | 7 | 0 |
| 120509 | 800 | 1 | 118 | 1 | 30 | 1 | 0 | 0 | 104 | 0 | 23 | 2 | 0 |
| 120509 | 900 | 1 | 111 | 0 | 27 | 2 | 1 | 0 | 132 | 1 | 19 | 4 | 1 |
| 120509 | 1000 | 0 | 102 | 0 | 27 | 3 | 1 | 1 | 116 | 0 | 17 | 3 | 1 |
| 120509 | 1100 | 2 | 112 | 0 | 21 | 3 | 0 | 0 | 109 | 0 | 23 | 6 | 1 |
| 120509 | 1200 | 0 | 102 | 1 | 25 | 5 | 0 | 0 | 96 | 0 | 22 | 5 | 1 |
| 120509 | 1300 | 0 | 86 | 0 | 17 | 1 | 0 | 0 | 95 | 0 | 26 | 5 | 0 |
| 120509 | 1400 | 0 | 113 | 0 | 22 | 7 | 0 | 0 | 112 | 0 | 19 | 3 | 1 |
| 120509 | 1500 | 1 | 116 | 0 | 31 | 3 | 0 | 0 | 115 | 0 | 31 | 8 | 0 |
| 120509 | 1600 | 1 | 110 | 0 | 26 | 6 | 0 | 0 | 114 | 0 | 24 | 2 | 1 |
| 120509 | 1700 | 1 | 121 | 0 | 36 | 10 | 0 | 0 | 112 | 0 | 23 | 4 | 2 |
| 120509 | 1800 | 1 | 142 | 0 | 32 | 7 | 0 | 1 | 123 | 0 | 28 | 7 | 1 |
| 120509 | 1900 | 0 | 110 | 0 | 18 | 6 | 0 | 0 | 120 | 0 | 24 | 9 | 0 |
| 120509 | 2000 | 1 | 102 | 0 | 27 | 3 | 0 | 0 | 86 | 1 | 15 | 2 | 4 |
| 120509 | 2100 | 0 | 64 | 0 | 16 | 7 | 1 | 0 | 60 | 0 | 22 | 1 | 0 |
| 120509 | 2200 | 0 | 40 | 0 | 11 | 7 | 0 | 0 | 26 | 1 | 21 | 2 | 0 |
| 120509 | 2300 | 0 | 24 | 0 | 7 | 3 | 1 | 0 | 21 | 0 | 8 | 2 | 2 |

# APPENDIX H

## Field Data 2 – Sample of Temperature Data

| AKUMADAN TEMPERATURE READINGS | | | | | |
|---|---|---|---|---|---|
| **Date and Time (PDT)** | **Ambient Temperature (°C)** | **Thermocouple 1 (°C)** | **Thermocouple 2 (°C)** | **Thermocouple 3 (°C)** | **Thermocouple 4 (°C)** |
| 9/29/09 0:00 | 26.3 | 34.32 | 32.34 | 30.88 | 28.36 |
| 9/29/09 1:00 | 26.3 | 34.32 | 32 | 30.48 | 28.24 |
| 9/29/09 2:00 | 26.4 | 34.36 | 31.62 | 30.46 | 28.34 |
| 9/29/09 3:00 | 26.9 | 34.28 | 31.48 | 30.32 | 29.02 |
| 9/29/09 4:00 | 27.8 | 34.12 | 31.38 | 30.62 | 30.33 |
| 9/29/09 5:00 | 30.4 | 33.96 | 31.45 | 31.68 | 33.73 |
| 9/29/09 6:00 | 34.1 | 33.98 | 32.41 | 34.16 | 38.82 |
| 9/29/09 7:00 | 36.4 | 33.89 | 33.72 | 36.69 | 42.97 |
| 9/29/09 8:00 | 37.5 | 33.88 | 35.46 | 39.37 | 45.46 |
| 9/29/09 9:00 | 38.4 | 33.79 | 36.88 | 40.96 | 47.98 |
| 9/29/09 10:00 | 37.9 | 33.82 | 38.3 | 42.61 | 47.83 |
| 9/29/09 11:00 | 36.4 | 33.83 | 39.37 | 42.74 | 46.05 |
| 9/29/09 12:00 | 34.4 | 33.87 | 39.64 | 41.97 | 43.13 |
| 9/29/09 13:00 | 32.2 | 33.89 | 39.31 | 40.47 | 39.49 |
| 9/29/09 14:00 | 30.5 | 34 | 38.72 | 38.72 | 36.8 |
| 9/29/09 15:00 | 29.6 | 34.03 | 37.82 | 37.35 | 35.08 |
| 9/29/09 16:00 | 29 | 34.08 | 37.05 | 36.23 | 33.73 |
| 9/29/09 17:00 | 28.6 | 34.21 | 36.36 | 35.26 | 32.81 |
| 9/29/09 18:00 | 28.1 | 34.29 | 35.7 | 34.41 | 31.85 |
| 9/29/09 19:00 | 27.8 | 34.41 | 34.94 | 33.65 | 31.15 |
| 9/29/09 20:00 | 27.5 | 34.41 | 34.35 | 32.94 | 30.5 |
| 9/29/09 21:00 | 27.2 | 34.51 | 33.93 | 32.47 | 29.9 |
| 9/29/09 22:00 | 26.9 | 34.57 | 33.35 | 31.94 | 29.37 |
| 9/29/09 23:00 | 27 | 34.49 | 32.92 | 31.45 | 29.3 |

