CRACKING OF JOINTED PLAIN CONCRETE PAVEMENT: A CASE STUDY OF KAGERE- NDUNYU-MUNYANGE- GITUIGA (E571) ROAD IN OTHAYA, KENYA

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JOMO KENYATTA UNIVERSITY OF AGRICULTURE AND TECHNOLOGY

2017

Cracking of Jointed Plain Concrete Pavement: A Case Study of Kagere- Ndunyu- Munyange- Gituiga (E571) Road in Othaya, Kenya

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A thesis submitted in partial fulfillment for the degree of

Master of Science in Construction Engineering and Management

in the Jomo Kenyatta University of Agriculture and Technology

2017

DECLARATION

This thesis is my original work and has not been presented for a degree in any other university.

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This thesis has been submitted for examination with our approval as the University Supervisors.

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DEDICATION

This work is dedicated to my mother Elizabeth Wakonyu Mwitari who strained to give me an education and to my wife Mrs. Emily Wangari Ndoria who has taken care of our children and those of my late sister Ann Wandia with selflessness, dedication and utmost commitment to their welfare in all aspects of their life and for encouraging me in my studies. May God bless them and keep them.

ACKNOWLEDGEMENT

I express my sincere gratitude to Jomo Kenyatta University of Agriculture and Technology for according me an opportunity to pursue the Master of Science in Construction Engineering and Management at the university and for creating a good atmosphere for self-actualization in academics and other fields of human growth and achievement.

My sincere thanks further go to my supervisors Professor James Wambua Kaluli and Eng. Charles Kabubo for their guidance and supervision in data collection, documentation and analysis as well as in paper writing for publication and compilation of this thesis. Their commitment has enabled me to get this far.

I cannot forget my family for standing with me during the entire period of my study. Their support and encouragement has been invaluable.

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ABBREVIATIONS AND ACRONYMS

AASHO	American Association of State Highway Officials
AC	Asphaltic Concrete
B S	British Standard
CTE	Coefficient of Thermal Expansion
DCP	Dynamic Cone Penetrometer
FWD	Falling Weight Deflectometer
GDP	Gloss Domestic Product
GNP	Gloss National Product
HDS	Hinged Dowelled System
HDS	Hinged Dowel System
JPCP	Jointed Plain Concrete Pavement
KeRRA	Kenya Rural Roads Authority.
LCCA	life cycle cost analysis.
MoT&I	Ministry of Transport & Infrastructure.
PCC	Portland Cement Concrete
SL	Separating Layer
TZ	Temperature Gradient

ABSTRACT

This study considered a 2.05 km section of the JPCP component of the Kagere – Ndunyu -Munyange–Njigari-Gitugi (E571) road in Othaya, Kenya. The road developed surface cracks within the first 2 years of construction. The study was carried to determine possible causes of the said cracking. The strength of the sub-base and subgrade layers were assessed in terms of California Bearing Ratio (CBR) by using the dynamic cone penetrometer (DCP). The Falling Weight Deflectometer (FWD) was used to assess the stiffness of the pavement through pavement deflection measurements. Concrete core samples from un-cracked slab sections were used to measure concrete strength, determine slab thickness and establish the constituent material proportions of the concrete. Other cores were cut at the saw-cut joints, to enable measurement of saw-cut joint depth. With a CBR of 10-17%, the subgrade layer met the basic strength requirement and was ruled out as a possible cause of cracking. However, half of the tested subbase layer samples did not meet the minimum CBR of 60%, and 24% of the pavement did not attain the recommended minimum layer modulus value of 345 MPa for neat gravel construction. The average strength of the slabs was less than the design crushing strength of 30N/mm^2. Only 5% of the slabs met the design slab thickness of 185mm and none of the saw-cut joints met the recommended depth of at least 25% of slab thickness. Inadequate slab thickness, inadequate depth of cut joints, and inadequate strength of concrete slabs and subbase layer could have been responsible for the development of surface cracks. To avoid these shortcomings, there is need for deliberate efforts to enhance capacity in concrete pavement design and construction in Kenya.

CHAPTER ONE

INTRODUCTION

1.1 Background Information

This paper is as a result of investigations of cracks manifested on the surface of a recently constructed Jointed Plain Concrete Pavement (JPCP), Kagere-Munyange-Njigari-Gituiga (E571) Road in Othaya sub-county, Nyeri County of Kenya. The road was completed in the year 2012. The pavement as designed comprises 300mm thick subgrade constructed in two layers each 150mm thick compacted to 100% MDD-AASHTO T99, 150mm cement improved gravel sub- base layer on which 185mm concrete slabs are constructed.

The road under study is a public road and it is the first and the longest JPCP road section documented in Kenya. The other concrete road documented in Kenya is the Mesh Reinforced, Mbagathi Road in Nairobi (Adoyo & Mwea, 2012).

Concrete pavements have been used globally in highly trafficked highways, however, construction of JPCP for Kagere-Ndunyu-Munyange-Njigari-Gituiga (E571) Road which has low volume of traffic (Class T5) was on the basis of practicality of construction in view of steep vertical gradients which made it impractical to use conventional and heavily mechanized road construction methods applicable for bituminized pavements.

The surface cracks on E571 Road are mainly transverse across the pavement lanes and are located at varying distances from the lateral saw-cut joints.

Research conducted on cracking of JPCP has shown that cracks on JPCP roads could be as a result of weak concrete slabs support layers (base, subbase and or subgrades). Chen and Won (2007), in their research on cracks on a recently constructed JPCP road in

Dallas in America it was established that the cause of cracks was poor support of JPCP. Xu and cebon (2017) in their research on cracking in JPCP observed that severity of longitudinal and transverse cracking was sensitive to slab thickness and base type and further observed that Cracks occurred earlier and were more severe in dry zones. In China Yao and Weng (2012) observed that weak road edge restraint resulted to deformation of subgrades which caused voids below the concrete slabs resulting to surface cracking of concrete slabs. Late and shallow saw-cutting of joints has been established to be another cause of surface cracks on JPCP. To accurately establish the vulnerability of the saw-cut joints, full pavement investigations is undertaken in which the other likely causes are eliminated. Chen and Won (2007) in their investigations established that cracks in one of the roads being investigated were caused by late cutting of saw-cut joints which resulted to poor development of intended full depth cracks at the joints while others developed on the surface of the slabs. Inadequate saw-cut joints results to poor or non-development of cracks at the joints as intended and instead manifest surface cracks on the pavement concrete slabs.

Method and strategy of construction influences significantly the level of cracking of JPCP. A study in China by Yao and Weng (2012) on a JPCP rehabilitated road in which four rehabilitation strategies of (1) overlay on existing pavement (2) replacement of subgrade (3) rubblization and (4) break and seat were applied in various sections of a road observed that replacement of subgrade produced the most cracks on the JPCP. This was attributed to variations in quality of subgrade materials used, varying moisture content and construction method adopted. It was established that the edges were poorly restrained and compaction was not uniform. This led to non- uniform deformation of subgrade layer on application of load resulting to massive cracking of JPCP.

The study further established that the strategy of applying a separation layer of a soft asphalt between JPCP and the base layer was more effective in preventing cracking on JPCP than when a geotextile material was applied or when the concrete slabs were directly placed on the base.

Operational aspects particularly overloading have been established to result to cracking of JPCP with the most loaded lane having more cracks along the wheel path. Yao and Weng (2012) on a study in Quinglian Highway in China further established that where the road had a cross-fall, most cracks occurred on the outer wheel on the downside wheel path this is as a result of the gravity of load shifting in the direction of the slope thus concentrating more loading on the outer wheel.

Environmental factors constituting temperatures and moisture also contribute to cracks on JPCP. The internal variations of temperature and moisture result to temperature and moisture gradients across the thickness of concrete slab. Nassiri and Vandenbossche (2012) identified the top of slab in contact with the atmosphere and bottom of slab in touch with base layer as the two boundary conditions with varying temperature and moisture conditions that cause temperature and moisture gradients resulting to warping and curling of concrete slabs and hence induction of stresses on concrete slabs resulting to surface cracks. Xu and cebon (2017) observed that premature cracking in plain jointed concrete pavements could result from depressurization of the soil caused by slab curling thus resulting to plastic deformation of unrestrained slab supporting layer on application of load. This deformation results to voiding in the foundation resulting to weak support of the slabs and eventual surface cracking. This mechanism, they observed was most prevalent in dry regions where sandy and other weakly cohesive soils were used as foundation soils for the concrete slabs.

Inherent material qualities which have significant behavior change on exposure to environmental variables have been established to cause cracks on JPCP. Chen and Won (2007) in a study on a bridge approach slabs observed that an approach slab that had surface cracks manifestation had been constructed using siliceous aggregates which has higher coefficient of thermal expansion than limestone aggregates which had been used on the other approach slab which had no cracks.

The type of joint dowels used at the construction joints which had restrained movement in one way or the other were noted to contribute to cracks at the joint due to load and environmental stresses. Zeinali et al. (2013) observed that the problems which are associated with conventional dowelled joints could be significantly reduced by application of a Hinged Dowelled Joint System.

Other causes identified to contribute to surface cracking on JPCP included uncontrolled shrinkage, spacing of the saw-cut joints and underlying pavement conditions for rehabilitated pavements.

Concrete pavements are expensive investment projects and it is imperative that sustainability of such projects are ensured in the entire project cycle. Cracking of JPCP is an engineering problem that requires to be understood and requisite action taken in design, construction and maintenance aspects of these pavements. The purpose of this study was to interrogate and diagnose the causes of cracks on E571 Road and lay a platform on which to build on further knowledge on JPCP including coming up with ways of resolving problems that influence cracking of Plain Jointed Concrete Pavements.

1.2 Statement of the Problem

Concrete pavement construction is an expensive developmental venture that requires clear and sober decisions to be made to justify the significantly high initial investment. Although the research road has low design traffic volume (0.25- 1million cumulative standard axles) and this type of construction is normally considered for higher traffic situations, the road started cracking within two years. The early cracking of the concrete pavement therefore raises pertinent concerns with regard to design and construction standards applied and raises the question of its residual strength and its sustainability over its expected service life. Cracking of concrete pavement is a manifestation of distress which can affect its structural integrity, performance and life cycle costs. It presents an engineering problem that requires to be examined, studied and understood. The purpose of this study was to diagnose and establish the causes of such cracks on a recently constructed concrete pavement, the "Kagere –Ndunyu -Munyange–Njigari-Gitugi (E571) Road" with an intention of building a knowledge base that would be available for reference in design of future JPCP.

1.3 Justification

In Kenya, few public roads pavements are of rigid concrete construction. The first of such roads is the mesh reinforced dual carriageway Mbagathi Road in Nairobi which is approximately 4Kms in length. Despite complaints of noise levels produced by traffic, the pavement has performed exemplary well in traffic conveyance since it was constructed over 6 years ago and it is expected to serve its full design life, however, distress signs and cracks have already started showing (Adoyo & Mwea, 2012). The other road worth of consideration is now the 9km road length constructed under Mt. Kenya Phase II Roads Project in Othaya subcounty of Nyeri County. This road comprise the road under study "Kagere –Ndunyu -Munyange–Njigari-Gitugi (E571) Road" which has 6km of Jointed Plain Concrete Pavement (JPCP). The other 3kms are of bituminous surface construction. Part of this road has developed many transverse cracks. Though longitudinal cracks are also witnessed they are few and were considered insignificant in the study and were thus ignored. The cracks under study developed a short while after construction of the JPCP. The experience of plain jointed concrete pavements in Kenya is quite limited and therefore there is limited local experience on performance of concrete pavements, emanating distresses and the consequential effects associated with these distresses. It is imperative that the design and construction of concrete pavements undertaken in Kenya are engineered to meet the highest operational and performance standards suitable for local conditions and at affordable costs in order to spar transportation and economic development in the country and the neighboring region. It is on this premise that this study is based and it is hoped it will contribute to the knowledge base on jointed concrete pavement and spur further research in this area and in addition promote acceptance and favorable consideration for use by Engineers, Contractors, Clients, Investors and other stakeholders in highways and transportation industry.

1.4 Objectives

1.4 1 General Objective

The general objective of the study was to establish the nature, causes and extent of cracks and understand the mechanisms responsible for the development of cracks in Plain Jointed Concrete pavement.

1.4.2 Specific Objectives

1. To establish the extent of cracking in the Kagere- Ndunyu- Munyange- Gituiga (E571) Road in Othaya, Kenya.
2. Evaluate the influence of saw-cut joint spacing and quality of concrete pavement on crack development.
3. Assess the strength and stiffness of the experimental road and evaluate their likely contribution to crack development.
4. To establish how topography contributes to development of cracks in concrete paved roads.

1.5 Research Questions

1. To what extent have the cracks invested in the pavement structure?
2. Could the cracks have been as a result of inadequate spacing of joints or by inadequate cut joint depth?
3. Could the concrete structural capacity be inadequate as to be unable to accommodate internal and external stress development thus causing cracks on concrete pavement?

4. Could the pavement layers have been inadequate as to compromise the rigidity of the concrete pavement slab resulting to cracks?
5. What are the likely effects of the cracks to the sustainability of the concrete pavement?
6. What were the extents of vertical alignment gradients and cracks in the research section?

1.6 Scope

This study covers the following:

- Identification, marking and mapping of the area of research.
- Field investigations on:
  - ➢ Extent of cracks by measurements length and width of discernible cracks
  - ➢ Adequacy of Saw-cut Joints
  - ➢ Strength of subbase and subgrade layers.
  - ➢ Stiffness of the entire pavement through deflection measurements
  - ➢ Thickness and status of concrete pavement through concrete core cutting.
- Laboratory tests to establish:
  - ➢ Quality of subbase material.
  - ➢ Strength and mix proportions of concrete making the concrete pavement slabs.
- Analysis of data collected from the field and laboratory tests results to establish the requisite parameters and co-relationships.
- Establishing the core factors likely to have influenced the development of the surface cracks on the concrete pavement slabs.

1.7 Limitations

Although this research was carefully conducted using currently available equipment and methods, there are clear and undisputed limitations. First because of the high cost of equipment hire and laboratory tests, the research road section was limited to a length of only 2.05km of concrete pavement and 2 days of equipment time. The limited section might not be fully representative of the entire length of the JPCP. The results obtained are therefore generalized to represent the entire length of concrete pavement. Secondly, the destructive nature of concrete core cutting to the pavement structure limited the number of concrete core samples that could be cut from the slabs for concrete slab depth measurements, saw-cut joints depth determination and the study of development of full depth cracks at the joints, concrete strength and mix proportion determination. This equally limited the extent of subbase and subgrade strength determination since access to these layers was through the openings made after removal of concrete cores. Thirdly, the non-exposure of a large surface from which subbase samples could be extracted limited the size of subbase material samples obtained and hence prevented the conduct of full spectrum of soil quality tests. A full depth test of an open subbase surface would have been preferred. Fourthly, the research was conducted at a high altitude area, which experiences variable temperatures and receives moderately high rainfall. The insitu soils are compressible and well drained. The research findings are therefore not representative of global weather and soil variations in Kenya and the region. Finally the research section selection was subjective to visually observed surface defects and sections which could have concealed defects worth of research could have been left out. It is however considered that the research section was adequate for purposes of this study.

CHAPTER TWO

LITERATURE REVIEW

The first recorded road system in the world was constructed around 600 BC in Tunisia in North Africa (Pavement Interactive, 2008). The practice of road construction was then taken up by Romans who constructed narrow and short length of pavements for use by soldiers on foot. The alignment followed the ground terrain and construction was labor intensive. Macadam roads became a reality in the year 1815 and were followed by bituminous roads in the form of tar macadam in the year 1848 and thereafter asphalt road surfacing in 1858. Although Portland cement was under production in UK as early as in 1824, it was not until 1894 that concrete was recorded to have been successfully used to construct long-lasting Portland Cement Concrete (PCC) pavement at Bellefontaine, Ohio in America (Pasko, 1998). The early use of concrete in pavement construction was hindered by lack of adequate knowledge on design, strength and jointing. With more understanding on concrete pavement design, construction and jointing and with legislative support, concrete pavement construction proliferated in America and worldwide and to date it is a major road construction material. The few kilometers of concrete paved roads in Kenya therefore make a minor fraction of the total length of concrete pavements constructed all over the world. This study therefore creates a good opportunity to understand the challenges being experience locally with concrete pavement roads and further understand the intricacies and requirements that need to be considered for the development of sustainable concrete pavement roads in Kenya.

2.1 Pavement Design

Pavement design is the process of developing economical combination of pavement layers with regard to the traffic load during a predetermined design period and taking into account the quality of the alignment soils, type of materials available for construction of pavement layers, the climatic cycles and other environmental factors (Werkmeister et al., 2004).

With time, pavement under use gradually losses its serviceability levels which is largely influenced by traffic and the environment. These factors should be taken into account during design process.

Evaluation of life cycle economic costs of pavements is used to compare the alternate pavement designs for a given design type and it is a major tool in the selection of the type of pavement and the construction approach (Babashamsi et al., 2016).

2.1.1 Traffic

Study of traffic types and volumes is conducted within the extents of influence of a proposed project. Categorization of traffic and computation of traffic loading based on the Equivalent Standard Axles is then undertaken. A standard equivalent axle load of 80KN has been adapted in Kenya based on a concept developed following the AASHTO Road Test carried out in the USA in the late 1950s. The standard axle is generally used for empirical method of pavement design. It is, however, noted that new methods of pavement design using mechanistic–empirical approach are evolving from the fact that today’s pavement design data exceed the data used in AASHTO road test (Ahmed & Erlingsson, 2015). In mechanistic models typical traffic data required for design include base year truck traffic volume, traffic volume adjustment factors, axle load distribution factors and general traffic inputs. The general traffic input include number of axles per truck and axle load configuration, tire pressure and wheel base (El-Badawy et al., 2012). Tire characteristics and inflation pressure are used to compute tire – pavement responses (El-Kholy & Galal, 2012)

Traffic is categorized as cars, light good vehicles, buses, medium good vehicles, heavy good vehicles and commercial vehicles (Ministry of transport and communication, Republic of Kenya, 1987). This classification is dependent on vehicle types, sizes and load carrying capacity of the vehicle. The damaging effect of a vehicle to pavement is measured in terms of the number of equivalent axle load each vehicle contributes. Cars and light good vehicles are considered inconsequential in pavement damage and they are not usually considered in the computation of cumulative axle loads for the design of a pavement structure.

For one directional multi-lane pavement, it is necessary to determine the distribution patterns of heavy traffic on the lanes in order to determine the design lane traffic which is the most heavily loaded lane. This process requires first, the estimation of heavy traffic from total traffic volume, followed by identification of lane traffic distribution pattern of heavy traffic and then determination of heavy vehicle type and axle load distribution of lane traffic volume (Fwa & Li, 1995). In the study of truck traffic distribution for pavement design Fwa and Li (1995) established that the heavy traffic distribution on the lanes was influenced by the number of traffic lanes in one direction, the functional class of the road, the hourly distribution of total volume of traffic and the hourly volume of heavy traffic.

The design axle load is projected to a set design period using growth factors based on traffic growth data obtained through vehicle census done from time to time as well as economic growth indicators including GNP or GDP, vehicle registration trend, fuel consumption or through growth projections in national and regional economic plans.

To compute cumulative axle load, daily axle load is obtained through aggregating the equivalent axle loads from daily traffic. The equivalent axle load is obtained for each vehicle axle considered by application of “Liddles” equation (2.1):

$$E_f = (L_s/80)^{4.5} \tag{2 .1}$$

Where: $\mathbf{E_f}$ is the equivalent factor for a single axle load less than 130 KN and

$\mathbf{L_s}$ is the load in KN on the single axle considered.

Vehicle axle loads are obtained by axle load surveys using manual mobile weigh bridges or weigh in motion (WIM) instruments. Further information can be obtained from weighbridge stations established in the region of interest. Overtime approximate equivalent load factors could be established for specific routes using historical data for specific types of vehicles.

The design period for pavement is based on the certainty to project traffic within very uncertain economic growth factors and the cost consideration of the selected pavement. Consideration for strengthening as circumstances becomes clearer also influences the design period considerations.

The cumulative axle load (T) in the projected design period (N) in years is computed using the equation (2.2):

$$T=365t_1 \{(1+i)^N-1\}/I \qquad (2.2)$$

$\mathbf{t_1}$ is the cumulative daily standard axles in the first year of design and **i** is the projected growth.

The computed cumulative axle load is then classified in terms of traffic class for application in the pavement design. According to Ministry of transport and communication, Republic of Kenya , five classes of traffic are specified (**Table 2.1**).

Table 2.1: Traffic classes in Kenya

Class	**Cumulative no of standard axle load**
T1	25- 60 million
T2	10- 25 million
T3	3- 10 million
T4	1- 3 million
T5	0.25 – 1 million

For new roads where no prior data exists, traffic inputs are considered from site specific data (traffic counts, classification and axle load measurements), sites which are likely to exhibit similar traffic characteristics and bolstered by analyst knowledge and expertise to create applicable data for pavement design. Geographic, agricultural, industrial and commercial patterns are considered in projecting truck loading patterns and growth over a specific design period (Li et-al., 2015). Consequently, Historical traffic data from existing roads can be used to model an axle road spectra for application in design of road pavements within specific geographic, economic and operational zones without going into the expensive, labor intensive, time consuming and complicated traffic survey and analysis exercise prior to design.

There has been shift from application of empirical design concepts of pavement design developed in the 1950s based on a standards equivalent axle load of 80KN developed by AASHTO to mechanistic– empirical approach on consideration of evolving changes in pavement loading configuration. In Kenya, however, empirical design approaches are still being applied in the design of new roads despite changes in vehicle characteristics and hence changes in local pavement loading regimes. There is therefore need to review our design approaches to accommodate the current pavement loading realities.

2.1.2 Subgrade soils and pavement materials

Design of road pavement requires a detailed study of the alignment and local soils which provide information on the quality of the soils and their suitability in road construction. The quality and strength profiling through testing of local and imported soils enables utilization of the material in subgrade, subbase and base construction.

To cut down on road pavement construction cost it is imperative that the naturally existing material be used to the maximum extent possible. The quality of subgrade soils has a huge impact on the thickness, quality treatment and cost of pavement layers of subbase, base and surfacing to be applied.

The classification of subgrade materials is done by establishing the inherent soil properties through grading tests and laboratory determination of atterberg limits which include shrinkage limit, plasticity limit and liquid limit. It is from these parameters that plastic and liquid indices are computed. Soils are further classified on the basis of strength values which include resilient modulus, CBR and UCS values and compaction. On new road alignment, other than trial holes excavation to profile and extract samples for testing, classification of road subgrade soils can be done simply by determination of deformation modulus through static plate test (SPT) and or by use of falling weight deflectometer (FWD). Vertical soil variations and thicknesses can be established through dynamic cone penetrometer (DCP) measurements. Grading of soils and bulk density values particularly in relation to fine particle contents of the soil can be used to classify the subgrade soils into their applicable bearing capacity categories (Zednick et al., 2015).

Volumetric changes of clays and expansive alignment soils with seasonal moisture variations is a critical consideration in pavement design and construction as it dictates the type of treatment that would be required to stabilize these soils against excessive deflections and differential movements which would otherwise cause early cracking and failure of pavement structure. To alter the physicochemical behavior of expansive clays various methods and approaches have been used including but not limited to mechanical stabilization, chemical treatment with lime, cement and fly ash and application of enzymes and polymers with various levels of success. To be able to economically utilize the right type of stabilizer it is critical to establish fundamental protocols of application. This is because some stabilizers are more active than others and time limitations apply and where combination of stabilizers is required the order of application might dictate the effectiveness of the process and the quality of the end result. The application of correct protocols is important in comparing candidate stabilizers in their effectiveness, identification of their shortcomings and deficiencies in soils treatment (Petry & Little, 2002).

In Kenya, pavement subgrade soils are classified on the criterion of strength based on California Bearing Ratio (CBR) values (**Table 2.2**). Apart from strength values, other parameters that determine suitability of subgrade material include quantity of organic matter which is normally limited to a maximum 5% by weight, swell limited to a maximum of 3% by volume, plasticity index, limited to a maximum of 50%, and moisture content of not more than 105% of optimum moisture content (Ministry of transport and communication, Republic of Kenya, 1987).

Table 2.2: Classification of subgrade soils in Kenya

Class	CBR Range (%)	Median CBR (%)
S1	2 - 5	3.5
S2	5 - 10	7.5
S3	7 - 13	10.5
S4	10 - 18	14
S5	15 - 30	22.5
S6	>30	

Subgrade quality impacts on its ability to carry traffic load transferred to it by the pavement layers. For weaker subgrade soils the subbase and base layers are made thicker than that of a stronger subgrade layer. To mitigate on the pavement layer thickness the subgrade layer may be improved either by importing better material, mechanical stabilization of insitu material with high quality imported material or by application of mineral filler like cement and lime whichever is considered cost effective. Cement and lime modify the soil properties through calcium exchange and pozolanic reactions to effect stabilization. Non-traditional stabilizers like sulfonated oils, enzymes and polymers can also be used to stabilize and strengthen the soils though their stabilization mechanisms are different (Petry & Little, 2002).

The specified minimum CBR values for neat natural material for subbase and base layers for Kenyan roads are 30% and 60% respectively (Ministry of transport and communication, Republic of Kenya, 1987). Subgrade class S6 (**Table 2.2**) is of subbase layer quality and if a section of a road is in a cut, no subbase layer would be required if the soils encountered are of S6 class. It is, however, rare to find Subgrade material of S6 class quality and often the subbase and base materials are sourced from material sites which in many instances are outside the road alignment or available in pockets along the road alignment. To meet the quality specifications granular subbase and base materials are improved with lime or cement to meet the strength range of 60% and 160% respectively specified for improved layers.

Care should be taken in the design of the amount of cement required for cement improved layers as excess application may lead to highly stiff layers that may be susceptible to cracking in a flexible pavement design. Reflective cracks from cracked base layer would evidently form on the riding surface of the road. Application of excessive moisture on the other hand would lead to excessive shrinkage resulting to cracking of treated layers. Moisture content should be controlled so as not to exceed the optimum moisture content (Chen et al., 2011).

Depending on the axle load considerations and availability of construction materials, graded crushed stones (GCS) bound together with at least 2% of cement is an alternative material for base construction for traffic class T2 (**Table 2.1**) and below. Grading, strength of stone, specific gravity, soundness, abrasion resistance, durability and compaction are key quality considerations in design of GCS base layer. Where unbound granular materials or GCS is used in the construction of pavement layers, only small permanent deformation would be considered in the design. Hence, application of load in design should be to that maximum value that ensures that the resilient response of the material is maintained for the requisite unbound layers during the design life of the pavement. It should, however, be noted that pavement materials are not purely elastic and always exhibit some permanent and resilient deformation under load (Werkmeister et al., 2004).

Construction time for improved layers is a critical consideration due to chemical reaction of the mixed materials with a specified maximum time of 2 hours to finish compaction after mixing. The finished layer should be protected against evaporation by covering within two hours of construction. Where evaporation is not rapid, watering of surface at intervals is undertaken. Application of bitumen emulsion seal coat or cut back bitumen (MC30) can be used for curing the base layer.

Bituminous mixes including sand bitumen mixes, dense bitumen macadam and dense emulsion macadam are used for base construction based on material availability limitations and traffic considerations.

