EFFECTS OF IN-SITU MOISTURE CONTENT ON PAVEMENT PERFORMANCE OF LOW VOLUME SEALED ROADS

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Effects of In-situ Moisture Content on Pavement Performance of Low Volume Sealed Roads

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DECLARATION

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

Signature**……………………………………….** Date………………………………..

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This thesis has been submitted for examination with our approval as university supervisors

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DEDICATION

This thesis is dedicated to my loving wife Regina and children for their patience, encouragement and understanding during my studies. I would also like to dedicate it to my late father Johannes, my late mother Agnes, whose value for education has been a source for inspiration to me.

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ABBREVIATIONS AND ACRONYMS

AADT	Annual Average Daily Traffic
AASHTO	American Association of State Highway and Transportation Officials
ABD	Average Balanced Deep Structure
ASTM	American Society for Testing Materials
BS	British standards
BS	British Standards
CBR	California Bearing Ratio
CESA	Cumulative Equivalent Standard Axles
COLTO	Committee of land transport officials
CUSUM	Cumulative Sum
DCP	Dynamic Cone Penetrometer
DCPI	Dynamic Cone Penetrometer Index
DESA	Design Equivalent Standard Axle
DN	Average Penetration Rate (mm/blow)
DSN 800	Number of blows required to penetrate the pavement depth of 800 mm (Pavement Structure Number)
EF	Equivalence Factor
EMC	Equilibrium Moisture Content
EOD	Environmentally Optimized Design
ESA	Equivalent Standard Axels (80 kN)
ETB	Emulsion Treated Base
FWD	Falling Weight Deflectimeter
FWD	Falling Weight Deflectometer
HDM-4	Highway Design and Maintenance Standards Model-4
IDD	In-situ Dry Density
IWT	Inner wheel-tracks
IWT	Inner Wheel Track
LCC	Live Circle Cost
LHS	Left Hand Side
LL	Liquid Limit
Log	Logarithm base 10

LSD	Layer Strength Diagram
LSP	Layer Strength Profile
LVSRs	Low Volume Sealed Roads
MDD	Maximum Dry Density
MESA	Million Equivalent Standard Axles
MoTI	Ministry of Transport and Infrastructure
NMC	Natural Moisture Content
OMC	Optimum Moisture Content
ORN	Overseas Road Note
OWT	Outer wheel-tracks
PI	Plasticity Index
PL	Plastic Limit
R^2	Coefficient of determination
RHS	Right Hand Side
SATTC	Southern Africa Traffic and Communications Commission
SCBR	Soaked CBR
TRH	Technical Recommendations for Highways
TRL	Transport Research Laboratory
UCBR	Unsoaked CBR
UCS	Unconfined Compressive Strength
UUCBR	Undisturbed Unsoaked CBR
VPD	Vehicle per Day
WBD	Well Balanced Deep Structure
WBD	Well Balanced Deep Structure

ABSTRACT

The introduction and appreciation of the low volume sealed road technique in Kenya has heralded a new era in construction of efficient and effective rural road network. The performance of such roads are affected by various environmental parameters including changes in the in-situ moisture. The objective of the research was to investigate the effects of in-situ moisture and density on the pavement performance of Low Volume Sealed Roads (LVSRs) before, during and after improvement by modification of the cross-section profile, side drains and surface sealing. This was done by conducting field Dynamic Cone Penetrometer (DCP) test and laboratory California Bearing Ratio (CBR) test and establishing the DCP-CBR correlation. Appropriate testing with the simple DCP device was used to assess the in-situ conditions including material quality and moisture regimes along the road alignment. This information was used to identify uniform sections; the in-situ layer strength diagrams of each of these sections were analyzed to determine the layer quality and thicknesses. The method of data analysis of pavement entailed assessment of the in-situ moisture, pavement strength and correlation of CBR-DCP results and moisture changes. Sampling was conducted on the alignment before construction, pavement and subgrade during construction and after allowing traffic flow were tested and analyzed as per the ASHTOO and BS standards. The research shows that, alignment soils before improvement the MDD and OMC for Wamumu - Karaba were 1090 kg/m³ and 46.5% respectively while the Maximum Dry Density (MDD) and the Optimum Moisture Content (OMC) for Kyeni - Karurumu were 1125 kg/m³ and 45.1% respectively and were predominantly borderline granular materials were susceptible to effects of moisture. As such they exhibited inferior characteristics but were improved by processing the subgrade and compacting heavily to improve the strength and reduce permeability. During construction, by adding sub-base and base layers, since the pavement had not fully consolidated it required 3 mm/blow to penetrate to 150 mm thickness and at a higher CBR of 102, hence reduction of moisture and increased pavement strength. A DCP –CBR correlation of 3.0 for Kyeni Karurumu and 2.7 for Wamumu Karaba was established hence along the road sections works could continue based on the DCP and CBR only used for confirmation on both sections. After construction and opening to traffic, the CBR achieved was 152% for the 0-150 mm layer and 106% for the 151-300 mm layer. This showed improved pavement strength due further consolidation and reduction in moisture content. In conclusion, the existing alignment materials can be used as the subgrade by processing and compacting at or near optimum moisture content (OMC). During construction, by adding sub-base and base layers, improving drains and sealing surface the resulting pavement becomes stronger. Further consolidation is achieved when the pavement is opened to traffic. It is recommended that the effects of in-situ moisture be assessed before, during and after traffic flow when upgrading or constructing new LVSRs as this will inform the use of existing alignment soils, required pavement strength and proper drainage.

CHAPTER ONE

INTRODUCTION

1.1 Background Information

Kenya, like other developing nations, has majority of its road network (90%) as low volume roads (Classes D, E and the unclassified). Over the past years, the main infrastructure investment has been towards high volume roads (Classes A, B and C roads), (Roads Design Manual Part 1). Low Volume Roads (LVSRs) principally involve sealing of existing gravel roads using a layer of bitumen with or without any improvements as preliminary strength tests may dictate. LVSRs, defined as those roads that carry less than 300 vehicles per day (VPD) and about one million equivalent standard axles over the design life, constitute a significant proportion of the Kenyan classified road network.

Special attention must be drawn to the LVSRs. Through the recently launched Road annuity Programme, the Kenyan Government was planning to improve on its roads infrastructure with emphasis on the low volume roads by sealing the existing gravel roads to the tune of 2,000 km within 2014-2015 fiscal year, (MoTI, 2014). The main focus of this policy intervention is to produce appropriate cost effective pavement design for low volume sealed roads using the Dynamic Cone Penetrometer (DCP) design method that will enable Kenya upgrade a larger proportion of the rural network than the current traditional methods can allow.

The new material specifications and design requirements for low volume sealed roads in these studies is derived from class D and E roads. The design and construction step applied recognizes the controlling influence of the road environment on the deterioration of lighter pavement.

The main focus of the project is on appropriate, cost effective pavement design for LVSRs using the DCP design method. However, appropriate geometric standards with regard to current and future traffic as well as traffic safety issues are also being considered. In order to successfully design long lasting LVSRs, information on the effects of the in-situ moisture content of the existing gravel roads is very important.

This is because moisture content usually has a significant effect on the performance of conventional roads.

1.2 Statement of Problem

Source of gravel as a wearing course is becoming more and more depleted and Kenya’s LVSRs are washed by floods due to poor internal and external drainage. Experience in Kenya has shown that the best wearing course gravels have either been depleted or are no longer available. The lack of suitable gravel may be a function of either economic or for environmental factors. Other aspects such as the need for regular grader maintenance, the generation of dust and the erosion of materials into water courses may also mitigate against the continued use of unsealed roads. The most beneficial solution to these problems is usually to reconstruct the roads with a bituminous surfacing, (Roads 2000 Programme Nyanza Report).

Optimizing the use of traffic loading, moisture variation, compaction of the pavement layers usually results in a reduced need to import large quantities of virgin gravel material.

Sealing of the existing gravel roads without giving due attention to the in-situ moisture and its influence on the performance of the road could lead to roads which have to be reconstructed after a very short time due to premature failure.

By making optimal use of the in situ conditions and minimizing the quantity of imported material required, a reduction in the construction cost with an immediate impact on the life-cycle costs of the pavement was achieved, provided that the maintenance needs for this new road are not significantly higher than a conventional pavement structure.

The goal of the study was to produce a design with respect to the effect of in situ moisture content on pavements that achieve construction and operational efficiency, but also cost effective, be aesthetically pleasing and minimize environmental impacts.

1.3 Objectives

By answering the above research questions, the study was to achieve the objectives shown below.

1.3.1 Overall Objective

To investigate the effect of in-situ moisture on the pavement performance of Low Volume Sealed Roads (LVSRs) before, during and after improvement by modification of cross section profile, side drainage and bituminous surfacing.

1.3.2 Specific Objectives

a) To assess in-situ moisture and pavement strengths of various sections of LVSRs before improvement by modification of cross section profile, side drainage and bituminous surfacing.
b) To assess in situ moisture, pavement layer strength and subgrade strength of various sections of LVSRs during improvement by modification of cross section profile, side drainage and bituminous surfacing.
c) To determine pavement strength and moisture after improvement to LVSRs and traffic flow.

1.4 Research Questions

This research seeks to answer the following questions:

i. How does in-situ moisture affect the pavement strength of LVSRs before improvement by modification of cross section profile, side drainage and bituminous surfacing?
ii. Does the moisture content of the pavement layers for LVSRs change during improvement by modification of cross section profile, side drainage and bituminous surfacing?
iii. Does the pavement strength and moisture change after improvement and traffic flow of the LVSRs?

1.5 Justification of the Study

Currently the government through the Roads Authorities through the Roads 10,000 km policy is emphasizing on enhancing the roads infrastructure as a tool to spur economic growth by improving LVSRs network to sealed standards. This is because the major challenges facing the Roads Authority has been the management of a large network of unpaved roads due to the use of non-renewable gravel resource which is being seriously depleted. The Authorities have thus adopted the design of LVSRs using the DCP design method to determine suitable pavement structure. It is used to a certain the strength and bearing capacity of the existing gravel road which is to be incorporated as part of the new pavement structure.

This study therefore seeks to establish the effect of in-situ-moisture on strength and bearing capacity of the LVSRs before and after construction especially now that the DCP technique is going to be used. That there is significant adoption of the LVSRs the study will come in handy to the decision makers on the influence of the environmental factors on the planning, design, construction and maintenance stages.

1.6 Scope and Limitations of the Study

1.6.1 Scope of Study

Whereas the DCP design for LVSRs encompasses traffic, geometrics, pavement and materials, the study will dwell with moisture and strength component of the pavement design. A little mention however, was made about other characteristic factors on pavement.

The scope of this research will include determining the effects of in-situ moisture contents and pavement strengths on two roads with varying climatic conditions, soil types but with almost similar traffic levels. The research was done in three stages namely during design, construction and after allowing traffic to flow.

1.6.2 Limitations of study

This research was confined to the two roads and was limited to the assessment of outcomes as a basis of acceptance of work accomplished. The results can be generalized to similar roads with caution.

The research is further limited with time as the performance of the road requires the study to take over two years. Collecting and analyzing all the data might prove difficult in terms of time and financial limitations, however, positive endeavors was put in place to strike this target.

CHAPTER TWO

LITERATURE REVIEW

2.1 Introduction on Low Volume Roads

Pratico and Giunta (2011) defined low-volume roads as facilities outside built-up areas with a traffic volume of less than 400 annual average daily traffic (AADT). In spite of the fact that these roads are of lower use because of their location, low-volume roads play an important social and economic role and sometimes represent a large part of the regional and national road network (Russell and Kornala, 2003; Pratico and Giunta, 2011).

It is well known that the unpaved roads are roads built without an asphalt or concrete wearing surface, so they derive all structural support from their aggregate base layers. In principle, the unpaved roads are used for many purposes including industrial and private roads, temporary roads, detours and rural roads. Because unpaved roads have no asphalt or concrete wearing surface to help support traffic loads, they require a greater depth of aggregate base than the paved roads. This is applicable in designing roads for the same traffic. However, it is important here to mention that because of the increasing cost associated with asphalt repaving, interest has been rising in turning back damaged asphalt roadways into maintainable aggregate driving surfaces (Shearer and Scheetz, 2011).

According to Wayne et al. (2010), the flexible pavement structure for a low traffic volume road (LVR) consists of a relatively thin asphalt-concrete wearing course and an aggregate base course constructed on subgrade layer. An asphalt wearing course provides a good riding surface and moisture protection for the base course. Service life of a thin asphalt pavement depends on material quality and thickness of granular layers (Wayne et al., 2011).

2.2.1 Design Principle for Low Traffic Volume Roads

In spite of the fact that there are similar basic principles for pavement design, each country has adopted individual methods and structures that suit their requirements according to their experience. In general, unpaved-road construction using gravel requires that the material meets certain basic engineering demands. These demands include the following: Adequate cohesion to resist erosion, a particle size distribution that assists a tight engagement of the individual material particles, and adequate strength to support the applied traffic loads without significant deformations.

Mahgoub et al. (2011) reported that motorists traveling on a gravel road will encounter the problem of corrugation. Nevertheless, there is no long-term solution for this problem which requires continual maintenance to control its appearance. Traffic volumes, subgrade and gravel properties, and vehicles speeds are factors that may cause corrugation. Research results have shown that when the aggregate base and subgrade soil intermix, this will reduce the effective thickness and in turn, the load-bearing capacity of the road structure will outcome ruts that must be periodically refilled with aggregate.

Pavement deterioration is controlled mainly by how the road responds to environmental factors, such as moisture changes in the pavement layers, fill and subgrade, rather than to traffic. The appropriate design options for LVSRs therefore need to be responsive to a wide range of factors as captured in the road environment with the most critical being internal and external drainage.

2.3 Typical Low Volume Road Pavement Structures

Pavement conditions are monitored in terms of moisture, strength (in-situ CBR measured with DCP), density, riding quality (roughness), deformation (rutting) and deflection at the end of each wet and dry surface conditions. The layout of a typical test section with measurement position is as shown in Figures 2.1 and 2.2.

According to Bradbury et al. (2005) cross-sections within each test section were tested with the DCP. The number of measurement positions chosen depended on the width of the road, but always included the outer and inner wheel-tracks (OWT and IWT, respectively) and the centre-line (CL). Further measurement positions were concentrated between the OWT and the shoulder. The longitudinal measurement position was relocated about 1 m further along the road in each successive survey. Thus, over time, there were only relatively small variations in the measurement position, and results from successive surveys are comparable. The transverse measurements at these locations were always made at the same offset positions.