| SOGAKOPE TEMPERATURE READINGS | | | | | |
|---|---|---|---|---|---|
| **Date and Time (PDT)** | **Ambient Temperature (°C)** | **Thermocouple 1 (°C)** | **Thermocouple 2 (°C)** | **Thermocouple 3 (°C)** | **Thermocouple 4 (°C)** |
| 2009-05-04 00:51 | 29.7 | 33.17 | 34.63 | 39.51 | 38.56 |
| 2009-05-04 01:51 | 29.4 | 33.37 | 34.75 | 38.80 | 37.16 |
| 2009-05-04 02:51 | 29.15 | 33.10 | 34.85 | 38.08 | 36.04 |
| 2009-05-04 03:51 | 28.95 | 33.13 | 34.88 | 37.44 | 35.11 |
| 2009-05-04 04:51 | 28.8 | 33.57 | 34.93 | 36.77 | 34.44 |
| 2009-05-04 05:51 | 28.7 | 33.70 | 34.98 | 36.26 | 33.90 |
| 2009-05-04 06:51 | 28.6 | 33.81 | 34.91 | 35.76 | 33.39 |
| 2009-05-04 07:51 | 28.55 | 33.84 | 34.83 | 35.30 | 33.06 |
| 2009-05-04 08:51 | 28.45 | 33.83 | 34.76 | 34.88 | 32.67 |
| 2009-05-04 09:51 | 28.35 | 33.91 | 34.72 | 34.52 | 32.16 |
| 2009-05-04 10:51 | 28.25 | 33.90 | 34.66 | 34.16 | 31.68 |
| 2009-05-04 11:51 | 28.15 | 33.89 | 34.56 | 33.73 | 31.29 |
| 2009-05-04 12:51 | 28.1 | 33.98 | 34.45 | 33.42 | 31.01 |
| 2009-05-04 13:51 | 28.05 | 33.81 | 34.37 | 33.14 | 30.73 |
| 2009-05-04 14:51 | 28 | 33.76 | 34.26 | 32.89 | 30.50 |
| 2009-05-04 15:51 | 27.95 | 33.97 | 34.18 | 32.61 | 30.30 |
| 2009-05-04 16:51 | 27.85 | 33.99 | 34.08 | 32.33 | 30.00 |
| 2009-05-04 17:51 | 27.8 | 33.98 | 34.00 | 32.13 | 29.78 |
| 2009-05-04 18:51 | 27.8 | 34.04 | 33.91 | 31.93 | 29.63 |
| 2009-05-04 19:51 | 27.8 | 34.01 | 33.74 | 31.76 | 29.54 |
| 2009-05-04 20:51 | 27.8 | 33.80 | 33.65 | 31.56 | 29.36 |
| 2009-05-04 21:51 | 27.8 | 33.71 | 33.62 | 31.38 | 29.12 |
| 2009-05-04 22:51 | 27.75 | 33.81 | 33.49 | 31.16 | 28.93 |
| 2009-05-04 23:51 | 27.65 | 33.85 | 33.36 | 30.94 | 28.74 |

# APPENDIX I

## Field Data 4 – Sample of Moisture Data

| AKUMADAN MOISTURE READINGS | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| **DATE** | **SUBBASE** | **BASE** | **DATE** | **SUBBASE** | **BASE** | **DATE** | **SUBBASE** | **BASE** |
| 29/09/2009 | 39 | 18 | 29/01/2010 | 41 | 26 | 01/06/2010 | 41 | 29 |
| 30/09/2009 | 39 | 19 | 30/01/2010 | 42 | 26 | 02/06/2010 | 40 | 29 |
| 01/10/2009 | 39 | 16 | 31/01/2010 | 41 | 26 | 03/06/2010 | 40 | 29 |
| 02/10/2009 | 39 | 16 | 01/02/2010 | 38 | 25 | 04/06/2010 | 41 | 29 |
| 03/10/2009 | 39 | 19 | 02/02/2010 | 39 | 24 | 05/06/2010 | 41 | 29 |
| 04/10/2009 | 52 | 15 | 03/02/2010 | 38 | 25 | 06/06/2010 | 40 | 29 |
| 05/10/2009 | 44 | 19 | 04/02/2010 | 37 | 26 | 07/06/2010 | 41 | 29 |
| 06/10/2009 | 42 | 14 | 05/02/2010 | 38 | 25 | 08/06/2010 | 44 | 30 |
| 07/10/2009 | 42 | 14 | 06/02/2010 | 37 | 26 | 09/06/2010 | 45 | 29 |
| 08/10/2009 | 39 | 16 | 07/02/2010 | 38 | 25 | 10/06/2010 | 44 | 29 |
| 09/10/2009 | 34 | 14 | 08/02/2010 | 37 | 26 | 11/06/2010 | 41 | 29 |
| 10/10/2009 | 42 | 14 | 09/02/2010 | 39 | 24 | 12/06/2010 | 44 | 33 |

| SOGAKOPE MOISTURE READINGS | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| **DATE** | **SUBBASE** | **BASE** | **DATE** | **SUBBASE** | **BASE** | **DATE** | **SUBBASE** | **BASE** |
| 26/10/2009 | 13 | 6 | 04/02/2010 | 8 | 3 | 18/05/2010 | 2 | 1 |
| 27/10/2009 | 12 | 6 | 05/02/2010 | 8 | 3 | 19/05/2010 | 3 | 2 |
| 28/10/2009 | 12 | 6 | 06/02/2010 | 8 | 3 | 20/05/2010 | 2 | 1 |
| 29/10/2009 | 12 | 6 | 07/02/2010 | 8 | 3 | 21/05/2010 | 2 | 1 |
| 30/10/2009 | 12 | 6 | 08/02/2010 | 8 | 3 | 22/05/2010 | 2 | 1 |
| 31/10/2009 | 12 | 6 | 09/02/2010 | 8 | 3 | 23/05/2010 | 2 | 1 |
| 01/11/2009 | 12 | 6 | 10/02/2010 | 8 | 3 | 24/05/2010 | 2 | 1 |
| 02/11/2009 | 13 | 6 | 11/02/2010 | 8 | 3 | 25/05/2010 | 2 | 1 |
| 03/11/2009 | 13 | 6 | 12/02/2010 | 5 | 2 | 26/05/2010 | 2 | 1 |
| 04/11/2009 | 13 | 6 | 13/02/2010 | 5 | 2 | 27/05/2010 | 2 | 1 |
| 05/11/2009 | 12 | 6 | 14/02/2010 | 5 | 2 | 28/05/2010 | 2 | 1 |
| 06/11/2009 | 12 | 6 | 15/02/2010 | 5 | 3 | 29/05/2010 | 2 | 1 |
| 07/11/2009 | 12 | 6 | 16/02/2010 | 3 | 3 | 30/05/2010 | 2 | 1 |
| 08/11/2009 | 12 | 6 | 17/02/2010 | 3 | 3 | 31/05/2010 | 2 | 1 |
| 09/11/2009 | 12 | 6 | 18/02/2010 | 3 | 3 | 01/06/2010 | 0 | 4 |
| 10/11/2009 | 11 | 6 | 19/02/2010 | 3 | 2 | 02/06/2010 | 0 | 4 |