In bituminous mixes for road construction the quality of the product is determined by the material quality, the method of manufacture and the laying process. In the design of the bituminous mixes, the resilient modulus is a fundamental property as it enables the estimation of their structural behavior in terms of stresses and deformation. The resilient modulus is influenced by the size and shape of the aggregates, the penetration of bitumen (type of bitumen) and the manufacture and application temperatures. The resilient modulus is higher for crushed aggregates than for rounded ones. The rounded aggregates have low internal friction thus causing reduction of voids and reduced stiffness of the mix. Bigger sized aggregates produce stiffer mixes than smaller sized aggregates due to reduced amount of voids in bigger aggregate mixes. The low penetration bitumen produce mixes with high resilient modulus and hence low deformation than high penetration bitumen. Bituminous mixes manufactured at a higher temperature and similarly applied have higher resilient modulus than correspondingly low temperature manufactured and applied bitumen mixes. With high temperatures in the rage of 160^{o}, a higher level of compaction and a stiffer mix is achieved than with lower temperatures (DelRio-Plat et al., 2011).

Study conducted to establish compaction quality of bituminous mixes in Olkohama in America established that compaction of asphalt varies randomly along the length and the breadth of pavements. This variation is dictated by the continuous reorientation of aggregates during compaction and the randomness of aggregates shape and texture. It was observed that density of asphalt mix does not increase linearly with the number of applied roller passes though a higher compactive effort is required to achieve a more uniform compaction. It was further established that thin layers below 50mm are likely to suffer over compaction which may result to lower density while thicker layers of 75mm are likely to suffer under compaction with similar low density results. Other variables that were noted to affect voids in mix thus affecting compaction included type of mix, binder type and weather conditions (Beainy et al., 2014).

Wu et al. (2014), in his paper on influence of aggregates on ageing of bitumen observed that the types of aggregates used in bituminous mixes could have a positive or a negative effect on ageing and hence the effective life of asphalt mixes. It was noted that mineral components on the surface of aggregates can act as catalysts on bitumen oxidation and hence ageing. Porous aggregates can on the other hand have high adsorption of binder fractions disrupting fractional proportional balance within the bitumen which may lead to a less well dispersed binder and thereby promoting the rate of bitumen ageing. On the positive side the mineral aggregate surface may adsorb some of the polar functional groups within the binder that contribute to the formation of viscosity build-up in bitumen. This will delay the increase of binder viscosity and therefore reduce the rate of ageing. Excessive voids in bituminous mixes result to higher oxygen content in the mix which results to oxidation and influencing a faster ageing of the mixes.

Lean concrete layer of at least 150mm thick and cement content of 3% to 6% is used in pavement construction as a rigid base layer. Lean concrete base is justifiable for heavy traffic only. Concrete pavements are made of a mixture of portland cement, naturally occurring or crushed aggregates, additives if required and water in proportions that are determined through a mix design to achieve the target strength. Ordinary Portland Cement (OPC) and High Strength Ordinary Portland Cement (HSOP) are the most widely used for concrete pavements. Rapid Hardening Portland Cement (RHPC) is used where time is a critical factor and the road pavement is required to be opened to traffic at an earlier date than would be possible if Ordinary Portland Cement or High Strength Ordinary Portland Cement was used. Two types of aggregates the fine and coarse aggregate are usually used. Maximum size of aggregate should not exceed 1/4 of the pavement slab thickness. Water used in mixing or curing of concrete should be clean and free from injurious amounts of oil, salt, acid, vegetable matter or other substances harmful to the finished concrete. Potable water is generally specified for use in concrete works. Equipment and tools used for concrete pavement construction include concrete pavers which lay, shape, level, compact and texture the concrete slabs. Otherwise equipment and tools for conventional construction for the different phases of concrete road construction include, concrete mixers, concrete delivery trucks, water bowsers, vibratory roller, poker vibrator and hand rammers for compaction purpose. Shovels and spades, hand brooms for texturing and joint cutting machine. Steel cutting machine would be required for cutting of steel for longitudinal joint tie bars and joint dowels where designated.

New methods of alignment and subgrade soil classification are now being used for soil classification and strength determination. The approaches include the use of static plate tests and Falling Weight Deflectometer equipment. For vertical soil profiling Dynamic Cone Penetrometer is being used while for classification to applicable bearing capacity categories, grading and bulk density values are being used. These methods offer speed and convenience resulting to reduced project design delivery times. Locally the traditional method of trial hole excavation for profiling and sampling for quality test is still prescribed as a the standard method despite its attendant delays. There should be a paradigm shift at the local level in accepting new technological approaches in soils and material investigation and quality assessment in order to be competitive in terms of delivery of projects.

2.1.3 Pavement types

Pavement design is the process of establishing the most economical combination of material type and layer thickness that constitute the pavement taking into account the properties of soil foundation and traffic to be carried during the service life of the road (Werkmeister et al., 2004). In order to satisfy pavement design requirements various types of pavements are considered. Highway pavement type could be flexible or rigid. Flexible pavement comprise bituminous layer/s as surfacing or binding layers in form of asphaltic Concrete or bituminous surface dressing. Other layers include, improved or neat material for base and or subbase. Rigid pavement comprise a rigid slab made of Portland Cement Concrete (PCC) as riding surface and include Jointed Plain Concrete Pavement (JPCP), Jointed Reinforced Concrete Pavement (JRCP) Continuously Reinforced Concrete Pavement (CRCP), Prestressed Concrete Pavement(PCP), Roller Compacted Concrete(RCC) and porous or pervious concrete pavements (Delatte, 2008). The concrete pavements may seat on a base layer or on subbase constructed similarly to that of flexible pavement.

Material layers for both flexible and rigid pavements are usually arranged in order of descending load bearing capacity with the highest load bearing capacity material and generally most expensive at the top and the lowest load bearing capacity material and least expensive at the bottom. Thus, the further down in the pavement structure a particular layer is, the less load it carries. The strength of subgrade layer primarily influences the thickness of other pavement layers.

Bituminous mixes in flexible pavements are classified dependent on the temperature at which it is applied, asphalt is categorized as hot mix asphalt (HMA), warm mix asphalt (WMA), or cold mix asphalt (CMA). Flexible pavement design is based on the load distributing characteristics of a layered system that transmits load to the subgrade through a combination of layers. The successive lower layer disperses the load to a winder bearing area than the layer above. The initial construction cost of a flexible pavement is lower than that of rigid pavement, however, this is inversely true for maintenance cost.

During construction of flexible pavement and before applying surfacing on the non-bitumen base layer, the surface is water proofed with a bituminous priming coat made of cut back bitumen and applied on the surface at a rate determined by texture and density of the material of the layer to be primed. The application could range between 0.8 to 1.2 liters per square meter. MC30 and MC70 are the commonly applied cutbacks. A tack coat is then applied to bind the primed surface with the consequent bituminous layers of surface dressing chippings or asphaltic concrete layer. In surface dressing the quantity of the binder and spread of chippings is determined by measurement of average least dimension of chippings and the number of spread layers. Cut back bitumen (MC 3000), bituminous emulsions (K160, K170) and straight run bitumen (80/100, 60/70) are used as binders (Ministry of transport and communication, Republic of Kenya, 1987). Asphalt concrete pavements are easier to construct and undertake maintenance operations than concrete pavements due to flexible nature of construction material.

Asphalt layer thickness is a function of applied axle loads. Thicker asphaltic concrete binder and surfacing layers are designed to accommodate greater axle loads. Asphaltic layers are designed to resist fatigue cracking, plastic deformation and loss of surfacing aggregates. Cracking could be as a result of fatigue from applied load and or environmental factors. Plastic deformation results from reduction of voids in mix to critical levels mainly due to secondary compaction which occurs under heavy traffic loading. Loss of surface aggregates could be due to poor particle size distribution, segregation of particles, inadequate compaction, low bitumen content, ageing of bitumen resulting to loss of adhesion and stripping due to ingress of water. Asphaltic mix design is usually based on material quality, appropriate particle size distribution, stability and strength of compacted mix and achievement of voids in mix on secondary compaction to a critical lower level of 3% to limit plastic deformation (Department for International Development: Overseas road note 19., 2002)

Performance level of flexible pavement can be predicted by fatigue life evaluation on the bituminous mix through measurement of tensile strain, stiffness modulus, fatigue resistance and bituminous layer thickness. Relevant models are applicable in the computation of critical strain which influences bottom up cracking of bitumen mix layer (Said et al., 2011). The design of flexible pavement layers is generally through the empirical approach. However, the bituminous layer design can be undertaken through the traditional fatigue failure criterion or an interactive approach which takes account of actual damage accumulation within the material. This is termed as the mechanistic-empirical approach (Oliveira et al., 2008)

Rigid pavement are mainly made of Portland cement concrete (PCC). Cobble stones, masonry blocks and paving blocks also form pavement system with a certain degree of rigidity. Rigid Concrete pavements resist traffic loads through flexure of the concrete. PCC rigid pavements contract due to drying shrinkage of the concrete, and expand and contract due to thermal effects. These movements must be considered in the design of the concrete pavements and taken care of during construction through jointing and or application of reinforcing steel to prevent the otherwise resultant cracking.

In Jointed Plain Concrete Pavement (JPCP) which are the most common type of rigid pavement, transverse joints are placed 3.6 - 6m apart while in Jointed Reinforced Concrete Pavement (JRCP) joints are generally spaced 7.5 – 9 m apart. Key performance issues of JPCP include; initial pavement smoothness which is a function of construction practices, adequate pavement thickness to prevent mid-slab cracking, limiting the joint spacing to also prevent mid-slab cracking, adequate joint design, detailing, and construction. In JPCP, expansion and contraction movements are addressed through provision of closely cut joints. The load is transferred through the joint by aggregate interlock and dowels.

In JRCP steel reinforcement in the rage of 0.1 – 0.25% of concrete slab cross-sectional area (temperature steel) is placed at the neutral axis of the slab purely to keep cracks together and not to resist flexural stresses. Joints in JRPC are generally dowelled to allow for load transfer and environmental movements (Delatte, 2008).

Continuously reinforced concrete pavement (CRCP), has no joints and has a heavy application of steel reinforcement in the range of 0.4 – 0.8% by volume of concrete. The cracks that develop are held together by the steel reinforcement and provide aggregate interlock and shear transfer of load to lower layers of pavement. In design of CRCP, key performance considerations include: initial pavement smoothness, adequate pavement thickness to prevent excessive transverse cracking and adequate reinforcing steel to hold cracks together and prevent punchouts which is distinct with CRCP. Continuously Reinforced Concrete Pavement have higher performance level and a longer life than the JRCP and JPCP but it is a more expensive pavement (Delatte, 2008).

Prestressed and precast concrete pavement like all conventional concrete pavements rely on the flexural strength of the concrete to resist traffic loads over time. Prestressing tendons in concrete induce a net compressive force and the traffic loads must overcome the compressive stress before inducing a net tensile stress and flexural fatigue into the pavement. Prestressed concrete pavements have considerably reduced thickness and have longer slabs and good performance levels. Prestressed concrete pavements have been used for original construction, overlays or full depth patchwork.

Precast concrete sections may be left in place as a permanent pavement, or may be a temporary remedy to allow flow of traffic until a permanent full depth patch is placed. Potential advantages of precast concrete panels include higher quality concrete, better curing control, less risk of weather disruption, and reduced delay before opening to traffic. Key issues to consider in precast slabs application include leveling the panels to avoid bumps at panel edges, and load transfer between precast panels or between precast panels and existing pavement. Precast panels are generally reinforced with mild steel, primarily to prevent damage during transportation and handling.

Roller Compacted Concrete (RCC) is a dry mixture often produced in a mixing plant. It is a low to no slump mixture that is closer in some respects to cement treated aggregate base than a conventional, flowing concrete. The material is delivered by dump trucks, placed into an asphalt paver and laid. It is then rolled with steel wheel rollers and then cured. RCC shrinks less than conventional concrete and the pavement may be allowed to crack naturally, or joints may be cut. The joints or cracks are further apart than those for JPCP.

The performance and life of jointed concrete pavements is largely dependent on the way joints perform. Distresses that may result from joint failure include faulting, pumping, spalling, corner breaks, blowups, and mid-panel cracking. Satisfactory joint performance is dependent on design for adequate load transfer, proper concrete consolidation during construction, appropriate pavement design standards, quality construction materials, and good construction and maintenance procedures.

Pervious Concrete Pavements (PCP) allow rapid water flow into and through the pavement structure and provides storm water management alternative in built environment. Pervious concrete has up to 30% voids and it is developed by limiting the amount of fines in aggregates. Pervious concrete has low compressive strength ranging between 10 N/mm^2 and 18 N/mm^2 and flexural strength of 1.5Mpa to 5.5 MPa. Pervious concrete is designed to have a permeability of up to 4m/s for passive and active drainage. Passive drainage is that storm generated from pervious surface while active drainage handles drainage from pervious surface and surrounding impervious areas (Herderson & Tighe, 2011). (PCP) are used for parking lots and light traffic streets and roads. An additional benefit of pervious concrete pavement is the reduction of noise (Delatte, 2008). The design of pervious concrete pavements should apart from permeability levels consider renewal of voids through appropriate methods of maintenance.

Construction of concrete pavement started over 120 years ago in America. The available literature on concrete pavement has no reference to any research conducted in any of the African countries. Despite the recorded advantages of concrete pavement over flexible pavements which include durability and lower life cycle costs, in Kenya all major pavements are constructed of bitumen. The little length of concrete roads so far developed in Kenya have attendant short comings. There is need therefore to promote the development of sustainable concrete pavements in Kenya and the region through research and building of capacity in order to reap the greater benefits associated with concrete pavements.

2.1.4 Environmental considerations in pavement design

Climatic factors influence design consideration of both flexible and rigid pavements in terms of selection of construction materials, design of drainage and anti-erosion systems in pavements and further influences the construction processes to be adopted. Internal and external drainage of pavement are critical consideration in wet areas and where the annual rainfall is expected to be above 500mm water should be kept out of the pavement. Where water table may rise, internal drainage methods should be applied. In dry areas and areas receiving less than 500mm of rainfall annually, open textured material for layer compaction may be used and dry compaction of layers can be considered as a construction method. However, appropriate measures against erosion and pavement drainage should be specified in case of occasional or unprecedented heavy rainfall (Ministry of transport and communication, Republic of Kenya, 1987).

In hot climates temperature variations across the pavement create temperature gradients resulting to thermal stresses in rigid pavements that require to be assessed and factored in the design. For flexible pavements asphalt is a viscoelastic material which tends to be plastic at low temperatures and viscous at high temperatures. As asphalt contracts, tensile stress develop due to friction between the base layer and contracting asphalt layer. Cracking of asphalt layer occurs if the internal stresses developed exceed the strength of asphalt concrete mixture. Conversely at high temperatures, asphalt concrete softens and develops permanent deformation on application of repeated loading (Rhamadhan et al., 1996).

Environmental factors of temperature and moisture which vary from time to time have contributory effects on cracking of concrete pavements. The top of concrete pavement slab is directly exposed to the atmosphere and experiences daily temperature fluctuations, evaporation of moisture and drying of the upper surface of the slab and it is constantly subject to ambient weather conditions while the bottom of concrete pavement slab in contact with the base is shielded against ambient weather conditions and therefore experiences little temperature and moisture fluctuations. These two boundary conditions result in temperature and moisture gradient within the concrete slab. Nassiri and Vandenbossche (2012) identified the two boundary conditions as the causes of curling and warping of concrete slabs. Curling and warping induces tensile stresses in the concrete slab which eventually results to cracking. The cracking can be exacerbated by the combination of curling and warping stresses and the stresses caused by traffic loading over time. Foundation soils or support layers which are less cohesive, curling of PJCP depressurizes the foundation layers exposing them to deformation and voiding near the external edges of the pavement slabs thus affecting the layer support strength resulting to eventual cracking of concrete slabs in voided areas on application of repeated traffic loading (Xu & cebon, 2017). In order to have a sustainable concrete pavement, stresses contributed by transient load, cyclic and permanent curling and warping of slabs require to be determined and incorporated in the concrete pavement design.

It has been observed that there is a permanent temperature gradient in concrete slab irrespective of existence of moisture and temperature fluctuations. This temperature gradient occurs immediately after construction as concrete sets and is generated from the heat of hydration and results to permanent warping in concrete slabs. Permanent temperature gradient occurs at the transition point in the strain - temperature response of the slab as concrete gradually transforms from thixotropic material capable of flow to hardened rigid material which is at the point when heat of hydration reaches 45% of total head of hydration (Nassiri &Vandenbossche, 2012). In concrete slabs, heat is produced through exothermic reaction of Portland cement and transferred through conduction. The heat changes across concrete pavement structure is dependent on, time, distance, thermal conductivity, weather condition and water cement ratio.

Irrespective of the type of pavement, rigid or flexible, the associated acoustic levels (noise generation) have been established to be comparable as long as the surface texture is of similar standards. However, the noise levels increase with increasing age of pavement as the pavement surface became worn down and other surface distresses develop. Exposure of construction material and constructed pavement to traffic loading and environmental variables greatly affect the properties of paving materials and progressively leads to deterioration and wearing of pavement surface leading to change of surface texture and increasing of pavement noise generation (Irali et al., 2015). Surface characteristics is influenced by aggregate gradation, age of pavement surface distresses, and macro texture. Ability of pavement material in vibration damping and attenuation influences tire –pavement noise production. Absolute noise levels of Portland concrete pavements (PCC) is usually higher than that of asphalt concrete pavements. PCC surface texture can however be modified to greatly reduce the tire noise production (Irali et al., 2015). Surface life cycle environmental benefits arising from a good pavement surface are reduced air and noise pollution and reduction of emitted greenhouse gasses. Vehicles on a rough surfaced road consume more fuel leading to higher emission of air pollutants and greenhouse gasses. Aging pavements produce more noise resulting in overall to a higher environmental cost. A well maintained pavement surface is more effective in mitigating environmental impacts thus creating long term environmental benefit (Pellecuer et al., 2014).

The anticipated climatic changes through global warming phenomenon may subject the road pavements to different and changing climatic conditions over the pavements life. Current designs are based on typical climatic patterns which consider known rages of temperature and rainfall. The climate change may bring drastic changes to known climatic parameters thus making the currently designed and constructed pavements inadequate to withstand future environmental stresses. Faced with the uncertain climatic changes and effects to the existing and future pavements an adaptation strategy should be considered in pavement design and construction (Li et al., 2013).

Changes in global warming will result in significant changes in temperatures, precipitation and water mass level rise with a profound effect on built environment. As these changes become more apparent, the traditional design philosophy and design standards in highways require to be re-evaluated to determine their adequacy to address climatic stresses that are beyond the currently considered environmental factors in pavement design (Li et al., 2013). Extreme climatic changes may result to excessive thermal expansion of pavement from extreme heat resulting to softening of asphalt and hence increased rutting, surface cracking and blowups in concrete pavement slabs. Increased precipitation and rising up of sea level may result to eroded road bases, washouts and flooded roads. Understanding how and when to respond to these changes would mitigate the effects and loss of investment in pavements from unpredicted environmental changes.

Emphasis has been made on the need to consider environmental as well as social costs in pavement investment decisions. Reduced air and noise pollution and reduction of emitted greenhouse gasses are some of the life cycle environmental benefits arising from a good pavement surface. These benefits should be converted to monetary values and be incorporated into other elemental costs in pavement construction for computing the life cycle costs and benefits for alternative pavement types. Rigid pavements have been cited to have a higher noise production levels than flexible pavements and effort should be made to come up with ingenious ways for the reduction of associated noise pollution. In other frontiers there is awakening to the realities of the likely effects of global warming phenomenon to the already constructed pavements. Global warming will have far reaching environmental impact. It is imperative that authorities and stakeholders in road sector in Kenya and the surrounding region enjoying similar climatic disposition, start focusing, developing and putting in place measures to mitigate against likely negative environmental impact occasioned by global warming phenomenon to road infrastructural network.

2.2 Jointed Plain Concrete Pavement (JPCP)

Jointed Plain Concrete Pavement (JPCP) consists of unreinforced concrete slabs with transverse contraction joints between the slabs spaced at an average distance of 4.5m.

The slabs are is generally limited to a maximum length of 6m (Kumar & Mathur, 2009). The joints are spaced closely so that the inevitable stress cracks in concrete pavements are guided to develop at the joints rather than everywhere on the slab surface.

JPCP is the cheapest type of concrete pavement compared to other concrete pavement types as it saves on steel and labor associated with reinforced concrete pavements and reduces the level of projects capital outlay demand thus promoting competition among a large pool of contractors who would otherwise be constrained by capital demand (Delatte, 2008). Most contractors are therefore likely to be familiar with JPCP than with other types of concrete pavement. In regions where corrosion of steel is a problem, application of JPCP largely eliminates this problem as steel is only provided at joints as steel dowels for load transfer across construction joints and at longitudinal joints to hold the pavement lanes together. Corrosion concerns are addressed by coating of the steel dowels with an epoxy or other appropriate corrosion resistant coatings.

Transverse contraction joints in JPCP are cut in concrete when it is still green and before the onset of plastic cracking. The recommended depth of the saw-cut joint is in in the range of 1/4 to 1/3 of the concrete slab thickness (Yao & Weng, 2012; Kumar & Mathur, 2009). A thin crack propagates through the remaining depth around the aggregates in the green concrete thus forming a complete contraction joint. The saw-cut joints transfer load across the slab through aggregate interlock in the thin crack below the cut joint. If the saw-cut joints fail in load transfer across to the other slab, the slab may fault creating bumps on the pavement and causing discomfort to the road users and leading to serviceability failure. JPCP use tie bars to connect adjacent traffic lanes. Tie bars are deformed reinforcing steel and, unlike dowels, are not intended to allow the joints to open and close.

Distresses in JPCP and more so cracking are associated with many but varying causes ranging from constructional, operational, physical and environmental. Amongst the causes of cracks in concrete, are those that have been considered as definitive while others are circumstantial. Some are within human control, in particular constructional aspects, while those which are concerned with physical characteristics of material and environmental influences are inherently present and the understanding of their effects to constructed works is critical to sustainability of JPCP.

JPCP should be founded on firm and quality supporting layers of base and or subbase to avoid cracks and other defects associated with deformation of supporting layers, the saw-cut joints should be cut to the specified depth and within the time limit in order to avoid surface cracks occurring due to environmental and intrinsic factors within the body of concrete including temperature and moisture gradients.

The construction process should ensure adequate quality control of construction materials and construction methods and movement joints should be functional and be able to transfer loads to adjacent slabs with efficiency.

Field investigations undertaken in Dallas District in America on observed cracks on JPCP, some of which had occurred on ‘recently’ reconstructed JPCP revealed that the base and the foundation soils had very poor compaction. The Dynamic Cone Penetration (DCP) tests returned very high penetration which is a sign of low strength on supporting layer (Chen & Won, 2007). A DCP penetration of 2.5 mm per blow was indicated to represent a good granular base with a CBR of approximately 100% and a modulus of approximately 345MPa. Further there were observed settled concrete slabs which further confirmed low base support. This manifestly resulted to cracks on the concrete slabs.

In a research in China, Yao and Weng, (2012) established that inadequate pavement edge support resulted to deformation of subgrades which caused voids below the slabs to develop and this resulted to cracking of concrete pavement along the wheel load. Research on concrete pavements in India indicated that settlement of subgrade and subbase was one of the causes of cracks appearing on concrete pavement (Kumar & Mathur, 2009).

History of JPCP and information on commencement and progression of cracks is prerequisite in making initial assessment of the likelihood of cracks occurring due to inadequate depth of saw-cut joints and or late saw-cutting of the joints. In absence of the time of occurrence of the cracks full investigation limited to field investigations is required. To undertake full investigation of JPCP, the history of the pavement, field observations and investigations are required. The history can be obtained from records of design and construction and or through interviews of the people concerned in its construction. The historical information required includes the structure of pavement which entails thickness of JPCP and type and thickness of base, design traffic, joint construction systems and age of pavement. Interventions undertaken in the course of the pavement life is also a requisite information. Field observations or condition survey involve visual examination of surface conditions of pavement including drainage and discernible distresses including cracks, faulting, popups, scaling, edge break ups, pumping at joints etc. For field investigations, DCP equipment is suitable for use in layer strength assessment due to its simplicity and ease of operation. To compute the strength of pavement from DCP equipment in terms of CBR and the layer modulus, Yao & Weng, (2012) in an investigation on a pavement in Dallas America used the equation (2.3):

$$CBR = 292/PR^{1.1/2} \text{ and Layer Modulus (E) MPa} = 17.58\ CBR^{0.64} \qquad (2.3)$$

Where PR = DCPs Penetration Rate of DCP in mm/blow (Penetration Index).

Dowelled joints in concrete pavements require careful design and construction considerations. Failure in performance of dowelled joints is a major cause of cracking of concrete pavement slabs. An investigation conducted in Beaumont District in America established cracks occurrence on either lanes of JPCP. There was neither information as to when the cracks occurred nor whether the concrete support was in good state. Hence full investigation was conducted with intention to assess existing pavement condition, identify causes of the irregular cracks and provide any possible solution. Information on pavement construction and the time when it was constructed was established through existing records. Field investigations were conducted using Falling Weight Deflectometer (FWD), DCP (Dynamic Cone Penetration), Ground Penetration Radar and Core cutting.

Analysis of the findings established the cracks were not caused by loss of support or subsidence of the base. Manifestation of the cracks at the saw-cut joints revealed the cracks had poorly developed and unexpected cracks had developed around the dowels. It was then theorized that the cause of the cracks was late cutting of saw-cut joints and restrictions at both ends of dowels (Chen & Won, 2007).

In rehabilitation of JPCP, the type of rehabilitation method is determinant as to whether an optimum rehabilitation strategy has been achieved in lowering or eliminating recurrence of cracks on JPCPs. In a research undertaken in China, a recently established JPCP was investigated and underlying causes of longitudinal cracks analyzed focusing on materials used for rehabilitation and traffic performance (overloading). Effects of use of separation layers of asphalt and geotextile in preventing development of longitudinal cracks was also investigated (Yao & Weng, 2012).

The pavements under investigation were constructed 10 years earlier and it was constituted of 260 mm thick JPCP, 150 mm thick cement crushed rocks, 150-200 mm subbase course of stone ballast/natural sand and soil subgrade. The rehabilitation of pavement took 2.5 years to complete and the weather was variable during that period.

The construction strategies/methods used during rehabilitation included:-

(i) **Strategy 1 -** Overlay of existing pavement with 190-240 mm PCC with a 70-100 mm slurry seal separating layer between old JPCP and the new concrete layer.

(ii) **Strategy 2** – Replacement of subgrade with crushed rock - This was considered where old JPCP had over 30% crack development. This involved demolition of existing pavement layers and then removal of existing subgrade to a depth of 300 mm and beyond followed by replacement with crushed rock for subgrade, application of 150 mm grade crushed rock bed course, 180 cement treated rock subbase, 200-220 mm of lean concrete base, 70 – 100 mm slurry seal separation, layer and 280 – 300 mm of PCC. During construction the edge restraint was not reconstructed and side slopes were left intact. The works were done during wet and dry seasons.

(iii) **Strategy 3** - Rubblization of existing Precast Concrete (PCC) Pavement – This involved breaking existing concrete pavement into small rubbles of size upto 380 mm in diameter. The rubblized layer was used for subbase followed by a lean concrete base, 70-100 mm slurry seal layer and 280-300 mm PCC were constructed in that order. This strategy was applied where cracking of existing PCC was less than 30%.

(iv) **Strategy 4** – Break and Seat – This involved breaking the existing pavement using impact compactor and using the broken slab as subbase course. Reconstruction of layer was as in strategy 3 was followed.

Two years after reconstruction, cracking status of sections constructed using the above strategies was as in **Table 2.3**.

Table 2.3: Cracking status of rehabilitated pavement

Strategy	Cracking level (%)	No. of cracked slabs (%)
i	6	7
ii	68	4
iii	8	1
iv	18	0.5

Strategy i – resulted in low level of cracking but also gave the highest number of cracked slabs. This was attributed to inadequate concrete overlay depth (190-240) mm as compared to 280 -300 mm in strategy ii – iv. The high level of cracking in strategy (ii) was attributed to poor edge restraint as the side slopes were not reconstructed and were left intact, dry and wet conditions which prevailed during construction period, variation of compactive effort across the pavement and plastic deformation of subgrade on loading. The low crack levels and low number of cracked slabs in strategy iii and iv were attributed to removal of voids that were previously below the concrete slab through breaking, followed by heavy compaction of the new subbase.