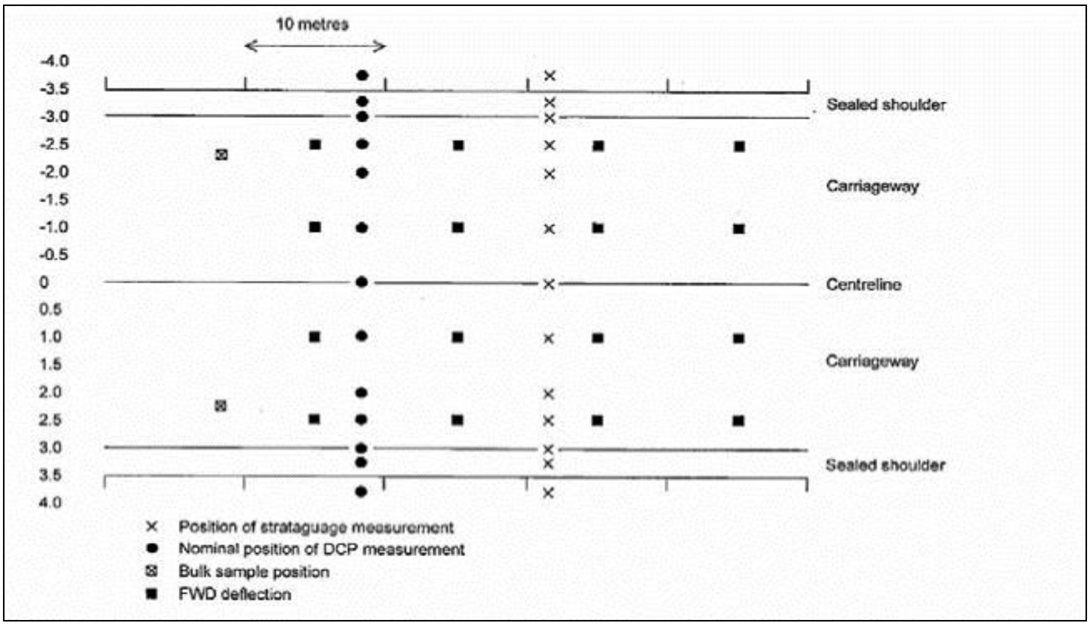


Figure 2.1: Typical section of LVSRs – Bradbury 2005

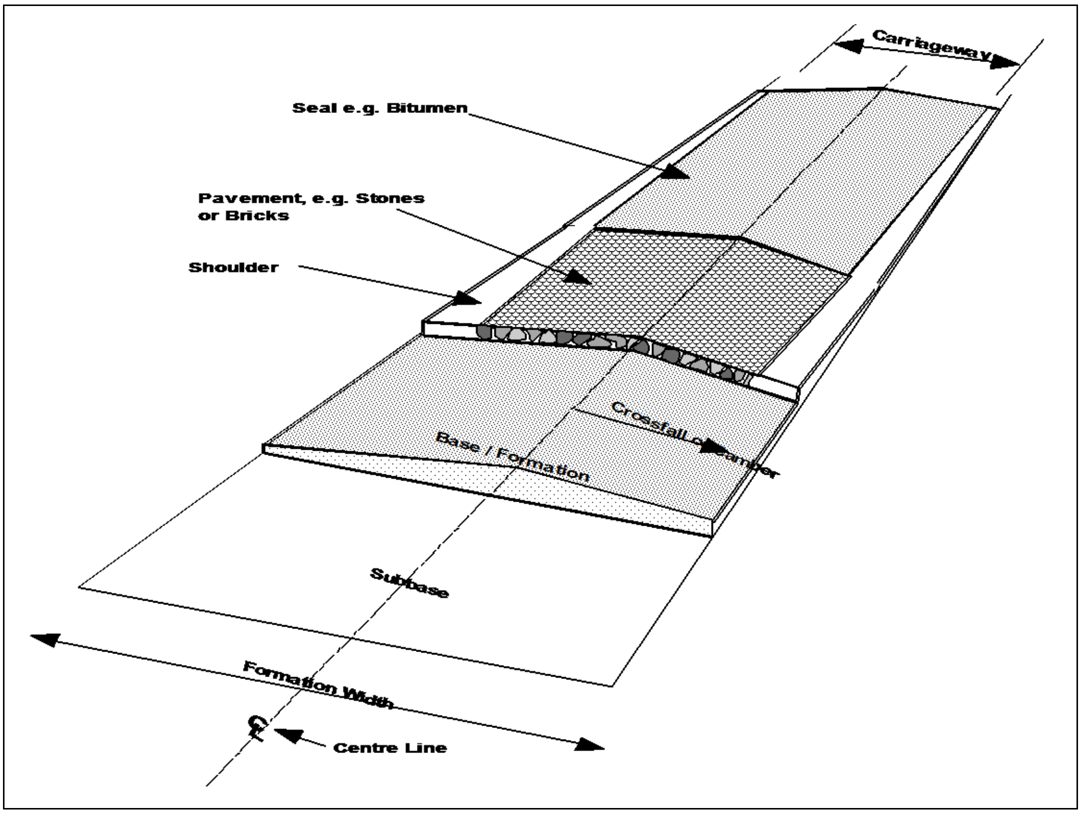


Figure 2.2: Typical LVSRs profile- Bradbury 2012

2.3 Road Level

The crown height of an LVSR, i.e. the vertical distance from the bottom of the side drain to the finished road level at the centre line, is a critical parameter that correlates well with the in-service performance of pavements constructed from naturally occurring materials. This height must be sufficiently great to prevent moisture ingress into the potentially vulnerable outer wheel track of the carriageway for which a minimum value of 0.75 m is recommended (SATCC 2003).

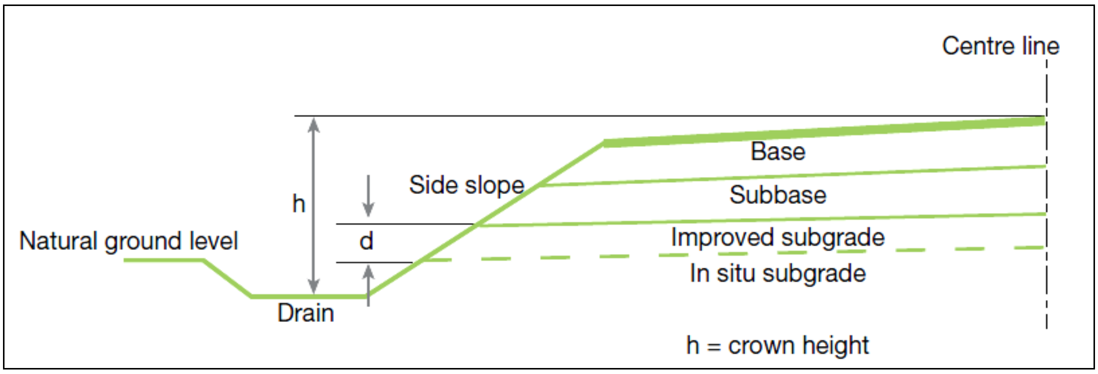


Figure 2.3: Crown height for LVSRs –SATCC 2013

The recommended minimum crown height of 0.75 m applies to unlined drains in relatively flat ground (longitudinal gradient, g, less than 1%). The recommended values for sloping ground (g > 1%) or where there are lined drains.

In addition to observing the crown height requirements, it is also equally important to ensure that the bottom of the sub-base is maintained at a height of at least 150 mm above the existing ground level. This is to minimize the likelihood of wetting up of this pavement layer due to moisture infiltration from the drain. Because of the critical importance of observing the minimum crown height and minimum height of the bottom of the sub-base above existing ground level, along the entire length of the road, the measurement of this parameter should form an important part of the drainage assessment carried out during and before construction. This is to ensure that any existing drainage problems associated with depressed pavement construction, often observed on gravel roads that have evolved over time with no strict adherence to observing minimum crown heights are avoided (SATCC 2003).

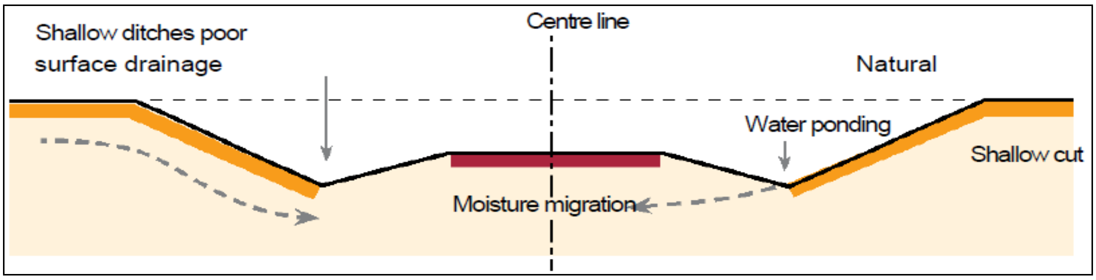


Figure 2.4: Potential drainage problems of LVSRs - SATCC 2013

2.5 Traffic and Loading

Accurate quantification of the traffic that the road will carry is essential. This, is however, considered to be a much bigger problem on lightly trafficked roads than on more highly trafficked roads. The main reason for this is that the small heavy vehicle counts normally obtained on such roads are dramatically affected by any intermittent or temporary (often seasonal) increases in traffic arising from the development of new infrastructure along the road, seasonal agricultural traffic and intermittent mining traffic. A sudden but small increase in heavy traffic can have a severe effect on the estimation of the overall cumulative standard axle estimation. It is thus vitally important that traffic counts capture all of the traffic using the road – this may require 24 hour counts, often during various seasons and at different times of the local commercial cycles, and should include axle weight surveys where necessary (Paige-Green & du Plessis, 2009).

Many better quality unsealed roads attract overloaded vehicles that avoid higher standard roads in order to minimize the possibility of being caught and prosecuted for overloading. Indeed, the possibility that unsealed roads may attract such traffic after sealing should also be assessed. The traffic counts need to be converted to cumulative standard axles (in terms of 80 kN axle loadings), which was used for classification of pavement structures within the design 'catalogues' or 'layer strength diagrams' that will form the basis of the DCP design method.

Research (Kleyn & Savage, 1982) has shown that, for balanced pavements (Paige-Green & du Plessis, 2009), the exponent (n) used to calculate the equivalency factor EF= $(P/80)^n$ can differ significantly from that normally used (i.e. 4.2 based on the AASHTO road test).

Recent studies (Paige-Green & Overby, 2010) have shown that roads on deep and strong subgrades can have n-exponents as low as between 1 and 2. This obviously has a major impact on determining a realistic cumulative axle count for the pavement design and will often reduce the estimated number of standard axles being carried significantly and a layer strength diagrams (LSD) for that traffic can be developed. This can be established from existing pavement design catalogues, although this is not really any more cost-effective than conventional pavement design as it would be based on traditional designs.

Typically, a series of catalogues or layer strength diagrams should be developed for specific roads, based on in situ moisture conditions (not soaked conditions, using the conventional CBR, assuming that the drainage standards are appropriate and drains are correctly maintained) and preferably making use of information collected from existing low volume roads in an area, such that the in situ conditions can be related to known performance.

2.6 The Design Process for LVR Pavements

The design fundamentals of low volume road pavements should not differ from any other pavement type. The standard procedures for appraising the pavement design loading, design strategy and analysis period must be followed (COLTO, 1996). It should, however, be noted that aspects such as traffic characterization may entail considerably more work. The actual design method still requires that the pavement structure bearing capacity must be appropriate for the estimated traffic that was carried over the life of the road.

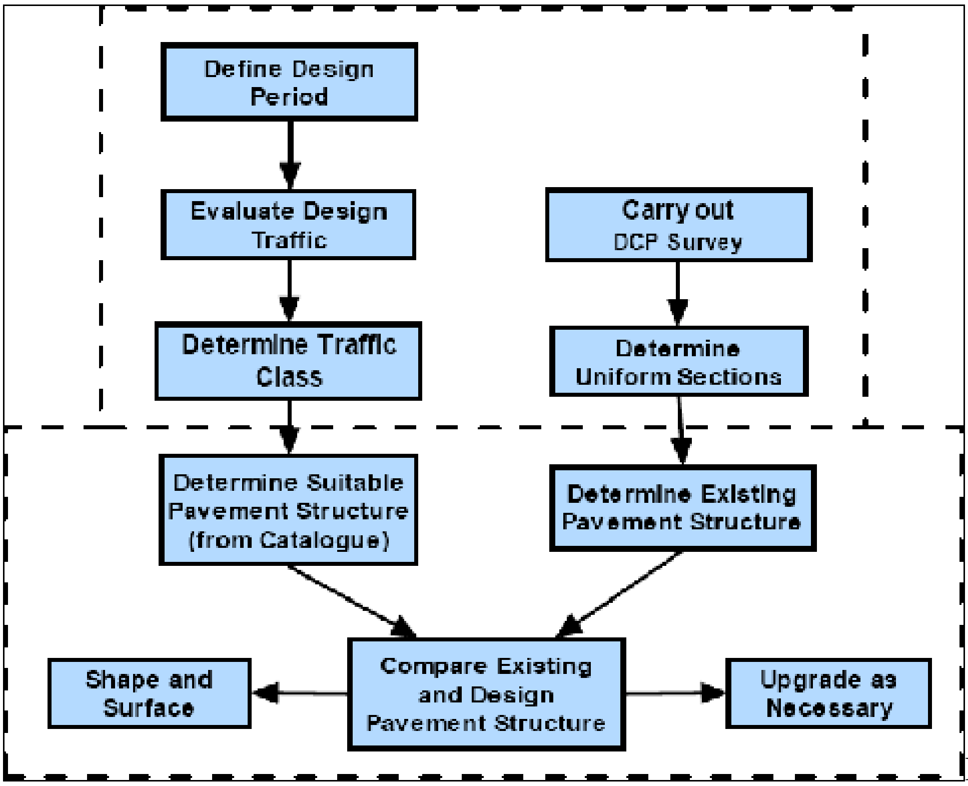


Figure 2.5: Flow chart of DCP pavement design process-Paige-Green 2012

2.6.1 Applicability of DCP Methods of Design

DCP-based methods, can be applied to most design situations found in practice in tropical and sub-tropical regions of the world. It should be noted, however, that the DCP method cannot be used directly if the proposed road is in cut or on fill, where the final formation level of the alignment would be outside the influence zone of an existing alignment DCP survey. In such cases, the material to be used for the embankment would need to be tested to determine its properties at varying densities and moisture contents. Fills can then be designed in accordance with the relevant catalogue to ensure that all the layers comply with the specifications of the respective design method. This will allow designers to go straight to design catalogues for contractual quantities (Paige-Green and Van Zyl, 2018).

As with all empirical methods of pavement design, the four main requirements of the design procedure are generally as follows (Rolt and Pinard, 2016); assessment of subgrade strength, design traffic loading, selection of pavement materials and determination of pavement layer requirements (thickness and/or strength).

2.6.2 DCP-DN method

The DCP-DN design method is empirical in nature and the findings are currently based on measurements and observations on a range of soil types and environmental conditions prevailing in South Africa. The method is now being commonly and effectively used in a number of countries in Africa, including Malawi, Tanzania, Ghana and Kenya and could be effectively used in geotechnical environments similar to those countries. In dissimilar environments, further verification and performance monitoring may be required (Kleyn, 1984; Paige-Green and Van Zyl, 2018).

2.6.3 Assessment of subgrade strength

This is based on the strength (DN value) of the subgrade layer at the anticipated long-term equilibrium moisture content (EMC) of the road after it has been upgraded or rehabilitated to a paved standard. Depending on environmental conditions, the EMC in the subgrade may be expected to equilibrate above, at or below OMC when compacted to the highest practicable field density, i.e. refusal density or "compaction to refusal" which is a specific feature of the DCP-DN method (Paige-Green and Van Zyl, 2018).

Testing to ascertain the durability properties of the material is undertaken separately from the DCP-DN test based on appropriate durability testing. Determination of pavement layer requirements is specified in a single DCP-DN structural catalogue that prescribes the pavement layer thicknesses and strengths in 150 mm increments to a depth of 800 mm, i.e. the required strength profile. The layer strengths are varied in relation to traffic loading and increase (decreasing DN value) gradually in relation to an increase in design traffic loading. The design method can be adapted for any selected layer thicknesses or materials available (Kleyn and van Zyl, 1989) and (Paige-Green, 1994).

2.6.4 DCP-CBR method

Assessment of subgrade strength is based on the in-situ worst-case long term conditions similar to that obtained in the laboratory soaked CBR test. However, in a dry/moderate climate it is assumed that the subgrade CBR strength value is halved which is equivalent to a shift upwards of one subgrade class (Gourley, 2002).

The DN values are converted to CBR values, based on the TRL DCP-CBR correlation, for input into a CBR catalogue. It should be noted, however, that the ratio between soaked and unsoaked CBRs is significantly less than the research-based ratios developed by both Emery (Emery, 1985) and Paige-Green (Paige-Green et al, 1999). This is likely to lead to the use of higher quality/thicker/costlier pavement layers.

Selection of pavement materials is based on the laboratory soaked CBR test, regardless of climate, and at a specified density likely to be attained in the field. Requirements are placed on the allowable plasticity and grading of the material, the limits of which are related to the class of material, i.e. the higher the class, the more stringent the limits and the type of material, i.e. different for pedogenic and non-pedogenic materials.

Determination of pavement requirements (thickness and/or strength) is based on the use of two structural design catalogues, one for dry-moderate climates (N-value > 4) and one for wet climates (N-value < 4). Pavement layer thicknesses are variable and range from 120 mm to 275 mm. For a given traffic loading, layer strengths and/or thicknesses are higher/greater in the wet zone than in the dry/moderate zone (TRL, 1993).