# APPENDIX J

## Sample results of the KENLAYER analysis

INPUT FILE NAME -C:\Users\user\Desktop\KENPAVE\SOGAKOPE PAVEMENT.DAT

NUMBER OF PROBLEMS TO BE SOLVED = 1

TITLE -simple

MATL = 4 FOR VISCOELASTIC AND NONLINEAR ELASTIC LAYERED SYSTEMS
NDAMA=2, SO DAMAGE ANALYSIS WITH DETAILED PRINTOUT WILL BE PERFORMED
NUMBER OF PERIODS PER YEAR (NPY) = 1
NUMBER OF LOAD GROUPS (NLG) = 7
TOLERANCE FOR INTEGRATION (DEL) -- = 0.985
NUMBER OF LAYERS (NL)------------- = 4
NUMBER OF Z COORDINATES (NZ)------ = 12
LIMIT OF INTEGRATION CYCLES (ICL)- = 80
COMPUTING CODE (NSTD)------------- = 9
SYSTEM OF UNITS (NUNIT)------------= 1

Length and displacement in cm, stress and modulus in kPa
unit weight in kN/m^3, and temperature in C

THICKNESSES OF LAYERS (TH) ARE : 16.5 20 20
POISSON'S RATIOS OF LAYERS (PR) ARE : 0.35 0.38 0.42 0.43
CONDITIONS OF INTERFACES (INT) ARE : 1 1 1

FOR PERIOD NO. 1 LAYER NO. AND MODULUS ARE : 1 2.055E+06 2 5.110E+05
3 2.820E+05 4 1.150E+05

LOAD GROUP NO. 1 HAS 1 CONTACT AREA
CONTACT RADIUS (CR)--------------- = 22
CONTACT PRESSURE (CP)------------- = 551
RADIAL COORDINATES OF 1 POINT(S) (RC) ARE : 0
LOAD GROUP NO. 2 HAS 1 CONTACT AREA
CONTACT RADIUS (CR)--------------- = 17
CONTACT PRESSURE (CP)------------- = 827
RADIAL COORDINATES OF 1 POINT(S) (RC) ARE : 0

LOAD GROUP NO. 3 HAS 2 CONTACT AREAS
CONTACT RADIUS (CR)--------------- = 20
CONTACT PRESSURE (CP)------------- = 827

NO. OF POINTS AT WHICH RESULTS ARE DESIRED (NPT)-- = 2
WHEEL SPACING ALONG X-AXIS (XW)------------------- = 0
WHEEL SPACING ALONG Y-AXIS (YW)------------------- = 33

RESPONSE PT. NO. AND (XPT, YPT) ARE: 1 0.000 0.000 2 0.000 16.500

LOAD GROUP NO. 4 HAS 4 CONTACT AREAS
CONTACT RADIUS (CR)--------------- = 21
CONTACT PRESSURE (CP)------------- = 827
NO. OF POINTS AT WHICH RESULTS ARE DESIRED (NPT)-- = 3
WHEEL SPACING ALONG X-AXIS (XW)------------------- = 130
WHEEL SPACING ALONG Y-AXIS (YW)------------------- = 33

RESPONSE PT. NO. AND (XPT, YPT) ARE: 1 0.000 0.000 2 0.000 16.500
3 65.000 16.500

LOAD GROUP NO. 5 HAS 4 CONTACT AREAS
CONTACT RADIUS (CR)--------------- = 22
CONTACT PRESSURE (CP)------------- = 827
NO. OF POINTS AT WHICH RESULTS ARE DESIRED (NPT)-- = 3
WHEEL SPACING ALONG X-AXIS (XW)------------------- = 130
WHEEL SPACING ALONG Y-AXIS (YW)------------------- = 33

RESPONSE PT. NO. AND (XPT, YPT) ARE: 1 0.000 0.000 2 0.000 16.500
3 65.000 16.500

LOAD GROUP NO. 6 HAS 4 CONTACT AREAS
CONTACT RADIUS (CR)--------------- = 23
CONTACT PRESSURE (CP)------------- = 827
NO. OF POINTS AT WHICH RESULTS ARE DESIRED (NPT)-- = 3
WHEEL SPACING ALONG X-AXIS (XW)------------------- = 130
WHEEL SPACING ALONG Y-AXIS (YW)------------------- = 33

RESPONSE PT. NO. AND (XPT, YPT) ARE: 1 0.000 0.000 2 0.000 16.500
3 65.000 16.500

LOAD GROUP NO. 7 HAS 6 CONTACT AREAS
CONTACT RADIUS (CR)--------------- = 21
CONTACT PRESSURE (CP)------------- = 827
NO. OF POINTS AT WHICH RESULTS ARE DESIRED (NPT)-- = 3
WHEEL SPACING ALONG X-AXIS (XW)------------------- = 130
WHEEL SPACING ALONG Y-AXIS (YW)------------------- = 33

RESPONSE PT. NO. AND (XPT, YPT) ARE: 1 0.000 0.000 2 0.000 16.500
3 65.000 16.500

NUMBER OF NONLINEAR LAYERS (NOLAY)-------------------------- = 3
MAXIMUM NUMBER OF ITERATIONS FOR NONLINEAR ANALYSIS (ITENOL) = 20

LAYER NUMBER (LAYNO) AND SOIL TYPE (NCLAY) ARE: 2 0 3 0 4 0

Z COORDINATES (ZCNOL) FOR COMPUTING ELASTIC MODULUS ARE: 26.5 46 60
R COORDINATE (RCNOL) FOR COMPUTING ELASTIC MODULUS ---------- = 0
X COORDINATE (XPTNOL) FOR COMPUTING ELASTIC MODULUS --------- = 0
Y COORDINATE (YPTNOL) FOR COMPUTING ELASTIC MODULUS --------- = 6.75
SLOPE OF LOAD DISTRIBUTION (SLD) ---------------------------- = 0
TOLERANCE (DELNOL) FOR NONLINEAR ANALYSIS ------------------- = 0.01
RELAXATION FACTORS (RELAX) FOR NONLINEAR ANALYSIS OF EACH PERIOD ARE: 0.5

UNIT WEIGHT OF LAYERS (GAM) ARE: 26.3 22.65 21.88 20.75

LAYER NO. = 2 NCLAY = 0 K2 = 0.6 K0 = 1
LAYER NO. = 3 NCLAY = 0 K2 = 0.6 K0 = 1
LAYER NO. = 4 NCLAY = 0 K2 = 0.51 K0 = 1

LAYER NUMBER AND GEOSTATIC STRESS (GEOS) ARE:
2 6.60450 3 10.94810 4 13.97175

FOR PERIOD 1 LAYER NO. = 2 NCLAY = 0 PHI = 0 K1 = 511000
FOR PERIOD 1 LAYER NO. = 3 NCLAY = 0 PHI = 0 K1 = 282000
FOR PERIOD 1 LAYER NO. = 4 NCLAY = 0 PHI = 0 K1 = 115000