Application of separation layer of Asphalt concrete (AC) 30 mm and Geotextile applied between the old JPCP and the new 250 mm PCC overlay resulted to cracks development scenario indicated in the **Table 2.4**. Survey was conducted after 2 years.

Table 2.4: Cracking attributed to separation layer

Section	Separation layer used	% of cracked slabs
1	AC	0
2	Geotextile	1.1
3	No – separation layer	3.2

In section 2 (**Table 2.4)**, it was observed after the removal of the cracked slabs that the cracks were reflective cracks implying that geotextile was not as effective as AC in breaking the bond between the two layers of concrete. Further tests showed the bond stress between the two pavement layers fell by 85% in section 1 and by 24% in section 2. The strain test showed a high strain at the bottom of PCC in section 1 as compared to section 2 and 3. The high strain development in section 1 indicated that the PCC layer was acting as a separate layer while the degree of homogeneity in section 2 and 3 varied. This demonstrated that material used in construction and method of construction applied has a high contributing effect to the development of cracks.

Operational aspects of pavement and mainly overloading of trucks has a critical role in cracks development on JPCP. Investigations by Yao and Weng, (2012) on Qinglian highway and Jiaoling projects in China revealed same and regular pattern of cracking on JPCPs in lanes carrying traffic in a particular direction. The cracks were longitudinal and were found to manifest in lanes with overloaded trucks.

The longitudinal cracks were along the wheel path. Where the road had a cross slope, the cracks were largely manifested along the outer wheel on the down side slope. This phenomenon was explained as due to shifting of the center of gravity from the center thus making the outer wheels on the downslope more loaded.

The findings of the investigation revealed that longitudinal cracks occurred on the lane that received the highest number of overloaded trucks and was manifested on the wheel path location. Where the road is inclined, most longitudinal cracks occurred on the downside wheel path. As long as the construction aspects are not compromised, longitudinal cracking resulting from overloading assumed similar pattern irrespective of method of construction. Consequently it was concluded that overloading was a high contributing factor in the cracking of JPCPs.

Inherent Material Qualities which have significant behavior change on exposure to environmental variables were found to contribute to cracking of JPCP. Materials which have high coefficient of expansion are known to cause cracking of concrete. Research conducted on approach slabs to a concrete bridge where one of the approach slab had cracked while the other one was still intact revealed that the side that had cracked was constructed using siliceous river gravel aggregates while the un-cracked side was constructed using limestone aggregates. Cores from cracked slabs indicated that cracks occurred around aggregates and coefficient of thermal expansion for siliceous aggregates was found to be higher than that of limestone aggregate. It was therefore concluded that the type of aggregates used greatly influenced the observed cracks (Chen & Won, 2007).

Dowel bars are commonly applied in JPCP to help transfer traffic loads across the concrete slabs at construction joints. Heavy wheel loads impose heavy level of bearing and shear stresses in joint areas. The high shear stresses under the dowel bars can lead to initiation of micro-cracks in the concrete slabs and propagation of these cracks can be by fatigue from traffic loads coupled by environmental loads leading to eventual joint failure (Zeinali et al., 2013). The conventional dowel joint construction it was observed, impose high shear stresses in joint areas and imposed high punching shear at the dowel tips due to impaired free slip movement of the dowel during contraction-expansion cycles of the concrete slabs resulting to eventual shattering of concrete around dowel tip. The curling and warping of pavement slabs due to temperature and moisture variations in JPCP resulted to a downward and upward cyclic movement of concrete slabs resulting to heavy imposition of relatively high moments in the dowel bars as well as heavy shear stresses at the concrete joint faces resulting to creation and propagation of micro-cracks and permanent deformation of dowel bars. It was further observed that there was a high chance of misalignment of the dowels doing construction causing additional shear and bearing stresses which are detrimental to joint performance.

The type of joint dowels has a significant influence on the performance of joints. Repairing of failed joints with dowel bar retrofit was observed to result to significant failure of the joints after some time due to shear and bearing stresses reinforced by environmental factors. It was established that the above short comings could be resolved by application of Hinged Dowelled System (HDS) which has a collective hinge separating two sets of dowels on either side (Zeinali et al., 2013).

The HDS allowed rotation of the joint resulting to zero moments. The HDS had flexible spacers at the bottom and at the ridge of HDS that fill the joint thus eliminating the need for saw-cut joint and ensures unimpeded free horizontal movement of concrete slabs. The HDS is strong enough to transmit shear stresses across the joint and it is fitted with sleeves at the end of dowels to facilitate free horizontal movement of the dowel thus eliminating punching stresses. The spine along the joint holds the dowels together in alignment thus eliminating the misalignment common with conventional dowels.

Zeinali et al. (2013) established that HDS reduced shear stress across joints by 15% as opposed to conventional dowelled joint thus significantly improving joint durability.

Further analysis indicated that HDS did not impose any risk of tensile or compressive failure on the joint system neither did it reduce load transfer efficiency.

Investigations on the cracking of JPCP has been widely investigated and documented in the developed world under their prevailing construction realities. The causes have been identified to result from constructional aspects including compaction of pavement layers and jointing, operational aspects mainly overloading, physical aspects which touches on quality and strength of construction materials and environmental aspects which touches on the stresses created by temperature and weather variations in the body of concrete slabs and response to variation in temperatures of inbuilt individual material properties. Currently bituminous pavements are the main type of road construction which are locally undertaken and as concrete pavements gets promoted and gains acceptance, there is need to undertake research and create acceptable standards for design, construction and material qualities based on local conditions. The inherent material qualities should be investigated and profiled for concrete pavement construction. Capacity building by road agencies and other stakeholders in the road and transportation sector should be deliberate and geared towards design, construction and maintenance of sustainable concrete pavements.

2.3 Pavement Construction and Maintenance

Pavement construction process encompasses both procurement and physical construction stages. Procurement process involve determination of the elements of construction, inputs, type and method of construction and selecting the entity to undertake physical construction. Procurement process may consider force account method of construction which is mainly adopted by a government through requisite agency by utilizing its internal capacity to undertake given construction works. The agency may procure materials and temporally labor to carry out the works. The other method of procurement is competitive tendering where independent establishments whether government agencies or private companies compete fairly through bidding for the work. The bidding and contracting documents form the foundation of this type of construction process (World Bank group, 2006).

Force account method of construction where adopted is known to produce works which are more expensive and of less quality than competitive tendering due to inherent inefficiencies and monopolistic tendencies by suppliers.

In competitive tendering, selection of contractors is on the basis of the lowest cost tender which conforms to contract specifications and conditions. The low cost tender only allows contractors to undertake calculated risks in contract performance and therefore inadvertently disallows contractors to come up with higher risk but innovative and probably sustainable practices in construction. This denies the industry ideas that would otherwise lead to new and sustainable construction methods and processes. It is therefore important that in assessment of contractor’s submissions, risk and cost benefit ratios should be well understood in order to allow and support innovation in the construction industry from the private prayers. Further, in the tender analysis, whole life cycle costs including road user costs, environmental costs and environmental mitigation costs should be assessed (Vorobieff, 2010).

Pavement construction can be labor based and or mechanized depending on the level of works sophistication, contract time limitations and the intended goals. Labor based construction programmes are adopted mainly in developing countries as a vehicle to create employment, impacting skills and developing institutional capacities.

In works contracting the level of risk has to be assessed by both the contractor and the employer in order to avoid failed contracts. The procurement of the works has to be in such a manner it motivates the contractor to bid at his best price while at the same time protecting the employer against irresponsibly low bids which latter resort to many claims and controversy.

Sustainable road construction practices should be encouraged at all levels. This requires that innovation and adoption of new technologies be encouraged in this age when environmental concerns and green gas emissions have become a concern for consideration in the construction processes. The governments and road agencies should be ready and willing to fund for research and development of new materials and technologies in view of depletion of high quality pavement construction materials that have traditionally been in use for a long time.

Maintaining a good road infrastructure contributes to reliable transport at reduced cost. A good road condition has a direct relationship to lower vehicle operating costs and reduced number of accidents which has an impact on both capital, human and property cost.

Maintenance of road infrastructure is categorized as routine, periodic, special or developmental. Routine maintenance could be on planned periods of maintenance referred to as cyclic or due to occurrence of a maintenance need or reactive maintenance. The works are mainly funded through recurrent budget. Periodic maintenance works are carried at intervals of several years in order to address deteriorating road condition and address changed motoring and loading circumstances through strengthening, widening, resurfacing or overlaying. Depending on the extent of intervention the works can be funded through recurrent or capital budgets. Special works are as a result of unforeseen circumstances like washouts or landslides that dictate maintenance. Funding of these works is from special funds or contingency funds in recurrent budget. Development works are identified as a national development agenda and are funded from capital budget.

Maintenance works in road projects can be undertaken through “force contract” (labour based), contracting through competitive bidding or through performance based contracting. Performance contracting encompasses both routine and periodic maintenance for a defined period of time in years and can range from 2 to 5 years. The contract could be price based, performance or result based and funding could be through tolling in which case the contractor provides management and maintenance works activities (World Bank group, 2006).

To help in road maintenance processes and in developing programmes that are sustainable and cost effective for existing pavements, pavement management systems (PMS) have been developed. These programmes consider different types and patterns of the pavement surface condition throughout the life of the pavement (Pellecuer et al 2014). Management of the road network requires the right information at different levels of decision-making process (planning, programming, design, and implementation). The data to be collected by an inspection system, and where, and how it should be collected depend largely on the use of the data. The choice of road management system is dependent on several factors including initial procurement cost, expertise requirement for collection of requisite data, costs of system management and data collection. Implementation of PMS could have far-reaching effects on all aspects of the operation of the road maintenance administration. Hence, proper understanding of the input requirements, the desired outputs and attendant pitfalls is necessary for establishment of an effective pavement management system.

In most circumstances, most road agencies have concentrated in development of new road networks without setting out maintenance and rehabilitation programmes. In Saudi Arabia, in the year 2000, the cost of road maintenance was only 0.51% per year compared to the cost of new development. In America maintenance allocation was 2.94% per year of new road development budget. At this low rate of expenditure in road maintenance programmes most of the developed roads suffer neglect and deterioration. Most agencies therefore resort to reactive maintenance rather than preventive maintenance and in the process loose about 25% of the cost that would have otherwise been saved if preventive maintenance was adopted (Mubaraki, 2012). In order to come up with suitable maintenance programmes it is important to characterize the urban and the rural roads in terms of the requisite technical and administrative issues associated with sharing of the road network, nature and types of pavement distresses and the size of network and the traffic volume. Mubaraki (2012) observed that pavement age has a dominant effect on pavement distress due to time related changes in traffic characteristics. Age it was observed prays a pivotal role in development of maintenance strategies. It was observed that corrective maintenance of most distresses can be undertaken when the distress levels are below 20% and preventive maintenance when distresses are below 35% otherwise beyond this level of pavement deterioration, structural maintenance and reconstruction which is quite expensive would most likely require to be undertaken.

As the good quality materials gets depleted from quarry sites, the cost of new materials for new construction and maintenance keep going up. There is also rising need for long term environmental management from waste material in stockpiles and dumpsites resulting from industrial activity, dumping of low quality materials and waste from roads under maintenance and reconstruction. Currently there is emphasis of waste minimization strategies which has given need to recycling of old pavements materials in construction of new pavements and in maintenance of existing pavements. The existing roads can now be considered as liner quarries from the fact that they carry the potential for reuse of the construction materials inbuilt in these roads. Consequently, insitu stabilization of existing pavements materials has become a common phenomenon in rehabilitation works. In asphalt, the reclaimed asphalt pavement(RAP) material is being used as a replacement for 'virgin' materials through currently limited to only 15% to ensure a durable asphalt is produced especially for heavily trafficked roads (Vorobieff, 2010). It is however noted that for sustainable utilization of recycled road construction materials in asphalt, this percentage should be increased through further research and innovative processes.

In concrete pavements, restoration of road surface roughness was through application of an overlay or removal and replacement of concrete slabs, however an innovative method of diamond grinding is becoming a common method in surface roughness level restoration. This is a sustainable approach to increasing longevity and service level of concrete pavements. Nevertheless, the design of the concrete slab has to now consider the emanating surface grinding rehabilitation strategy by increasing the layer thickness during construction to allow for grinding off of a layer without loss of the structural capacity of pavement (Vorobieff, 2010). The design consideration however increases the initial pavement construction cost. More research should be put in advancing recycling of concrete pavement materials for construction and maintenance of road pavements.

In Kenya, road agencies use competitive tendering in line with Government procurement policies. Despite competitive tendering limitations expressed in the literature of innovation and advancement of new ideas and experimentation of new construction materials, it gives better value for money and it is risk averse. Previously the only road agency in Kenya that was in existence (Ministry of Roads) was using force account for road maintenance. This has however now changed to competitive tendering with creation of several road agencies and in line with the above stated Government policy on procurement. Furthermore in recent time, performance term contracting for routine and periodic maintenance has been rolled out by Kenya National Highways Authority. Pavement Management Systems (PMS) have also been rolled out by all road agencies in Kenya according to recent technical and performance audits. As observed in literature, investment in road maintenance is universally low even in developed economies. This trend similarly applies to Kenya and has resulted to dilapidated or poorly maintained roads due to limited fund allocation. In advent of the concrete roads in Kenya, the method of maintenance should be clear right at the planning and design stages in order to enable the correct design approaches that will consider maintenance processes and enable the life cycle cost of the pavement to be determined.

2.4 Pavement Failure

Pavement fails when the serviceability levels decrease due to development of surface distresses. The most common pavement distresses include surface cracking, pavement deformation, pavement disintegration and other Surface defects. The primary function of a pavement is to provide a reasonably smooth riding surface, provide adequate surface friction and protect the subgrade and provide water proofing to lower pavement layers (Adlinge & Gupta, 2013).

Pavement failure can be attributed to various factors that impact on the performance of pavements including traffic, pavement construction material quality, ground water effects, construction practices and maintenance (Adlinge &Gupta, 2013; Chen et al., 2006).

Pavement performance is heavily influenced by traffic. The loading magnitude from commercial vehicles, configuration of axles and the repetitive application of load on the pavement determine the damaging levels of traffic to pavement structure. A pavement structure is designed to withstand a certain number of equivalent standard axle load repetitions for a predetermined design period. Overloading of axles by commercial goods vehicles and increase of average equivalent standard axles of heavy goods beyond predicted growth, distresses the pavement structure and contributes to rutting of flexible pavements (Al-Suleiman et al., 2000). In airfields with flexible pavement, repeated trafficking by flights may result to excessive vertical subgrade stresses resulting to generation of rutting (Kasthurirangan & Marshal, 2007). Repeated application of heavy wheel load would also lead to fatigue or alligator cracking in asphaltic concrete pavements. Shear cracks on pavement is also as a result of heavy traffic loading on the wheel path. In rigid pavements or concrete slab pavements traffic lanes carrying overloaded trucks have been observed to be more cracked than the lanes carrying less loaded traffic leading to the conclusion that overloaded traffic contribute heavily to cracking of rigid pavements (Yao &Weng, 2012). Heavy truck loads result to differential settlement of subbase and subgrade layers resulting to cracking of concrete slabs and faulting of slabs (Kumar & Mathur, 2009).

Quality construction material and construction quality control are prerequisite to a sustainable pavement. Pavement layers contribute individually to the total deformation of pavement structure. Excessive deformation resort to rutting and eventual failure of pavement. In flexible pavement, strength of asphaltic concrete and its resistance to deformation can be achieved through proper material specification, mix design and construction specifications. The structural capacity of the granular base and subbase and hence resistance to deformation can be addressed through specifying gradation, strength of granular material (CBR or UCS values), layer thickness and the atterberg limits of the fine fraction. The subgrade contribution to deformation is dictated by stress levels reaching the subgrade from the upper layers, soil strength (CBR) and the magnitude of repetitive loading. Subgrade vertical strain levels are critical to development of pavement surface deformations or rutting. (Kasthurirangan & Marshal, 2007; Al-Suleiman et al., 2000). Non uniform compaction of pavement layers and material variations particularly in rehabilitation projects has been observed to contribute to rutting and longitudinal cracking of rehabilitated pavements. Hence, proper construction practices, quality control and quality assurance during road construction process should be maintained in order to avoid and eliminate the formation of undesirable pavement failure.

Moisture significantly weakens the strength and integrity of pavement layers resulting to pavement failure. Moisture above the optimum moisture content of molded pavement layers of base, subbase and subgrades reduces the inherent strength parameters including the layer modulus which is a measure of structural strength of pavement. Saturated pavement layers receiving heavy application of axle loads can result to sudden total structural and functional failure of pavement (Burman, 2006).

Moisture could enter the pavement structure vertically through surface cracks, voids in asphalt concrete layer and porous concrete slabs, unsealed joints in concrete pavement and on unsealed pavement shoulders. Moisture also ingress laterally through the porous embankments of pavement layers, subgrade, and from the underlying water table through capillary action or elevated water table during wet seasons. The result of moisture ingress is the lubrication of particles in unbound gravel layers leading to loss of particle interlock and subsequent reduction in layer strength. Pumping out of loosened particles occurs in rigid pavements through open joints creating voids below the pavement thus weakening the support system of the slab leading to cracking and faulting of pavement slabs.

Penetration of water through cracks in asphalt concrete pavements which have overlays can other than softening of base and subgrade layers result to loss of bond between AC overlay and the layer below resulting to stripping of asphalt layers (Hong & Chen, 2009).

To ensure moisture is kept out of pavement layers, appropriate designs, quality construction materials and adoption of appropriate construction practices for both new pavements and rehabilitation projects should be observed (Chen et al., 2006).

Pavement performance and sustainability depends on properly programmed maintenance. However well a pavement has been constructed it will deteriorate over time from functional and environmental loadings. Timely maintenance increases the surface life of pavement and saves the cost of reconstruction that would have been used over and above if the road had been left unmaintained (Adlinge & Gupta, 2013). Funding for routine, periodic and emergency maintenance should be incorporated in annual budgets for recurrent, capital and contingency expenditure.

Pavement distresses and failure can be attributed to among other factors, high traffic, over-loading, compromised quality of construction material, high moisture, uncontrolled construction practices and lack of proper maintenance. Quality control of construction processes are prerequisite to a sustainable pavement. Material variations and non-uniform compaction of pavement layers and material variations contribute to cracking of pavements. Lack of adequate funding for maintenance works has led to deterioration and failure of pavements. In Kenya the only other concrete road the Mbagathi way has developed surface distress in form of cracks. It is imperative therefore to diagnose the likely causative effects that are responsible for cracking of concrete pavements in the confines of the local conditions

2.5 Pavement Cost

The cost of road construction comprise expenses for planning, design, construction, maintenance, rehabilitation and socio-economic costs. The socio economic costs comprise road agency management costs, user costs and environmental costs. User costs include travel delay costs, accident costs and vehicle operation cost. To be able to capture the entire cost of a road project, life cycle cost analysis (LCCA) is required. LCCA is an appropriate tool for selecting the most suitable pavement for an economically reasonable long term investment. The LCCA has evolved since the year 1960 when it was first put to use in America and today, computer based systems and pavement selection optimization models have been developed to analyse life cycle costs for both flexible and rigid pavements. LCCA can additionally be used to analyse the current net worth of pavements by discounting from the initial investment costs the future costs of maintenance, rehabilitation, reconstruction, resurfacing and pavement restoration costs (Babashamsi et al., 2016). Life cycle cost analysis method can be used to analyse the cost effectiveness of preservation or preventive methods of pavements which are the methods used to extend pavement life without increasing its structural capacity (Wang et al., 2012). For flexible pavements, preventive treatment include cracks sealing, chip sealing, slurry sealing, micro-surfacing and thin hot mix- Asphalt overlay. Wang et al. (2012) observed that life extension of pavement through preventive treatment was directly proportional to the cost of the preservation method. The effectiveness of the applied method in asphalt concrete pavements was found to follow the order of thin hot mix- asphalt overlay, chip sealing, cracks sealing and slurry sealing. The effectiveness or the performance level was assessed based on the change in the International Roughness Index (IRI) from the time of intervention to the time of assessment.

To capture the entire life cycle cost of pavement, environmental costs related to the production of new pavement materials, materials deposited in landfills associated with construction, maintenance, rehabilitation and reconstruction and pavement material recycling should also be considered (Santos & Ferreira, 2013).

Concrete pavements in road development are usually selected for higher traffic routes and are comparatively more expensive at construction stage but have lower full life cycle cost compared to asphalt pavements. This holds true if the preservation programmes are well planned and executed (Robert, 1994). Distresses in concrete pavements is a concern and a threat to the heavy initial investment particularly if the distresses result to structural failure of the pavement before the attainment of the designed service life. The life cycle costs depend on the structural adequacy of pavement and it is vital that realistic and equitable designs be used. Simple and quick methods of alternative pavement assessment in terms of cost and reliability are necessary and can easily be developed for selection of the most suitable and sustainable pavements in varying circumstances of traffic and pavement layers as demonstrated by Pabitra and Animesh (2008) in optimal asphalt pavement designs.

Concrete pavements require comparatively higher initial cost than flexible pavements and are generally selected for roads of higher traffic categories. To justify the investment in concrete pavement, life cycle cost analysis is necessary .There is need therefore for a local criteria to be set up by road agencies to guide in the investment of concrete pavements and which will be used locally to justify investment in rigid pavements. Further, design, construction and maintenance standards for concrete pavements require to be developed as it is with flexible pavements. These will enable comparable life cycle costs to be carried out for various road alternatives both for rigid and flexible pavements.

2.6 Topography

One of the factors that aid in long survival of pavements is a flat topography (Seong et al., 2016). To enhance slope stability, reduced erosion, and provide sufficient space for road construction, in Kenya it is recommended that benches terraces be constructed where the slope of the road is more than 1:3 (Ministry of transport and communication, Republic of Kenya. 1987). Roads in hilly areas are prone to effects of landslides. In relatively wet areas the chances of landslides are increased.. Roads passing through hills need more care as far as drainage is concerned. There are no “catch water drains” to divert and intercept the water from the hill slope. The rainwater from the slopes rundown and cause deterioration of pavement surface through erosion and differential settlements that is caused due to the retention of moisture in the subgrade soil.

Mobilization of resources and transportation of pavement construction materials in hilly areas is a difficult and an expensive task due to fuel consumption and time taken. The heavy earthworks arising from cutting and blasting and protection works to arrest erosion and increase slope stability add to the cost of construction in such hilly area (Rashid et al., 2017).

The construction of roads on steep slopes often calls for substantial grading, extra-wide rights-of-way to accommodate road slopes, retaining walls, and steeply sloping embankments, which can also require expensive long-term maintenance. The poor soils often found on steep slopes can impact buildings and driveways. Shifting foundations, cracked walls, and cracked pavement and roadways are some of the potential problems associated with slope instability. These problems often result in increased development and maintenance costs or, in extreme cases, structural failure (Lehigh Valley Planning Commission, 2008).

2.7 Research Gaps

Cracks in concrete pavement is an expression of distress arising from likely singular or multiple causes. Traffic overloading of pavement overstretches the structural capacity of the road resulting to surface cracking, pavement deformations and failure. Overloading of pavement can be through application of high axle loads beyond design limits or growth in volume of traffic beyond the projected design traffic (Adlinge & Gupta, 2013: Chen et al., 2006; Al-Suleiman et al, 2000; Kasthurirangan & Marshal, 2007; Yao &Weng, 2012; Kumar & Mathur, 2009).

Similar distresses occur when the concrete slab, base, subbase and subgrade layers are of inadequate strength in terms of layer thickness and material quality (Petry & Little, 2002; Werkmeister et al., 2004; DelRio-Plat et al., 2011; Beainy et al., 2014; Wu et al., 2014). Concrete pavements are a new concept in Kenya and so little pavement cracking data exists in the country.

Transverse contraction joints in JPCP are cut in concrete when it is still green and before the onset of plastic cracking. The recommended depth of the saw-cut joint is in in the range of 1/4 to 1/3 of the concrete slab thickness (Yao & Weng, 2012: Kumar & Mathur, 2009). A thin crack propagates through the remaining depth around the aggregates in the green concrete thus forming a complete contraction joint. In Kenya, no data exists on the effect of saw-cut joints on crack development.

The strength of a pavement is dictated by the quality of individual pavement layers and its stiffness (Said *et al.*, 2011) which is also a measure of pavement strength. Moisture significantly weakens pavement layers, significantly reducing strength parameters such as CBR and layer modulus (Burman, 2006). There exists no data in Kenya on strength and stiffness of concrete pavements.

The construction of roads on steep slopes often calls for substantial grading, extra-wide rights-of-way to accommodate road slopes, retaining walls, and steeply sloping embankments, which can also require expensive long-term maintenance. These problems often result in increased development and maintenance costs or, in extreme cases, structural failure (Lehigh Valley Planning Commission, 2008). The life of pavement is affected by the ground terrain. One of the factors that aid in long survival of pavements is a flat topography (Bea et al., 2016). Studies in Alabama, USA, found that the main cause of the premature failure was attributed to material and construction practices (Chen et al., 2006). Hardly anyone associates premature pavement failure to topography.

Therefore this study investigated these gaps within the Kenyan Context.

CHAPTER THREE

METHODOLOGY

3.1 The Research Road

The Kagere-Ndunyu-Gituiga (E571/E1684) Road considered in this study is a Jointed Plain Concrete Pavement (JPCP) and it was constructed in the year 2011. The JPCP slabs acts as the base layer and riding surface. The JPCP road section has a length of 6kms thus making it the longest concrete road in Kenya to-date. The road runs in a hilly topography along the edge of Aberdare Forest (Fig. 1). The altitude of the project area is at least 2000m above sea level.

The width of JPCP is 7m comprising two lanes with a longitudinal joint at the middle. The pavement is constituted of 300mm thick double layered subgrade and 150mm thick cement improved gravel layer subbase on which concrete slabs are casted. The concrete slabs have transverse saw-cut joints at an average spacing of 4.65m. The central longitudinal joint is fixed with steel dowels. The slabs were designed to be 185mm thick. Lined side drains are constructed on either side of the pavement where applicable (Fig. 2). The experimental JPCP section is 2.05km in length.

The design traffic load on the experimental road is class T5 (0.25 -1 million cumulative standard axles). Ordinarily concrete pavements are used for high traffic volume roads, however, the hilly terrain with steep vertical gradients made it impractical to use conventional and heavily mechanized road construction equipment applicable for laying bitumen on roads. Therefore, JPCP was used in the steep road section.

Construction of the research road was completed in the year 2011and By the year 2014 when the site studies were conducted and 2 years after the road was opened to traffic, the concrete pavement slabs had developed substantial surface cracks.