2.6.5 TRH4 method

Assessment of subgrade strength is based on the soaked CBR value, regardless of climatic zone. A minimum CBR value of 3% at 95% Mod. AASHTO is assumed for design purposes, but lower layers in the catalogue may be omitted if the subgrade CBR strength is higher than 3% or, conversely, added if the subgrade CBR strength is lower than 3%. Also, the selection of pavement materials is based on the soaked CBR value. In addition, requirements are placed on the allowable plasticity and grading of the material, the limits of which are related to the class of the material, i.e. the higher the class, the more stringent the limits as stipulated in TRH4 (Theyse et al. 2006).

Determination of pavement requirements (thickness and/or strength) is based on the use of two structural design catalogues, one for dry-moderate climates (N-value > 2) and one for wet climates (Nvalue <2). Pavement layer thickness varies between 100 and 200 mm and layer strengths are varied in relation to the geo-climatic zones – dry/moderate (Weinert N value > 2) and wet (Weinert N value < 2). Thus, for a given traffic loading, layer strengths are higher in the wet zone than in the dry/moderate zone (TRL, 1993).

Assessment of subgrade strength is based on the moisture content equal to the wettest moisture condition likely to occur in the subgrade after the road is opened to traffic, i.e. the long-term, in-service, equilibrium moisture content. Three categories of subgrade condition are assessed:

Category 1 Subgrade where the water table is sufficiently close to the ground surface to control the subgrade moisture content. In this case, the moisture content is determined from similar roads in the vicinity or from a knowledge of the relationship between suction and moisture content for the subgrade soil. In practice, this moisture content is likely to be at or above OMC.

Category 2 Subgrade with deep water tables and where rainfall is sufficient (> 250mm) to produce significant changes in moisture conditions under the road. The moisture condition for design purposes can be taken as the optimum moisture content given by the BS Standard (Light) Compaction Test (2.5 kg rammer method).

Category 3 Subgrade in areas with no permanent water table and where the climate is dry throughout most of the year (annual rainfall 250 mm or less). For design purposes a value of 0.80 OMC obtained in the BS Standard (light) Compaction test (2.5 kg rammer method).

Selection of pavement materials is based on the soaked CBR of 80% for the base course and 30% for the subbase, regardless of climatic zone. Requirements are placed on the allowable plasticity and grading of the pavement materials, the limits of which are related to the design traffic class and moisture regime, i.e. the higher the class and the wetter the anticipated moisture regime, the more stringent the limits (TRL, 1993).

Determination of pavement requirements (thickness and/or strength) is based on the use of one structural design catalogue. Pavement layer thickness varies between 100 and 350 mm and layer strengths are varied as discussed above. The layer strengths are varied in relation to traffic loading and increase (decreasing DN value) gradually in relation to an increase in design traffic loading.

2.6.6 The DCP Test

The Dynamic Cone Penetrometer (DCP) is a simple yet robust piece of equipment that can characterize the ground conditions in and beneath an unsealed road quickly and with accuracy appropriate to the requirements of the design procedure. Since its development in the 1950s (Scala, 1956), the apparatus and its use and interpretation of results has increased significantly (Paige-Green & Du Plessis, 2009). The result is that the DCP is used extensively for many road projects, including the rehabilitation design of even heavily trafficked roads (COLTO, 1997). Although its use for the design of light pavement structures was first proposed in 1987 (Kleyn & Van Zyl, 1987), its implementation was minimal. Paige-Green (2011) reported on his personal experience with its use and presented a slightly refined technology for its implementation.

The DCP equipment consists of a steel cone (20 mm diameter with a 60º angle) that is driven into the ground under a fixed energy (an 8 kg mass falling through 575 mm). The rate of penetration into the gravel or soil material (DN in mm/blow) has been found to be a reasonably good predictor of the California Bearing Ratio (CBR) at the prevailing in situ moisture and density conditions. Of major significance is the fact that the DCP test assesses the material conditions at their in-situ density and moisture content. Both of these need to be taken into consideration when interpreting the results of the DCP survey (Kleyn, 1984)

Tables to convert DCP penetration rates into in-situ CBR values (in terms of the South African G-class materials classification system) for unsealed road wearing courses and subgrades as well as sealed low volume road structural layers and subgrades have been developed and published (Paige-Green 2011). As the DCP penetration rate (DN) and the CBR values are essentially interchangeable, either of these can be used in the pavement design process.

2.6.7 DCP Survey

For an old unsealed road that has carried many heavy vehicles over its service life, the underlying in situ material is often highly compacted giving strong support over considerable depth (deep pavements usually with a well-balanced structure) (Paige-Green & Du Plessis, 2009). This deep pavement structure should be retained and used to greatest benefit in the new pavement structure.

In order to determine the condition of these materials beneath the proposed pavement structure, a DCP survey should be carried out. This will normally consist of a DCP test (to 800 mm depth) at a spacing of between 100 and 500 m depending on the typical material types, variability, drainage conditions, etc. When in doubt, or in the absence of data regarding the in situ conditions, the smallest spacing should be used.

It is imperative that an estimate of the in situ moisture condition is made at the time of DCP testing. This should assess the moisture condition in and beneath the unsealed road in terms of whether it is at the expected in-service moisture condition (outer wheel track) or much wetter or much drier. Based on this assessment, a statistical estimate of the likely strength in the final road pavement was made. As many DCP tests as possible should be carried out during the survey and it is suggested that at least 20 test results should be available for each uniform section. This will ensure some statistical reliability.

DCP tests are designed to eliminate the structural capacity of pavement layers and embankments. Livneh et al. (1989) demonstrated that the results from penetration test correlate well with the in-situ CBR values. Livneh and Ishai (1987) conducted a correlative study between the DCP values and the CBR value. During their studies, both CBR and DCP tests were done on a wide range of undisturbed and compacted fine grained soil samples, with and without saturation in the laboratory. Field tests were performed on natural and compacted layers representing a wide range potential pavement and subgrade materials. It is generally recognized that the performance of a pavement is strongly affected by the characteristics of the subgrade soil. The presence and variations of moisture affect the durability and strength characteristics of soil, subsequently the ability of the pavement to support the pavement. Based on the moisture regime at the time of testing, a percentile value for the DN value of each layer can, however, be determined. The recommended percentiles used are those shown in Table 2.1, although these can be modified depending on the experience and judgment of the designer.

Table 2.1: Percentile values of DN

	Percentile of strength profile (maximum penetration rate – DN)	
	Materials with strengths not moisture sensitive*	**Materials with strengths that are moisture**
Wetter than expected in	20	20 - 50
service	50	50 - 80
Expected in service moisture	80	80 - 90

* Moisture sensitivity can be estimated by assessing the 'flatness' of the CBR moisture content curve. Materials with a flat curve are considered to be of low moisture sensitivity while steep curves indicate highly moisture sensitive

The rationale behind these percentile values is shown in Table 2.1, where the dry and wet seasonal distributions of the DN values and their 20th and 80th percentiles are plotted and compared with the average in service condition. It can be seen that DCP data collected during the dry season was stronger (lower DN) than that collected during the wet season. The use of the respective 80th and 20th percentiles effectively results in an estimate of the expected in-service moisture conditions. It should also be noted that by using the weighted average DN values, thicker parts of the layer with weak materials will result in a higher overall penetration rate. This ensures that the effects of weak sections within the layer are adequately taken into account (Livneh and Ishai, 1987).

2.6.8 Representative DN Values

In practice, many methods rely on the use of the DCP to determine uniform sections of the road under design by undertaking a CUSUM analysis of the range of values within that uniform section as follows:

DCP-DN uses the 80th, 50th or 20th percentile of the range of values depending on whether the anticipated long-term EMC in the pavement is respectively wetter than, the same or drier than at the time of the DCP survey. TRH4 uses the 90th/10th percentile of the range of CBR/DN values found along the road, as determined from a DCP survey. DCP-CBR uses the mean, lower quartile or lower decile value of the range of CBR/DN values. ORN31 uses the 90th/10th percentile of the range of CBR/DN values within a uniform section as determined from a DCP survey.

The above percentile values for the different design methods were used in the determination of the design subgrade strength in a uniform section of road for pavement design purposes (Gourley and Greening, 1999).

2.6.9 Conversion from DN to CBR

The following relationships (Equations 2.1 and 2.2) were used to convert DN values to CBR values as developed by Kleyn (Kleyn, 1984) and TRL (Samuel and Done, 2005).

Kleyn: $CBR = 410\ x\ DN\ 1.27$ Equation 2.1

TRL: $DN\ =\ 10\text{^}\ (2.48 - Log\ CBR)/1.057$ Equation 2.2

Where DN = the average penetration rate in mm/blow.

It should be appreciated however, that the conversion from DCP-DN values to equivalent CBR values at any stage of the design process will introduce errors due to the relatively poor correlation between DCP and CBR measurements (material specific correlation coefficients range from 0.67 – 0.79 (Sampson and Netterberg, 1990).

2.6.10 Conversion from in-situ DCP CBR to laboratory soaked CBR values

A key requirement for comparing the pavement structures derived from the various pavement design methods is to convert the in-situ DCP-CBR values to equivalent laboratory soaked CBR values for use in the DCP-CBR, TRH4 and ORN31 methods. This was achieved by using the relationship between soaked CBR values and field DCP-CBR values (Paige-Green et al 1999).

2.7 Borrow Pit Materials as Structural Layers

Comparing the in-situ strength and the required strength of a pavement to carry the design load very often indicates the need to import additional layers e.g. subbase and/or base of appropriate quality. Historically, using the DCP CBR design approach, the required strengths of these additional layers were expressed in terms of CBR and suitable materials sourced. The drive towards optimising the use of local materials resulted in testing the DN of available materials at different moisture contents and compaction efforts in the laboratory. Using the DN value determined at different moisture contents and densities for selecting materials as the sole determinant of layer strength would also permit the use of a wide range of locally occurring materials that would otherwise have been rejected. The influence of grading and plasticity of such materials is indirectly measured by the resistance to penetration and would thus not have to be specified separately as discussed earlier (Paige Green and Van Zyl 2019).

The conventional limits for grading and plasticity would be considered in this decision although the primary selection criterion would be the strength (as DN in mm/blow) as measured in the laboratory by the DCP. Materials with a grading modulus of less than 1 are normally considered to be unacceptable for structural layers in roads during routine testing and control.

2.8 Move towards an Environmentally Optimized Design (EOD) Approach

A further development with the DCP-DN method was combining it with the Environmentally Optimized Design (EOD) concept (TLL, 2008) for low volume roads. This entails the definition of uniform sections along the road based on the DCP DN value, including local environmental issues (e.g. topography, drainage, etc.) that will affect the pavement design and performance. In this way, the pavement design (particularly in terms of the expected moisture conditions) can be varied to accommodate localised drainage conditions. Although it has been clearly shown that the moisture content in roads seldom exceeds the optimum moisture content for the materials involved (Emery 1985)], localized impeded drainage conditions may require the use of soaked material results over limited sections of the roads.

2.9 In-Situ Moisture Content and Pavement Strength of Low Volume Roads

Subgrade soil strength and/or stiffness are major factors that affect the design and performance of pavements, particularly low-volume pavements. A practical method of realistically estimating in situ moisture content significantly improves the determination of the appropriate resilient modulus to be used for pavement design. Because of the variability in soil properties and soil behavior under repeated traffic loads, environmental factors, geometric factors, and site conditions, and because of the complexity of moisture movement in soils, the prediction of subgrade moisture content has been unreliable and complicated.

The upper and lower equilibrium limits for subgrade moisture contents are estimated. These equilibrium values are independent of environmental factors and are solely dependent on soil properties and site conditions. Regression equations to predict upper and lower equilibrium values from soil properties are developed. It is shown that reasonable predictions of in situ moisture content may be developed, given the range of subgrade moisture content variation for a given soil type and the trends of moisture variation with temperature, precipitation, and depth. In addition, guidelines and issues to be considered when establishing a subgrade moisture content monitoring program are given. The information presented could provide agencies with responsibility for low volume roads valuable tools for obtaining reasonable estimates of subgrade moisture conditions without the need for extensive (and expensive) soil sampling and testing programs (Paige-Green and Van Zyl, 2018)

Drainage is undoubtedly one of the most important factors that affects the long-term performance of a LVR, given adequate construction practice, maintenance attention and control of overloading. Thus, the assumed long-term equilibrium moisture content (EMC) is critical in that it affects the strength of the material in the pavement layers and the subgrade.

For purposes of the pavement design and LCC analyses, it has been assumed that, for all four design methods under consideration, adequate drainage prevails. In terms of currently recommended practice, this means that the level difference between the crown of the road and the invert of the drain (gradient dependent), should be about 0.75 m on relatively flat ground and slightly less on steeper ground) and, where feasible, the level distance between the original ground level and the underside of the subbase layer should be about 0.15 m (Emery,1985).

2.10 Moisture Regime

The moisture regime in which a LVSR pavement must operate has a particularly significant impact on its performance due to the use of locally occurring unprocessed materials which tend to be relatively moisture sensitive. This places extra emphasis on drainage and moisture control for achieving satisfactory pavement life.

Each climatic zone will generally provide a different moisture regime which, other than in localized areas of micro climate, would be related to the Weinert N-value – the lower the N-value, the greater the availability of moisture during the year to wet up the pavement, and vice versa. The various sources of moisture infiltration into a pavement are illustrated in Figure 2.6.

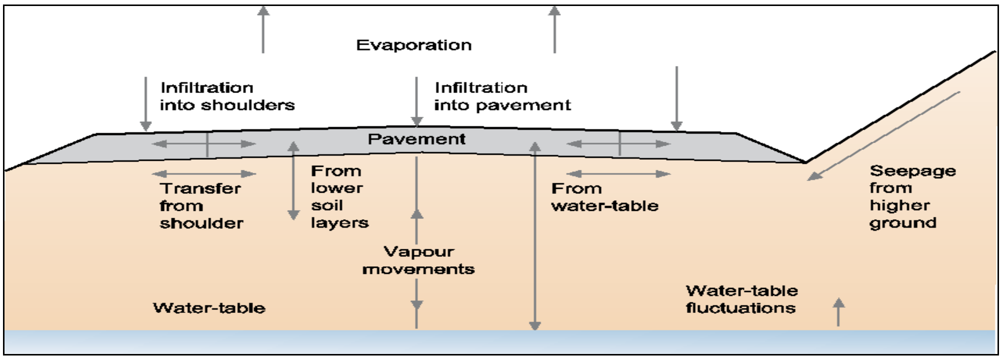


Figure 2.6: Moisture movements in pavements and subgrades -SATCC 2003

2.11 Summary of Literature Review and Research Gap

LVSRs studies by Pratico and Ginta (2011), and Russell and Kornala (2003) have contributed on the need to seal LVSRs for social economic roles. Shearer and Scheete (2011) and Wayne et al, (2011) have discussed the flexible pavement structure of LVSRs constructed on subgrade layer and that service life depends on material quality and thickness of granular layer.

Paige Green and Du Plessis (2009) have researched on the need to retain and use in-situ material for the new, deep pavement structure through the use of DCP. Livheh et al, (1989) has also demonstrated the correlation of DCP values and CBR values on undisturbed and compacted fine grained soil samples.

Bradbury 2012, Paige-Green (2009), Overby (2010), Colto (1996) and SATCO (2003) have carried out studies that delved with the effects traffic and loading on the strength of LVSRs pavements.

Paige-Green (2019), Rolt and Pinard (2016), and Grourley (2002) have researched on the effects of material densities and moisture on performance of pavement.

This research intends to study the effects of moisture changes on pavement layers' performance before, during and after improvement by modification of cross section, side drainage and bituminous surfacing of LVSRs.

2.11 Conceptual framework

This study intends to determine the effect of In-situ moisture in the low volume sealed roads by additional sub base and base layer, sealing of the surface and enhanced drainage system. The method includes analysis of the in-situ moisture, the material densities and the DCP-CBR relationship. The performance correlations will thus indicate how better performances can be obtained.

In this study, the approach would focus on two key critical characteristics that affect the pavement performance and this include the In-situ moisture and density. A range of other variables that affect performance like plasticity and grading were also recorded.