FOR LOAD GROUP 1 LAYER NO. AND R COORDINATE FOR COMPUTING MODULUS ARE:
2 0 3 0 4 0

FOR LOAD GROUP 2 LAYER NO. AND R COORDINATE FOR COMPUTING MODULUS ARE:
2 0 3 0 4 0

FOR LOAD GROUP 3 LAYER NO. AND X COORDINATE FOR COMPUTING MODULUS ARE:
2 0 3 0 4 0
FOR LOAD GROUP 3 LAYER NO. AND Y COORDINATE FOR COMPUTING MODULUS ARE:
2 6.75 3 6.75 4 6.75
FOR LOAD GROUP 4 LAYER NO. AND X COORDINATE FOR COMPUTING MODULUS ARE:
2 0 3 0 4 0
FOR LOAD GROUP 4 LAYER NO. AND Y COORDINATE FOR COMPUTING MODULUS ARE:
2 6.75 3 6.75 4 6.75
FOR LOAD GROUP 5 LAYER NO. AND X COORDINATE FOR COMPUTING MODULUS ARE:
2 0 3 0 4 0
FOR LOAD GROUP 5 LAYER NO. AND Y COORDINATE FOR COMPUTING MODULUS ARE:
2 6.75 3 6.75 4 6.75

FOR LOAD GROUP 6 LAYER NO. AND X COORDINATE FOR COMPUTING MODULUS ARE:
2 0 3 0 4 0
FOR LOAD GROUP 6 LAYER NO. AND Y COORDINATE FOR COMPUTING MODULUS ARE:
2 6.75 3 6.75 4 6.75
FOR LOAD GROUP 7 LAYER NO. AND X COORDINATE FOR COMPUTING MODULUS ARE:
2 0 3 0 4 0
FOR LOAD GROUP 7 LAYER NO. AND Y COORDINATE FOR COMPUTING MODULUS ARE:
2 6.75 3 6.75 4 6.75

DURATION OF MOVING LOAD (DUR) = 0.1
NUMBER OF VISCOELASTIC LAYER (NVL) = 1
LAYER NUMBERS WHICH ARE VISCOELASTIC (LNV) = 1
CREEP TIMES (TYME) ARE: 0.1 0.5 1 5 10 50 100 500 1000

FOR LAYER 1 TIME TEMPERATURE SHIFT FACTOR (BETA) = 3.00085
REFERENCE TEMPERATURE (TEMREF) = 36

CREEP COMPLIANCES (CREEP) AT REFERENCE TEMP. (TEMREF) OF 36 ARE:
4.590E-07 8.070E-07 1.010E-06 1.830E-06 2.030E-06 2.600E-06 2.700E-06
3.220E-06 3.380E-06

LAYER NO. 1 DIRICHLET SERIES AT REFERENCE TEMPERATURE (TEMREF) OF 36 ARE:
-3.664E-01 -2.070E-04 6.784E-05 -2.854E-06 1.411E-06 -2.422E-06 3.121E-06

COMPUTED COMPLIANCES (CREEP) AT REFERENCE TEMP.(TEMREF) OF 36 ARE:
4.590E-07 8.072E-07 1.008E-06 1.907E-06 1.905E-06 2.673E-06 3.035E-06
3.121E-06 3.121E-06

FOR PERIOD NO. 1 LAYER NO. AND TEMPERATURE ARE: 1 37

CREEP COMPLIANCES (CREEP) OF LAYER 1 AT TEMPERATURE (TEMP) OF 37 ARE:
3.035E-06 3.121E-06 3.121E-06 3.121E-06 3.121E-06 3.121E-06 3.121E-06
3.121E-06 3.121E-06

NUMBER OF LAYERS FOR BOTTOM TENSION (NLBT)---- = 1
NUMBER OF LAYERS FOR TOP COMPRESSION (NLTC)--- = 1
LAYER NO. FOR BOTTOM TENSION (LNBT) ARE: 1
LAYER NO. FOR TOP COMPRESSION (LNTC) ARE: 4

LOAD REPETITIONS (TNLR) IN PERIOD 1 FOR EACH LOAD GROUP ARE : 2035 25841
88027 393229 32142 23531 19100

DAMAGE COEF.'S (FT) FOR BOTTOM TENSION OF LAYER 1 ARE: 0.414 3.291 0.854

DAMAGE COEFICIENTS (FT) FOR TOP COMPRESSION OF LAYER 4 ARE: 1.365E-09 4.477

DAMAGE ANALYSIS OF PERIOD NO. 1 LOAD GROUP NO. 1

```
  RADIAL     VERTICAL     VERTICAL   VERTICAL    RADIAL  TANGENTIAL   SHEAR
 COORDINATE  COORDINATE  DISPLACEMENT  STRESS     STRESS    STRESS     STRESS
                         (STRAIN)   (STRAIN)  (STRAIN)   (STRAIN)
  0.00000    16.50000    0.02928   1786.481   1002.700   1002.700     0.000
 (STRAIN)                -1.530E-03  3.325E-05  3.325E-05  .000E+00
  0.00000    56.50010    0.02753   1009.255    259.810    259.810     0.000
 (STRAIN)                 1.652E-04 -6.009E-05 -6.009E-05  .000E+00
```

AT BOTTOM OF LAYER 1 TENSILE STRAIN = 0.000E+00

ALLOWABLE LOAD REPETITIONS = 1.000E+30 DAMAGE RATIO = 0.000E+00

AT TOP OF LAYER 4 COMPRESSIVE STRAIN = 1.652E-04

ALLOWABLE LOAD REPETITIONS = 1.168E+08 DAMAGE RATIO = 1.743E-05

DAMAGE ANALYSIS OF PERIOD NO. 1 LOAD GROUP NO. 2

```
  RADIAL     VERTICAL     VERTICAL   VERTICAL    RADIAL  TANGENTIAL   SHEAR
 COORDINATE  COORDINATE  DISPLACEMENT  STRESS     STRESS    STRESS     STRESS
                         (STRAIN)   (STRAIN)  (STRAIN)   (STRAIN)
  0.00000    16.50000    0.02873   2634.949   1469.843   1469.843     0.000
 (STRAIN)                -2.252E-03  4.160E-05  4.160E-05  .000E+00
  0.00000    56.50010    0.02656   1107.299    212.608    212.608     0.000
 (STRAIN)                 1.955E-04 -7.507E-05 -7.507E-05  .000E+00
```