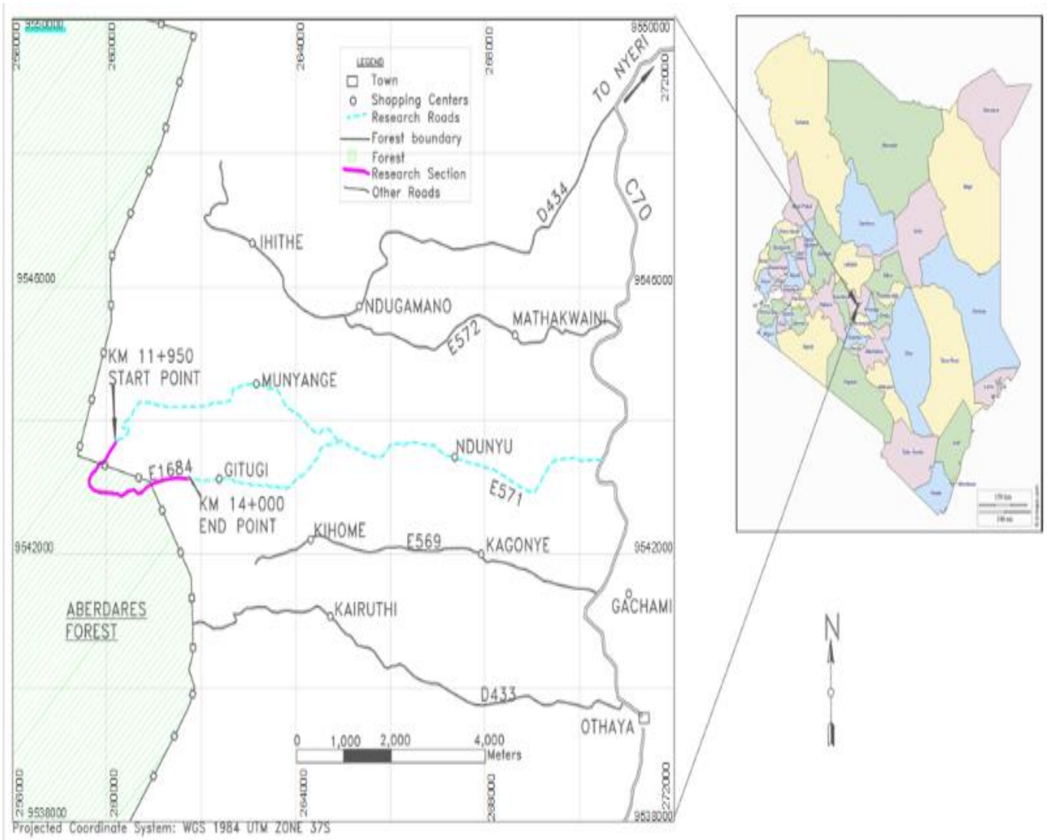


Figure 3.1: Kagere-Munyange-Njigari-Gituiga (E571/E1684) Road

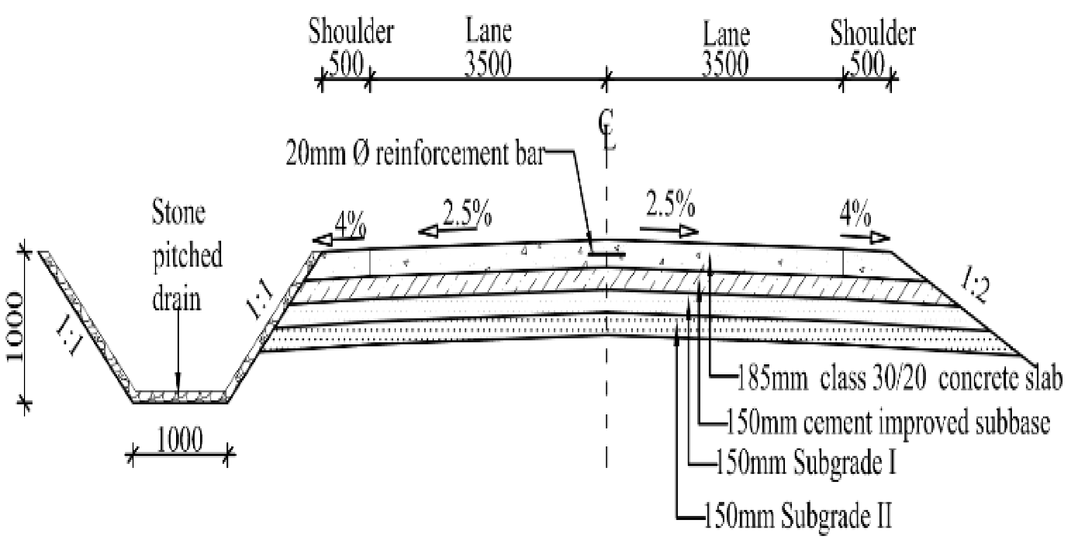


Figure 3.2: Typical concrete pavement structure

3.2 Research Design

3.2.1 The study approach.

The study was conducted after the long rains in the months of May, June and July, 2014. The Ministry of Roads and infrastructure of Kenya, Materials Department, provided the field and laboratory testing equipment and expertise. The selection of the study section was based on the outcome of surface condition survey. The road section with most visually discernible cracks was chosen for the research. The focus of the study was on the quality of pavement layers. The quality of subgrade and subbase was assessed by measuring their soil properties and related strength. The quality of concrete slabs was measured through crushing strength, concrete mix ratios and slabs thickness determination. Performance of saw-cut joints was established by measurement of the depth of saw cut joint and their ability to propagate full depth crack. The overall residual strength of pavement was assessed trough deflection measurements.

The research was designed to cover analytical process for quantitative approach to come up with the likely causes of cracks and mechanisms responsible for the development of the surface cracks on concrete pavement slabs. The research focused on the study of the available records and collection and compilation of data from the field for quantitative analysis.

3.2.2 Study of existing concrete pavement construction records

The following site records were obtained and studied:-

i. Tests results for pavement layers including alignment soils, subgrade layers, subbase and base courses which included checking of CBR values tested on neat and chemically improved materials, atterberg limits and grading.

ii. Quality of concrete materials used. This included course and fine aggregate parameters and strength of cement used.
iii. Concrete mix design/s and class of concrete used.
iv. Study of design drawings and specifications for approved concrete pavement structure.
v. Weather and climatic records during construction period and the general climatic condition of the area.
vi. Topography and soil types along the road alignment.

3.2.3 Field and laboratory investigations

The following investigations and processes were conducted:

a) Mapping of test location and determining the extents of cracks in the research section through crack width and length measurements.
b) Measurement of saw-cut joints to establish their spacing.
c) Cutting of concrete cores at the saw-cut joints, at crack locations and at intact sections to establish the depth of saw-cut joints, the nature and depth of developed cracks and the thicknesses of the concrete slabs.
d) Deflection testing of the pavement to establish its strength and stiffness.
e) DCP testing to pavement layers underlying concrete slabs to establish their strength and establishing of layer thickness.
f) Sampling of gravel material immediately underlying the concrete pavement slab for quality analysis (subbase layer material).
g) Laboratory testing for strength on the concrete cores (crushing test) and chemical analysis on samples of crushed concrete cores to establish constituent proportions of cement and aggregates.
h) Laboratory testing to establish soil and gravel characteristics and strength properties of samples collected from subbase layer.

33 Sample size

The sample size in this research comprised a 2.05km of the 6km concrete paved road length of Kagere- Ndunyu- Munyange- Gituiga (E571) Road which had discernible surface cracks on concrete pavement slabs. A total of 886 consecutive concrete slabs half on each lane of the pavement were studied. The sample included cracked and non-cracked slabs. The selection of the study section was based on the outcome of visual surface condition survey. The road section with highest level of discernible cracks was considered most suitable and was chosen for the research.

3.4 Sampling Techniques and illustrations

Sampling techniques used in identification of samples for research was the Non – Probability sampling methods. Infrastructural bias was used to identify the road to be researched this was so because only one road was a candidate. Purposive sampling was used to select the 2.05km location of the concrete pavement on which the research was to be conducted. The researcher observed the visual manifestation of the cracks on the surface of concrete slabs and was of the opinion that the selected section was a more representative sample of the JPCP road section as it had cracked and non-cracked concrete slabs and the alignment had flat and steep grades.

In order to obtain viable volume of data for research purposes, concrete core cutting and extraction points in which DCP tests were conducted and soil samples extracted, and the FWD drop points where deflection measurement were picked, were conveniently selected on site. The cracked and non-cracked slabs on which tests were conducted were opportunistically selected on the ground. The basis of this method of selection was for purpose and convenience. Therefore the sampling method manifestly was purposive and convenience sampling.

3.5 Instruments for Sampling and Testing of Pavement Layers

The instruments, apparatus and plants that were used for data collection in the research include:

a. **Concrete core cutting machine-** this was used to cut cores in the concrete slab for concrete strength testing, concrete slab thickness determination and for the study of cut joint behavior.
b. **Concrete crushing Machine** – was used at the laboratory to crush the concrete cores for concrete strength determination.
c. **Dynamic Cone Penetrometer (DCP)** – used in the field for strength testing of pavement layer of subbase and subgrade on which the concrete slabs bears. These layers were accessed through the openings made by extracted concrete cores.
d. **Falling Weight Deflectometer (FWD)** - this equipment was used for pavement deflection measurement. It is mounted in a vehicle and it simulates load produced by moving traffic on pavement. It has deflection sensors which pick and record deflection of pavement on load impact from the falling weight dropped from the deflection measurement equipment configuration.
e. **Measuring Tape/Ruler** - for measuring length, depth and width of cracks, core depth and slab dimensions.
f. **Camera** - for photographic recording of the research process.
g. **Chemical analysis equipment-** used in the laboratory for extraction and determination of proportions of constituent material of concrete from crushed cores.
h. **Core trimming machine-** used to trim the cores in the laboratory lengthwise to have a uniform length and create uniform faces.
i. **BS sieves, Soil spectrometer, weighing balance, washing jags, oven and shrinkage gauge-** for laboratory testing of soil parameters including grading and atterberg limits.
j. **A water can with a hose mounted on a pickup-** for cooling of concrete cutting cylinder during core cutting operation.

3.6 Data Collection

3.6.1 Determination of the extents of the cracks in concrete pavement on the Kagere –Ndunyu -Munyange–Njigari-Gitugi (E571) Road

i Identification, Marking and Mapping of the study area

The research section of JPCP was identified through visual inspection of the pavement for discernible cracks and related defects. The section with most intense and balanced cracks was selected as the most suitable for the study.

The pavement was made of concrete pavement slabs demarcated by saw-cut joints in the transverse direction, and a construction joint in the longitudinal direction which also marked the center line of the pavement. The slabs in the selected study area were marked for identification with indelible marks on the slabs external longitudinal edge. The slabs in the North East Lane were marked from A1 to A443 **(Figure 3.3)** and corresponding slabs on the South West Lane were marked from B1 to B443. The marks were made at intervals of approximately 5 slabs.

Figure 3.3: Slab Identity marking

The slabs and associated distresses were entered in specially designed tables namely:-

a. **Concrete slab mapping table -** this table captured the location and identity of slabs, number of observed cracks and location where concrete cores were cut.

b. **Slab identification and cracks mapping record-** this table captured in more details the characteristics of the cracks identified in (i) above, including length and orientation of the cracks. A sketch of the crack/s on each affected slab was incorporated in the record.

ii. Determination of intensity of cracks

The intensity of the cracks was determined by aggregating the length of measured cracks to and the area of slabs in the research section. The total length of cracks in millimeters (mm) was divided by the total area of slabs in square meters (m^2) to give the crack intensity in mm/m^2

3.6.2 Evaluation of the influence of saw cut joints and quality of concrete pavement on crack development.

3.6.2.1 Investigation of saw cut joints and cracks development.

The saw-cut joints are transverse grooves which are cut on the concrete slab before the concrete fully sets (green concrete) to facilitate development of thermal cracks at the guided location along the saw-cut joint. Saw-cut joints should be cut within the first 12 hours of casting and the depth of the cut groove should have a depth of 1/4 to 1/3 of the concrete slab thickness (Yao & Weng, 2012). The designed depth of the concrete pavement slabs was 185mm hence the groove depth should have been at least 46.25 mm. The adequacy of the saw-cut joint was determined by cutting of cores at the saw-cut joints randomly at selected joints along the study section and measuring and recording the depth of the saw-cut joint (groove) and the overall depth of the core which gave a measure of slab thickness at that point. Development of full depth crack below the cut groove or lack of it was checked on the core and recorded accordingly.

The cores were cut using core cutting machine (**Figure 3.4**). The core cutter comprise a cylinder of 100mm internal diameter with a core cutting edge. During the cutting process the core is housed inside the cylinder and it is retrieved from the cylinder after the cutting operation is complete. The cutting cylinder is mechanically withdrawn from the slab. A slight tamping of the cylinder releases the cut concrete core from the housing.

Figure 3.4: Core cutting process on concrete slab

The details of the cut cores and the groove were recorded in concrete core extraction record form (**Appendix A).**

The depth of the cut joint was analysed to establish if it met the criteria of 1/4 to 1/3 depth of the slab thickness. The level of development of guided cracks at the joints was recorded **(Appendix B**). Measurement undertaken on site showed that spacing of the cut joints (slab length) were within the 3.5m to 6 m range.

3.6.2.2 Investigation of Quality of Plain Jointed Concrete Slabs.

Establishment of the quality of concrete slabs was undertaken by determining the strength of concrete through compressive strength testing of solid cores, mix proportion of concrete and grading of aggregates making the concrete. Mix proportions were determined through chemical analysis of randomly selected crushed concrete cores while grading was done according to requirements of BS 8882:1992.

(i) Determination of Strength of Concrete (ASTM C42)

Concrete strength determination was based on solid concrete cores extracted from un-cracked concrete slabs within the research section. The concrete cores extracted were cleaned and marked with requisite core identification numbers (**Figure 3.5**).

Figure 3.5: Extracted, cleaned and marked concrete cores

The cores were measured and their length (depth) and other characteristics recorded in concrete extraction core record (**Appendix A**). The cores were tagged and put in polythene sample bags to preserve their insitu characteristics as they were transported to the Ministry of Transport and Infrastructure Laboratory in Nairobi where they were examined and ten solid cores selected, measured and recorded for strength testing.

The cores were trimmed lengthwise using a cutting machine to produce uniform length and flat faces suitable for compressive testing (**Figure 3.6).**

Core CA7 after extraction Core CA7 after Trimming to size

Figure 3.6: Sample concrete core

In order to unify the core surface conditions and balance their moisture condition, the trimmed core samples were soaked in water for a period of approximately 30 minutes after which they were surface dried and then crushed in a crushing machine **(Figure 3.7)** to get the concrete crushing strength**.**

The machine crushing load applied was recorded and converted to cube crushing strength in N/mm^2. Three crushed core samples were randomly selected and put into sample bags for further testing to establish constituent material proportions of concrete through chemical testing.

From the test cube strength results, concrete strength achieved from core samples were compared against designed/ expected strength. The quality of concrete making the concrete slabs in terms of concrete strength was hence determined by studying the obtained strength results.

Figure 3.7: Crushing concrete Cores.

(ii) Chemical Analysis to Determine Concrete Mix Proportions and Grading of Aggregates.(ASTM C114 & BS 882)

Three of the crushed cores were randomly selected for chemical analysis to determine the proportions of cement, sand and coarse aggregates. The samples were examined and their visual appearance recorded. Part of each sample was kept aside for reference and the rest was put in a muffle furnace and heated to 900^{o}C for at least 30 minutes. The sample was then allowed to cool, then broken down carefully not to break the aggregates. The pulverized samples were then riffled and samples for further testing taken.

The riffled samples were weighted to the nearest gram and put in 2 liter nikely beakers. One liter of water and 200 ml of concentrated hydrochloric acid were added and content stirred for at least one hour intermittently. The liquid was decanted into a Winchester bottle. The solid residue left in the nikely beaker was added 500 ml of distilled water and 200 ml of hydrochloric acid, stirred and gently boiled for at least one hour. The liquid after cooling was decanted into a Winchester bottle and the residue treated as previously detailed until the hydrochloric acid used was approximately equal to weight of sample. The residue finally obtained was washed overnight in a stream of water until it was free of acid. The residue was then oven dried.

The extracted liquid was well mixed and its volume determined. 10 ml of liquid was pipetted into a standard flask and volume made up to 100 ml by adding 90 ml of distilled water. 10 ml of EDTA buffer solution followed by 10 ml of Sodium Cyanide and upto 10 drops of solochrome indicator was added. The obtained solution **(Figure 3.8)** was then titrated with standard EDTA solution until the pink solution changed to pale clear blue.

The cement content was determined through equation (3.1):

$$C = X * Y * 0.8 \qquad (3.1)$$

Where: C – is the cement content in gms

X - is total volume of liquid extract

Y - is the volume of EDTA solution used

The assumption made is that the cement contained 63% of Calcium Oxide which is the case for class 42.5 N/mm^2 cement or OPC cement. According to site records, OPC cement was used. The solid residue was then sieved to separate course aggregates and fine aggregates (sand) depending on limiting sizes **(Figure 3.9).** According to the testing laboratory (MoT&I), past experiments had shown that in chemical analysis, 3% and 7% of aggregates and sand respectively are lost in the process of testing. Consequently the obtained weight of aggregates and sand were corrected by multiplying by factors of 100/97 and 100/93 respectively. Having obtained the quantity of cement, sand and aggregates in terms of weight, the ratios of cement: sand: aggregates were computed.

Figure 3.8: Solution used to determine the amount of cement in concrete

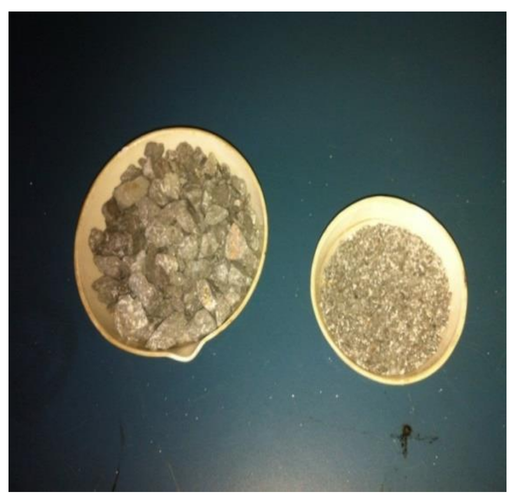


Figure 3.9: Coarse and fine aggregates

(i) Grading of Aggregates obtained from Chemical Analysis

To establish conformance of aggregates with grading standards for all in aggregates as specified in the Standard Specification for Road and Bridge construction in Kenya, the aggregates so obtained were combined and subjected to sieving using BS 882 complying sieves. The grading values were plotted against the specified grading envelop for all in aggregates. Study of curves was made to establish whether the aggregates complied with the size requirements of aggregates for use in concrete for concrete pavement slabs.

3.6.2.3 Assessment of Strength and Quality of Subbase and Subgrade Layers (ASTM D6951-03).

The parameters considered for assessment of subbase and subgrade layers were:-

(i) Strength of the layers expressed in CBR values.

(ii) Atterberg Limits and Plasticity Modulus (PM) for gravel subbase layer.

The procedures are detailed here below:-

3.6.2.3.1 Strength Determination of Subbase and Subgrade Layers – Dynamic Cone Penetrometer (DCP) Method (ASTM D6951-03).

DCP equipment from the MoT&I Laboratory was assembled on site. The assembled equipment comprised the handle, the hammer (8 kg), the anvil, the cone, upper and lower shafts and the ruler.

The cone, the 60° type, was inspected to ensure it was in good working state. The DCP tests were conducted on layers underlying concrete pavement slabs through the openings made by extracted concrete cores. The DCP tests were conducted immediately after the concrete core was extracted to avoid change of layer condition. Any water core cutting cooling water that had ponded into the cored space during coring operation was immediately dried out using cotton rugs. The DCP equipment was centered in the hole with cone tip at the center. The equipment was then held vertically at the handle by one person. The other operated the equipment by raising the hammer to the upper limit of the upper shaft and dropping it to hit the top of the anvil thus driving the cone into the soil. The third person read the penetration measuring scale (ruler) attached to the anvil parallel to the lower shaft and recorded the penetration data in DCP form **(Figure 3.10).**

Each drop of the hammer produced a corresponding penetration into the underlying layers which was recorded. Several hammer blows were applied to produce a discernible reading when the test layers become increasingly stiff. The number of blows for each penetration reading was recorded. The test was considered complete after the cone penetrated to the full depth of the lower shaft or at least 500 mm of the shaft had been achieved or the effort to drive the cone further into the soil was too high as to likely damage the cone.

The cone was extracted through gentle vertical blows to the handle. For successive tests the cone was examined to ensure that it was still in good state.

Figure 3.10: Field testing using DCP equipment

DCP data collected on site was recorded in DCP test forms. The data captured the location of test which correlated to the concrete core hole, the number of blows and corresponding penetration reading on the ruler scale. The zero blow scale reading was considered as the ruler reading when the cone was at the top of the layer immediately below the concrete slab and this was the start reading. A sample test form with entries is presented in **Appendix C.**

The data was further evaluated to provide Penetration Index (DPI) which is penetration in mm/blow. Invert cumulative reading of penetration per blow was tabulated jointly with DPI to facilitate computer generation of soil profile graph (**Appendix C, Figure 4.12**).

From the soil profile graph the depth of subbase and subgrade layers were isolated on consideration of the DPI transition zones. The DCP graph therefore helped in the determination of concealed pavement layer thickness of subbase and subgrade.

The strength of soil or pavement layer at any particular DPI point expressed as California Bearing Ratio (CBR) in percentage was computed from equation (3.2):

$$\text{Log CBR} = 2.46 - 1.12 \log \text{DPI} \qquad (3.2)$$

Values of CBR for subbase and subgrade were computed as the average value of CBR values corresponding to DPI points within established layer thickness of subbase and subgrade respectively.

3.6.2.3.2 Determination of atterberg limits of gravel subbase layer material to BS 1377 &1924.

Gravel samples from subbase layer were extracted from the layer immediately underneath the concrete slabs by chiseling and scooping out material through extracted concrete core openings. The gravel harvesting from cored holes followed immediately after the DCP test. The samples were put in tightly tied sample bags, labeled according to the source (hole/core) and transported for testing at MoT&I laboratories in Nairobi. Due to small samples extracted from each hole which were inadequate for individual sample quality tests, the samples were grouped based on sample source location. Samples from core holes within the same location were considered to be from same source. These samples were mixed together to produce six quantitative samples which were consequently graded and atterberg limits including Plasticity Index (PI) and Linear Shrinkage (LS) determined. Plasticity modulus (PM) was computed using equation (3.3):

$$\text{PM} = \text{Percentage passing sieve 425 Microns x PI.} \qquad (3.3)$$

Non-plastic samples had a PI of zero and hence did not have a PM value.

3.6.3 Assessment of pavement strength and stiffness and their likely contribution to crack development (ASTM D4694 & D46 95).

Stiffness of pavement can be determined by establishing the amount of deflection on application of standard load. The resultant deflection gives a means to evaluate the residual strength of pavement.

Falling Weight Deflection ((FWD) method was applied in the assessment of deflection of Jointed Plain Concrete Pavement.

FWD entailed dropping a standard load from a standard height. The dropped weight is matched to a typical wheel load standardized to 50 KN for FWD survey purposes. The load and loading mechanisms simulates the loading and loading time similar to that of a moving vehicle. Deflection measurements were taken from the loading point ($\mathbf{d_o}$) and at standard distances from $\mathbf{d_o}$ point using geophone probe sensors resting on the road surface and directly attached to loading plate (**Figure 3.11)**.

Figure 3.11: Geophone probe sensors on pavement surface

The geophones which span a distance from the loading points on a geophone frame enable a "deflection bowl" to be developed thus making it possible to evaluate a particular pavement structure for layer stiffness.

To start the FWD measuring process, inspection was made on the status of geophones, weight dropping mechanism and the computerized data recording devices all housed in the FWD van to ensure they were in good working conditions. Temperature of air was picked by sensors in the van while surface temperature of the pavement was taken using a surface probe thermometer. It is recognized that deflection of pavement is influenced by temperature variations. The weights on a platform in the van were then hydraulically dropped on a loading plate resting on pavement surface.

Measuring cycle consisted of 3 drops at each measuring point. The first drop is for adjustment of the plate so that it sits flat on the pavement surface while the other two drops were for loading and deflection measurements. The measured results were stored in the data collection file through a computer system connected to the loading and deflection measuring devices. The interval of FWD testing averaged a distance of 25 m on either lane.

The data collected was converted to deflection in mm and the deflection bowl for d_0, d_{200}, d_{300}, d_{600}, d_{900}, d_{1200}, d_{1500}, d_{1800} and d_{2100} recorded automatically. The subscript refer to the distance of probe from the dropped off point d_o. Other details obtained included pavement layer modulus and prevailing air and surface temperatures. Further analysis of the FWD data at the load drop off point (deflection data at d_o) was undertaken to establish sections of pavement with similar characteristics. Cumulative Sum Method (Cusum) on FWD data (d_o) was used. Cusum values were computed from equation (3.4):

$$Si = FWD_i - FWD\ mean + S_{i-1} \tag{3.4}$$

Where: S_i = Cumulative sum of deviation from mean deflection at point i, FWDi = FWD deflection at chainage i and

FWD mean = Mean FWD deflection of the road structure.

A sample cusum values calculation from FWD data is presented in **Table 3.1**.

From the cusum data a graph of cusum values against chainage was plotted (**Figure 3.12)** and subjected to further interpretation for pavement strength

Table 3.1: Cusum calculation on FWD central deflections (d_o)

Test No.	**Chainage From start point in meters**	**Deflection at loading point (d0) in Micrometer (a)**	**do- mean of deflection= (d0-∑d0/x). (b)**	**Cusum=cumulative sum of (b)**
1	0	170	-55	-55
2	5	234	9	-46
3	10	163	-62	-108
4	25	204	-21	-129
5	38	185	-40	-169
6	53	198	-27	-196
7	56	181	-44	-240
8	73	208	-17	-257
9	88	287	62	-195
10	97	192	-33	-228

A further evaluation of strength of pavement was undertaken by calculating the structural number which is the strength of pavement in terms of traffic carrying capacity. Three strength parameters or structural numbers were calculated as detailed below (Overlay Design Manual for Roads and Bridges part 4, 2009)

(i) Structural Number (SN) equation (3.5):

$$SN = 0.0394\ \Sigma(a_i\ h_i) \qquad (3.5)$$

Where: a_i = layer coefficient of layer i and

h_i = thickness of layer i in millimeters (mm)

Layer coefficients for the concrete pavement were considered to match that of Asphaltic Concrete (AC) layer surface (in absence of alternative reference) and were based on the stiffness of Asphaltic Concrete while for granular materials the layer coefficient was based on CBR values for stabilized base. Values of $\mathbf{a_i}$ for concrete and for subbase 0.35 and 0.2 respectively were adopted from Overlay Design Manual for Roads and Bridges part 4 (2009) on overlay designs in Kenya.

(ii) Modified Structural Number (SNC) equation (3.5):

The Modified Structural Number takes care of subgrade strength variations in determining the strength of pavement. SNC is computed through equation (3.6):

$$SNC = SNSG + SN \qquad (3.6)$$

Where: SNSG = Structural number contribution from subgrade computed from equation (3.7).

$$SNSG = 3.51 \log10 (CBR) – 0.85 (\log10 (CBR) 2) – 1.43 \qquad (3.7)$$

(iii) Adjusted Structural Number (SNP Existing)

This strength parameter is a contribution of all pavement layers including subgrade and it is wholly based on FWD data. It is suitable for assessing strength of existing pavements and the correlation is computed in equation (3.8):

$$SNP = 1.394 + 4.548\,(d_o*08)^{-0.5} - 1.760*\left[\frac{d_{900} - d_{1200}}{d_{900}}\right]^{-0.5} \qquad (3.8)$$

Where: d_o = Central deflection in (mm)- deflection at the load drop off point

d_{900} = Deflection at 900 mm from the load drop off point in mm

d_{1200} = Deflection at 1200 mm from the load drop off point in mm

Parameters of pavement strength calculated from the above equations are presented in **Appendix E.**

3.6.4 Establishment of how topography contributes to development of cracks in concrete paved roads.

The as built design drawings of the research road were studied and the design details of the study section identified and picked for analysis. A vertical profile of the section was plotted. The design allignment and the existing ground profile were superimposed. A table was drawn and details of chainages and length of sections between selected points along the profile and associated variation of vertical elevation between the points tabulated. The gradient of the designed profile within the selected points was computed as a percentage gradient by dividing the variation in height against variation in horizontal distance. From the Slab identification and cracks mapping record obtained from the ground survey the number of cracked slabs within the various identified points were enumerated for each lane. The quantum of cracked slabs against the length of sections for each lane were computed and recorded in units of slabs/m2. The Intensity of cracked slabs/m2 per unit gradient was then computed by dividing the intensity of cracked slabs against the gradient. Graphs of intensity of cracked slabs per unit gradient against % gradient were drawn for further assessment and interpretation.