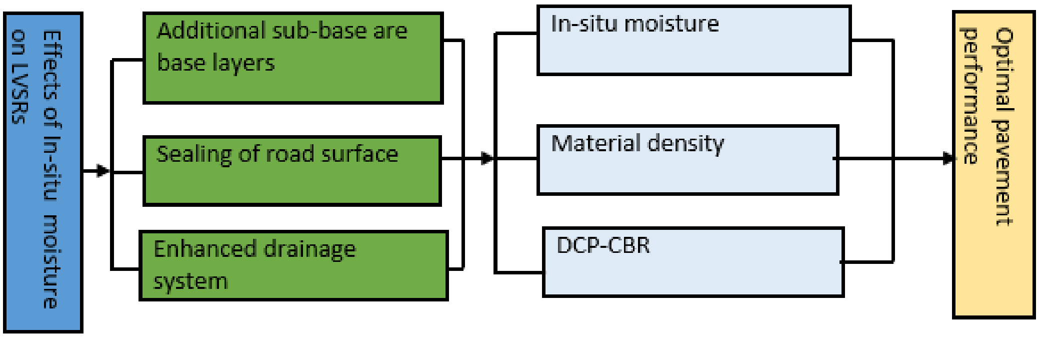


Figure 2.7: Flow diagram of LVSR concept

CHAPTER THREE

MATERIALS AND METHODS

3.1 Introduction

Two roads with varying climatic conditions, soil types but almost similar traffic levels were identified for this research. The Road sections were D470 Kyeni – Karurumo and E628 Wamumu Karaba. The tests on the roads pavement were carried out in three stages: Before construction, during construction and after allowing traffic to flow.

Tests were carried out during construction stage: In-situ moisture content test, classification tests namely: sieve analysis, plasticity limit, liquid limit test and linear shrinkage test, pavement strength tests namely: Dynamic Cone Penetrometer Test (DCP) and California Bearing Ratio Test (CBR).

The following tests were carried out after allowing traffic flow on the road: In-situ moisture content test, Pavement strength tests

3.2 Data Collection Equipment

The following equipment were used to collect and analyze this data: The DCP win tool, camera for taking still pictures of the various machines and laboratory testing equipment

3.3 In-situ moisture and pavement strength of LVSRs sections before improvement

Classification test including grading, atterberg limits and linear shrinkage, compaction test including MDD and OMC, strength test including CBR soaked, OMC and 0.75 OMC, pavement layer properties including IDD and MC and field confirmation of road side drainage

3.4 In-situ moisture, pavement layer strength and improved subgrade strength of LVSRs section during improvement

Classification test including grading, atterberg limits and linear shrinkage, Compaction test including MDD and OMC, strength test including CBR soaked, OMC and 0.75 OMC, pavement layer properties including IDD and MC, field confirmation of road side drainage and DCP and CBR correlation

3.5 Determination of pavement strength and moisture after improvement to LVSR and traffic flow

DCP correlation, visual assessment, surface deflection test, rut depth comparison and skid resistance and riding quality

3.6 Data analysis

Assessment of the in-situ moisture and pavement strength through tests before the improvement, assessment of the in-situ moisture and pavement strength through tests during the improvement, the pavement strength and moisture change tests after the improvement and traffic flow and correlation of CBR and DCP results and moisture change before, during and after the improvement.

3.7 Compaction of samples

Some natural, particularly pedogenic, gravels (e.g. laterite, calcrete) exhibit a self-cementing property in service, i.e. they gain strength with time after compaction. This effect was evaluated as part of the test procedure by allowing the samples to cure prior to testing in the manner prescribed below.

The samples were thoroughly mixed and split each borrow pit sample into nine sub-samples for DN testing in a CBR mould at three moisture contents: (a) soaked, (b) at OMC and (c) at 0.75 OMC and three compactive efforts: (a) BS Light, (b) BS Intermediate and (c) BS heavy.

The sample was allowed to equilibrate for the periods shown below before DN testing was carried out to dissipate compaction stresses and to allow the samples to cure.

(a) 4 days soaked: After compaction, soak for 4 days, allow draining for at least 15 minutes, then undertaking a DCP test in the CBR mould to determine the soaked DN value.

(b) At OMC: After compaction, seal in a plastic bag and allow to “cure” for 7days (relatively plastic, especially pedogenic, materials (PI > 6), or for 4 days (relatively non-plastic materials (PI < 6)), then undertake a DCP test in the CBR mould to determine the DN value at OMC. The curing period is required to dissipate pore pressure generated during the compaction process).

(c) At 0.75 OMC: Air dry the sample in the sun (pedogenic materials) or place the sample in the oven to maximum 50 degrees Celsius (non-pedogenic materials) to remove moisture. Check from time to time to determine when sufficient moisture has been dried out to produce a sample moisture content of about 0.75 OMC (it doesn't have to be exactly 0.75 OMC, but as close as possible). Once this moisture content is reached, seal the sample in a plastic bag and allow curing for 7 days (pedogenic materials) or for 4 days (non-pedogenic materials) to allow moisture equilibration before undertaking the DCP test at approximately 0.75 OMC. Weigh again before DCP testing to determine the exact moisture content at which the DN value was determined.

3.8 Particle Size Distribution

Two methods used ware the sieve analysis generally used for coarse grained soils and the sedimentation method, using a hydrometer, used for analyzing fine grained soils or part thereof. The difference between the two methods lies only in the way of making the observations, and sieving is a more direct method for determining particle sizes.

3.8.1 Sieve analysis

A soil was divided into some fractions of percent finer than or passing (AASHTO T88) using a series of sieves; sieves are wire screens made to different standards and having square openings. The finest sieve used in the sieve analysis is 75micron (0.075mm) opening. Wet sieve analysis was carried out where the soil contained a substantial portion of fines passing the 75µm sieve (over 10% by dry weight). Here, the soil of known initial dry weight was washed over this75µm sieve, dried in the oven at 105±5°C and weighed again on cooling to provide the amount of fines as the difference between the initial and final weight. Dry sieve analysis then followed as described below.

In dry sieve analysis, the final weight of washed and oven-dried soil sample was poured into a series of sieves stacked according to their sizes, with the larger aperture sieves over the smaller ones. A receiver or pan was placed at the bottom and a cover to the topmost sieve of the stack. The soil sample was passed through the stack of sieves by shaking manually for about 10 minutes.

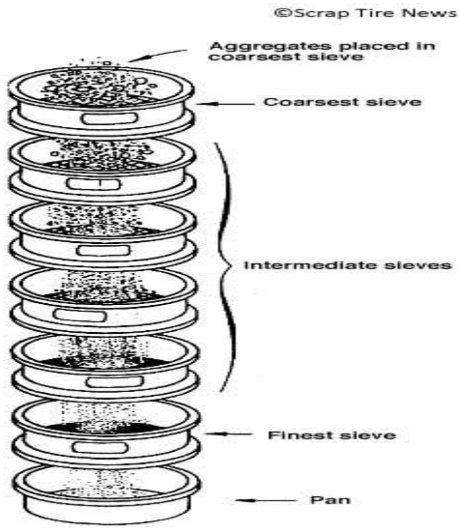


Figure 3.1: An illustration of sieve analysis

The amount of soil retained on each sieve was determined and weighed separately. The weight of material retained on each sieve was converted to a percentage of the total dry weight of sample.

Grain sizes corresponding to 60% finer (D_{60}), 30% finer (D_{30}), and 10% finer (D_{10}) are significant and are called *grading characteristics*. They were, where possible, read off the grading curve. To indicate gradations or uniformity in a soil sample, these values were used to compute the uniformity coefficient and the coefficient of curvature thus:

i) Uniformity coefficient, $C_u = \frac{D_{60}}{D_{10}}$ (Eq.3.1)

ii) Coefficient of curvature, $C_g = \frac{(D_{30})^2}{D_{60} \times D_{10}}$ (Eq. 3.2)

Grading was performed on the two neat materials only with the resulting grading curves defining the limits of the grading envelope for the composite materials.

Curves for the individual composite materials at step percentages were not carried out.

3.8.2 Sedimentation analysis

The sedimentation analysis is most convenient for determining the grain size distribution of silts and clays with soil fraction finer than 75µm size. This was carried out by the *hydrometer method as shown in figure 3.2,* which is simpler and does not require very accurate weighing like the alternative pipette method. Assuming that the soil grains are spherical in shape and have the same specific gravity, the Stokes equation for spheres falling freely in a fluid of known properties was used for this purpose. Stokes law is applicable for spheres of diameter between 0.2 mm and 0.0002 mm (Taylor, 1948).

This is because spheres of diameter larger than 0.2 mm, falling through the water cause turbulence whereas the velocity of settlement for spheres of diameter less than 0.0002mm is too small for accurate measurement (Ranjan and Rao, 1993).

This application is not absolutely correct since most fine grained soil particles are not rounded but rather flat or plate-shaped. However, the method is still in wide use since the resulting effects are of little engineering significance and it is also felt to be a practical way of obtaining reasonable approximations of particle sizes for cohesive soils (McCarthy, 1988).

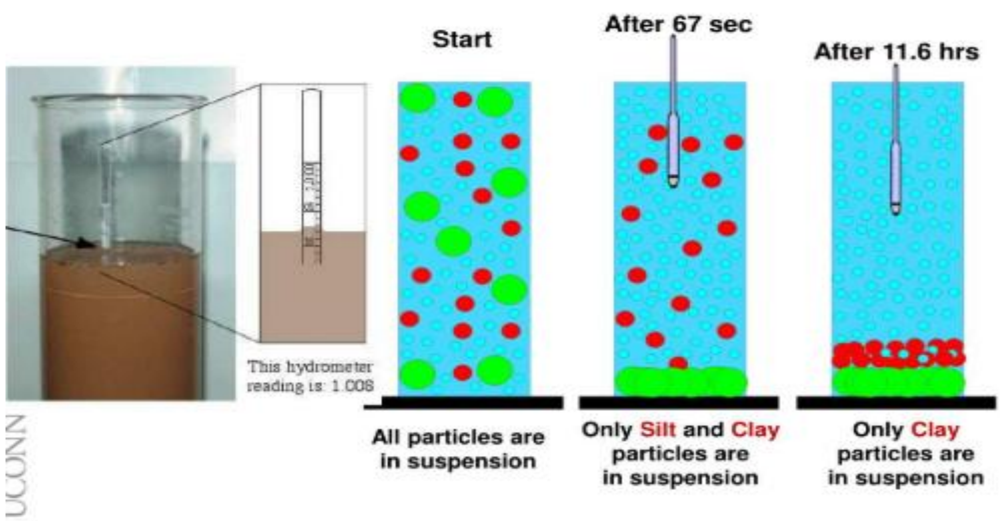


Figure 3.2: Use hydrometer in sedimentation test

The soil was made into a solution with distilled water. 50g of the dry soil sample passing through 75 µm sieve was taken, put in a beaker and then 100cc solution of a deflocculating agent (33 g sodium hexametaphosphate and 7 g sodium carbonate per liter of distilled water) added to it. The sample was allowed to soak for 5 minutes then transferred into the dispersion cup of a mechanical stirrer. The cup was filled to a depth not more than ¾ with distilled water and the soil sample stirred for 10 minutes. The suspension was then transferred into a special 1000 cc glass cylinder of constant cross-sectional area. The total volume in the cylinder was brought to the 1000cc ring mark by adding more distilled water. A duplicate cylinder with the deflocculating agent only was made to the same concentration and level. The two cylinders were immersed in a water bath and the suspensions allowed adequate time to attain the temperature of water in the bath.

The soil suspension in the cylinder was agitated and mixed thoroughly by firmly placing the palm of the hand on the open end and turning the cylinder upside down and back several times for about 1 minute. Once the suspension is mixed well, the cylinder was replaced inside the water bath and a stopwatch started at the instance when the cylinder is held upright prior to its replacement in the bath. The soil particle was permitted to settle out of the suspension and as settling occurred, the average specific gravity of the suspension decreased. Readings were made using a hydrometer at different time intervals, as the specific gravity of the suspension at the centre of volume of the hydrometer, to provide an indication of the weight of soil remaining in suspension and also the information on the particle sizes that have settled out of the suspension. The hydrometer was carefully inserted into the suspension immediately and its readings taken after 1 and 2 minutes; the hydrometer was then removed from the suspension and placed in the duplicate cylinder where it stayed when not in use. Further readings of the hydrometer were taken after 5, 15, 30 minutes, 1, 4 and 24 hours with the hydrometer being inserted into the suspension about 30 seconds before each reading so that it is stable by the time the reading was due. Since the suspension is opaque, the hydrometer reading corresponded to the upper level of the meniscus.

Most conventionally, the test data are reduced to provide particle diameters and the percentage (by weight) that is finer than a particle size. The weight of solids present at any time is calculated indirectly by using the density readings of soil suspension. However, the following corrections were first applied to the observed hydrometer readings:

i) A meniscus correction (C_m) – added to bring the hydrometer reading to the true lower level of meniscus, and

ii) The temperature correction (F) – read off a table if temperature of the suspension was other than the calibration temperature, usually 20°C.

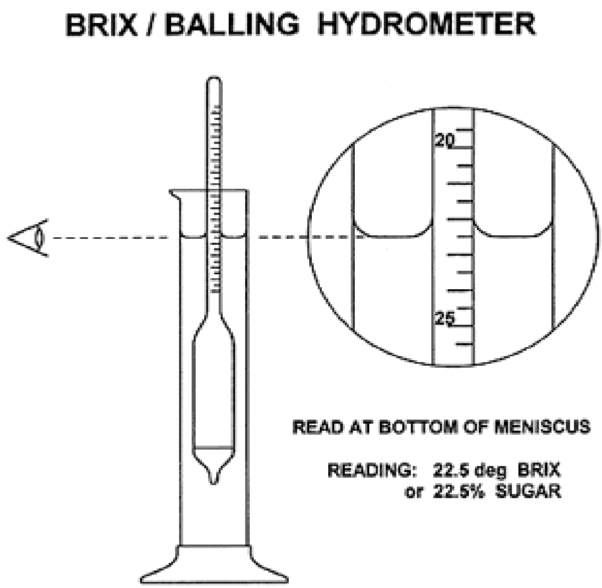


Figure 3.3: Hydrometer calibration

The grain size D corresponding to elapsed time t and effective depth was determined thus:

$$D\,(\mathrm{mm}) = \left[\frac{(18\eta \times H_e)^2}{(\gamma_s - \gamma_w) \times 60t}\right] \qquad \text{(Eq. 3.3)}$$

or $$D\,(\mathrm{mm}) = \left[\frac{30\eta \times H_e}{(\rho_s - \rho_w) \times t}\right]^{1/2} \qquad \text{(Eq. 3.4)}$$

The percentage of particles finer than D is equal to:

$$\frac{\text{Weight of solids per cc at } H_e \text{ after time } t}{\text{Weight of solids per cc in original suspension}} \times 100 = \frac{W_d}{W} \times 100 \qquad \text{(Eq. 3.5)}$$

If R_c was the corrected hydrometer reading, then the percentage of particles smaller than D was:

$$P\% = \frac{100}{W}\left(\frac{G_s}{G_s - 1}\right) \times R_c \qquad \text{(Eq. 3.6)}$$

3.9 Free Swell

The free-swell test is a fairly simple test that was suggested by Holtz and Gibbs (1956). The test is performed using dry soil passing through the 425-micron sieve. Composite materials for the separate step percentages were also be prepared. Exactly 10cc of each soil specimen was poured slowly into 100cc graduated glass cylinder filled with distilled water. The volume of swell was read after 24 hours, from the graduation of the cylinder.

The free swell value is then determined as:

$$\text{Free Swell}(\%) = \frac{\text{Final Volume} - \text{Initial Volume}}{\text{Initial Volume}} \times 100 \qquad \text{(Eq. 3.7)}$$

3.10 Specific Gravity

Also known as *relative density*, specific gravity is the ratio of the mass of a given volume of material (soil particles) to mass of the same (absolute) volume of water. It is the density or unit weight of a soil relative to that of water. Thus,

$$G_s = \frac{M_s}{V_s \rho_w} = \frac{\rho_s}{\rho_w} \qquad \text{(Eq. 3.8)}$$

For the fine soils, a density bottle of 100ml capacity is used to determine the specific gravity of soil particles. The weight of a clean and dry density bottle is determined. An appropriate quantity of oven-dried soil is placed in the bottle and weighed. The soil is then submerged in de-aired distilled water and agitated to remove air bubbles. After carefully topping up the bottle with water and drying the surface, the bottle is weighed again with soil and water. Finally, the bottle is emptied, cleaned, and topped up with de-aired water and then weighed again full of water.