AT BOTTOM OF LAYER 1 TENSILE STRAIN = 0.000E+00

ALLOWABLE LOAD REPETITIONS = 1.000E+30 DAMAGE RATIO = 0.000E+00

AT TOP OF LAYER 4 COMPRESSIVE STRAIN = 1.955E-04

ALLOWABLE LOAD REPETITIONS = 5.490E+07 DAMAGE RATIO = 4.707E-04

DAMAGE ANALYSIS OF PERIOD NO. 1 LOAD GROUP NO. 3

```
 POINT    VERTICAL   VERTICAL    VERTICAL     MAJOR      MINOR   INTERMEDIATE
                DISPL.             PRINCIPAL   PRINCIPAL   PRINCIPAL
  NO.    COORDINATE (HORIZONTAL   STRESS      STRESS      STRESS      STRESS
               P. STRAIN)  (STRAIN)    (STRAIN)    (STRAIN)    (STRAIN)

  1     16.50000    0.04258    3635.017    3635.555    1991.192    2027.397
       (STRAIN)   4.237E-05  -3.078E-03  -3.078E-03  -3.103E-05   1.356E-06
```

1 56.50010 0.04053 2031.019 2033.880 574.047 575.467
(STRAIN) -7.646E-05 2.136E-04 2.141E-04 -7.621E-05 -2.445E-06

2 16.50000 0.04787 5144.535 5144.705 2923.221 2761.133
(STRAIN) 5.797E-05 -4.357E-03 -4.356E-03 4.001E-04 2.135E-06
2 56.50010 0.04501 2564.353 2564.353 602.076 620.902
(STRAIN) -1.068E-04 2.835E-04 2.835E-04 -1.068E-04 -3.330E-06

AT BOTTOM OF LAYER 1 TENSILE STRAIN = 0.000E+00
ALLOWABLE LOAD REPETITIONS = 1.000E+30 DAMAGE RATIO = 0.000E+00

AT TOP OF LAYER 4 COMPRESSIVE STRAIN = 2.835E-04
ALLOWABLE LOAD REPETITIONS = 1.040E+07 DAMAGE RATIO = 8.465E-03

DAMAGE ANALYSIS OF PERIOD NO. 1 LOAD GROUP NO. 4

| POINT NO. | VERTICAL COORDINATE | VERTICAL DISPL. (HORIZONTAL P. STRAIN) | VERTICAL STRESS (STRAIN) | MAJOR PRINCIPAL STRESS (STRAIN) | MINOR PRINCIPAL STRESS (STRAIN) | INTERMEDIATE PRINCIPAL STRESS (STRAIN) |
|---|---|---|---|---|---|---|
| 1 | 16.50000 | 0.05383 | 3877.483 | 3878.068 | 2139.364 | 2139.365 |
| | (STRAIN) | 4.211E-05 | -3.277E-03 | -3.278E-03 | 4.289E-05 | 1.338E-06 |
| 1 | 56.50010 | 0.05180 | 2319.101 | 2322.348 | 727.415 | 731.663 |
| | (STRAIN) | -7.470E-05 | 2.156E-04 | 2.162E-04 | -7.473E-05 | -2.390E-06 |
| 2 | 16.50000 | 0.05888 | 5316.802 | 5316.973 | 3008.471 | 2856.937 |
| | (STRAIN) | 5.632E-05 | -4.495E-03 | -4.495E-03 | 3.761E-04 | 2.061E-06 |
| 2 | 56.50010 | 0.05613 | 2859.684 | 2859.688 | 765.375 | 783.130 |
| | (STRAIN) | -1.022E-04 | 2.799E-04 | 2.799E-04 | -1.022E-04 | -3.198E-06 |
| 3 | 16.50000 | 0.04209 | 1049.926 | 1049.932 | 583.058 | 570.904 |
| | (STRAIN) | 9.286E-06 | -8.826E-04 | -8.826E-04 | 3.514E-05 | 3.252E-07 |
| 3 | 56.50010 | 0.04163 | 975.064 | 975.070 | 537.003 | 543.122 |
| | (STRAIN) | -1.466E-05 | 6.514E-05 | 6.514E-05 | -1.478E-05 | -4.438E-07 |

AT BOTTOM OF LAYER 1 TENSILE STRAIN = 0.000E+00
ALLOWABLE LOAD REPETITIONS = 1.000E+30 DAMAGE RATIO = 0.000E+00

AT TOP OF LAYER 4 COMPRESSIVE STRAIN = 2.799E-04
ALLOWABLE LOAD REPETITIONS = 1.102E+07 DAMAGE RATIO = 3.569E-02

DAMAGE ANALYSIS OF PERIOD NO. 1 LOAD GROUP NO. 4
MULTIPLE AXLES WITH RESPONSE POINT HALFWAY BETWEEN TWO AXLES

| POINT NO. | VERTICAL COORDINATE | VERTICAL DISPL. (HORIZONTAL P. STRAIN) | VERTICAL STRESS (STRAIN) | MAJOR PRINCIPAL STRESS (STRAIN) | MINOR PRINCIPAL STRESS (STRAIN) | INTERMEDIATE PRINCIPAL STRESS (STRAIN) |
|---|---|---|---|---|---|---|
| 1 | 16.50000 | 0.04116 | 1003.277 | 1003.318 | 555.032 | 547.429 |
| | (STRAIN) | 8.818E-06 | -8.433E-04 | -8.432E-04 | 2.503E-05 | 3.017E-07 |
| 1 | 56.50010 | 0.04072 | 933.989 | 934.309 | 518.424 | 523.588 |
| | (STRAIN) | -1.373E-05 | 6.197E-05 | 6.203E-05 | -1.384E-05 | -4.171E-07 |
| 2 | 16.50000 | 0.04209 | 1049.926 | 1049.932 | 583.058 | 570.904 |
| | (STRAIN) | 9.286E-06 | -8.826E-04 | -8.826E-04 | 3.514E-05 | 3.252E-07 |
| 2 | 56.50010 | 0.04163 | 975.064 | 975.070 | 537.003 | 543.122 |
| | (STRAIN) | -1.466E-05 | 6.514E-05 | 6.514E-05 | -1.478E-05 | -4.438E-07 |
| 3 | 16.50000 | 0.05888 | 5316.827 | 5316.755 | 2932.805 | 2932.806 |
| | (STRAIN) | 5.628E-05 | -4.495E-03 | -4.496E-03 | 5.733E-05 | 1.858E-06 |
| 3 | 56.50010 | 0.05613 | 2859.696 | 2859.725 | 765.209 | 783.293 |
| | (STRAIN) | -1.022E-04 | 2.799E-04 | 2.799E-04 | -1.022E-04 | -3.198E-06 |