3.6.5 Sealing of Core Holes from where the Cores were extracted

Core cutting is a destructive method of investigation as it permanently cuts off part of the pavement and exposes the supporting layers underneath to destructive agents like water. To safeguard the integrity of the pavement the core holes were backfilled with concrete of mix proportions 1:1:2 presumed to be class 30 (30 $N/mm^{2)}$ concrete. The concrete was filled into the holes in layers that were compacted using a tamping rod. Once full the surface was leveled and treated to match existing concrete pavement surface. The season during which the study was conducted was rainy and the concrete filled spots were naturally cured.

3.6.6 Application of Study Data and Test Results.

The information obtained from field, laboratory tests and desktop computations was further assessed to establish the correlationships of the various variables obtained in the study. These relationships have been presented as study results in chapter Four.

CHAPTER FOUR

RESULTS AND DISCUSSION

4.1 Extents of the cracks in concrete pavement

Most of the surface cracks were found to be on NE lane. The width of the cracks varied from 4.93 mm to a maximum of 8.75 mm with an average of 6.84mm. The intensity of cracks on the cracked concrete panels varied from 40 to 84mm/m^2. Cracks in the range of 5 mm are considered tight and may remain so for a long time (Chen & Won, 2007). Accordingly cracks of width less than 6 mm are considered of low severity and between 6mm and 19mm of moderate severity level (United States Department of Transportation, 2003). In this study, most cracks were found to be of low severity level, and only a few are within the moderate severity range. All cracks in the moderate range boarder on low severity boundary. The developed cracks are generally tight and of low severity range and therefore have negligible effect to performance of the pavement.

Study of cores cut at the saw-cut joints randomly selected revealed that:-

(i) A proportion of 37.5% of the cut joints induced full depth cracks at the cut grooves as intended (**Figure 4.1)**.

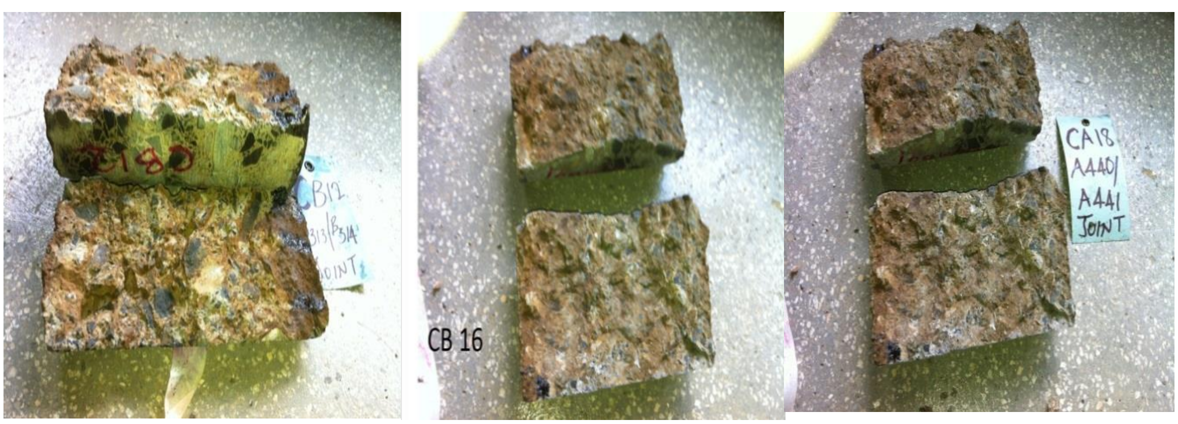
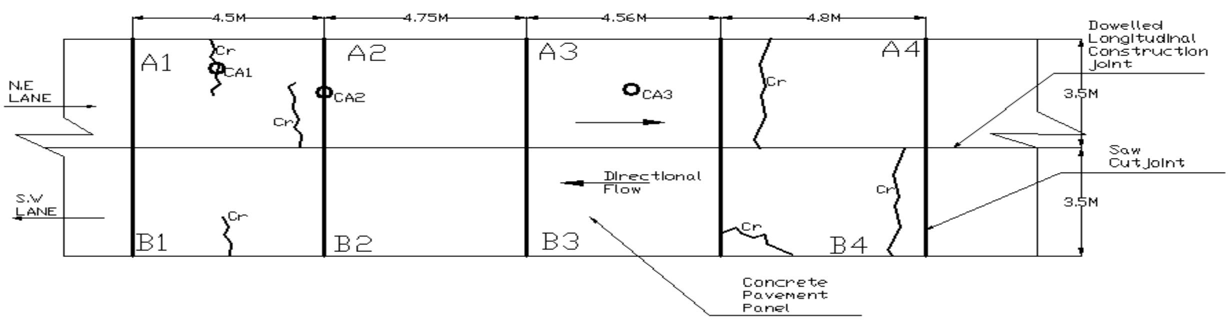


Figure 4.1: Full cracks development under saw-cut joints

(ii) Cracks developed on the surface of the slabs at varying distances from the cut groove and almost always parallel to the transversely cut joints. **Figure 4.2** shows two sets of cores that were cut from slabs at varying locations from the saw-cut joints. Cracks which developed on surface of the slabs are also illustrated. The two sets had concrete cores CA1, CA2, CA3 and CA7, CA8 and CA9 respectively. Cores CA2 and CA8 were extracted from the saw-cut joints, they are solid and hence the saw-cut joints did not induce the intended cracks.



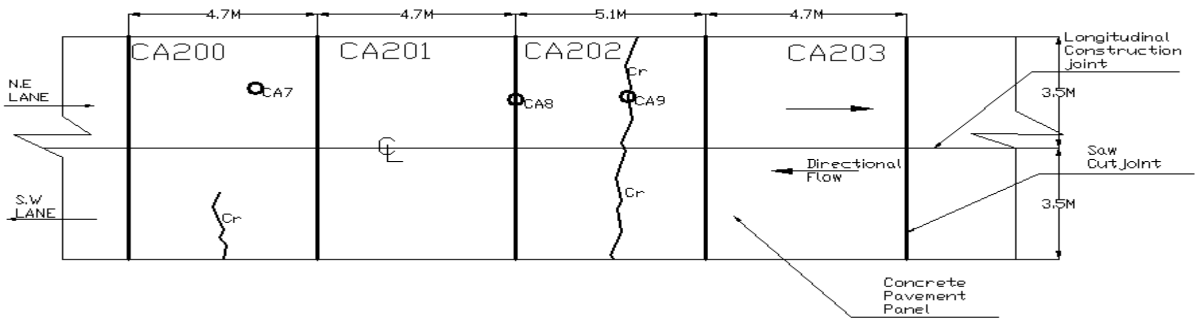


Figure 4.2: Cracking patterns in the experimental pavement

(iii) Surface cracks developed on thicker concrete slabs nearer to the cut joints than for otherwise thinner slabs (**Figure 4.3)**. The cut groove at the intended joint which unfortunately did not induce full depth crack could have weakened the slab at the point of the cut groove and hence the cracks could have concentrated at the weaker location of the thicker slabs. The shallow saw-cut joints were not effective in the induction of full depth crack and hence did not seem to have any significant influence on the location where the surface cracks on the slabs occurred (**Figure 4.4).**

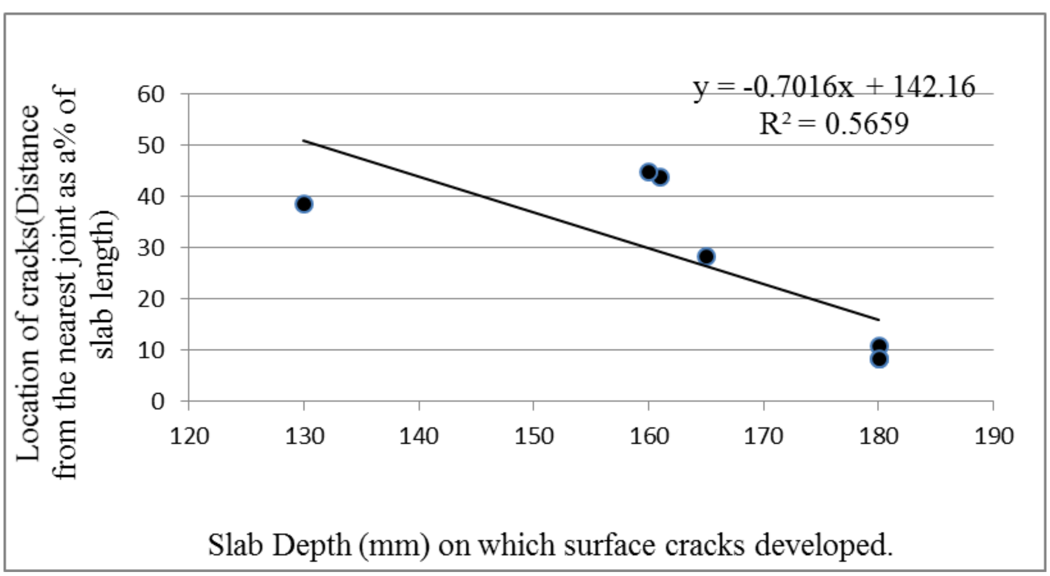


Figure 4.3: Slab thickness and location of surface cracks

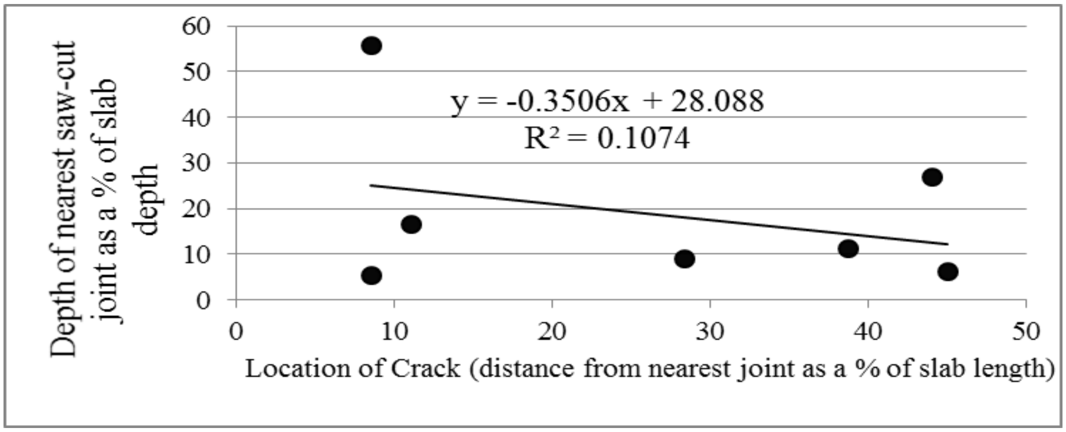


Figure 4.4: Cut joints and surface cracks development

4.2 Influence of saw cut joints and quality of concrete pavement on crack development

4.2.1 Influence of Saw-cut Joints on crack development.

A total of 208 (23.5%) slabs out of 886 slabs studied had at least one surface cracks which run parallel to the saw-cut joints (cut grooves) and transversely to the longitudinal construction joint which marked the center of the pavement. The cracked slabs exceeded the 5% percentile range acceptable in normal distribution. Status of cut joints from concrete cores cut at the joints and development of the intended cracks is presented in **Table 4.1**. Some cores were found to be honey combed at the lower depth. The effective thickness of such cores was considered to be to the extent of solid section of the core.

Table 4.1: Effectiveness of saw-cut joints

Concrete Core No.	Concrete Core Depth(mm)	Effective Core Depth(mm)	Depth of Saw-cut Joint (mm)	% Depth of Saw-cut Joint	Recommended cut depth of groove (mm)	Status of Crack Development
CA2	170	170	15	8.82	42.5 - 57	No crack
CA5	170	170	10	5.88	42.5 - 57	No crack
CA8	160	160	10	6.25	40 - 53	No crack
CB12	180	180	30	16.67	45 - 60	Full depth crack
CB13	170	100	15	15.00	25 - 33	No crack
CB16	180	140	15	10.71	35-47	Full depth crack
CB17	190	190	10	5.26	47.5 - 63	No Crack
CA18	145	145	15	10.34	36 - 48	Full depth crack

The study revealed that none of the saw-cut joints met the minimum recommended depth of cut joint of 1/4 to 1/3 of concrete slab thickness for it to be sufficient in inducing development of the crack at the joint (Yao & Weng, 2012). It was, however, observed that 37.5% of the test cores with insufficient saw-cut joints still induced cracks at the saw-cut joints. This is a deviation from the established norm and further research should be undertaken to establish the likely causes of this behavior. The lateral saw-cut joints were spaced at a distance of 4.65m on average which is a normal spacing for JPCP and this is not considered as an influencing factor in the development of surface cracks on the concrete slabs.

The failure of the saw-cut joints to induce full depth cracks must have been a contributory factor in the development of irregular surface cracks on concrete pavement slabs. A similar investigation conducted on Quinglian Highway (Yao & Weng, 2012) observed that the saw-cut joints satisfied the depth requirements and cracks developed below all cut joints and therefore the joints were discounted as the cause of the cracks.

4.2.2 Quality of concrete Pavement on crack development.

(i) Concrete strength and quality

Quality tests conducted on concrete pavement slabs were crushing test of concrete cores to establish insitu concrete strength and chemical tests to establish concrete mix proportions of aggregates and cement and the grading of the all in aggregates.

The crushing tests gave concrete strength values which varied substantially. None of the tested samples met the design strength threshold of 30 N/mm^2 **(Figure 4.5).** Forty percent (40%) of the concrete samples tested gave results of non-structural concrete (strength values below 15 N/mm^2). The average concrete strength achieved was 16.75 N/mm^2 which is only 56% of desired concrete strength.

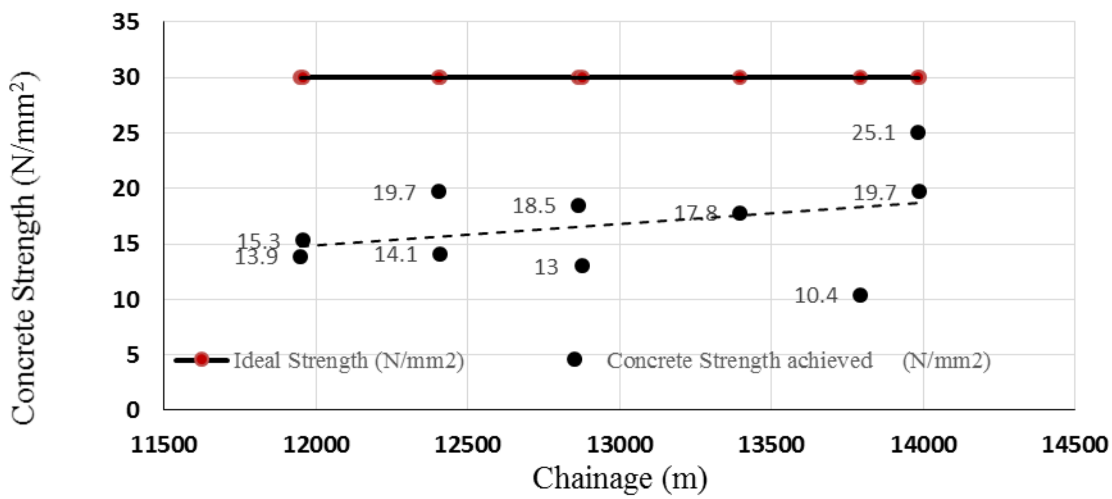


Figure 4.5: Concrete slab crushing strength

Concrete samples recovered from crushing of cores, CB6, CA8 and CB11 were collected separately and each subjected to chemical analysis to establish the proportions of solid ingredients of cement, fine and coarse aggregates in concrete. Recovered ingredients from concrete cores by chemical method and grading of aggregates using BS 882:1992 guidelines gave proportions presented in **Table 4. 2**.

Table 4.2: Concrete mix proportions

Core Sample Identity	Mix Proportions		
	Cement	**Sand**	**Aggregates**
CB6	1	1.94	2.91
CB8	1	2	2.91
CA 11	1	2.20	3.4

The approximate average proportions from the analysis is in the ratio of 1:2:3 (cement: fine aggregates: coarse aggregates). The trial mix proportions for class 30/20 concrete (ideal mix) conducted before construction was 1:1.45:2.7. Comparison of design mix proportions to actual mix proportions clearly indicate over application of fine aggregates by approximately 38% and coarse aggregates by 15%. Overall the aggregates were 23% more than required and therefore an equal proportion of cement was not applied. The proportions for ideal mix (mix design) are used to guide in manufacture of concrete. The ideal mix proportions were not adhered to during production and construction of the concrete pavement. This could have contributed to the development of the low strength of concrete. The low strength concrete may not have been strong enough to counteract stresses induced by vehicular and environmental loadings resulting to induction of cracks on the pavement slabs.

Aggregates that were chemically extracted from concrete cores did not meet the grading specifications. The grading curves fell below or on the lower limit curve indicating the aggregates were more course than required (**Figure 4.6**). The aggregates used therefore did not comply with aggregates grading requirements for concrete manufacture. Misappropriation of design mix proportions and apparent non-conformity to size specifications of aggregates could have been contributory to low insitu concrete slabs strength.

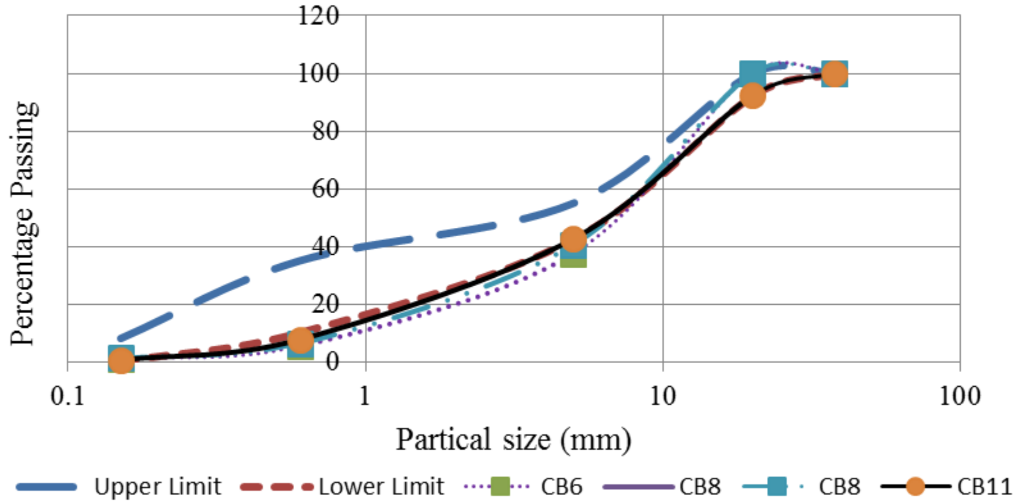


Figure 4.6: Grading curves of aggregates.

Evaluation of concrete strength and the intensity of the cracks on the surface of the concrete slabs showed that cracks intensity was inversely proportional to concrete strength (**Figure 4.7).**

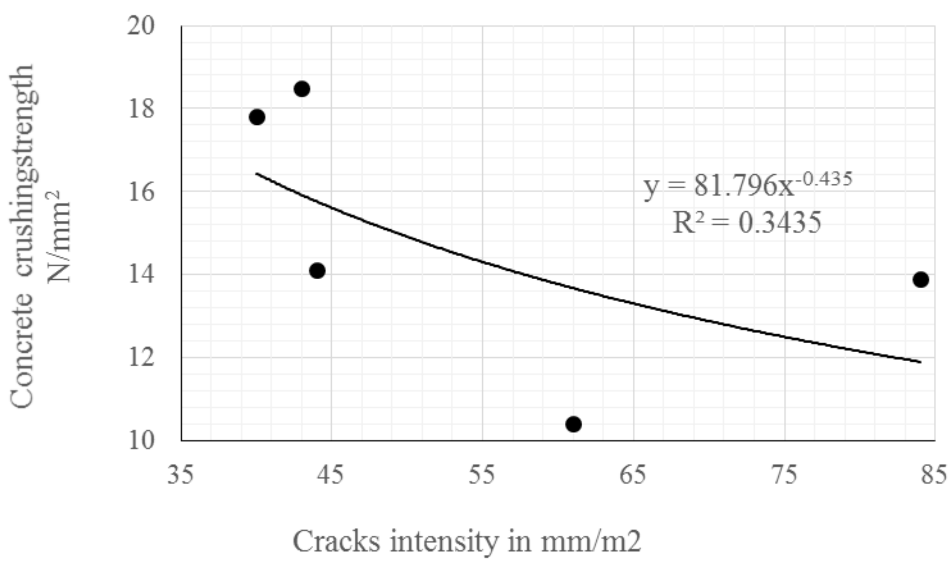


Figure 4.7: Concrete strength and intensity of cracks.

Construction materials of low quality and poor workmanship contributes significantly to development and propagation of pavement distress which include surface cracks as the constructed pavement does not meet the design strength and quality requirements to resist applied operational and environmental stresses (Yao & Weng, 2012; Hong et al, 1997). Quality of concrete pavement slabs in terms of compressive strength (**Figure 4.5**) and quality of constituent materials **(Table 4.2, Figure 4.6**) revealed that concrete did not meet specified strength, the proportions for design mix were not adhered to and the grading of aggregates was outside the specified grading curves (Ministry of transport and communication, Republic of Kenya. 1987) were not fulfilled. Some cores had honey combs which affects quality of end product. These unmet quality parameters revealed poor quality control during construction of the works resulting to a low quality product which once subjected to operational and environmental stresses could not resist development of cracks. It is most likely that non-conformance to specified concrete strength contributed to the generation and propagation of cracks reflected on the surface of concrete pavement.

(ii) Subbase and Subgrade Quality

The strength of subbase and subgrade layers was determined through DCP probe test values and converted to CBR values through application of requisite empirical equation. CBR values for subgrade layer (**Table 4.3)** were found to conform with CBR values of natural red coffee soil (**subgrade class S3 and S4: table 2.2**). The observed subgrade layer modulus from FWD test **(Figure 4.8)** varied between 49 and 385MPa**.** The modulus exceeded 39 MPa which is the desirable minimum strength for a suitable subgrade (Gaspard et al., 2013). The design records revealed that the pavement design was based on subgrade class S3 (CBR range 7-13). It was observed that subgrade met specified strength requirements and therefore did not influence the development of cracks on the concrete pavement slabs.

Table 4.3: Strength of subbase and subgrade layers

Chainage	DCP Probe Mark	Achieved strength (CBR values (%))			
		Subgrade Observed	Subgrade Specs	Subbase observed	Subbase (RDM Part 3)
11+950	CA1	12.64	Class S3 (CBR range 10 – 14) Average 10 Class S4 (CBR range 10-18) Average 14	20.78	60 minimum
11+959	CA3	17.5		35.7	
12+405	B100	9.5		60.6	
12+866	CA200	11.6		51.5	
13+393	B314/631S	16.57		69.16	
13+783	B400	14.98		65.1	

Minimum CBR value of 60% is specified for chemically improved subbase layer in Kenya. Tested CBR values for subbase layer varied significantly from those expected. It was observed that 50% of tests fell below the specified minimum strength. The subbase layer modulus varied from 122 to 12,260MPa. Quality neat gravel subbase layer would be expected to have a minimum layer modulus value of 345 MPa (Chen & Won, 2007). Improved gravel subbase layer would be expected to have higher values. The cement improved subbase layer had 24% of test sections with layer modulus values of less than 345MPa (**Figure. 4.9).** This is an indication of weak sections of subbase layer**.**

Sections of subbase layer with low CBR and low layer modulus values imply sections of soft layer formation which could have been as a result of poor workmanship, application of low quality material or there could be pockets of material with high water content.

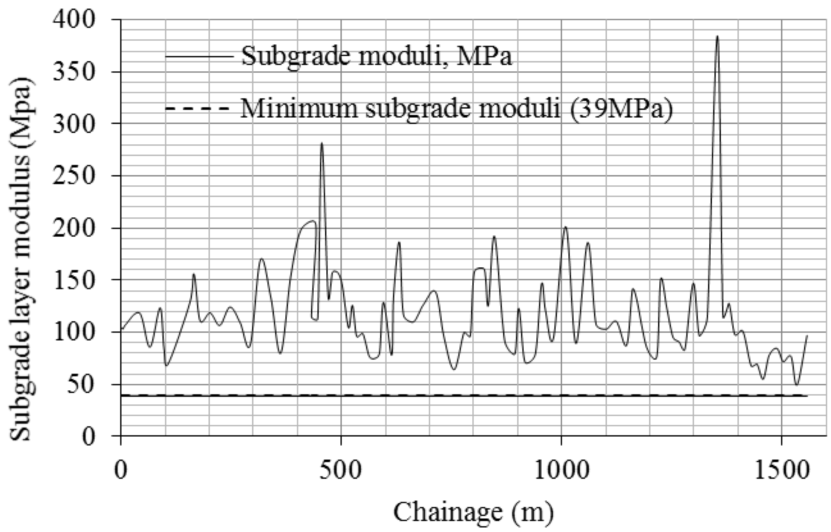


Figure 4.8: Variation of subgrade layer modulus along the research pavement section

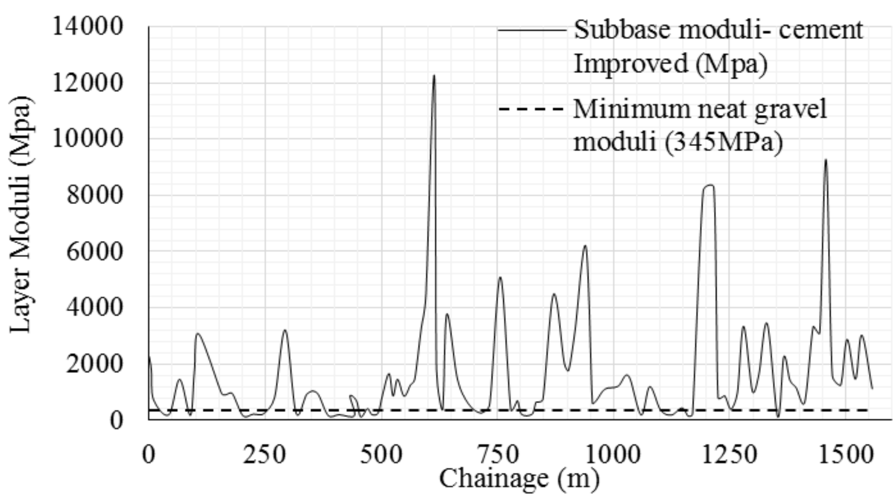


Figure 4.9: Variation of subbase layer modulus along the research pavement section

The road side drains of the experimental section were lined with stone pitching on the sides and concrete slab at the bottom. The drain had good gradients and it was clean with no siltation. The side drains therefore protected the pavement from external ingress of water from the sides. Consequently, wet subbase was discounted as a factor influencing the strength of the subbase layer on the assumption that the water content was controlled to specified limits during construction.

Grading and atterberg limit tests on subbase samples revealed that some of the subbase material had Plasticity Index (PI) of 17 and Plasticity Modulus (PM) of 374. These values were beyond the specified maximum PI of 15 and PM of 250 for improved subbase layer (**Table 4.4).** The high values gave an indication of weak subbase layer.