The specific gravity of the soil particles is obtained thus:

$$\text{Specific gravity, } G_s = \frac{\text{Mass of soil}}{\text{Mass of water displaced by soil}} = \frac{W_2 - W_1}{(W_4 - W_1) - (W_3 - W_2)} \qquad \text{(Eq. 3.9)}$$

Where W_1 = Weight of empty density bottle

W_2 = Weight of bottle + dry soil

W_3 = Weight of bottle + soil + water

W_4= Weight of bottle full of water

3.11 Indices of Consistency

Consistency is that property of a soil which is manifested by its resistance to mechanical deformation or flow, and it varies in proportion to the water content. Thus, a clay sample may behave like a liquid, exhibit plastic behavior or may be very stiff depending upon the water content. The numerical values assigned to the boundary moisture content between these states of a soil are popularly known as *Atterberg Limits*.

The consistency limits is measured by a standard procedure and expressed in percentages of water content as follows:

a) *Liquid Limit* – Denoted as w_L, this is the boundary between the liquid and plastic states when a soil mixed thoroughly with some water changes from liquid to plastic state.

b) *Plastic Limit* – Denoted as w_P, this is another boundary where the soil sample changes from plastic state to the semi-solid state when the water content is reduced further.

c) *Shrinkage Limit* – Denoted as w_L, this is the boundary where the soil changes from a semi-solid to a solid state with a further reduction in the water content.

Figure 3.4: The Casagrande Apparatus

3.11.1 Determination of liquid limit

The liquid limit is determined in the laboratory using a standard apparatus, popularly known as the Casagrande apparatus (Figure 3.4) that is universal. This consists of a brass cup, suitably mounted and resting on a vulcanized rubber compound base. The cup can be raised and made to fall on the rubber base through a cum arrangement operated by a handle. The height of fall of the cup is adjusted to 1cm by means of an adjusting screw. The apparatus comes with a grooving tool that is used to cut a groove in a pat of soil.

About 120 g of air-dried soil passing 425 micron is taken and mixed with distilled water until it attained a putty-like consistency. A portion of the paste is carefully placed in the cup to a maximum depth of about 1cm. A groove is cut in the soil pat using the grooving tool resulting in a groove 2mm wide at the bottom, 11mm wide at the top and 8mm deep. In cutting the groove, the tool is drawn through the sample along the symmetrical axis of the cup towards the front, always holding perpendicular to the surface of the cup. The handle is rotated at the rate of 2 revolutions per second. The number of blows necessary to close the groove for a length of 12.5mm is noted and recorded, ensuring that the groove closed by flow and not by slipping of soil. About 10g of soil near the closed groove was taken to determine moisture content by oven-drying at 105± 5°C.

By altering the water content of the sample, the procedure is repeated and 4 to 5 readings of water content in the range of 10 to 40 blows obtained. A graph known as *flow curve* is then plotted for the water content on a natural scale against the number of blows on a logarithmic scale. The liquid limit is read off as the water content corresponding to 25 blows on the flow curve. The slope of this curve, I_f, is called the flow index.

3.11.2 Determination of plastic limit

About 50g of air-dried soil passing 425 micron is taken and mixed with a sufficient quantity of distilled water to make a soil mass plastic enough to be easily shaped into a ball. A portion of the ball was taken and rolled on a glass plate with the palm of the hand into a thread of uniform diameter throughout its length. On reaching a diameter of 3mm, the soil is remolded into a ball and rolled again. The process of remolding and making the thread is repeated until the sample at a diameter of 3mm just start crumbling. Some of the crumbled portion of thread is dried in the oven at 105±5°C for water content determination.

The test is repeated twice with fresh samples. The average of the three values of moisture content is taken as the plastic limit of the soil.

3.11.3 Determination of linear shrinkage

The actual shrinkage limit test uses the liquid metal mercury and it was not performed because of the environmental hazards involved. The simpler and safer linear shrinkage test that uses a standard brass mould was performed.

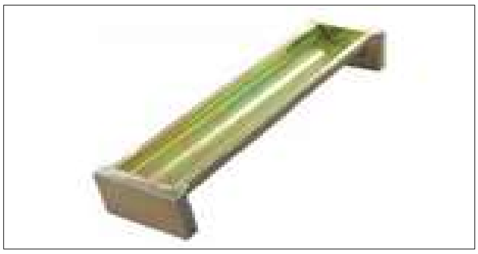


Figure 3.5: Linear shrinkage brass mould

The brass mould was cleaned and a thin film of oil was applied internally; its internal length was also measured and recorded. 150g of air-dried soil passing 425 micron sieve was taken for the linear shrinkage test. This portion was thoroughly re-mixed with distilled water to form a smooth homogeneous paste at approximately the liquid limit of the soil. The soil paste was placed into the mould, taking care not to entrap air, and the surface was then struck off level. The soil was allowed to air-dry until it had shrunk clear of the mould and then placed in the oven to complete the drying at 105±5°C. After cooling, the final length of the sample was measured and the linear shrinkage obtained the equation 3.10:

$$\text{Linear shrinkage, LS} = \frac{\text{Initial length} - \text{Final dry Length}}{\text{Initial Length}} \times 100 \ldots\ldots \text{(Eq. 3.10)}$$

If the soils have very small clay content like the laterite soil, the liquid and plastic limit tests results are not reliable.

According to Whitlow (1996), an approximation of plasticity index may be obtained in such cases from the linear shrinkage using the equation 3.11:

$$I_p = 2.13 \times LS \ldots\ldots\ldots\ldots\ldots\ldots \quad \text{(Eq. 3.11)}$$

3.12 Compaction

Compaction is a process that typically employs an input of mechanical energy to force soil particles, lubricated with some water, into a closer state of packing with a corresponding reduction in volume, and the expulsion of air.

To assess the compaction potential of a soil, one of three standard laboratory tests – the Proctor test – was used on the neat black cotton soil and its composites with the laterite soil. The state of compaction of soil was conveniently measured using the dry density, the attainable values of which were related to water content.

Figure 3.6: Manual Proctor compaction apparatus

3.11 Proctor Compaction Test

The *standard compaction* (AASHTO T99) test was employed to express the moisture/density relations of soils using a standard mould. The volume of the mould was not more than 1000cm^3 (it was actually 997.5cm^3), together with a 2.5 kg rammer having a free-fall of 30 cm. About 5-6 batches of soil passing 20mm sieve and each weighing 2.5kg were mixed with water that is varied by 5% intervals from the initial air-dry condition of the soil. Fresh soil specimens were used each time since materials susceptible to crushing like the laterite soil must not be re-used for the test. The fraction of material retained on 20mm, was recorded for purposes of adjusting the final results. Each batch was compacted in the mould in 3 equal layers that received 25 blows each from the 2.5 kg rammer, using an automatic compaction machine. The bulk density of the compacted specimen is computed thus:

$$\text{Bulk density, } \rho = \frac{\text{Mass of Mould and Wet Specimen} - \text{Mass of mould}}{\text{Volume of mould}} = \frac{M_2 - M_1}{V} \quad \text{(Eq. 3.13)}$$

From the values of bulk density (ρ) and water content (w) obtained, the dry density is calculated thus:

$$\text{Dry density}, \rho_d = \frac{\rho}{(1+w)} \qquad \text{(Eq. 3.14)}$$

The graph of the dry density against water content is plotted for each soil or admixture, and the all-important parameters of the Maximum Dry Density (MDD) and Optimum Moisture Content (OMC) was read off.

3.13 California Bearing Ratio

The California Bearing Ratio, popularly referred to as the CBR, is a special and simple strength test; it compares the bearing capacity of a material with that of a standard well-graded crushed stone. Actually, it is a comparative measure of the shearing resistance of a soil.

The CBR test is basically a laboratory penetration test. It is normally performed on samples compacted at 100% MDD (AASHTO T99) after 4 days' soak where its swelling is monitored.

3.13.1 CBR Laboratory Test

For CBR compaction, batches of soil with stabilizer were prepared by mixing with the desired proportion of potable water obtained from the moisture-density relationship. The mixes consisted of 0, 25, 50, 75 and 100 % by weight of laterite soil. Also, a bigger mould measuring 150 mm in diameter and 175 mm high (2360 cm^3 mould) and of known weight was used. However, a 50mm thick circular spacer disk was placed inside the mould to create a void at the bottom of the specimen. Then a filter paper, with details of the specimen written on it and facing down, was placed on top of the disk before using the same standard rammer weight and drop height to compact the soil in three equal layers. Each layer received 62 blows to allow for the larger surface area of compaction, trimming the final layer level with the top of the mould and then inverting the mould so that the void space came to the top. To obtain the compacted density, the mould with sample was weighted and moisture content of the soil determined.

The mould was assembled for soaking of the compacted sample in a water tank by inserting a swell plate in the void space and mounting a special tripod stand with dial gauge. The mould was raised from the tank base such that the specimen would imbibe water only from the bottom side of the baseplate. Water was carefully added to the tank to a level just below the top of the mould and swell monitored daily for 4 days, noting whether or not water would appear on the surface of the specimen within the first 3 days. If water did not appear by end of day 3, the specimen would be submerged in water for the whole of day 4.

The amount of swell was expressed in terms of *expansion ratio* as a percentage of the initial specimen height.

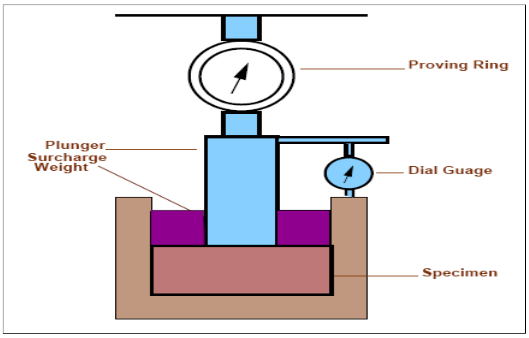


Figure 3.7: Schematic illustration of CBR test

At the end of day 4, the specimen was removed from the water tank and allowed to drain freely for about 15 minutes. It was then transferred to a motorized CBR testing machine where a small initial seating load was applied to a standard piston size measuring 1935mm^2. The penetration test followed and involved the application of load to the piston so as to penetrate the soil specimen at a constant rate of 1mm per minute. The total load was measured by means of a proving ring and was recorded at intervals of 0.5mm between 0 and 7.5mm of penetration. However, an initial reading was made at 0.1mm to help check on the need for curve correction. A plot was drawn of the force against penetration, corrections made if necessary, on the initial stage of the graph and the forces corresponding to the 2.5 and 5.0mm read off.

The loads at penetration depths of 2.5 and 5mm was used to obtain the CBR value by expressing each as a percentage of the corresponding standard load, which are 13.24kN and 19.96 kN respectively. Thus:

$$\text{CBR} = \frac{\text{Force (load) on test soil}}{\text{Force for same penetration on standard sample}} \qquad \text{(Eq. 3.15)}$$

The CBR value is taken as the greater of the two, usually at 2.5 mm penetration

CHAPTER FOUR

RESULTS AND DISCUSSION

In this chapter, results of several material properties of selected pavement section sections have been presented and discussed. The in-situ moistures and densities before improvement, during improvement and changes after improvement and traffic flow tests was conducted as shown in chapter 3.

4.1 In-situ moisture and pavement strength of LVSRs before improvement

Alignment soils samples were collected from trial pits dug to depths ranging from 1m to 1.5 m below existing road level at intervals of 500m. The samples were tested at the Norken Ltd. materials testing laboratory for the following:

1) Atterberg Limits
2) Linear Shrinkage
3) Particle Size Distribution
4) Standard Compaction Test (AASHTO T99)
5) Particle Density
6) Standard CBR Test (4 days soak)

The data are given in Appendix A and analyzed results discussed in the sections that follow.

4.1.1 Classification of alignment soils

Alignment soils on the project roads are predominantly silts and sandy silts of intermediate to high plasticity (PI 10 to 39), interposed with short sections of sandy clays.

Under the AASHTO classification system, subgrade soils are classified and rated according to the parameter given in Table 4.1 and in the graphs of PI verses LL given in Figure 4.1 and 4.2.

Table 4.1: Classification of the Soils

General Classification	Granular Materials (35% or Less Passing 0.075 mm)							Silt-Clay Materials (More than 35% Passing 0.075)			
Group Classification	A-1		A-3	A-2				A-4	A-5	A-6	A-7
	A-1-a	A-1-b		A-2-4	A-2-5	A-2-6	A-2-7				A-7-5 A-7-6
Usual types of significant constituent materials	Stone fragments, gravel and sand		Fine sand	Silty or clayey gravel and sand				Silty soils		Clayey soils	
General Ratting as Subgrade	**Excellent to Good**							**Fair to Poor**			

Plasticity index of A-7-5 subgroup is equal to or less than the LL - 30. Plasticity index of A-7-6 subgroup is greater than LL – 30. The AASHTO classifications of the soils are as shown in the figures below.

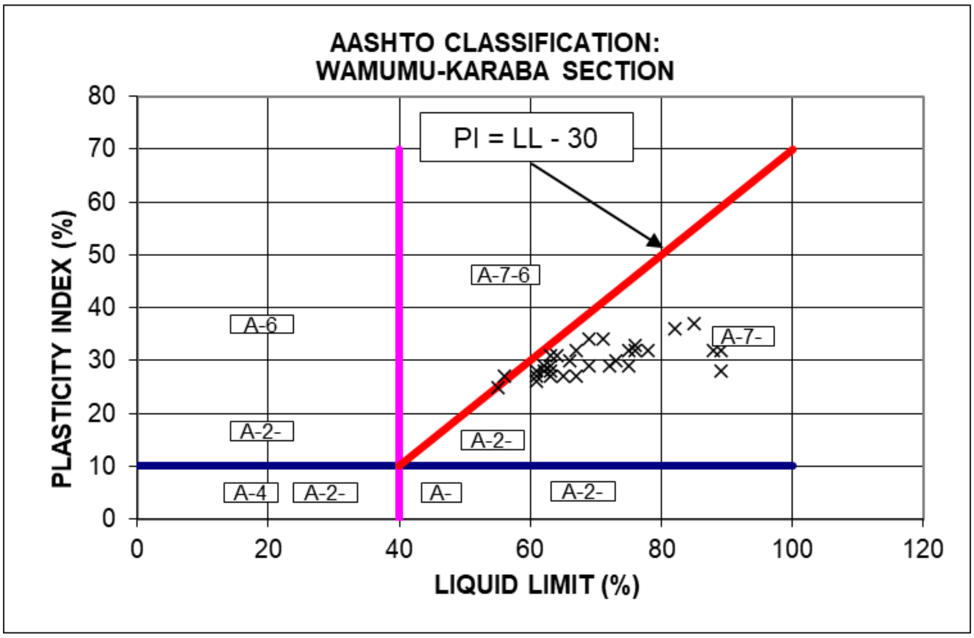


Figure 4.1: AASHTO classification of the soils (Wamumu - Karaba)

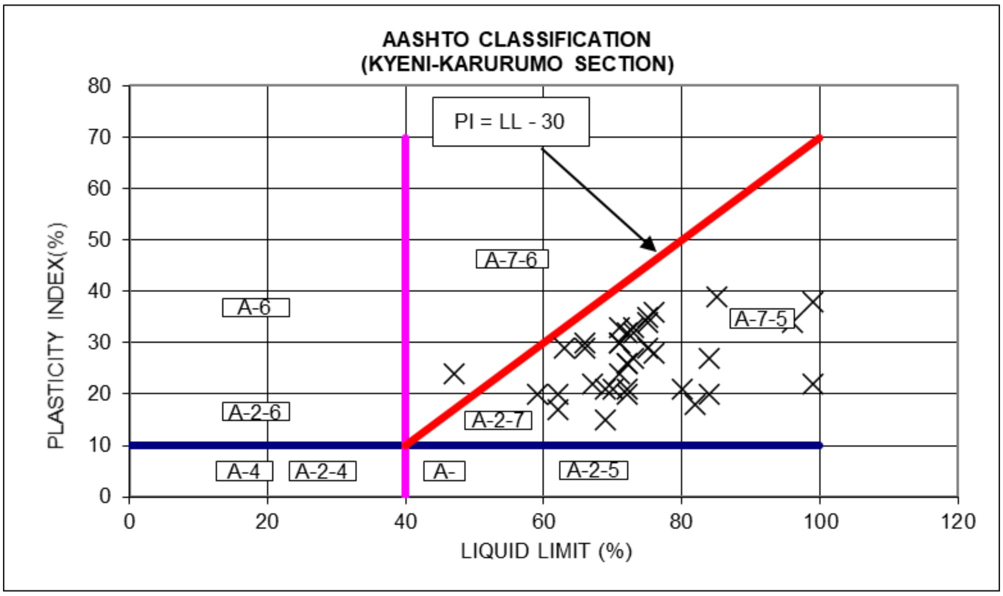


Figure 4.2: AASHTO classification of the soils (Kyeni-Karurumo)

The materials for both road sections are predominantly A-2 containing borderline granular materials. From the above findings the results of the tests for classification for the alignment soils conformed to the requirements of the BS standards and LVS design guidelines.