AT BOTTOM OF LAYER 1 DIFFERENTIAL TENSILE STRAIN = 0.000E+00
ALLOWABLE LOAD REPETITIONS = 1.000E+30 DAMAGE RATIO = 0.000E+00

AT TOP OF LAYER 4 DIFFERENTIAL COMPRESSIVE STRAIN = 2.147E-04
ALLOWABLE LOAD REPETITIONS = 3.608E+07 DAMAGE RATIO = 1.090E-02

DAMAGE ANALYSIS OF PERIOD NO. 1 LOAD GROUP NO. 5

| POINT NO. | VERTICAL COORDINATE | VERTICAL DISPL. (HORIZONTAL P. STRAIN) | VERTICAL STRESS (STRAIN) | MAJOR PRINCIPAL STRESS (STRAIN) | MINOR PRINCIPAL STRESS (STRAIN) | INTERMEDIATE PRINCIPAL STRESS (STRAIN) |
|---|---|---|---|---|---|---|
| 1 | 16.50000 | 0.05623 | 4000.553 | 4001.242 | 2205.902 | 2205.902 |
| | (STRAIN) | 4.232E-05 | -3.379E-03 | -3.380E-03 | 4.323E-05 | 1.340E-06 |
| 1 | 56.50010 | 0.05421 | 2476.502 | 2480.258 | 799.994 | 803.830 |
| | (STRAIN) | -7.506E-05 | 2.188E-04 | 2.195E-04 | -7.504E-05 | -2.401E-06 |
| 2 | 16.50000 | 0.06112 | 5359.952 | 5359.881 | 2955.173 | 2955.174 |
| | (STRAIN) | 5.575E-05 | -4.529E-03 | -4.529E-03 | 5.668E-05 | 1.835E-06 |
| 2 | 56.50010 | 0.05842 | 3010.732 | 3010.752 | 843.258 | 860.957 |
| | (STRAIN) | -1.006E-04 | 2.792E-04 | 2.792E-04 | -1.007E-04 | -3.156E-06 |

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 3     16.50000    0.04438    1152.016    1151.986     620.837     644.388
       (STRAIN)   9.797E-06  -9.680E-04  -9.681E-04  -3.811E-05   3.261E-07
 3     56.50010    0.04390    1069.977    1070.007     591.273     594.332
       (STRAIN)  -1.544E-05   6.866E-05   6.867E-05  -1.525E-05  -4.674E-07

AT BOTTOM OF LAYER  1   TENSILE STRAIN =   0.000E+00
ALLOWABLE LOAD REPETITIONS =   1.000E+30  DAMAGE RATIO =   0.000E+00

AT TOP OF LAYER  4   COMPRESSIVE STRAIN =   2.792E-04
ALLOWABLE LOAD REPETITIONS =   1.113E+07  DAMAGE RATIO =   2.887E-03

DAMAGE ANALYSIS OF PERIOD NO.  1  LOAD GROUP NO.  5
MULTIPLE AXLES WITH RESPONSE POINT HALFWAY BETWEEN TWO AXLES


 POINT    VERTICAL   VERTICAL    VERTICAL     MAJOR       MINOR   INTERMEDIATE
               DISPL.             PRINCIPAL   PRINCIPAL   PRINCIPAL
  NO.    COORDINATE (HORIZONTAL   STRESS      STRESS      STRESS      STRESS
              P. STRAIN)  (STRAIN)    (STRAIN)    (STRAIN)    (STRAIN)

 1     16.50000    0.04338    1100.827    1100.827     604.383     604.381
       (STRAIN)   9.299E-06  -9.249E-04  -9.250E-04   9.597E-06   3.076E-07
 1     56.50010    0.04294    1024.867    1025.167     567.302     576.488
       (STRAIN)  -1.446E-05   6.532E-05   6.538E-05  -1.488E-05  -4.395E-07

 2     16.50000    0.04438    1152.016    1151.986     620.837     644.388
       (STRAIN)   9.797E-06  -9.680E-04  -9.681E-04  -3.811E-05   3.261E-07
 2     56.50010    0.04390    1069.977    1070.007     591.273     594.332
       (STRAIN)  -1.544E-05   6.866E-05   6.867E-05  -1.525E-05  -4.674E-07

 3     16.50000    0.06112    5359.977    5360.051    2977.029    2933.186
       (STRAIN)   5.573E-05  -4.529E-03  -4.528E-03   1.487E-04   1.893E-06
 3     56.50010    0.05842    3010.720    3010.764     844.128     860.068
       (STRAIN)  -1.006E-04   2.792E-04   2.792E-04  -1.006E-04  -3.156E-06

AT BOTTOM OF LAYER  1   DIFFERENTIAL TENSILE STRAIN =   0.000E+00
ALLOWABLE LOAD REPETITIONS =   1.000E+30  DAMAGE RATIO =   0.000E+00

AT TOP OF LAYER  4   DIFFERENTIAL COMPRESSIVE STRAIN =   2.106E-04
ALLOWABLE LOAD REPETITIONS =   3.939E+07  DAMAGE RATIO =   8.160E-04

DAMAGE ANALYSIS OF PERIOD NO.  1  LOAD GROUP NO.  6
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| POINT NO. | VERTICAL COORDINATE | VERTICAL DISPL. (HORIZONTAL P. STRAIN) | VERTICAL STRESS (STRAIN) | MAJOR PRINCIPAL STRESS (STRAIN) | MINOR PRINCIPAL STRESS (STRAIN) | INTERMEDIATE PRINCIPAL STRESS (STRAIN) |
|---|---|---|---|---|---|---|
| 1 | 16.50000 | 0.05865 | 4127.824 | 4128.662 | 2274.634 | 2274.635 |
| | (STRAIN) | 4.253E-05 | -3.484E-03 | -3.486E-03 | 4.345E-05 | 1.340E-06 |
| 1 | 56.50010 | 0.05661 | 2634.585 | 2638.808 | 872.752 | 879.731 |
| | (STRAIN) | -7.535E-05 | 2.218E-04 | 2.225E-04 | -7.557E-05 | -2.409E-06 |
| 2 | 16.50000 | 0.06332 | 5400.230 | 5400.231 | 2975.819 | 2975.832 |
| | (STRAIN) | 5.499E-05 | -4.560E-03 | -4.560E-03 | 5.570E-05 | 1.808E-06 |
| 2 | 56.50010 | 0.06067 | 3155.875 | 3155.910 | 923.413 | 938.619 |
| | (STRAIN) | -9.888E-05 | 2.780E-04 | 2.780E-04 | -9.883E-05 | -3.107E-06 |
| 3 | 16.50000 | 0.04669 | 1258.785 | 1258.803 | 693.400 | 688.075 |
| | (STRAIN) | 1.032E-05 | -1.057E-03 | -1.057E-03 | 2.346E-05 | 3.889E-07 |
| 3 | 56.50010 | 0.04620 | 1169.248 | 1169.230 | 645.190 | 650.828 |
| | (STRAIN) | -1.624E-05 | 7.224E-05 | 7.223E-05 | -1.622E-05 | -4.916E-07 |