A comparison of the strength of subbase layer (CBR values) and the intensity of the cracks on the concrete pavement revealed that intensity of surface cracks increased with reducing strength of subbase layer (**Figure 4.10).**

Chen and Won (2007) observed that irregular surface cracks on concrete slabs whose supporting layer had a layer moduli of 345MPa and above remained tight even after decades of trafficking. It is apparent that the subbase layer on the research road did not fully comply with strength criteria in terms of CBR and Layer Moduli and to this extent the concrete slabs support was compromised. The weak subbase layer could have most likely contributed to cracks development in concrete pavement slabs.

Table 4.4: Insitu subbase layer grading and atterberg limits

Sample No	Sample Location	Grading								Atterberg Limits				
		BS Sieve Size								LL	PL	PI	LS	PM
		50 mm	28 mm	20 mm	10 mm	5 mm	2 mm	425 µm	75 µm	(%)	(%)	(%)	(%)	
1074	11950-11959	100	96	89	57	44	35	27	20	Non Plastic				
1075	12405-12410	100	86	71	55	38	30	22	16	48	31	17	9	374
1076	12867-12875	100	100	87	58	43	34	23	17	Non Plastic				
1077	13389-13394	100	90	80	50	33	24	16	12	Non Plastic				
1078	13783-13803	100	90	79	62	48	38	26	18	Non Plastic				
1079	13977-13986	100	100	98	69	55	43	20	15	Non Plastic				

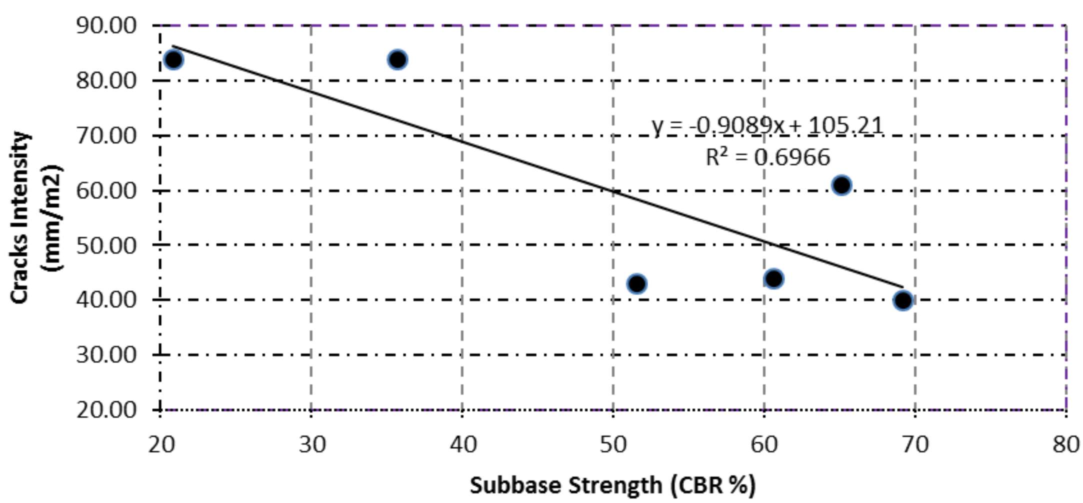


Figure 4.10: Effect of strength of Subbase layer to the development of cracks

4.3 pavement strength and stiffness in crack development

4.3.1 Pavement strength

Combined strength of concrete pavement slab, subbase and subgrade layers were assessed though FWD tests which analyses deflection induced to pavement by a standardized weight falling from a constant height. The overall strength of pavement is given by a computed parameter called Structural Number (SNP). Structural number computation considers deflection values in a bowl for a distance of 2100 mm from the point of falling weight (point 0). The SNP aggregates deflections from individual layers making the pavement. A high SNP value gives an indication of a strong pavement structure. Combined layer structural number for subgrade and subbase layers is denoted as SNC while subbase layer structural number alone is denoted as SN.

Parameters taken into account in Structural number computations include deflections of pavement at various points from weight drop off point (d_o), subbase thickness, CBR values of subbase and subgrade layers (**Table 4.5).**

Distances d_0, d_{900} and d_{1200} are distances of deflection probes from the falling weight point that are taken into account in computation of SNP. The variation of deflection along the pavement road at the drop off point is as presented in **Figure 4.10a**.

Assessment of how the overall pavement strength related to the cracks on the surface of concrete pavement slabs was evaluated by comparing width of cracks to SNP. The assessment revealed that the width of the cracks reduced in size as the SNP (overall pavement strength) increased **(Figure 4.11).**

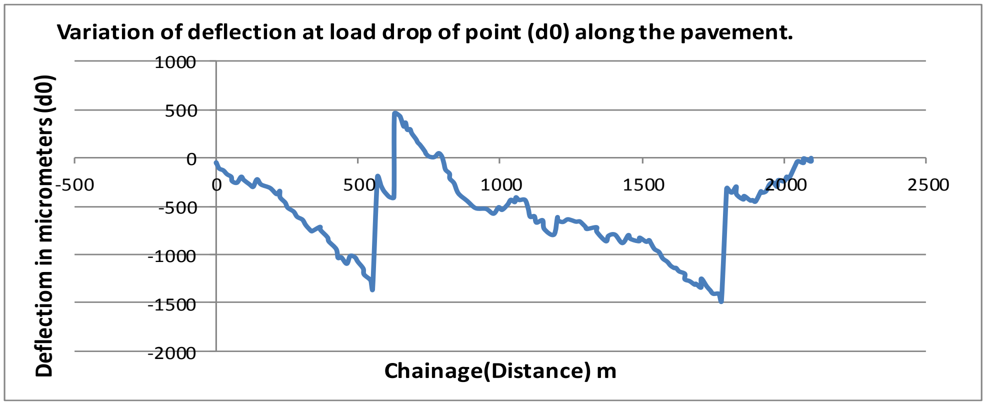


Figure 4.10a: Variation of pavement strength characteristics along the experimental road.

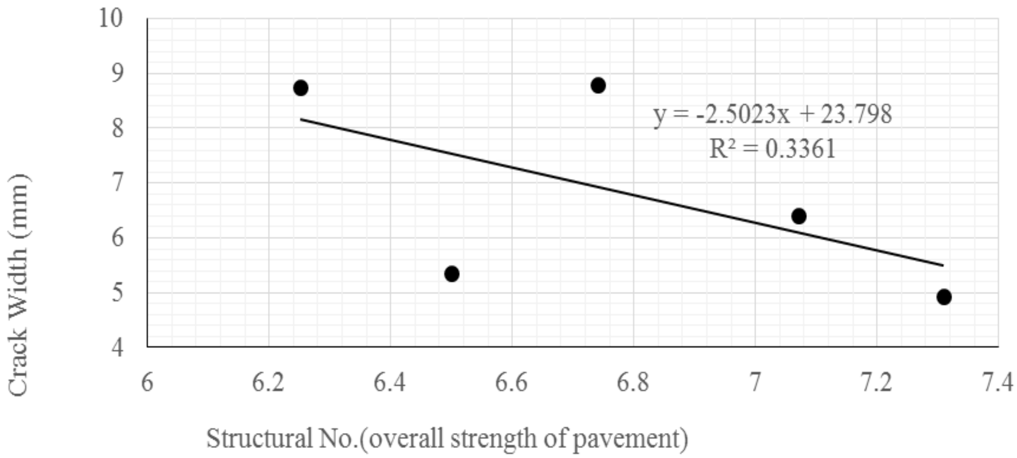


Figure 4.11: Strength of pavement and the width of cracks

From the relationship it can be inferred that the stronger the pavement the smaller the width of generated cracks. The generated cracks were relatively tight and may not have an effect on long term performance of the JPCP. The strength of entire pavement is a sum total of individual pavement layer strength, material quality and adherence to construction standards. Variations in homogeneity of pavement layer (**Figure 4.12)** portray changes in pavement characteristics which impacts on overall quality and strength of the pavement. The width of the non-uniform surface cracks on concrete pavement were found to be small but were established to run full depth in the concrete slab (**Figure 4.2)** .These cracks could pose potential danger to underlying pavement layers if water ingresses through the cracks to the subbase and subgrade layers. This would result to reduction of individual layer strength hence undermining the support to the concrete slabs thus and consequently inducing wider and more intensive cracks on JPCP on continuous application of load. It is therefore prudent to seal the cracks at the earliest opportunity when they develop.

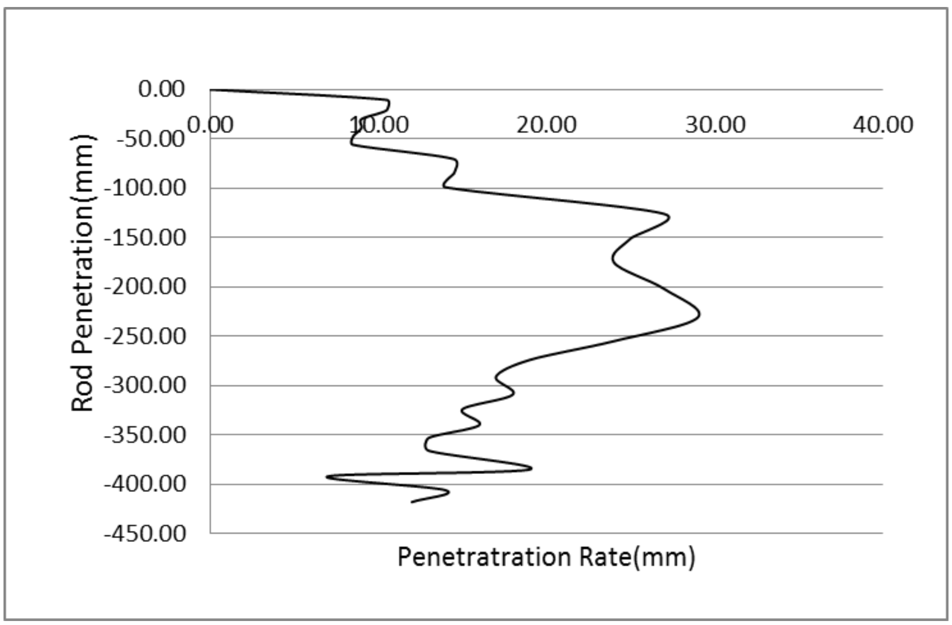


Figure 4.12: Typical Soil profile for the study site

4.3.2 Pavement Stiffness

(i) **Homogeneous sections of pavement.**

Assessment of deflection variations of pavement structure was conducted by application of FWD.. Other than deflection values, other values that were obtained from the FWD test include base and subbase moduli in Mpa, applied surface pressure in Kpa and air and surface temperatures.

From the available deflection data, at the drop off point (d_0), homogeneous sections (sections of pavement with similar characteristics) were determined through a plot of cusum graph (**Figure 4.13)**. Sections of similar or near similar graphical slope are interpreted as homogeneous.

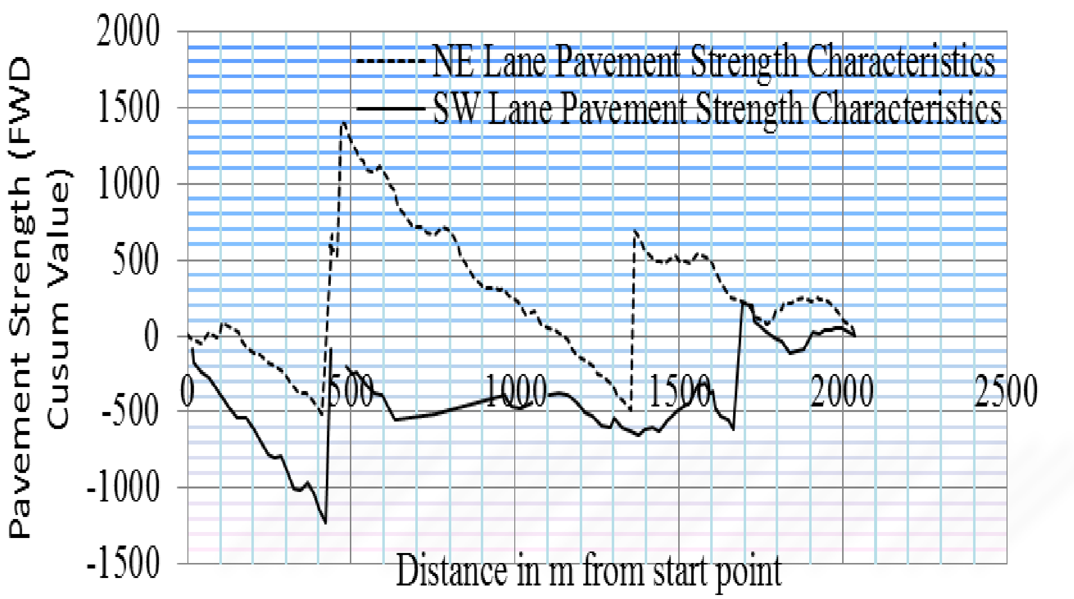


Figure 4.13: Graph showing homogeneous sections on NE and SW lanes

The homogenous sections have similar construction characteristics and are expected to behave in a similar way under similar conditions of exposure. On this presumption it was theorized that the crack characteristics on the pavement surface would then follow a similar pattern on the identified homogeneous sections. The sharp vertical jumps in the graph were found to be locations of steel dowelled construction joints on concrete pavement slabs.

Table 4.5: Variation of pavement strength in the experimental road

Test No.	Concrete slab no.	Test location chainage	FWD deflection values (mm)			Estimated depth of Subbase layer (mm)	CBR Subbase (%)	CBR (%) subgrade	Pavement strength values		
									SN	SNC	SNP
			d_0	d_{900}	d_{1200}				Subbase only	Subbase &subgradeDE grade	Overall strength
1	CA1	11950	0.202	0.12	0.107	126	20.8	12.6	0.5	1.06	7.36
2	CA2/CA3	11954	0.202	0.12	0.107	122	47	21.5	0.48	1.46	7.36
3	CA3	11959	0.163	0.119	0.111	138	35.7	17.5	0.54	1.36	7.2
4	CA101	12410	0.18	0.119	0.109	180	41.9	11.3	0.71	1.19	7.31
5	CA100/101	12405	0.18	0.119	0.109	169	80.4	9	0.67	0.96	7.31
6	B100	12405A	0.18	0.119	0.109	142	40.8	10	0.56	0.94	7.31
7	CA200	12866	0.208	0.152	0.138	176	51.5	11.6	0.69	1.19	6.74
8	CA201/202	12871	0.208	0.152	0.138	172	62.8	13.5	0.68	1.29	6.74
9	CA202	12871A	0.208	0.152	0.138	210	37.1	12.7	0.83	1.4	6.74
10	B314	13384	0.22	0.162	0.148	194	68.7	22.6	0.76	1.79	6.25
11	B315	13388	0.22	0.162	0.148	215	68.1	15.7	0.85	1.58	6.25
12	B313/314	13388A	0.22	0.162	0.148	120	40.2	8.6	0.47	0.74	6.25
13	B314/315	13393	0.22	0.162	0.148	162	69.2	16.5	0.64	1.41	6.25
14	B400	13783	0.194	0.134	0.124	246	65.1	15	0.97	1.67	6.5
15	B402	13786	0.194	0.134	0.124	145	115.2	610	0.57	4.18	6.5
16	B402/403	13793	0.194	0.134	0.124	192	164.3	11.7	0.76	1,26	6.5
17	B401/402	13793A	0.194	0.134	0.124	237	95.3	21.5	0.93	1.92	6.5

Falling Weight Deflectometer (FWD) test measures deflection of pavement under a standard falling weight and is a useful tool in characterizing overall pavement stiffness. This can usefully be used to assess extent of damage caused to pavement under applied loading and prevailing environmental conditions (Deblois et al, 2010: Suleiman et al, 2011). FWD Cusum plots generated from pavement deflection studies of the research section, portrayed different characteristics for each lane assessed **(Figure 4.13).** Construction record informs that the two lanes were constructed at different periods and most likely under different conditions. NE lane was constructed first and had higher variations in pavement characteristics and similarly a higher intensity of cracks than the SW lane **(Figure 4.18).** Considering the general learning curve and the process of constructing the NE lane, it could be that the workmanship and overall quality control of works had improved when constructing the SW Lane hence less variability in pavement characteristics as compared to the NE lane.

(ii) **Stiffness of pavement in relation to its strength**

Pavement deflection is a direct expression of its stiffness. Comparison of Pavement stiffness (deflection) and pavement overall structural number (SNP) revealed that deflection of the pavement reduced as the structural number (overall strength) of pavement increased (**Figure 4.14)**. It can therefore be stated that stiffness of Jointed Plain Concrete Pavement is a direct expression of its strength.

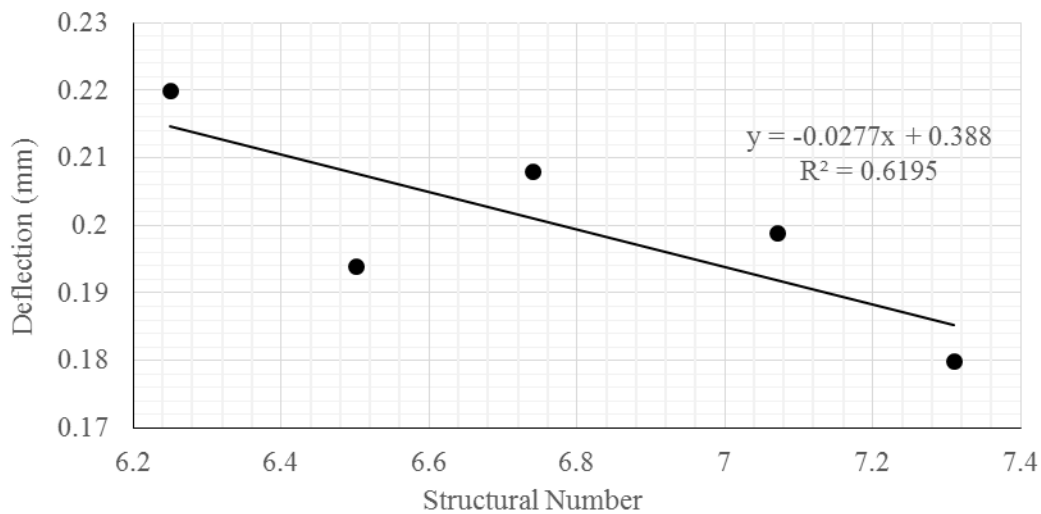


Figure 4.14: Pavement strength (SNP) and pavement deflection

(iii) Influence of concrete slab thickness on crack development

Thickness of concrete pavement contributes to the stiffness of the overall pavement and hence defines the strength regime of the pavement. Comparison of thickness of concrete pavement slabs measured from concrete cores to the crack intensity in locations of cores extraction indicated that surface cracks significantly reduced with increased slab thickness (**Figure 4.15).** A 20 mm increase in slab thickness reduced the cracks intensity by approximately 50%. Increasing slab thickness of JPCP increases pavement stiffness and hence reduces pavement deflection. It has further been established that crack width on the concrete pavement surface proportionately increased with pavement deflection (**Figure 4.16).** Adequate thickness of concrete slabs is a prerequisite in preventing premature development of surface cracks.

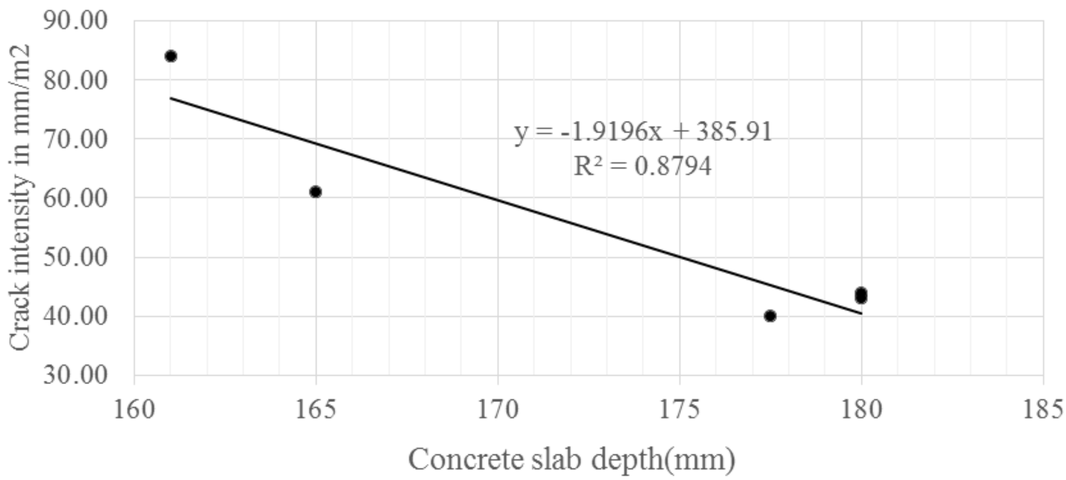


Figure 4.15: Effect of slab thickness to crack intensity

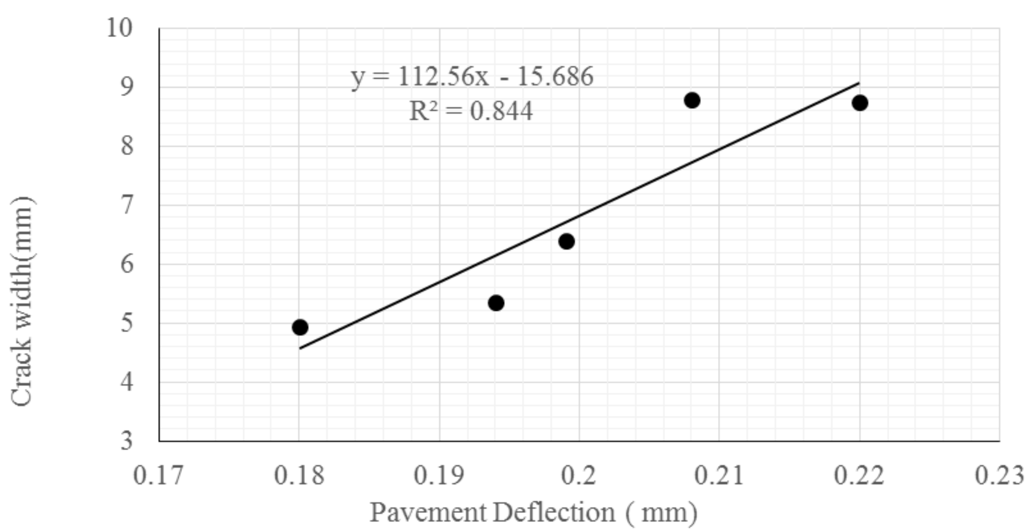


Figure 4.16: Pavement deflection and crack width on pavement surface

4.4 Pavement Gradients and Surface Cracks Development.

Assessment of vertical road alignment of the study section (**Figure 4.17, Table 4.6)** indicates that the alignment varies from negative to positive slopes on either lane and thus the lanes are well balanced in terms of vertical alignment variations. A study of the intensity of cracks on slabs in relation to alignment slopes indicated that the intensity of cracks reduced with increased vertical gradient (**Figure 4.18**). History of construction indicate that the NE lane was constructed first followed by the SW lane. It is considered that in the process of construction of the NE lane, the contractor and the supervision team went through a learning process and had acquired adequate skills in construction process of concrete pavement during the SW lane construction and hence the reduced crack intensity.

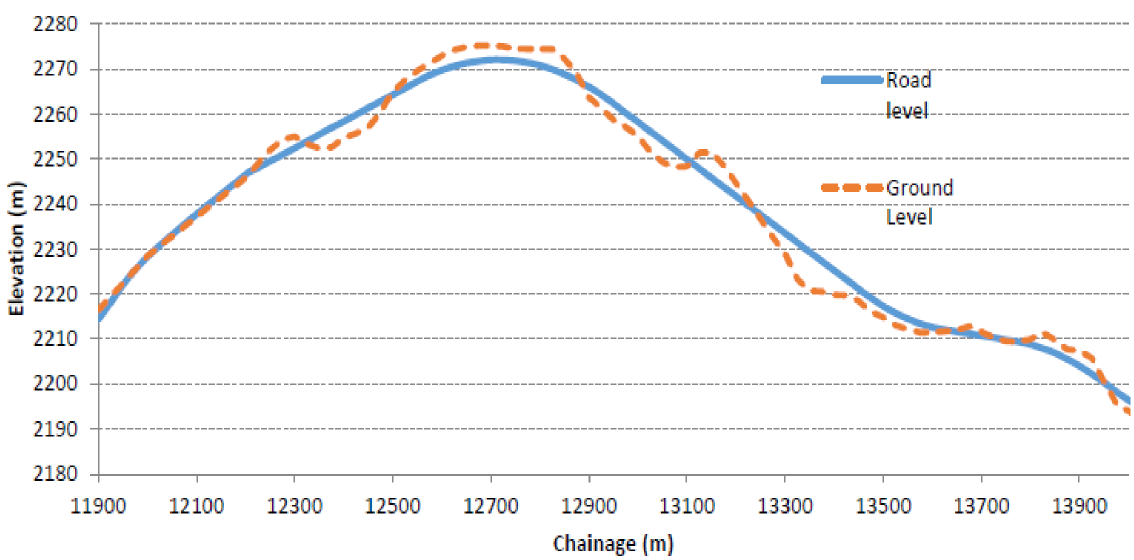


Figure 4.17: Vertical alignment of the Jointed Plain Concrete Pavement

Table 4.6: vertical alignment variables for topography related cracks assessment

Item No	Length of Section (m) from 11900.	Distance from start point(m)	Change in Level (m)	Gradient %	No. of cracked slabs NE Lane	No. of cracked slabs SW lane	Intensity Cracked slabs/m2 NE lane	Intensity Cracked slabs/m2SW lane	Intensity cracked slabs/m2 per unit gradient NE Lane	Intensity cracked slabs/m2 per unit gradient SW lane
1	400	400	27.1	6.8	32	18	0.01	0.01	0.17	0.09
2	275	675	1.4	0.51	16	4	0.01	0.00	1.63	0.41
3	625	1300	57.5	9.2	41	14	0.01	0.00	0.10	0.03
4	325	1625	6.9	2.12	19	11	0.01	0.00	0.39	0.23
5	275	1900	19.3	7	18	2	0.01	0.00	0.13	0.01

It can therefore be concluded that topography and vertical disposition of the alignment had direct influence in the development of the cracks. In sections with high vertical gradients it is highly likely that the road was generally in cut or had limited fill and the drainage was good and therefore the pavement foundation was on compacted insitu soils with minimum fills while on lower gradients the road was mainly in compacted fills. The firmness of the insitu ground and the good drainage therefore could have provided a firm and well drained road foundation resulting to a stiff pavement thus reducing the development of surface cracks. The made up fills could have undergone differential settlement over time thus influencing occurrence of surface cracks on concrete pavement slabs.