4.1.2 Analysis of MDD and OMC of Alignment Soils

Road Design Manual Part III provides guidelines for classification of subgrade soils for pavement design based on CBR. However, Table 6.1.2 of this Manual warns that some ash and pumice soils (standard Compaction MDD < 1400 Kg/m3) cannot be classified for pavement design purposes on the basis of CBR alone. The soils in the project area belong to this category of soils as illustrated in the frequency analyses of MDD and OMC shown in 4.3.

For figure 4.3, the MDD and OMC for Wamumu – Karaba were 1090kg/m³ and 46.5% while from figure 4.4 the MDD and the OMC for Kyeni - Karurumu were 1125 kg/m³ and 45.1% respectively. In both the sections the alignment soils exhibited inferior characteristics that were improved by processing the subgrade and compacting heavily. It was observed that with proper compactive effort the soils improved in strength, density and decreased permeability. The findings met the compaction requirement of subgrade as per the BS standands.

The MDD and OMC were indicative of the compactability and moisture sensitivity of materials and were dependent on the grading and plasticity (Paige-Green, 1999).

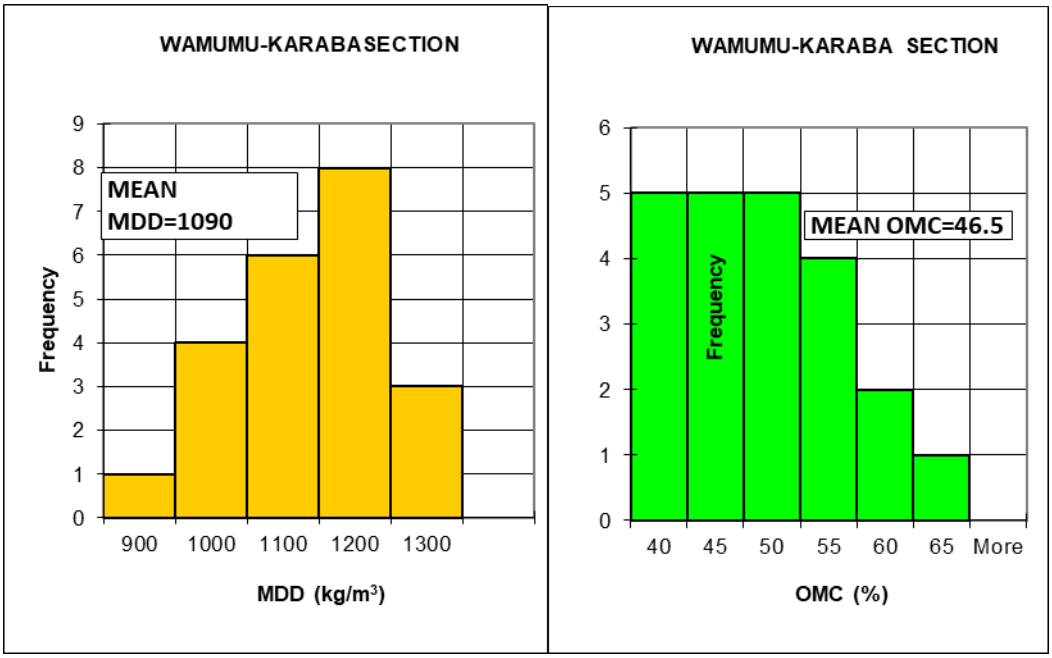


Figure 4.3: MDD and OMC frequency Analysis

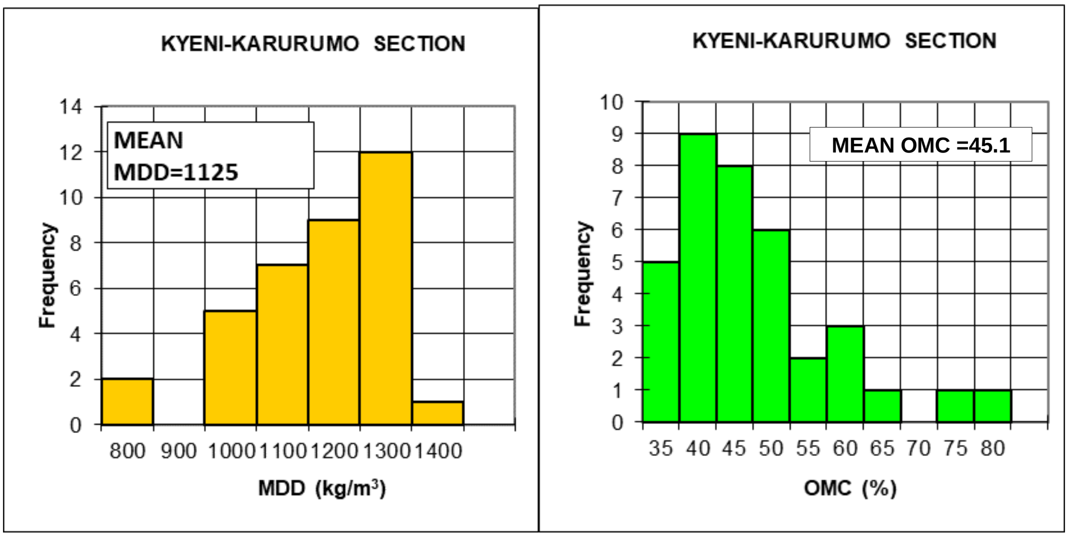


Figure 4.4: MDD and OMC frequency Analysis

4.1.3 In-situ alignment CBR before improvement

A plot of CBRs and resultant CUSUMs has been used to demarcate uniform sections. The 90th Percentile CBR (CBR $_{design}$) values for the demarcated uniform sections for each road section are summarized in the table 4.2 below:

Table 4.2: Percentile Design CBR

Road Section	Uniform Section No.	Chainage From km	Chainage To km	Length km	90th -%ile CBR (%)	CBR $_{design}$ (%)	Subgrade Class
Wamumu-Karaba	1	0+000	1+500	1.5	5.0	5	S2
	2	1+500	3+000	1.5	8.0	8	S3
	3	3+000	5+000	2.0	8.8	9	S4
Kyeni-Karurumo	1	0+000	5+000	5.0	8.0	8	S2
	2	5+000	10+500	5.5	5.0	5	

From the table 4.2, it is apparent that in spite of the AASHTO class and low MDDs of the soils, certain road sections classify as having bearing strengths higher than S1. The higher subgrade classes (S2, S3 and S4) have therefore been ignored and subgrade class S1 used for all road sections. CBR values also increased with increase in compaction and therefore substandard CBR can be improved by calling for higher than normal compaction. The findings compared to the requirements of the Roads 2000 Manual and the Road Design Manual Part 1.

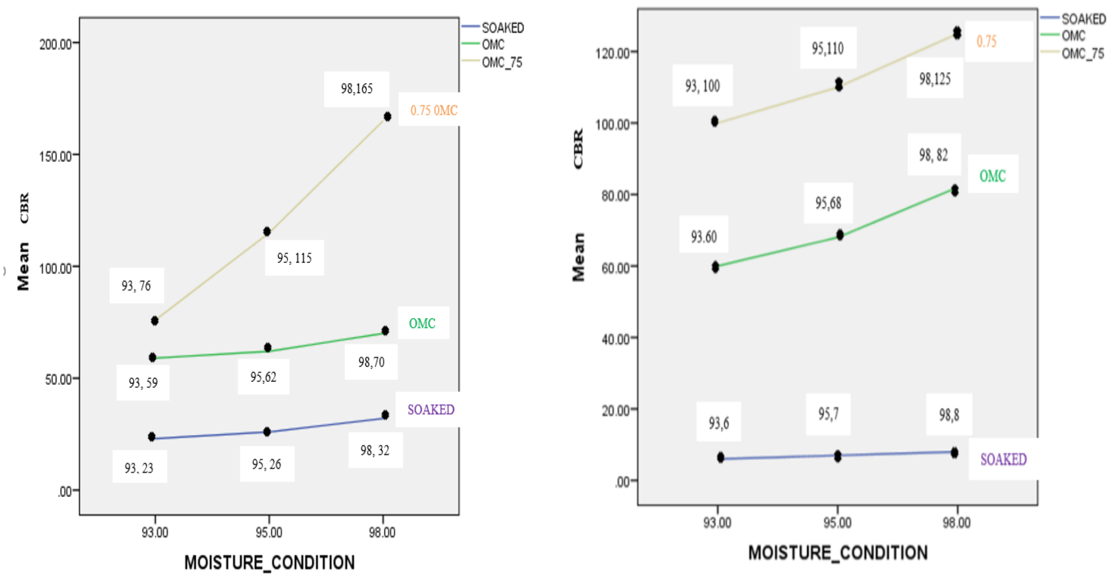
4.1.4 DCP analysis before improvement

DCP tests for D470 and E628 roads were carried and material samples were taken of the gravel and subgrade layers for determination of in situ moisture content and the CBR at various compaction efforts and moisture contents as shown in Table 4.3 below.

Table 4.3 Moisture contents at various compaction efforts

Road Name: D470											
Layer	**Atterberg Limits**				**Compaction T180**		**Moisture condition**	**CBR at various compaction levels**			**Increase from 93% to 98%**
					MDD	OMC					
	LL	PL	PI	LS	(Kg/m)	(%)		93%	95%	98%	
Lateritic Gravel (for base)	46	24	22	11	1890	13.6	4-Days soak	23	26	32	39%
							OMC	59	62	70	19%
							0.75 OMC	76	115	165	117%
Subgrade	56	26	30	15	1510	25.5	4-Days soak	6	7	8	33%
							OMC	60	68	82	37%
							0.75 OMC	100	110	125	25%

From the Figure. 4.5 a and b below, it can be seen that at 0.75 of OMC the mean CBR values are higher than both the OMC and the 4-day soaked condition.



Base (a) **Subgrade (b)**

Figure 4.5: The CBR and moisture condition at base and subgrade

For the lateritic gravel, the moisture conditions of 4-days soak results in an average increase of 39% at the various compaction levels. The 0.75-OMC condition gives a CBR of 165 at 98% compaction level giving an average increases of 117%.

In both cases, the 0.75 of OMC results in higher CBR values than the OMC and the 4-day soaked conditions.

From the above findings the relationship between the CBRs of different materials compacted to specified densities (93%, 95% and 98%) at OMC in soaked condition results in scatter between 0.75OMC and soaked CBR (Pinard, 2011).

4.1.5 DCP Correlation

Correlation between the laboratory soaked CBRs and the DCP-CBRs gave a factor of 0.6 for the subgrade and 0.58 for the Lateritic gravel base layer. It has been found that the moisture content of the pavement layers in a sealed road normally fluctuates between 0.75 of OMC and OMC. Non-standard, in situ materials used for Low Volume Sealed Roads, LVSR, have a significantly higher strength at these moisture contents compared to the 4-day soaked CBR values normally used in the pavement design. These materials were found to perform well with adequate compaction and that the bituminous surfacing and drainage system prevents the materials from being soaked during the wet season.

Table 4.3 clearly shows the inherent strength of the in situ subgrade and high PI laterite gravel at expected in service moisture contents. It is this strength one wants to utilize in low cost pavement design of LVSR.

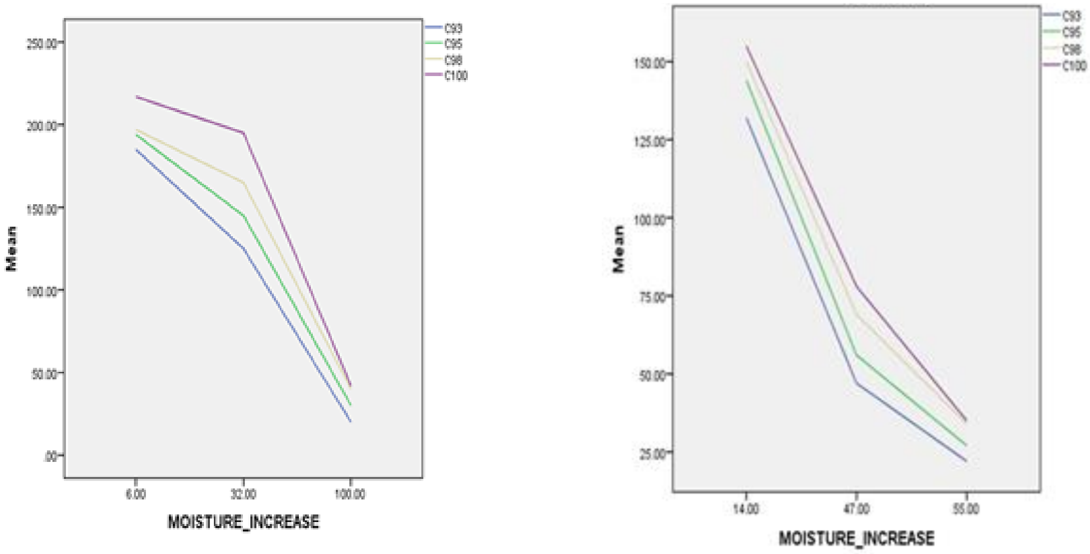
From the graphs the strength of the pavement is optimum when the in-situ moisture averages at 47% and above results in low CBRs of below 50.

From the above findings, it was deduced that for a specific material the relationship between the DN value, density and in-situ moisture content can be established. The above data produced a correlation.