AT BOTTOM OF LAYER 1 TENSILE STRAIN = 0.000E+00
ALLOWABLE LOAD REPETITIONS = 1.000E+30 DAMAGE RATIO = 0.000E+00

AT TOP OF LAYER 4 COMPRESSIVE STRAIN = 2.780E-04
ALLOWABLE LOAD REPETITIONS = 1.135E+07 DAMAGE RATIO = 2.073E-03

DAMAGE ANALYSIS OF PERIOD NO. 1 LOAD GROUP NO. 6
MULTIPLE AXLES WITH RESPONSE POINT HALFWAY BETWEEN TWO AXLES

| POINT NO. | VERTICAL COORDINATE | VERTICAL DISPL. (HORIZONTAL P. STRAIN) | VERTICAL STRESS (STRAIN) | MAJOR PRINCIPAL STRESS (STRAIN) | MINOR PRINCIPAL STRESS (STRAIN) | INTERMEDIATE PRINCIPAL STRESS (STRAIN) |
|---|---|---|---|---|---|---|
| 1 | 16.50000 | 0.04565 | 1202.874 | 1202.955 | 676.892 | 642.960 |
| | (STRAIN) | 9.794E-06 | -1.010E-03 | -1.010E-03 | 8.145E-05 | 3.693E-07 |
| 1 | 56.50010 | 0.04519 | 1119.950 | 1120.270 | 619.869 | 630.388 |
| | (STRAIN) | -1.521E-05 | 6.872E-05 | 6.878E-05 | -1.569E-05 | -4.621E-07 |
| 2 | 16.50000 | 0.04669 | 1258.785 | 1258.803 | 693.400 | 688.075 |
| | (STRAIN) | 1.032E-05 | -1.057E-03 | -1.057E-03 | 2.346E-05 | 3.889E-07 |
| 2 | 56.50010 | 0.04620 | 1169.248 | 1169.230 | 645.190 | 650.828 |
| | (STRAIN) | -1.624E-05 | 7.224E-05 | 7.223E-05 | -1.622E-05 | -4.916E-07 |

3 16.50000 0.06332 5400.255 5400.231 2975.819 2975.832
(STRAIN) 5.494E-05 -4.560E-03 -4.560E-03 5.570E-05 1.808E-06
3 56.50010 0.06067 3155.887 3155.898 923.142 938.906
(STRAIN) -9.888E-05 2.780E-04 2.780E-04 -9.887E-05 -3.107E-06

AT BOTTOM OF LAYER 1 DIFFERENTIAL TENSILE STRAIN = 0.000E+00
ALLOWABLE LOAD REPETITIONS = 1.000E+30 DAMAGE RATIO = 0.000E+00

AT TOP OF LAYER 4 DIFFERENTIAL COMPRESSIVE STRAIN = 2.058E-04
ALLOWABLE LOAD REPETITIONS = 4.366E+07 DAMAGE RATIO = 5.389E-04

DAMAGE ANALYSIS OF PERIOD NO. 1 LOAD GROUP NO. 7

POINT VERTICAL VERTICAL VERTICAL MAJOR MINOR INTERMEDIATE
DISPL. PRINCIPAL PRINCIPAL PRINCIPAL
NO. COORDINATE (HORIZONTAL STRESS STRESS STRESS STRESS
P. STRAIN) (STRAIN) (STRAIN) (STRAIN) (STRAIN)

1 16.50000 0.05887 3912.365 3912.969 2157.891 2157.892
(STRAIN) 4.185E-05 -3.306E-03 -3.307E-03 4.263E-05 1.332E-06
1 56.50010 0.05686 2356.477 2359.747 756.509 756.632
(STRAIN) -7.392E-05 2.149E-04 2.155E-04 -7.358E-05 -2.365E-06

2 16.50000 0.06389 5351.837 5352.009 2951.172 2951.173
(STRAIN) 5.600E-05 -4.524E-03 -4.523E-03 5.658E-05 1.848E-06
2 56.50010 0.06115 2898.527 2898.538 792.470 810.701
(STRAIN) -1.011E-04 2.786E-04 2.786E-04 -1.012E-04 -3.165E-06

3 16.50000 0.04908 1111.929 1111.940 613.015 608.471
(STRAIN) 9.595E-06 -9.345E-04 -9.344E-04 2.111E-05 3.642E-07
3 56.50010 0.04862 1036.586 1036.571 581.887 587.804
(STRAIN) -1.461E-05 6.729E-05 6.729E-05 -1.470E-05 -4.422E-07