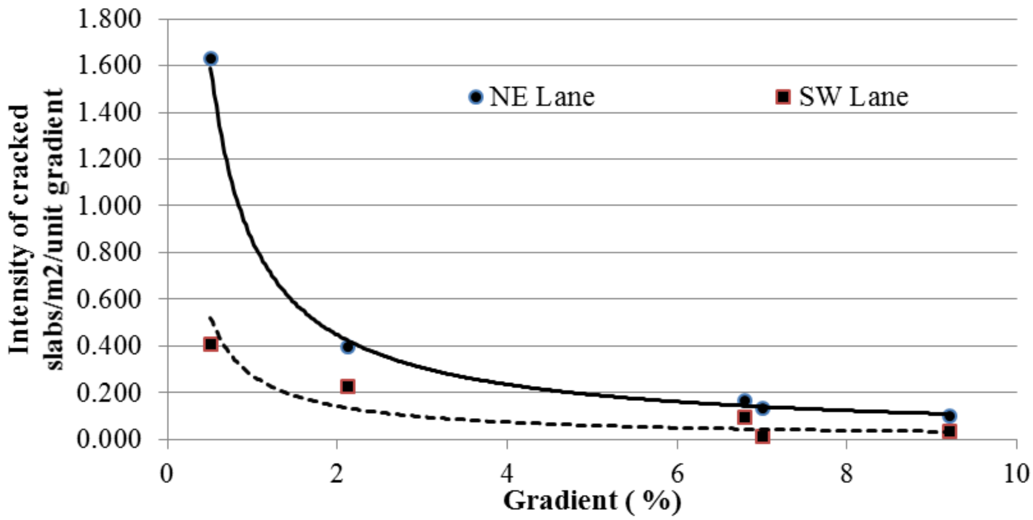


Figure 4.18: Variation of intensity of surface cracks with vertical gradient on NE & SW Lanes

Concrete pavements are selected for routes with high traffic (Robert, 1994), however, the research road has low volume of traffic and the main reason for selecting JPCP was on consideration of prevailing topography. The road alignment has high vertical gradients and use of conventional bitumen laying equipment was found not practical. It was therefore important to establish the influence of steep gradients on cracks development. The study established that steep gradients had no significant influence on surface cracks development.

From condition survey it was observed that the performance of the concrete pavement in its primary role of carrying traffic was good despite the existence of surface cracks. The pavement was found not to have developed faults or other distress signs other than the surface cracks. The cracks were found to be tight and did not seem to reduce the performance levels under the operating conditions. It is postulated that the pavement life would not be reduced by the current level of cracks as long as the operating conditions remained the same over the design life. The strength of concrete slabs averaged 16.5 N/mm^2 which was just above 50% of the designed strength. From the observed good performance of the pavement at the time of research it can be argued that the low strength concrete slabs did not affect the pavement performance in its primary role of carrying traffic. The design concrete strength could therefore have been an over design and a lower strength concrete could have been successfully used. It is however not clear how the low strength concrete pavement now in position will behave in the right of increased traffic with time and the cyclic stresses from environmental factors. A further study to evaluate the pavement performance might be necessary in the future..

CHAPTER FIVE

CONCLUSSIONS AND RECOMMENDATIONS

5.1 Conclussions

1. The cracks width varied between 4.93 to 8.75mm. The average crack width was 6.84mm which was within the moderate severity level range of 6 to 19mm. All the crack width were either on the low severity level bracket (< 6mm) or near the low intensity limit boundary. The cracks were therefore tight and had little or no adverse effect to the structural integrity of the pavement.
2. The depth of saw-cut joints was less than the recommended depth which should be at least 25-33% of concrete slab thickness. As a result, two thirds of the investigated joints were not effective in inducing a vertical crack at the saw cut joint, in order to prevent pavement surface cracking.
3. The subgrade layer of the experimental road had adequate strength (CBR of 9.5% to 16.6% and layer modulus greater than 39 MPa) and it is therefore unlikely to have contributed in development of surface crack on JPCP.
4. A significant proportion of the subbase layer tested did not meet the minimum strength (CBR of 60% and minimum layer modulus of 345 MPa) and could have contributed to induction of surface cracks on concrete pavement slabs.
5. The crushing strength of concrete slabs was below the mix design strength of 30N/mm2. This could have contributed towards increased surface cracking.

6. Most of the concrete slabs (95%) did not meet the design thickness, thus reducing the stiffness and overall strength of pavement thus compromising its response to cracking.

7. High grades were found to favor reduced incidence of surface cracks on concrete pavement in the study section. The topography on which the road traversed was generally hilly. Therefore it can be concluded that the high vertical gradients did not contribute in generation of surface cracks on concrete pavement slabs.

5.2 Recommendation

In order to ensure good performance of JPCP, it is imperative to ensure that the specified quality parameters are achieved. There should be continuous competence assessment of contractors, construction managers and quality control personnel and building up of capacity in new areas of construction.

Further studies should be undertaken to establish the combination of factors that would give a design that if implemented would give a sustainable pavement in terms of serviceability, structural adequacy and favourable in terms of life cycle costs.

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APPENDICES

Appendix A: Experimental concrete core extraction record

Core no.	Concrete slab no./Mark.	Location of core extraction (mark where appropriate)				Size of core		Details of crack development			Comments
		From solid slab.	From cracked	At the joint	At the crack	Diameter (mm)	Length(mm)	full crack developed	no crack developed	depth of cut groove(joi	
CA1	A1		√		√	100	161	√			
CA2	A1/A2			√		100	170		√	15mm	
CA3	A3	√				100	165		√		
CA4	A101		√		√	100	180	√			
CA5	A100/A101			√		100	170		√	10mm	
CB6	B100	√				100	180		√		
CA7	A200	√				100	140		√		
CA8	A201/A202			√		100	160		√	10mm	
CA9	A202		√		√	100	160	√			
CB10	B314		√		√	100	180	√			shear crack
CB11	B315	√				100	175		√		
CB12	B313/B314			√		100	180	√		30mm	lower 70mm H/C
CB13	B314/B315			√		100	170		√	15mm	
CB14	B400	√				100	165		√		
CB15	B402		√		√	100	180	√			
CB16	B402/B403			√		100	180	√		15mm	lower 40mm H/C
CB17	B401/B402			√		100	190		√	10mm	
CB18	A440/A441			√		100	145	√		15mm	
CA19	A441		√		√	100	130	√			shallow slab
CA20	A443	√				100	175		√		
CA21	A442		√			100	165		√		

Appendix B: Characteristics of cracks and cut joints

Slab Marks	Chainage		Distance from start point (m)	Total no of slabs	No of slabs with cracks	No of slabs with cracks LHS	No of slabs with cracks RHS	Total no. of cracks	Total length of cracks (m)	Average length of cracks per slab (m)	Average width of cracks (mm)	No of cores cut	Average Depth of cores. (mm)	No of core cut at joints	Has the crack developed at the joint (yes or no)	Depth of cut groove at joint (mm)	Effective depth of core for cracks at joints	Ratio of cut joint depth to effective depth of slab.
	From	To																
A1	11950	-		-		-	-	-	-	-	-	-	-	-	-	-		
A/B, 1– 23	11950	12050	100	46	18	14	4	29	58.52	2.02	6.4	3	165.3	1	No	15	170	0.09
A/B,24-45	12050	12150	200	44	16	9	7	23	59.4	2.58	7.3	-	-	-	-	-		
A/B,46- 67	12150	12250	300	44	15	10	5	26	69.4	1.58	9.4	-	-	-	-	-		
A/B,68- 88	12250	12350	400	42	16	10	6	20	52.1	2.64	15.25	-	-	-	-	-		
A/B,89-110	12350	12450	500	44	14	9	5	15	30.6	0.7	4.93	3	176.7	1	No	10	170	0.06
A/B, 90-131	12450	12550	600	42	5	5	0	7	17.2	0.41	5.29	-	-	-	-	-		
A/B,132-153	12550	12650	700	44	6	3	3	7	19	0.43	8.86	-	-	-	-	-		
A/B,154-175	12650	12750	800	44	8	7	1	8	23.5	0.53	9.13	-	-	-	-	-		
A/B,176-197	12750	12850	900	44	7	6	1	7	24.3	0.55	7.43	-	-	-	-	-		
A/B,198-218	12850	12950	1000	42	10	8	2	10	30.3	0.72	8.8	3	153.3	1	No	10	160	0.06
A/B,219-240	12950	13050	1100	44	6	5	1	9	23	0.52	10	-	-	-	-	-		
A/B,241-262	13050	13150	1200	44	9	5	4	9	23.4	0.53	8.67	-	-	-	-	-		
A/B,263-283	13150	13250	1300	42	10	8	2	12	36.2	0.86	7.83	-	-	-	-	-		
A/B,284-306	13250	13350	1400	46	13	10	3	14	48.5	0.05	6.64	-	-	-	-	-		
A/B,307-328	13350	13450	1500	44	4	2	2	4	13.5	0.31	8.75	4	176.3	1	Yes	30	180	0.17
A/B,307-328	13350	13450	1500	44	4	2	2	4	13.5	0.31	8.75	4	176.3	1	no	15	170	0.09
A/B,329-349	13450	13550	1600	42	9	6	3	9	28.1	0.67	6.33	-	-	-	-	-		
A/B,350-371	13550	13650	1700	44	3	1	2	3	10.5	0.23	8	-	-	-	-	-		
A/B,372-393	13650	13750	1800	44	12	8	4	13	38.1	0.87	7.77	-	-	-	-	-		
A/B,394-414	13750	13850	1900	42	12	9	3	14	43	1.02	5.36	4	133.75	1	Yes	10	190	0.05
A/B,394-414	13750	13850	1900	42	12	9	3	14	43	1.02	5.36	4	133.75	1	Yes	15	140	0.11
A/B,415-443	13850	13950	2000	58	15	14	1	24	73.8	1.27	4.33	4	153.75	1	Yes	15	145	0.10
TOTALS				**886**	**208**	**149**	**59**	**263**	**566.7**			**21**						

% TOTALS					**23.5**	**16.8**	**6.7**											

Appendix C: DCP Site data entry form

Test Location/ Test Identity	No of blows	Rod reading (mm)	Penetration (mm)
HOLE CA1	0.00	382.00	0.00
	2.00	403.00	21.00
	2.00	421.00	18.00
	2.00	438.00	17.00
	2.00	467.00	29.00
	1.00	481.00	14.00
	1.00	508.00	27.00
	1.00	533.00	25.00
	1.00	557.00	24.00
	1.00	584.00	27.00
	1.00	613.00	29.00
	1.00	637.00	24.00
	1.00	656.00	19.00
	1.00	673.00	17.00
	1.00	691.00	18.00
	1.00	706.00	15.00
	1.00	722.00	16.00
	1.00	735.00	13.00
	1.00	748.00	13.00
	1.00	767.00	19.00
	1.00	774.00	7.00
	1.00	788.00	14.00
	1.00	800.00	12.00

Appendix D : Thickness and CBR values of Subbase and Subgrade

No of blows	Rod reading (mm)	Blow units	Penetration/ blow in mm (DPI)	Invert Cumulative reading(mm)	(2.46-1.12log DPI)	Estimated CBR=10^2.46-1.12log DPI)	Average CBR	Thickness of base
0	382	0	0.00	0.00	0.00	0.00		
2	403	1	10.50	-10.50	1.32	20.71		
2	421	2	10.50	-21.00	1.32	20.71		
2	438	3	9.00	-30.00	1.39	24.62	20.78	126
2	467	4	9.00	-39.00	1.39	24.62		
1	481	5	8.50	-47.50	1.42	26.25		
1	508	6	8.50	-56.00	1.42	26.25		
1	533	7	14.50	-70.50	1.16	14.43		
1	557	8	14.50	-85.00	1.16	14.43		
1	584	9	14.00	-99.00	1.18	15.01		
1	613	10	27.00	-126.00	0.86	7.19		
1	637	11	25.00	-151.00	0.89	7.84		
1	656	12	24.00	-175.00	0.91	8.21		
1	673	13	27.00	-202.00	0.86	7.19		
1	691	14	29.00	-231.00	0.82	6.64		
1	706	15	24.00	-255.00	0.91	8.21		
1	722	16	19.00	-274.00	1.03	10.66		
1	735	17	17.00	-291.00	1.08	12.08		
1	748	18	18.00	-309.00	1.05	11.33	12.64	
1	767	19	15.00	-324.00	1.14	13.89		
1	774	20	16.00	-340.00	1.11	12.92		
1	788	21	13.00	-353.00	1.21	16.31		
1	800	22	13.00	-366.00	1.21	16.31		
		23	19.00	-385.00	1.03	10.66		
		24	7.00	-392.00	1.51	32.62		
		25	14.00	-406.00	1.18	15.01		
		26	12.00	-418.00	1.25	17.84		

Appendix E: Pavement strength parameters from FWD data

No.	Slab No.	Test Location Chainage	Distance From 11+950	Fwd Distances(mm)			Estimated depth of Subbase Layer(mm)	CBR Subbase (%)	CBR Sub-Grade (%)	State of Cored Test Location	Pavement Strength		
				D0	D900	D1200					SN (Subbase Only)	SNC Subbase And Subgrade	SNP Adjusted Structural Number (Existing Strength)
1	CA1	11+950	0	0.202	0.12	0.107	126	20.78	12.64	at crack	0.50	1.06	7.36
2	CA2/CA3	11+954	4	0.202	0.12	0.107	122	47.01	21.5	at joint	0.48	1.46	7.36
3	CA3	11+959	9	0.163	0.119	0.111	138	35.7	17.5	solid	0.54	1.36	7.20
4	CA101	12+410	460	0.18	0.119	0.109	180	41.9	11.3	at crack	0.71	1.19	7.31
5	CA100/101	12+405	455	0.18	0.119	0.109	169	80.4	9	at joint	0.67	0.96	7.31
6	B100	12+405A	455	0.18	0.119	0.109	142	40.75	10	solid	0.56	0.94	7.31
7	CA200	12+866	916	0.208	0.152	0.138	176	51.5	11.6	solid	0.69	1.19	6.74
8	CA201/202	12+871	921	0.208	0.152	0.138	172.3	62.8	13.5	at joint	0.68	1.29	6.74
9	CA202	12+871A	921	0.208	0.152	0.138	210	37.1	12.7	at crack	0.83	1.40	6.74
10	B314	13+384	1434	0.22	0.162	0.148	194	68.7	22.6	at crack	0.76	1.79	6.25
11	B315	13+388	1438	0.22	0.162	0.148	215	68.1	15.7	solid	0.85	1.58	6.25
12	B313/314	13+388A	1438	0.22	0.162	0.148	120	40.15	8.64	at joint	0.47	0.74	6.25
13	B314/315	13+393	1443	0.22	0.162	0.148	161.5	69.16	16.51	at joint	0.64	1.41	6.25
14	B400	13+783	1833	0.194	0.134	0.124	245.7	65.1	14.98	solid	0.97	1.67	6.50
15	B402	13+786	1836	0.194	0.134	0.124	145	115.23	610	at crack	0.57	4.18	6.50
16	B402/403	13+793	1843	0.194	0.134	0.124	192	164.3	11.74	at crack	0.76	1.26	6.50
17	B401/402	13+793A	1843	0.194	0.134	0.124	237	95.32	21.53	at joint	0.93	1.92	6.50

Appendix F: Letter from BPS Approving Research Proposal and Supervisors

JOMO KENYATTA UNIVERSITY
OF
AGRICULTURE AND TECHNOLOGY

DIRECTOR, BOARD OF POSTGRADUATE STUDIES

P.O. BOX 62000
NAIROBI – 00200
KENYA
Email: director@bps.jkuat.ac.ke

TEL: 254-067-52711/52181-4
FAX: 254-067-52164/52030

REF: BPS/ EN352-0768/2013 **27th August, 2014**

Mr. James N. Mwitari
C/o SMARTEC
JKUAT

Dear Mr. Mwitari

RE: APPROVAL OF RESEARCH PROPOSAL AND SUPERVISORS

Kindly note that your M.Sc. research proposal entitled: **"Establishment of the nature and extent of cracks on concrete pavement and determination of culpability of joints and pavement support layers on the development of the cracks: A case study of Kagere-Ndunyu-Munyane-Gituiga (E571) Road, Othaya Constituency"** has been approved. The following are your approved supervisors:-

1. Prof. Wambua Kaluli
2. Eng. Charles Kabubo

Yours sincerely

PROF. MATHEW KINYANJUI
DIRECTOR, BOARD OF POSTGRADUATE STUDIES

Copy to: Director, SMARTEC

/pw

JKUAT is ISO 9001:2008 Certified
Setting Trends in Higher Education, Research and Innovation

Appendix G: Authority to undertake Research on the road by KeRRA

KENYA RURAL ROADS AUTHORITY

RESIDENT ENGINEER
KAGERE – MUNYANGE - GITUGI
KAIRO – KIRIAINI ROADSPROJECT
CONTRACT No. RWC 001
P.O. BOX 411-10106,
OTHAYA

Our Ref.:KMGK/RWC001/1/253

4th April, 2014

Eng. James Ndoria Mwitari
SMARTEC, JKUAT
P.O. Box 62000-00200
NAIROBI

RE: CONSTRUCTION OF KAGERE–MUNYANGE–GITUGI (E571), NDUNYU–MIRIINI GITUIGA (E1685) & KAIRO–KIRIA-INI (D428) ROADS PROJECT. CONTRACT No. RWC 001

AUTHORITY TO DO RESEARCH ON CONCRETE PAVEMENT ON KAGERE – NDUNYU – MUNYANGE – GITUIGA ROAD – OTHAYA CONSTITUENCY

Reference is made to the Director General's Letter Ref. No. **KeRRA/20/1** dated **1st April, 2014** addressed to the Director SMARTEC, JKUAT and copied to you on the above captioned subject *(copy attached)*.

Through the letter under reference, the Director General granted consent for you to undertake research on the cracked sections of the concrete pavement along the aforementioned road.

The purpose of this letter is therefore to forward the requested details to you to assist in carrying out the proposed research between **Km 13+000** to **Km 14+000** of the aforementioned road.

Eng. J. N. Githui
RESIDENT ENGINEER

RESIDENT ENGINEER
KAGERE - NDUNYU - GITUGI &
KAIRO - KIRIAINI ROADS PROJECT
CONTRACT No. RWC 001

c.c. General Manager (Design & Construction) KeRRA – *for information*

Appendix H: Letter releasing requested details from KeRRA

KENYA RURAL ROADS AUTHORITY

RESIDENT ENGINEER
KAGERE – MUNYANGE - GITUGI
KAIRO – KIRIAINI ROADSPROJECT
CONTRACT No. RWC 001
P.O. BOX 411-10106,
OTHAYA

Our Ref.: KMGK/RWC001/1/253

4th April, 2014

Eng. James Ndoria Mwitari
SMARTEC, JKUAT
P.O. Box 62000-00200
NAIROBI

RE: CONSTRUCTION OF KAGERE–MUNYANGE–GITUGI (E571), NDUNYU–MIRIINI GITUIGA (E1685) & KAIRO–KIRIA-INI (D428) ROADS PROJECT. CONTRACT No. RWC 001

AUTHORITY TO DO RESEARCH ON CONCRETE PAVEMENT ON KAGERE – NDUNYU – MUNYANGE – GITUIGA ROAD – OTHAYA CONSTITUENCY

Reference is made to the Director General's Letter Ref. No. **KeRRA/20/1** dated **1st April, 2014** addressed to the Director SMARTEC, JKUAT and copied to you on the above captioned subject *(copy attached)*.

Through the letter under reference, the Director General granted consent for you to undertake research on the cracked sections of the concrete pavement along the aforementioned road.

The purpose of this letter is therefore to forward the requested details to you to assist in carrying out the proposed research between **Km 13+000** to **Km 14+000** of the aforementioned road.

Eng. J. N. Githui
RESIDENT ENGINEER

RESIDENT ENGINEER
KAGERE - NDUNYU - GITUGI &
KAIRO - KIRIAINI ROADS PROJECT
CONTRACT No. RWC 001

c.c. General Manager (Design & Construction) KeRRA – *for information*

Appendix I: Test results from MoR&I Laboratory on

(1)Concrete crushing strength Test Results

MOR/QMS/MD 368

MINISTRY OF TRANSPORT & INFRASTRUCTURE

Telegraphic Address: "MINWORKS", Nairobi
Telephone: Nairobi 554950/3/4 Fax: 554877
E-mail: chief.engineer@materials.go.ke
If calling or telephoning ask for
When replying please quote

MATERIALS TESTING & RESEARCH DEPARTMENT
MACHAKOS ROAD
INDUSTRIAL AREA
P. O. Box 11873
NAIROBI

Client Name: **James Ndoria Mwitari PO Box 393-00618 I**
Job Name: **Kagere Munyange - Githiga (E571) Rd**
Structure: **Concrete Road**
Our Job No. **001430/B/14**

Ref. No. **M.0362/35/J/13**
Concrete Mix:
Date core casted:
Date of test: **23/6/2014**

Laboratory sample No.	717	718	719	720	721	722		
Identification mark	CA3	CA2	CA20	CA8	CA5	CB6		
Age of concrete at test (days)	OVER 28 DAYS							
Average diameter (mm)	99	99	99	99	99	99		
Cross section area (mm^2)	7,698	7,698	7,698	7,698	7,698	7,698		
Length of core (mm) as received (max)	160	170	164	160	165	180		
(min)								
After trimming	100	100	100	107	100	100		
(min)								
After capping								
Density (Kg/m^3) as received								
Saturated	2450	2420	2500	2430	2420	2490		
Materials used for capping								
Failing Load (kN)	128.2	116.1	164.7	108.4	117.7	164.4		
Measured core strength (N/mm^2)	16.6	15.1	21.4	14.1	15.3	21.4		
Length: diameter ratio	1	1	1	1.07	1	1		
D (2.3/2.5) Factor. 2.3	0.92	0.92	0.92	0.94	0.92	0.92		
In-situ cube strength (N/mm^2)	15.3	13.9	19.7	13	14.1	19.7		
Mode of failure	STF	STF	STF	STF	STF	STF		
Type of aggregate	CRUSHED							
Maximum size aggregate (mm)	20	20	20	20	20	20		
Distribution of materials	UNIFORM							
Compaction: Excess voidate (%)	0.5	0.5	0.5	0.5	0.5	0.5		
Reinforcement present (Yes/No)	No	No	No	No	No	No		
Diameter of bar (mm) or,								
Diameter of core (mm) 0c								
Length of capped core (mm) l								
Distance of axis of bar (mm) d								
Correction for reinforcement								
Photograph reference number								
In-situ cube strength (N/mm^2)	15.3	13.9	19.7	13	14.1	19.7		

REMARKS: Cores cut by the client

CHIEF ENGINEER MATERIALS
MINISTRY OF TRANSPORT AND INFRASTRUCTURE
P. O. Box 11873-00400
NAIROBI

Eng. S.K Kogi
CHIEF ENGINEER (MATERIALS)

(2)- CONCRETE CORE CHEMICAL ANALYSIS RESULT

MINISTRY OF TRANSPORT AND INFRASTRUCTURE
STATE DEPARTMENT OF INFRASTRUCTURE

Telegraphic Address: "MINWORKS", Nairobi
Telephone: Nairobi 554950/3/4 Fax: 554877
E-mail: chief.engineer@materials.go.ke
If calling or telephoning ask for
When replying please quote

MATERIALS TESTING AND RESEARCH DEPARTMENT
MACHAKOS ROAD
INDUSTRIAL AREA
P. O. Box 11873
NAIROBI

Date
8 July, 2014

Mr. Mwitari

CHEMICAL ANALYSIS OF CONCRETE CORES

The samples you submitted gave the results shown below:-

Your Ref	Mixing Proportion (Ratio)				
	Cement	:	Sand	:	Aggregates
SOLID CB6 (i)	1	:	0.9	:	3.7
SOLID CB6 (ii)	1	:	0.8	:	4.3
CB 11 (i)	1	:	1.1	:	3.5
CB 11 (ii)	1	:	1.2	:	5.4
CA8 JOINT A201/A202 (i)	1	:	0.9	:	3.7
CA8 JOINT A201/A202 (ii)	1	:	0.8	:	4.3

J. N. Ngurungu
Section Head- Chemistry Laboratory

(3) – CONCRETE MIX DESIGN FOR CLASS 30/20 N/mm^2

MINISTRY OF ROADS AND PUBLIC WORKS

CONCRETE CUBE SUMMARY FOR LAB DESGN WORKS

Date of casting	Location & chainage Km	Cube marking	Age at test (days)	Slump (mm)	Weight of cube (g)	Area mm^2	Density (Kg/m^3)	Load (KN)	Compressive Strength (N/mm^2)	Class of concrete	Specification N/mm^2	Remarks
8.05.10	LAB MIX	47	28	67	8510	22500	2521	985	43.8	30/20	30	
"	"	48	28	67	8505	22500	2520	1000	44.4	"	30	
"	"	51	28	78	8515	22500	2523	955	42.8	"	30	
"	"	52	28	78	8490	22500	2516	960	42.7	"	30	

N/B

CUBE NO 47 & 48:
- W/C – 0.48.
- CEMENT – 440 Kg/m3.
- WATER – 211 Kg/m3.
- SAND – 612 Kg/m3.
- 6/14 mm – 682 Kg/m3.
- 14/20 mm – 455 Kg/m3.

CUBE NO 51 & 52:
- W/C –
- CEMENT – 420 Kg/m3
- WATER – 197 Kg/m3
- SAND – 624 Kg/m3
- 6/14 – 695 Kg/m3
- 14/20 – 464 Kg/m3

China Overseas Engineering Group Co., Ltd
COVEC

Senior Tech. [signature] 06:06:10

Material Engineer [signature]

Site Agent: [signature]

Senior Tech.