Table 4.4 a: CBR at various moisture content and Compaction (Wamumu-Karaba)

	Sample No	Layer	Ref.	Moisture condition	CBR at various compaction levels				Increase from 93% to 98%
					93%	95%	98%	100%	
Wamumu-Karaba	555/S/2016	Gravel	0+000	4-Dsoak	20	30	40	42	100%
				OMC	125	145	165	195	32%
				.75 OMC	185	194	197	217	6%
	557/S/2016	Gravel	0+200	4-Dsoak	22	27	34	35	55%
				OMC	47	56	69	78	47%
				.75 OMC	132	144	150	155	14%
	558/S/2016	Subgrade	0+350	4-Dsoak	9	12	15	17	67%
				OMC	62	64	68	70	10%
				.75 OMC	128	135	146	150	14%
	559/S/2016	Gravel	0+350	4-Dsoak	5	7	11	12	120%
				OMC	92	102	120	132	30%
				.75 OMC	154	160	167	170	8%

WAMUMU-KARABA



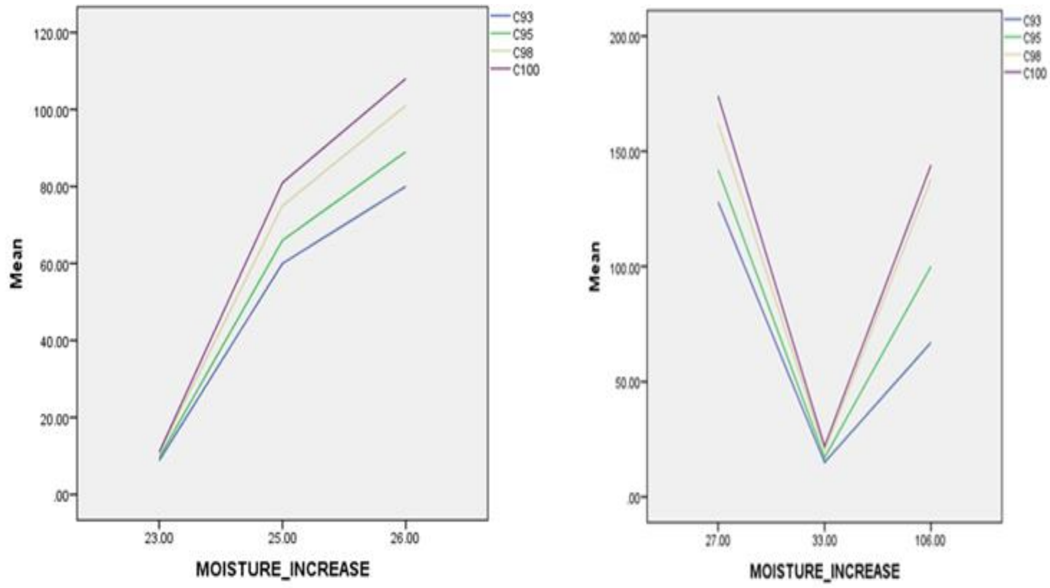
Base **Subgrade**

Figure 4.6a: The CBR and Moisture Increase on Base and Subgrade

Table 4.4 b: CBR at various moisture content and Compaction (Kyeni-Karurumo)

Kyeni-Karurumo	560/S/16	Gravel	0+000	4-Dsoak	15	17	20	22	33%
				OMC	67	100	138	144	106%
				.75 OMC	128	142	162	174	27%
	562/S/16	Gravel	0+450	4-Dsoak	15	18	24	26	60%
				OMC	80	116	160	209	100%
				.75 OMC	124	136	148	158	19%
	564/S/16	Subgrade	0+900	4-Dsoak	15	19	24	25	60%
				OMC	90	94	100	105	11%
				.75 OMC	134	140	146	155	9%
Kyeni-Karurumo	680/S/16	Subgrade	0+000	4-Dsoak	8.8	9.6	10.8	11	23%
				OMC	60	66	75	81	25%
				.75 OMC	80	89	101	108	26%
	684/S/16	Subgrade	0+450	4-Dsoak	24	25	27	28	13%
				OMC	60	66	75	81	25%
				.75 OMC	80	89	100	108	25%

KYENI-KARURUMO



Base **Subgrade**

Figure 4.6: The CBR and Moisture increase on base and subgrade (Kyeni-Karurumo)

It implies that for the road to function normally there needs to be a balanced in-situ moisture by ensuring that the road has proper drainage system so that ingress of rain water into the pavement is minimized.

4.2 Assessment of In-situ Moisture and Layer Strengths During Improvement

4.2.1 Wamumu-Karaba DCP data

The DCP analysis for Wamumu-Karaba in soaked conditions gave the following results as given in Table 4.5. From the table, it can be seen that to penetrate top 150 mm layer 3.2 mm/blow is required and eventually up to 50 mm/blow to penetrate to 800 mm depth. The resulting layer strength diagrams as defined by the DN, the CBR and pavement depth are as shown in Figure 4.5.

From the results it can be seen that using DCP design there a reduction in layers and this is comparable to OR-31 method and SATCC method (SATCC,1998).

Table 4.5: DN values

Depth (mm)	0-150	151-300	301-450	451-600	601-800
DN (mm/blow)	≤3.2	≤6	≤12	≤36	≤50

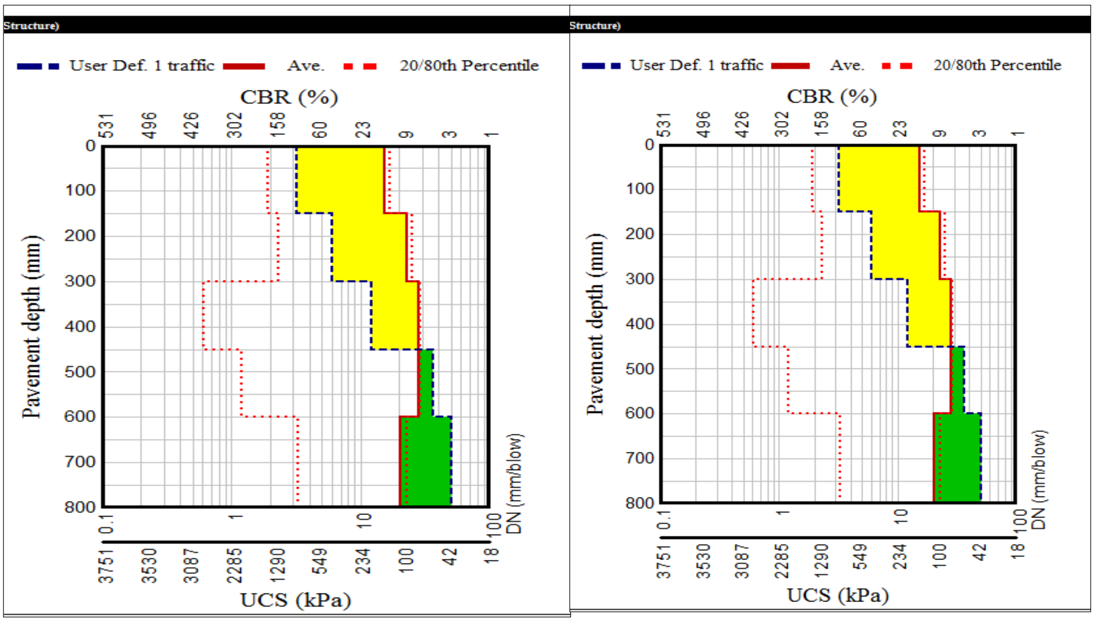


Figure 4.7: Layer strength diagrams during construction

From the layer strength diagram in Figure 4.5 a depth of 450mm had been constructed by adding an improved subgrade, subbase and base and this layer had reduced moisture through compaction and gained strength. It can also be seen that the moisture had not ingressed the lower sections beyond 600mm as they gained even more strength. Further improvement by sealing the road and improving the drainage would protect the layers from wetting and keep the road at the required strength.

From Table 4.5 the DCP tests showed a high variability in the DSN_{800} (the number of blows to penetrate to a depth of 800mm). From a visual assessment, the subgrade (500 – 600 mm) on the whole section is quite uniform. The variability is ascribed to the fact that due to the irregular surface, water was ponding in certain spots and effectively soaking the underlying layers. Other high spots on the other hand remained drier and consequently stronger. Some variability was also due to stones in the top layer.

The existing pavement does not show signs of deep structural failure even in unsealed condition. This is a good indication that the in situ subgrade has adequate strength for the existing traffic. The top layer however became very wet and disturbed during the rains and must be reshaped with more material added to form a uniform base layer. Additional DCP test were done towards the end of the dry season. The final pavement design is based on the weakest spot identified in these tests as shown Figure 4.5. For all layers the 20^{th} percentile (dotted red line to the right of the whole red line for the average weighted DN value) is at or to the left of the DCP design curve, i.e. the design is on the safe side.

From the above findings it can be found that 80% or 20% percentiles are DN values depending on whether the EMC in the pavement was wetter than, the same or drier than at the time of DCP survey (Paige-Green, 2003).

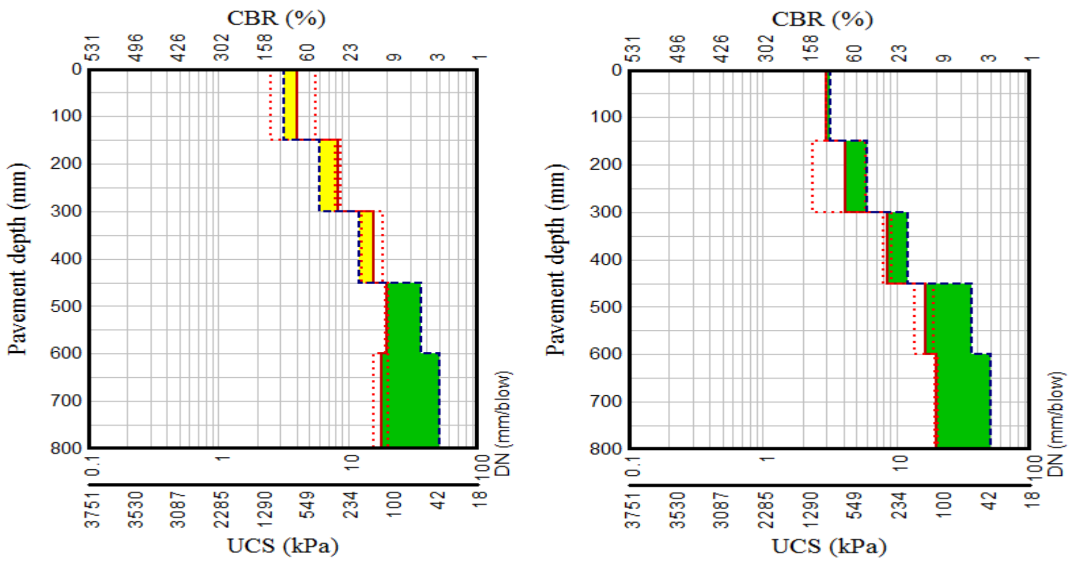


Figure 4.8: Layer Strength diagram with additional 150mm base layer

Table 4.6 below shows the data reported for this spot before and after addition of a 150mm base layer:

Table 4.6: Data report before and after adding 150mm base layer

	Before adding 150mm base		After adding 150mm base		DCP Design Curve Pavement Class LV 0.3
Layer depth mm	Avg. DN (mm/blow)	CBR	Avg. DN (mm/blow)	CBR	
0-150	4.05	69	3	102	≤3.2
151-300	8.39	28	4.11	68	≤6
301-450	15.67	12	8.5	27	≤12
451-600	19.84	9	16.28	12	≤36
601-800	17.92	10	19.70	9	≤50
DSN 800		93		135	
Structural capacity MESA		0.5		1.8	
Pavement balance		ABD		ABD	

From the graph 4.9, it can be seen that the road before improvement has consolidated as it requires 4.5 mm/blows to penetrate 150mm thickness at a CBR of 69. After improvement the road has not fully consolidated as it requires 3mm/blow to penetrate to 150mm thickness and at a higher CBR of 102. The correlation before adding 150mm base is 2.7.

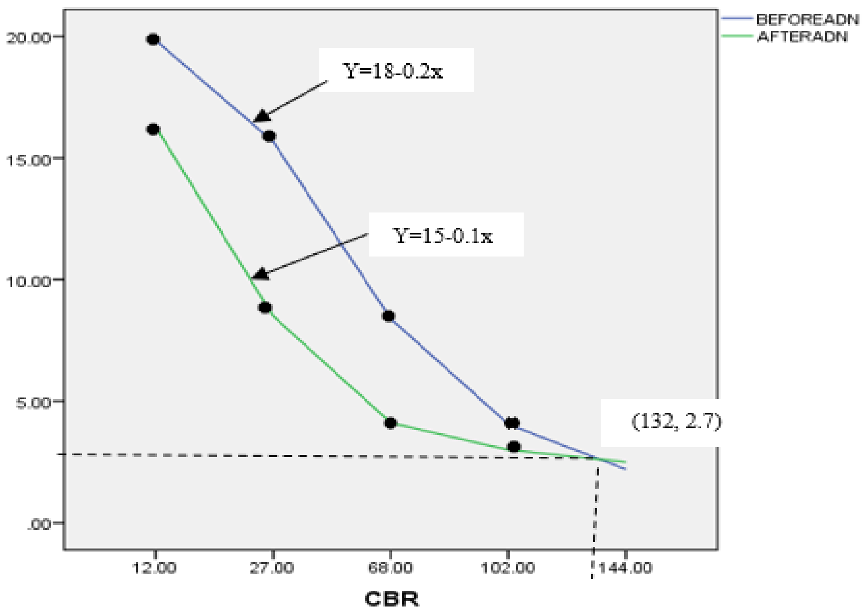


Figure 4.9: CBR before and after adding 150mm base

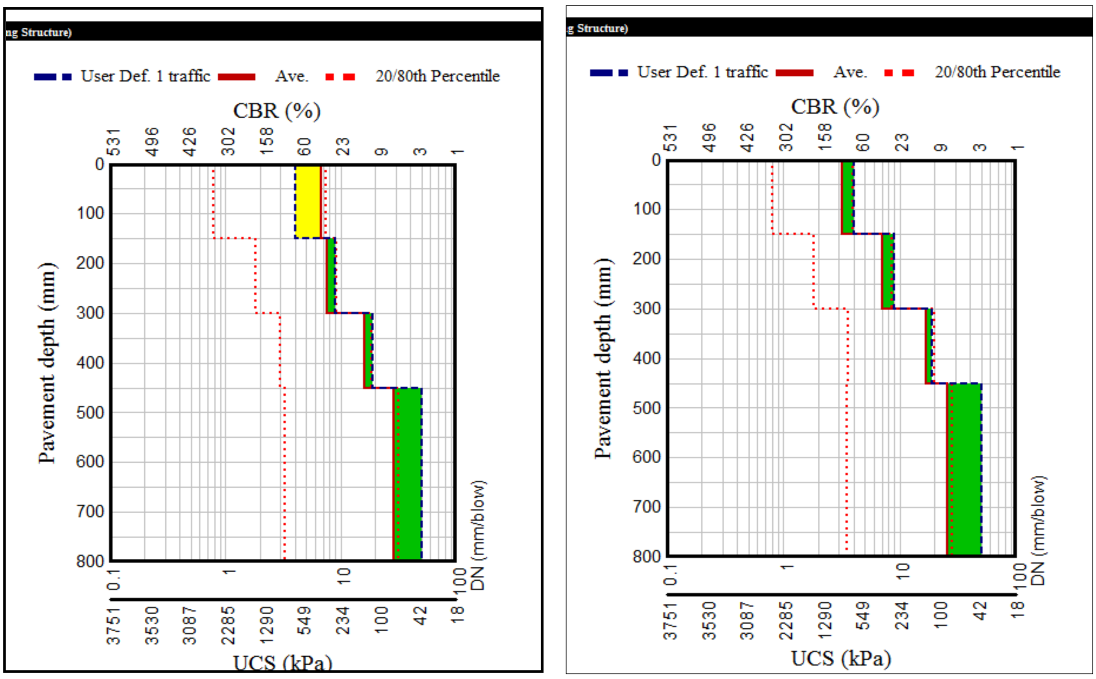


Figure 4.10: Layer strength diagrams during construction

4.2.2 Kyeni - Karurumu DCP data

The DCP analysis for Kyeni - Karurumu in wet and optimum conditions gave the following results as given in Table 4.4 below. From the table, it can be seen that to penetrate top 150 mm layer 4 mm/blow is required and eventually up to 50 mmm/blow to penetrate to 800mm depth.

Table 4.7 DN values

DN (mm/blow)	0-150	151-300	301-450	451-800
DN (mm/blow)	≤4	≤9	≤19	≤50

The resulting layer strength diagrams as defined by the DN, the CBR and Pavement depth are as shown in Figure 4.8. From Fig 4.11, the upper 150 mm of the subgrade had dried out and gained strength, whereas the layers from 150 mm down were still more or less at the same moisture content and strength. Most of the moisture had penetrated from top, and by sealing the road and improving the drainage, the layers no longer wet up and lose strength to the same extent as before sealing.