AT BOTTOM OF LAYER 1 TENSILE STRAIN = 0.000E+00
ALLOWABLE LOAD REPETITIONS = 1.000E+30 DAMAGE RATIO = 0.000E+00

AT TOP OF LAYER 4 COMPRESSIVE STRAIN = 2.786E-04
ALLOWABLE LOAD REPETITIONS = 1.125E+07 DAMAGE RATIO = 1.698E-03

DAMAGE ANALYSIS OF PERIOD NO. 1 LOAD GROUP NO. 7
MULTIPLE AXLES WITH RESPONSE POINT HALFWAY BETWEEN TWO AXLES

POINT VERTICAL VERTICAL VERTICAL MAJOR MINOR INTERMEDIATE

| NO. | DISPL. COORDINATE | (HORIZONTAL P. STRAIN) | PRINCIPAL STRESS (STRAIN) | PRINCIPAL STRESS (STRAIN) | PRINCIPAL STRESS (STRAIN) | STRESS (STRAIN) |
|---|---|---|---|---|---|---|
| 1 | 16.50000 | 0.04813 | 1064.853 | 1064.827 | 564.505 | 605.092 |
| | (STRAIN) | 9.110E-06 | -8.948E-04 | -8.950E-04 | -7.369E-05 | 3.017E-07 |
| 1 | 56.50010 | 0.04770 | 995.097 | 995.414 | 563.109 | 567.880 |
| | (STRAIN) | -1.369E-05 | 6.414E-05 | 6.419E-05 | -1.376E-05 | -4.157E-07 |
| 2 | 16.50000 | 0.04908 | 1111.929 | 1111.940 | 613.015 | 608.471 |
| | (STRAIN) | 9.595E-06 | -9.345E-04 | -9.344E-04 | 2.111E-05 | 3.642E-07 |
| 2 | 56.50010 | 0.04862 | 1036.586 | 1036.571 | 581.887 | 587.804 |
| | (STRAIN) | -1.461E-05 | 6.729E-05 | 6.729E-05 | -1.470E-05 | -4.422E-07 |
| 3 | 16.50000 | 0.06932 | 5455.004 | 5454.932 | 3007.684 | 3007.685 |
| | (STRAIN) | 5.673E-05 | -4.610E-03 | -4.611E-03 | 5.772E-05 | 1.871E-06 |
| 3 | 56.50010 | 0.06654 | 2999.108 | 2999.121 | 858.096 | 874.813 |
| | (STRAIN) | -1.019E-04 | 2.842E-04 | 2.842E-04 | -1.018E-04 | -3.191E-06 |

AT BOTTOM OF LAYER 1 DIFFERENTIAL TENSILE STRAIN = 0.000E+00
ALLOWABLE LOAD REPETITIONS = 1.000E+30 DAMAGE RATIO = 0.000E+00

AT TOP OF LAYER 4 DIFFERENTIAL COMPRESSIVE STRAIN = 2.113E-04
ALLOWABLE LOAD REPETITIONS = 3.879E+07 DAMAGE RATIO = 9.847E-04

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* SUMMARY OF DAMAGE ANALYSIS *
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AT BOTTOM OF LAYER 1 SUM OF DAMAGE RATIO = 0.000E+00
AT TOP OF LAYER 4 SUM OF DAMAGE RATIO = 6.454E-02

MAXIMUM DAMAGE RATO = 6.454E-02 DESIGN LIFE IN YEARS = 15.49

# APPENDIX K

## Sample of the applied unit rates for asphaltic concrete overlay

**ASPHALTIC OVERLAY (7.5M WIDTH) OF 1KM**

| ITEM CODE | DESCRIPTION | QUANTITY | UNIT | RATE | AMOUNT |
|---|---|---|---|---|---|
| | **BILL NO.1 - GENERAL ITEMS** | | | | |
| A110 | Performance Bond | Item | Sum | | 12,000.00 |
| A120 | Insurance of the works | Item | Sum | | 10,000.00 |
| A229 | Provide monthly progress photographs. | Item | Sum | | 200.00 |
| A250 | Testing of materials as specified in specification | Item | Sum | | 600.00 |
| A272 | Provide and maintain traffic safety signs and systems during construction | Item | Sum | | 400.00 |
| | **Provisional Sums** | | | | |
| A221 | Allow for provision Engineer's facilities | Item | **P. S.** | | **20,000.00** |
| A420.2 | Allow for reinstatement and relocation of services | Item | **P. S.** | | **1,000.00** |
| A420.3 | Percentage adjustment to A221 & A420.2 | 20 | % | | 4,200.00 |
| A420.4 | Provide First Aid Kit and train First Aider | | **P. S.** | | **300.00** |
| A420.5 | Provide assistance to facilitate site visits by MOH personnel to educate workers and local communities in STDS, HIV/AIDS awareness and consultation meetings, including MOH personnel honorarium | | **P. S.** | | **2,500.00** |
| A420.6 | Percentage adjustment for item A420.4 & A420.5 | 20 | % | | 560.00 |
| A420.7 | Bonus for full compliance with obligation in respect of item A420.5 to be awarded at the discretion of the Engineer | | **P. S.** | | **400.00** |
| | **BILL NO.1 - GENERAL ITEMS** **TO GENERAL SUMMARY** | | | | **52,160.00** |

**ASPHALTIC OVERLAY (7.5M WIDTH) OF 1KM**

| ITEM CODE | DESCRIPTION | QUANTITY | UNIT | RATE | AMOUNT |
|---|---|---|---|---|---|
| R115 | Provide,lay and compact granular sub-base material 300mm thick. | 2,250 | m3 | 17.50 | 39,375.00 |
| R900.1 | Extra over for haulage of granular sub-base materials. | 2,250 | $m^3$ km | 0.85 | 1,912.50 |
| R115 | Provide,lay and compact crush rock base material material 250mm thick. | 1,875 | m3 | 73.50 | 137,812.50 |
| R900.1 | Extra over for haulage of gravel base materials. | 1,875 | $m^3$ km | 0.85 | 1,593.75 |
| R350.1 | Prime surface of base course with MC 250 cutback at nominal spray rate of 1.0 litre/$m^2$ and blind with sand ( applied 1.21/$m^2$ ) | 7,500 | $m^2$ | 1.45 | 10,875.00 |
| R350.6 | Tack coat consisting of MC 250 cutback at nominal spray rate 0.15 litre/$m^2$ over area to receive asphaltic concrete (applied 0.34litre/$m^2$) | 7,500 | $m^2$ | 0.52 | 3,881.25 |
| R380 | Asphaltic concrete in a layer not exceeding 180mm compacted thickness (Provisional) | 1,350 | $m^3$ | 210.00 | 283,500.00 |
| R322.1 | Asphaltic concrete wearing course layer 180mm compacted thickness type 1 | 7,500 | $m^2$ | 9.25 | 69,375.00 |
| R329.1 | Adjustment of items R380 and R322.1 for increase or decrease in binder content by weight of total mix by 0.1% | 3,287 | t | 0.75 | 2,465.44 |
| R329.2 | Extra over items R380 and R322.1 for the addition of ordinary Portland Cement as filler to the asphaltic mix | 3,287 | t | 191.20 | 628,508.11 |
| | | **Total** | | | **1,179,298.55** |

| | | | | | |
|---|---|---|---|---|---|
| | **SUMMARY** | | | | |
| | Road Works | | | | 1,179,298.55 |
| | Sub- Total | | | | 1,179,298.55 |
| | **Add** General Items | | | | 52,160.00 |
| | Contingencies (20%) | | | | 246,291.71 |
| | **ESTIMATED UNIT COST** | | | | **¢1,477,750.26** |