Material Engineer [signature]

Resident Engineer:

82

MINISTRY OF TRANSPORT AND INFRASTRUCTURE

MATERIALS TESTING & RESEARCH DEPARTMENT

P.O.Box 11873 - 00400 NAIROBI. Tel 554950/3/4, Fax 554877, e-mail chief.engineer@materials.go.ke

Test Certificate for Soil samples

(Testing as per BS 1377 & 1924 :1990)

CLIENT: JAMES NDORIA MWITARI

ADDRESS P.O BOX 393 00618 NAIROBI

MOTI/I/QMS/MD 038

Ref No: M.4529/35/L/1

Date: 4th July,2014

Email: ndoriamwitari@yahoo.co.uk **Phone:** 0723850838

Sample No	Reference	Grading								Atterberg Limits					Compaction T 99		Compaction T 180		CBR (%)		
		BS Sieve Size								LL	PL	PI	LS	PM	MDD	OMC	MDD	OMC	4 days	7 days cure	Swell
		50 mm	28 mm	20 mm	10 mm	5 mm	2mm	425 µm	75 µm	(%)	(%)	(%)	(%)		(Kg/m^3)	(%)	(Kg/m^3)	(%)	soak	7 day soak	
1074/S/2014	C B6,A100/A101,C5	100	96	89	57	44	35	27	20	NON PLASTIC											
1075/S/2014	C A2,S/B,Joint A1/A2,C A4 ,A101 Crack.C A3 A3 5B-C A1,S/B	100	86	71	55	38	30	22	16	48	31	17	9	374							
1076/S/2014	C A21,A402,Multiple Crack S/B.C A19,A 441 Multiple Crack S/B,C A20,aA443,SOLID S/B.A A18 A440/A441 Joint S/B	100	100	87	58	43	34	23	17	NON PLASTIC											
1077/S/2014	C A7,A200 S/B SOLID.C AqA2O2 S/B Crack.CA8,A201/A202 SB JOINTCB14,B400 SOLID S/B .C B15,B402 CRACK S/B	100	90	80	50	33	24	16	12	NON PLASTIC											
1078/S/2014	C B17,B 401/B 902 JOINT S/B.CB16,B402/B405 JOINT	100	90	79	62	48	38	26	18	NON PLASTIC											
1079/S/2014	C B11,B315 Solid,C B12,B313/B314 Joint,C B10,B314 Crack Shear,C B13,B34/B 315 Joint	100	100	98	69	55	43	20	15	NON PLASTIC											

CHIEF ENGINEER MATERIALS
MINISTRY OF TRANSPORT AND INFRASTRUCTURE
P. O. Box 11873-00400
NAIROBI

CHIEF ENGINEER (MATERIALS)

PROJECT NAME : Kagere-Ndunyu-Munyange-Gituiga(E571) Road

Date of sampling : Not provided

Sampling Location : Not provided

Sampled & Submitted By : Client

JOB CARD No. 01429/S/14

RECEIPT No.4297140 of 16/06/2014

AMOUNT PAID: Kshs 7,500

(4) :-SELECTED FWD DATA SAMPLE											
CHAINAGE IN M FROM ZERO POINT		DEFLECTION			TEMPERATURE			PRESSURE APPLIED	LAYER MODULI		
	D0	D300	D900	D1200	Air	Surface	Pavement	Pressure KPa	Base moduli, MPa	Subbase moduli, MPa	Subgrade moduli, MPa
0.0	234	175	113	97	17.4	20	21	613	8526	2266	105
4.5	204	172	124	110	18.2	20.1	21	612	12599	1787	103
9.0	198	175	130	117	18	20.4	21	623	14901	746	107
42.1	181	165	131	121	17.8	20.7	21	647	21558	198	118
65.5	287	214	150	132	18	20.8	21	621	7439	1458	86
88.5	180	162	131	122	17.9	20.7	21	611	18483	177	124
97.1	233	200	151	135	18	20.7	21	644	11589	1705	85
106.3	298	229	159	138	18	20.5	21	609	8285	3083	69
156.0	162	141	101	90	17.7	20.5	21	659	18088	950	128
165.4	160	131	91	82	18.1	21	21	609	14475	916	156
179.2	169	153	119	109	18.2	21	21	616	23010	949	111
201.8	184	172	134	126	18.2	21.1	21	612	18601	169	119
224.0	194	177	141	131	18.2	21.7	21	623	20194	223	107
246.8	165	149	118	109	18.1	21.4	21	637	26738	244	124
269.9	195	170	125	114	18.1	21.5	21	610	15076	805	110
292.9	181	164	128	117	18.1	21.6	21	636	22623	3216	88

316.1	136	121	91	84	18	21.9	21	614	26758	267	170
339.7	159	138	102	91	17.8	22	21	644	19258	937	134
362.0	211	194	155	144	17.5	21.9	21	612	21284	968	80
385.2	164	151	120	113	17.3	21.9	21	607	9311	176	154
408.6	136	122	92	85	17.5	22.1	21	646	12362	204	199
441.9	138	124	96	89	17.2	21.5	21	612	13446	167	204
432.2	191	164	120	108	16.6	20.5	21	620	14739	865	115
446.3	188	163	123	112	16.8	20.7	21	619	16895	697	113
455.4	1094	158	115	103	17.2	20.5	21	656	1476	122	282
470.0	203	161	118	107	17.3	20.5	21	639	12030	413	134
479.3	143	134	105	99	17.5	20.2	21	615	18610	205	157
492.9	151	138	103	94	17.5	20.1	21	613	17533	265	157
502.3	182	144	98	87	17.6	20	21	613	11926	893	145
516.3	164	149	115	105	17.6	20.1	21	647	25636	1666	104

CHAINAGE		DEFLECTION			TEMPERATURE			PRESSURE	LAYER MODULI		
								APPLIED			
	D0	D300	D900	D1200	Air	Surface	Pavement	Pressure KPa	Base moduli, MPa	Subbase moduli, MPa	Subgrade moduli, MPa
525.06	196.77	155.3415	112.0341	101.678	17.8	20.3	21	615	12938.79687	867.0016	125.930002
534.56	161.05	150.5017	121.7434	113.1159	18	20.3	21	604	32192.42647	1467.7363	96.2930838
548.7	198	177	136	122	18.2	20.7	21	643	17099	859	99
562.5	216	196	157	142	18.4	20.9	21	642	20512	1259	77
571.7	260	210	154	137	18.6	21.2	21	600	12628	1464	74
586.1	191	176	143	130	18.8	21.4	21	632	22676	3310	79
595.7	156	125	87	75	18.3	21.2	21	652	17400	4434	129
614.0	174	154	117	104	18.5	21.3	21	646	24674	12260	78
618.6	176	141	100	89	18.6	21.6	21	618	12814	1787	135
631.6	121	106	81	75	18.6	21.8	21	660	30092	374	187
640.8	147	129	100	90	18.5	22.7	21	640	25117	3781	117
663.5	150	137	110	101	18.5	22.9	21	642	31751	1483	110
686.5	217	169	120	108	18.3	23.2	21	618	10087	609	127
714.0	176	159	117	106	18.1	23.3	23.7	590	15567	260	139
733.0	198	179	142	131	17.	23.	23.7	599	21625	564	95

					9	4					
756.0	273	228	167	148	17.9	23.3	23.7	595	10556	5099	64
778.6	201	177	141	130	17.9	23.7	23.7	621	20525	380	100
792.6	197	177	140	128	17.9	23.8	23.7	605	20957	705	96
801.7	137	129	102	95	18.2	24	23.7	616	24057	217	158
824.4	139	126	99	93	18.7	23.6	23.7	624	25379	222	161
833.4	159	143	110	102	19.2	23.5	23.7	608	24775	652	126
847.6	134	114	78	70	19	23.6	23.7	627	18235	739	192
870.8	167	150	114	101	19.1	23	23.7	631	26208	4489	92
893.6	404	176	133	120	20.3	26.4	27.8	647	8980	2043	79
902.8	195	152	106	94	19.4	23.1	23.7	631	12132	1767	123
916.6	208	194	152	138	19	23.2	23.7	623	21629	3150	72
940.3	218	187	135	118	18.4	23.6	23.7	647	13154	6182	79
954.4	209	157	108	97	18.6	23.5	23.7	613	8926	616	146
963.3	170	159	114	102	18.6	23.5	23.7	657	17905	718	121
981.4	177	164	132	122	18.2	24.7	23.7	640	27833	1114	94
1009.2	128	101	65	55	18.1	24.3	23.7	647	17235	1210	202
1032.2	237	194	147	133	18.2	25.6	23.7	645	11817	1582	89
1059.1	138	127	100	94	18.7	24.1	23.7	621	12414	194	186

CHAINAGE		DEFLECTION			TEMPERATURE			PRESSURE	LAYER MODULI		
								APPLIED			
	D0	D300	D900	D1200	Air	Surface	Pavement	Pressure KPa	Base moduli, MPa	Sub base moduli, MPa	Subgrade moduli, MPa
1077.76	185.32	158.5796	118.9347	108.793	18.8	25.1	23.7	628	18802.18265	1201.085	107.95811
1101.0	202	178	139	128	19	24.5	23.7	631	18783	372	103
1124.2	183	171	140	132	19.3	24.3	23.7	636	21270	189	110
1147.5	196	183	150	139	19.1	24.5	23.7	631	26095	442	87
1160.8	161	153	128	122	18.5	24.4	23.7	603	15426	144	141
1169.9	160	147	117	108	18.2	24.5	23.7	635	24772	216	133
1193.0	184	163	122	108	18.1	24.4	23.7	638	18136	8166	85
1215.8	201	176	127	110	18.3	24.1	23.7	629	17872	8284	76
1224.9	190	145	98	86	18.3	23.8	23.7	626	10572	808	151
1238.8	168	150	114	103	17.8	24.5	23.7	626	20008	881	123
1252.8	206	181	145	134	17.7	24	23.7	622	20989	396	95
1266.5	188	175	139	128	17.7	24.3	23.7	627	24060	968	91
1279.8	185	170	131	120	17.6	24	23.7	635	23642	3350	84
1298.8	145	130	92	83	17.9	23.7	23.7	621	20107	1037	147
1312.7	188	164	118	103	18.1	24.3	23.7	627	18294	1625	96

1330.6	150	136	104	93	18.5	23.7	23.7	636	24463	3445	114
1353.5	1392	127	93	84	19	23.6	23.7	624	1538	122	385
1366.5	175	151	109	96	19.2	23.6	23.7	633	16223	2255	115
1379.6	144	130	99	88	19.8	23.8	23.7	622	26832	1432	128
1393.2	172	157	126	115	19.6	23.6	23.7	644	28024	1160	98
1411.4	178	164	132	122	19.5	23.7	23.7	627	25823	642	101
1429.9	210	195	158	144	19.8	24	23.7	656	23611	3344	68
1443.7	211	195	155	141	19.3	24	23.7	613	23032	3110	69
1457.4	238	217	170	154	19.2	23.7	23.7	625	18671	9276	55
1470.9	247	200	145	128	18.7	24.4	23.7	620	13497	1598	78
1488.9	181	171	148	128	18.5	24.1	23.7	640	30617	1245	85
1502.9	213	194	156	145	18.3	24	23.7	644	20883	2881	72
1520.7	207	189	152	139	18.6	24.2	23.7	615	24174	1463	77
1534.4	270	250	202	184	18.3	24.4	23.7	636	20719	3032	49
1557.0	207	174	134	121	18.5	24.7	23.7	658	17400	1123	97

Appendix J(i): Slab Identification and Cracks Mapping Record

<u>*SLAB IDENTIFICATION AND CRACKS MAPPING RECORDFORM*</u>

Name Of The Road: KAGERE- NDUNYU –MUNYANGE-GITUIGA(E571)ROAD *Date:*

Slab no./ mark	*Size of slab L x W (m) and location (L or R)*	*No of cracks in the slab*	*Crack identification Mark*	*Length of Crack(m)*	*Width of Crack (mm)*	*Crack Orientation- Transverse(T),/ Longitudinal(L) /Diagonal(D)*	*Sketch of the cracks on concrete slab*
B63	R 5·1X3·5	2	1	3·5	12	T	3·5 (12mm)
			2	3·5	13	T	
			3				
			4				3·5 (13mm)
A64	L 3·7X2·5	4	1	0·9	4	T	1·8 (15mm)
			2	1·1	4	T	0·8 (9mm) 1·1 (4mm)
			3	0·8	9	T	0·9 (4mm)
			4	1·8	15	T	
B64	R 3·7X3·5	1	1	3·5	8	T	3·5 (8mm)
			2				
			3				
			4				
B66	L 4·4X3·5	1	1	3·5	10	T	3·5 (10mm)
			2				
			3				
			4				

APPENDIX J(ii)- CORE CUTTING MAPPING RECORD

CORE CUTTING MAPPING RECORDFORM

Name Of The Road: KAGERE- NDUNYU –MUNYANGE-GITUIGA(E571)ROAD *Date:*

Slab no./ mark	*Core No.*	*Sketch of the core on concrete slab*
B400	CB14	2.21 CB14 2.0
B401	CB17	1.3 CB17
B402.	CB 17 CB15 CB16	1.1 CB15 0.41 CB16
B403	CB16	0.8

Appendix K: Cusum Computations from FWD Data

APPENDIX 5(i)- CUSUM COMPUTATIONS FROM FWD DATA

DISTANCE(m)	DEFLECTION IN MICROMETERS (D0)(LHS)	D0-MEAN	CUSUM	DISTANCE	DEFLECTION IN MICROMETERS (D0)(RHS)	D0-MEAN	CUSUM
0	234.24	17.89658	17.90	13.81	170.458	-81.620	-81.620
4.5	204.35	-11.9952	5.90	18.61	163.019	-89.059	-170.679
9	197.96	-18.3915	-12.49	42.06	185.280	-66.798	-237.477
42.06	180.77	-35.5784	-48.07	65.46	208.402	-43.676	-281.153
65.46	287.17	70.82096	22.75	83.86	192.068	-60.010	-341.163
88.46	180.05	-36.299	-13.55	106.26	197.344	-54.734	-395.897
97.06	232.86	16.51058	2.96	128.06	171.215	-80.863	-476.760
106.26	297.58	81.23316	84.20	151.36	188.338	-63.740	-540.500
155.96	161.66	-54.685	29.51	179.16	258.702	6.624	-533.876
165.36	159.72	-56.624	-27.11	201.76	172.084	-79.994	-613.870
179.16	169.19	-47.1598	-74.27	223.96	172.310	-79.768	-693.638
201.76	184.49	-31.8628	-106.13	246.76	167.704	-84.374	-778.012
223.96	194.24	-22.109	-128.24	265.36	227.633	-24.445	-802.457
246.76	164.52	-51.8285	-180.07	284.06	266.860	14.782	-787.675
269.86	194.58	-21.7662	-201.84	306.56	147.751	-104.327	-892.002
292.86	181.17	-35.183	-237.02	325.66	145.449	-106.629	-998.630
316.06	135.79	-80.5566	-317.58	344.26	229.732	-22.346	-1,020.976
339.66	159.14	-57.213	-374.79	366.66	305.796	53.718	-967.258
361.96	210.98	-5.37263	-380.16	385.16	179.528	-72.550	-1,039.808
385.16	164.07	-52.2822	-432.45	403.96	154.276	-97.802	-1,137.610
408.56	136.24	-80.1128	-512.56	422.86	160.626	-91.452	-1,229.062
441.86	1,392.16	1175.813	663.25	436.96	1,392.161	1,140.083	-88.979
432.16	138.13	-78.2207	585.03	455.36	172.877	-79.201	-168.180
446.26	191.44	-24.9046	560.13	483.66	222.482	-29.596	-197.777
455.36	188.01	-28.3368	531.79	502.26	195.413	-56.666	-254.442
469.96	1,093.57	877.2205	1409.01	516.26	266.959	14.881	-239.561
479.26	202.97	-13.381	1395.63	525.06	230.383	-21.695	-261.256
492.86	143.10	-73.2457	1322.39	548.66	192.395	-59.683	-320.939
502.26	151.13	-65.2225	1257.16	571.66	200.035	-52.043	-372.982
516.26	182.30	-34.0529	1223.11	595.66	233.632	-18.446	-391.429
525.06	163.77	-52.5815	1170.53	618.56	157.120	-94.958	-486.387
534.56	196.77	-19.5823	1150.95	631.56	184.545	-67.533	-553.920
548.66	161.05	-55.3018	1095.65	755.96	291.869	39.791	-514.129
562.46	198.10	-18.2455	1077.40	778.56	281.471	29.393	-484.735
571.66	215.55	-0.80139	1076.60	806.06	274.795	22.717	-462.018
586.06	259.59	43.23688	1119.84	824.36	226.129	-25.949	-487.967

595.66	190.56	-25.7912	1094.04	852.26	229.951	-22.127	-510.094
613.96	156.29	-60.0536	1033.99	870.76	249.524	-2.554	-512.648
618.56	173.88	-42.4689	991.52	875.46	404.495	152.417	-360.232
631.56	176.14	-40.2122	951.31	940.26	252.131	0.053	-360.179
640.76	121.06	-95.2845	856.03	967.96	227.656	-24.422	-384.601
663.46	146.56	-69.7887	786.24	972.46	223.262	-28.816	-413.416
686.46	149.71	-66.6378	719.60	990.56	203.604	-48.474	-461.890
713.96	217.36	1.011104	720.61	1018.36	242.949	-9.129	-471.018
732.96	175.66	-40.6854	679.92	1050.06	281.549	29.471	-441.547
755.96	198.16	-18.1929	661.73	1100.96	307.799	55.721	-385.826
778.56	273.44	57.09558	718.83	1133.56	261.663	9.585	-376.241
792.56	201.39	-14.9568	703.87	1160.76	242.094	-9.984	-386.225
801.66	197.15	-19.201	684.67	1188.46	194.182	-57.896	-444.121
824.36	137.23	-79.1176	605.55	1211.46	189.984	-62.094	-506.214
833.36	139.18	-77.1654	528.39	1234.16	226.486	-25.592	-531.806
847.56	159.03	-57.3136	471.07	1261.96	199.986	-52.092	-583.898
870.76	133.90	-82.4486	388.62	1289.26	235.276	-16.802	-600.700
893.56	167.00	-49.3465	339.28	1303.36	318.495	66.417	-534.283
902.76	194.53	-21.8188	317.46	1325.86	184.395	-67.683	-601.966
916.56	208.18	-8.16834	309.29	1353.46	223.394	-28.684	-630.651
940.26	218.36	2.007368	311.30	1374.96	235.525	-16.553	-647.203
954.36	208.74	-7.60587	303.69	1397.56	288.569	36.491	-610.712
963.26	223.26	6.914281	310.61	1420.56	260.945	8.867	-601.845
981.36	170.09	-46.2614	264.34	1438.96	229.924	-22.154	-623.998
1009.16	177.32	-39.0294	225.32	1466.36	330.857	78.779	-545.219
1032.16	127.97	-88.3775	136.94	1488.86	296.783	44.705	-500.514
1059.06	236.99	20.63793	157.58	1511.66	293.052	40.974	-459.540
1077.76	137.99	-78.3578	79.22	1534.36	273.776	21.698	-437.842
1100.96	185.32	-31.0312	48.19	1556.96	358.869	106.791	-331.051
1124.16	201.87	-14.4781	33.71	1580.16	272.851	20.773	-310.277
1147.46	182.99	-33.3623	0.35	1593.96	192.094	-59.984	-370.261
1160.76	196.36	-19.9836	-19.64	1603.06	262.444	10.366	-359.895
1169.86	161.31	-55.0347	-74.67	1612.36	135.221	-116.857	-476.752
1192.96	159.57	-56.7812	-131.45	1626.16	199.624	-52.454	-529.206
1215.76	184.23	-32.1208	-163.57	1649.26	230.205	-21.873	-551.080
1224.86	200.67	-15.6772	-179.25	1667.96	191.765	-60.313	-611.393
1238.76	189.61	-26.7395	-205.99	1695.26	1,081.791	829.713	218.321
1252.76	168.34	-48.0127	-254.00	1723.86	229.832	-22.246	196.075
1266.46	205.72	-10.6263	-264.63	1728.46	154.220	-97.858	98.216
1279.76	188.38	-27.9653	-292.59	1746.96	221.465	-30.613	67.603

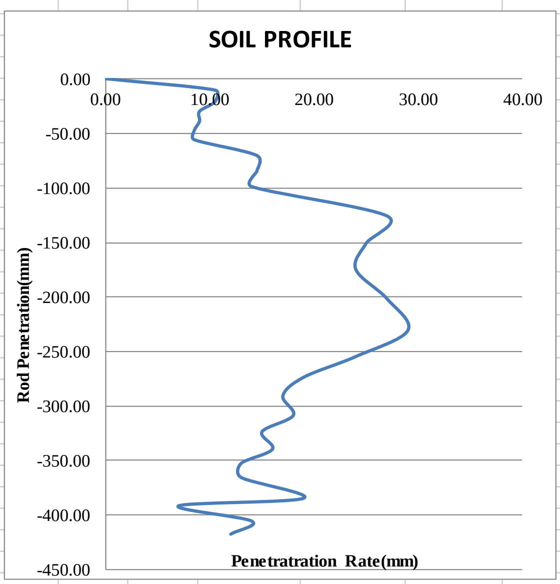
1298.76	185.10	-31.2505	-323.85	1769.76	214.654	-37.424	30.179
1312.66	145.45	-70.8988	-394.74	1792.16	214.038	-38.040	-7.861
1330.56	188.38	-27.9653	-422.71	1814.46	229.035	-23.043	-30.904
1353.46	150.21	-66.1359	-488.85	1838.46	174.691	-77.387	-108.292
1366.46	1,391.83	1175.479	686.63	1858.56	264.686	12.608	-95.684
1379.56	174.71	-41.642	644.99	1882.16	255.182	3.104	-92.580
1393.16	144.28	-72.0636	572.93	1910.46	370.992	118.914	26.335
1411.36	171.72	-44.6261	528.30	1929.26	240.551	-11.527	14.807
1429.86	178.22	-38.1232	490.18	1943.36	276.056	23.978	38.785
1443.66	210.06	-6.28409	483.89	1962.06	253.649	1.571	40.356
1457.36	210.63	-5.7168	478.18	1975.56	261.456	9.378	49.734
1470.86	238.08	21.73668	499.91	1998.46	251.022	-1.056	48.678
1488.86	247.48	31.12769	531.04	2017.66	228.666	-23.412	25.266
1502.86	180.94	-35.4106	495.63	2035.96	226.834	-25.244	0.022
1520.66	213.08	-3.26893	492.36				
1534.36	207.12	-9.22617	483.14				
1556.96	270.38	54.03396	537.17				
1570.96	206.79	-9.56241	527.61				
1593.96	192.09	-24.2542	503.35				
1612.36	135.22	-81.1271	422.23				
1626.16	150.96	-65.3847	356.84				
1639.96	170.35	-45.9992	310.84				
1658.96	160.15	-56.1992	254.64				
1676.86	194.58	-21.7662	232.88				
1695.26	207.80	-8.5504	224.33				
1709.56	205.57	-10.7748	213.55				
1728.46	154.22	-62.1285	151.42				
1728.46	187.19	-29.1595	122.26				
1760.46	194.27	-22.0784	100.19				
1769.76	198.16	-18.1929	81.99				
1787.76	243.20	26.85059	108.84				
1796.16	270.26	53.91296	162.76				
1814.46	235.16	18.81207	181.57				
1818.86	252.17	35.82641	217.39				
1843.16	213.90	-2.45003	214.94				
1858.56	239.29	22.94686	237.89				

1882.16	227.89	11.54628	249.44				
1905.86	198.30	-18.0476	231.39				
1923.96	240.27	23.92333	255.31				
1929.26	196.61	-19.7406	235.57				
1943.36	217.24	0.891944	236.47				
1957.16	207.92	-8.43024	228.03				
1966.26	185.32	-31.0312	197.00				
1984.76	183.37	-32.9831	164.02				
2003.16	156.49	-59.8616	104.16				
2017.66	192.39	-23.9531	80.21				
2027.16	199.42	-16.9255	63.28				
2031.56	201.03	-15.3193	47.96				
2035.96	168.39	-47.9609	0.00				
TOTALS	**27,043.51**				**22,687.04**		
MEAN	**216.3481193**				**252.0782436**		

APPENDIX 5(ii)- DCP ANALYSIS SAMPLE

DCP ANALYSIS -KM 11+950(CA1)

No of blows	Rod reading (mm)	Penetration (mm)	Rate of penetration per blow(mm)	Blow count	PENETRATION - RATE	Cummulative Rod Reading(mm)	Invert Cummulative reading(mm)	penetration reading(mm/blow)	Invert Cummulative reading(mm)	(2.46-1.12log DPI)	Estimated CBR	Average CBR
0.00	382.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	
2.00	403.00	-21.00	10.50	1.00	10.50	10.50	-10.50	10.50	-10.50	1.32	20.71	
2.00	421.00	-18.00	9.00	2.00	10.50	21.00	-21.00	10.50	-21.00	1.32	20.71	
2.00	438.00	-17.00	8.50	3.00	9.00	30.00	-30.00	9.00	-30.00	1.39	24.62	20.78
2.00	467.00	-29.00	14.50	4.00	9.00	39.00	-39.00	9.00	-39.00	1.39	24.62	
1.00	481.00	-14.00	14.00	5.00	8.50	47.50	-47.50	8.50	-47.50	1.42	26.25	
1.00	508.00	-27.00	27.00	6.00	8.50	56.00	-56.00	8.50	-56.00	1.42	26.25	
1.00	533.00	-25.00	25.00	7.00	14.50	70.50	-70.50	14.50	-70.50	1.16	14.43	
1.00	557.00	-24.00	24.00	8.00	14.50	85.00	-85.00	14.50	-85.00	1.16	14.43	
1.00	584.00	-27.00	27.00	9.00	14.00	99.00	-99.00	14.00	-99.00	1.18	15.01	
1.00	613.00	-29.00	29.00	10.00	27.00	126.00	-126.00	27.00	-126.00	0.86	7.19	
1.00	637.00	-24.00	24.00	11.00	25.00	151.00	-151.00	25.00	-151.00	0.89	7.84	
1.00	656.00	-19.00	19.00	12.00	24.00	175.00	-175.00	24.00	-175.00	0.91	8.21	
1.00	673.00	-17.00	17.00	13.00	27.00	202.00	-202.00	27.00	-202.00	0.86	7.19	
1.00	691.00	-18.00	18.00	14.00	29.00	231.00	-231.00	29.00	-231.00	0.82	6.64	
1.00	706.00	-15.00	15.00	15.00	24.00	255.00	-255.00	24.00	-255.00	0.91	8.21	
1.00	722.00	-16.00	16.00	16.00	19.00	274.00	-274.00	19.00	-274.00	1.03	10.66	
1.00	735.00	-13.00	13.00	17.00	17.00	291.00	-291.00	17.00	-291.00	1.08	12.08	
1.00	748.00	-13.00	13.00	18.00	18.00	309.00	-309.00	18.00	-309.00	1.05	11.33	12.64
1.00	767.00	-19.00	19.00	19.00	15.00	324.00	-324.00	15.00	-324.00	1.14	13.89	
1.00	774.00	-7.00	7.00	20.00	16.00	340.00	-340.00	16.00	-340.00	1.11	12.92	
1.00	788.00	-14.00	14.00	21.00	13.00	353.00	-353.00	13.00	-353.00	1.21	16.31	
1.00	800.00	-12.00	12.00	22.00	13.00	366.00	-366.00	13.00	-366.00	1.21	16.31	
				23.00	19.00	385.00	-385.00	19.00	-385.00	1.03	10.66	
				24.00	7.00	392.00	-392.00	7.00	-392.00	1.51	32.62	
				25.00	14.00	406.00	-406.00	14.00	-406.00	1.18	15.01	
				26.00	12.00	418.00	-418.00	12.00	-418.00	1.25	17.84	



APPENDIX 5(iii)	DCP ANALYSIS AND FWD ANALYSIS(PAVEMENT STRENGTH BELOW CONCRETE SLAB)													
				FWD DISTANCES(MM)							PAVEMENT STRENGTH		
											SN	SNC	SNP
NO.	**SLAB NO.**	**TEST LOCATION ON CHAINAGE**	**DISTANCE FROM 11+950**	**D0**	**D900**	**D1200**	**ESTIMATED DEPTH OF SUB-BASE LAYER(MM)**	**CBR SUB-BASE(%)**	**CBR SUB-GRADE(%)**	**STATE OF CORED TEST LOCATION**	**(SUBBASE ONLY)**	**SUB-BASE AND SUBGRADE**	**ADJUSTED STRUCTURAL NUMBER(EXISTING STRENGTH)**
1	CA1	11+950	0	0.202	0.12	0.107	126	20.78	12.64	at crack	0.50	1.06	7.36
2	CA2/CA3	11+954	4	0.202	0.12	0.107	122	47.01	21.5	at joint	0.48	1.46	7.36
3	CA3	11+959	9	0.163	0.119	0.111	138	35.7	17.5	solid	0.54	1.36	7.20
4	CA101	12+410	460	0.18	0.119	0.109	180	41.9	11.3	at crack	0.71	1.19	7.31
5	CA100/10	12+405	455	0.18	0.119	0.109	169	80.4	9	at joint	0.67	0.96	7.31
6	B100	12+405A	455	0.18	0.119	0.109	142	40.75	10	solid	0.56	0.94	7.31
7	CA200	12+866	916	0.208	0.152	0.138	176	51.5	11.6	solid	0.69	1.19	6.74
8	CA201/20	12+871	921	0.208	0.152	0.138	172.3	62.8	13.5	at joint	0.68	1.29	6.74
9	CA202	12+871A	921	0.208	0.152	0.138	210	37.1	12.7	at crack	0.83	1.40	6.74
10	B314	13+384	1434	0.22	0.162	0.148	194	68.7	22.6	at crack	0.76	1.79	6.25
11	B315	13+388	1438	0.22	0.162	0.148	215	68.1	15.7	solid	0.85	1.58	6.25
12	B313/314	13+388A	1438	0.22	0.162	0.148	120	40.15	8.64	at joint	0.47	0.74	6.25
13	B314/315	13+393	1443	0.22	0.162	0.148	161.5	69.16	16.51	at joint	0.64	1.41	6.25
14	B400	13+783	1833	0.194	0.134	0.124	245.7	65.1	14.98	solid	0.97	1.67	6.50
15	B402	13+786	1836	0.194	0.134	0.124	145	115.23	610	at crack	0.57	4.18	6.50
16	B402/403	13+793	1843	0.194	0.134	0.124	192	164.3	11.74	at crack	0.76	1.26	6.50
17	B401/402	13+793A	1843	0.194	0.134	0.124	237	95.32	21.53	at joint	0.93	1.92	6.50