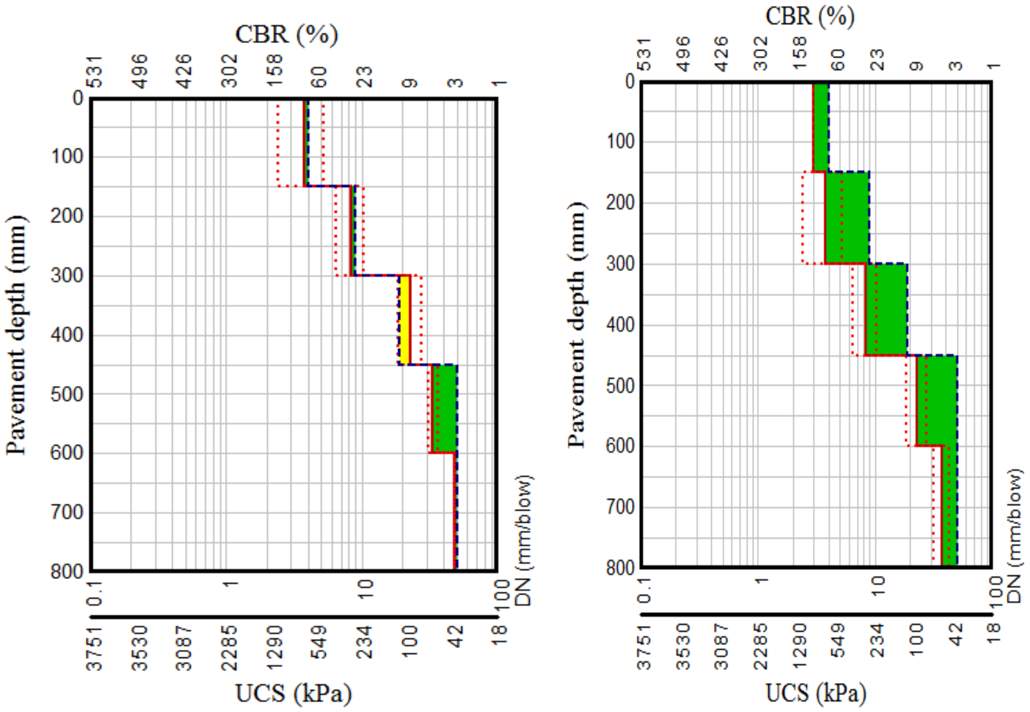


Figure 4.11: Layer Strength diagram during dry period

The existing pavement did not show signs of structural failure even in unsealed condition. This was a good indication that the pavement was adequate for the existing traffic. The following analysis of the weakest spot identified on the section was used to determine the final design.

The pavement strength at this point was on the borderline of the requirement. It was thus recommended to reshape the existing wearing course and add enough gravel to form a 150 mm thick base layer. The top 50mm was emulsion treated for practical reasons during construction and additional water proofing of the pavement.

Table 4.8 below shows the situation after adding a 150mm base layer with a DN = 3mm. The layer strengths are then on the safe side all through the pavement structure.

Table 4.8: DCP data before and after adding a 150mm base layer.

	Before adding 150mm base		After adding 150mm base		DCP Design Curve Pavement Class LV 0.3
Layer depth mm	Avg. DN (mm/blow)	CBR	Avg. DN (mm/blow)	CBR	
0-150	3.79	76	3	102	≤4
151-300	8.34	28	3.79	76	≤9
301-450	22.94	8	8.34	28	≤19
451-600	33.42	5	22.94	8	≤50
601-800	48.40	3	37.17	4	≤50
DSN 800		89		136	
Structural capacity MESA		0.2		0.9	
Pavement balance		WBS		PBD	

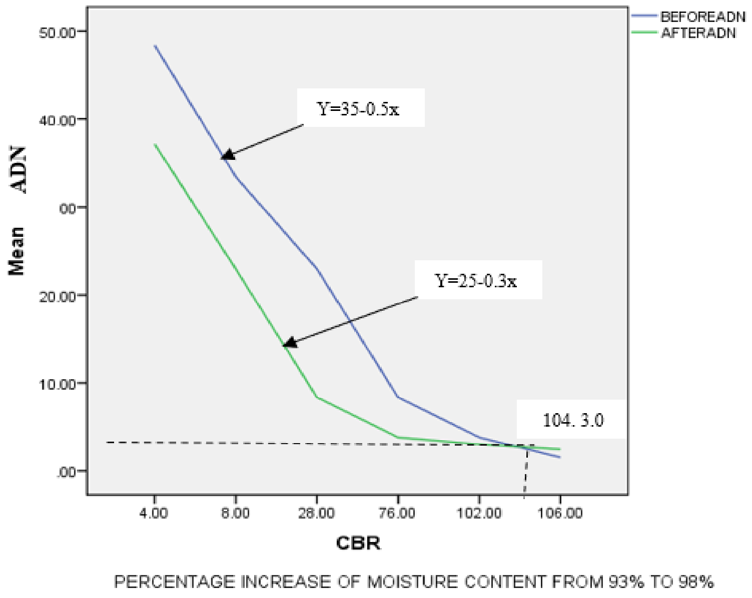


Figure 4.12: CBR before and after adding 150mm base

From the Table 4.8, it can be seen the before adding 150mm base the structure is well balanced but shallow while on addition of 150mm base the structure is poorly balanced but deep structure. Because of the compaction of the added base layer the CBR value increases from 72 to 102. The other layers also increase in CBR as can be seen on the table above. The correlation before adding 150mm base is 3.0.

Also, from the Figure 4.12, the DSN800 increases from 89 to 136 while the structural capacity also increases from 0.2 to 0.9 by adding 150mm base showing that due to the improvement the pavement can handle heavier traffic. From the CBR values down to depth of 800mm it was seen that the layer strengths decreased progressively and smoothly from 76 to 3 hence well balanced structure before adding 150mm base. The above findings conformed to the requirement of the LVS Design guidelines.

4.2.3 Kyeni - Karurumu as Built DCP

The DCP analysis for Kyeni - Karurumu in wet and optimum conditions gave the following results as given in Table 4.7

Table 4.9: DCP Design curve and 80-percentile DN value

Layer (mm)	Design Curve (DCP) (mm)	80-percentile (mm)
0 -150	4	3.9
151- 300	9	5.4
301 – 450	19	10.3
451 – 800	50	20.4

The resulting DN of each layer in the structure gave values that met the specification limits. The resulting layer strength diagrams as defined by the DN, the CBR and Pavement depth are as shown in **Figure 4.13.**

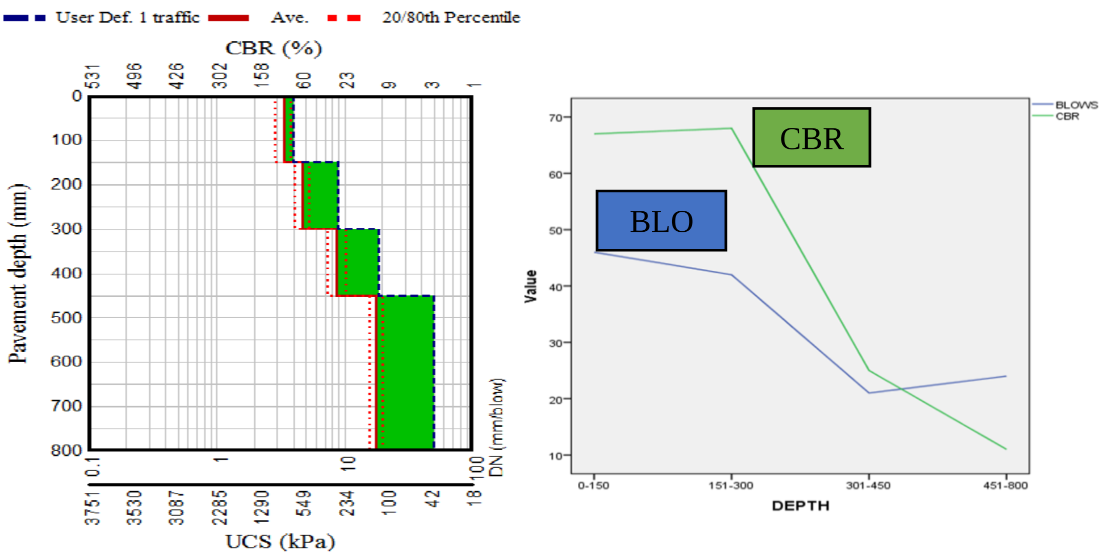


Figure 4.13: As built DCP analysis – Average all points

From **Table 4.8,** the 80-percentile (confidence level meaning that maximum 20% of values for the layer strength measured anywhere within the section may be weaker than the shown values, which is deemed to be reasonable level of safety for this class of road) is within the design curve specifications as in Table 4.7

Figure 4.13 shows the layer strength diagrams and output tables of weighted average penetration, 80-percentile, DCP CBR and UCS compared to the DCP analysis used for the design. From **Figure 4.13** above, it was seen that an overall strengthening of the pavement has been achieved, mostly for the upper 150mm base layer, compared to the Design DCP analysis by reworking the top of the layer and re-compacting, as recommended. The structural capacity has increased from 0.8

MESA (Million Equivalent Standard Axles) to 1.0 MESA and this compares the requirement of the Road Design Manual Part 1.

The apparent reduced strength of the layer from 151-300 mm, DCP CBR 57% reduced from 68%, is ascribed to the fact that the Design DCP tests were taken on the consolidated pavement whereas the as built DCP tests include points on the widened pavement constructed from bottom up with in situ subgrade material. The underlying layers from 301 mm depth down to 800 mm show no significant change.

4.2.4 Built design DCP average all points

The resulting layer strength diagrams for Kyeni – Karurumo as defined by the DN, the CBR and Pavement depth for the sections after improvement and trafficking are as shown in **Figure 4.14.** The figure shows the strength of the pavement measured at 3 points on the centre line. This illustrates the importance of maintaining as much as possible the strength of the existing pavement that has been consolidated under years of trafficking. The CBR achieved is 152% for the 0-150 mm layer and 106% for the 151-300 mm layer. This shows a stronger pavement because it lies on a well trafficked subgrade.

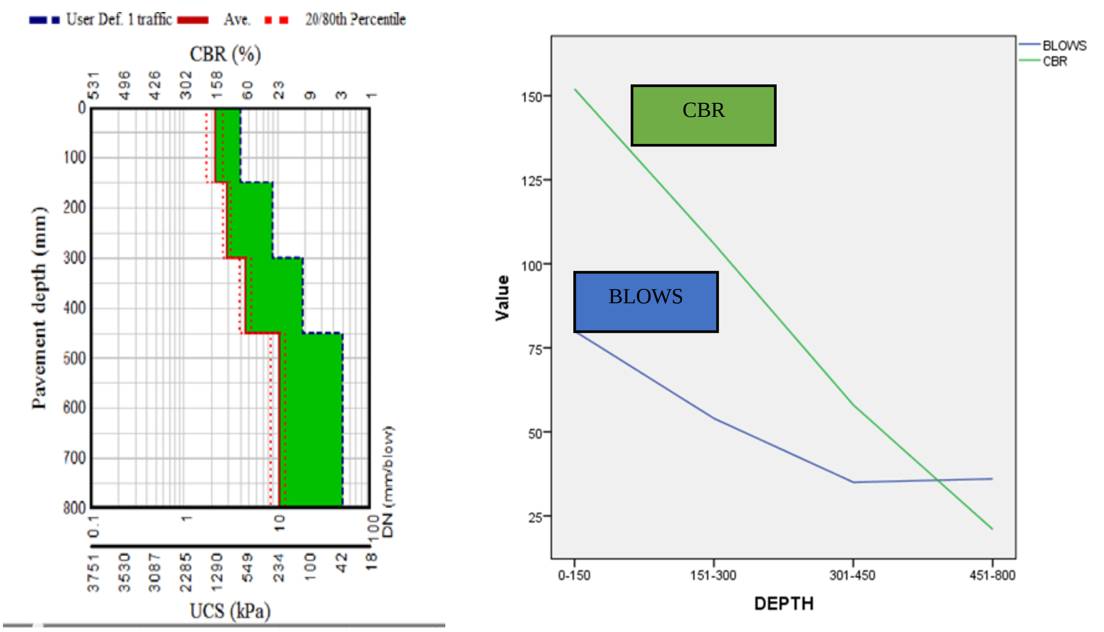


Figure 4.14: As built DCP analysis – Average points CL

Figure 4.15 shows the strength of the pavement in the outer wheel paths (LHS and RHS combined), which fall in the widened sections of the pavement. The 80-

percentile for the upper layer 0 -150 mm is marginally outside the Design Curve specification (4.6 mm penetration vs. 4.0 mm as specified). This was because the field densities achieved ware marginally below the minimum requirement of 98% Mod AASHTO on some points, showing that compaction to refusal was not done and the moisture content was too high at the time of compaction. The imported lateritic gravel was weaker than specified (32% at 98% compaction verses. the specified 45% soaked CBR).

The OWT results showed reduction in strength with CBR values of 70% for 0 -150mm Layer and 45% for layer 151-300 mm layer. This was lower than the results of the Center line because the road was widened at the sides to achieve the required carriageway width of 7 m. Despite this, the structural capacity in the outer wheel paths is 0.4 MESA, which was still well above the 15-year design traffic load for this road. The road was also expected to consolidate further when exposed to traffic.

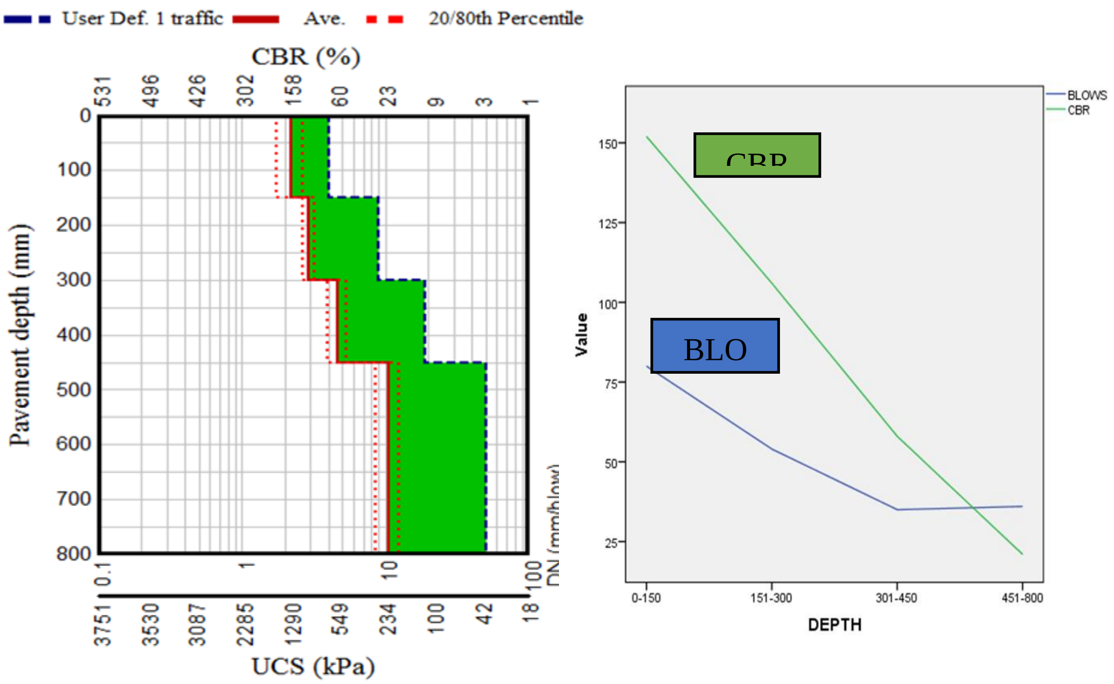


Figure 4.15: the strength of the pavement in the outer wheel paths (LHS and RHS combined

The in-situ DCP CBR strengths show a corresponding pattern to the in-situ moiture pattern. That is the higher the in-situ moisture content the lower the in-situ strength and this observation compares to the requirements TRL Design Manual (TRL, 1988), and ERA Design Manuals (ERA, 2013).

Table 4.10: As built and opening to traffic analysis

		Depth (mm)	W.Ave.Pen (mm/blow)	Blows	SD (mm/blow)	80P (mm/blow)	CBR (%)	UCS (kPa)
Structure number (DSN80)	109	0-150	4.00	46	0.70	4.60	70	634
Struct. Cap (MISA)	0.4	151-300	5.67	30	1.00	6.50	45	429
RUT limit	20mm	301-450	10.93	16	2.20	12.80	20	206
Balance curve is where	B=31 A=1059	451-800	21.98	18	3.00	24.50	8	95

MISA= Million Standard Axels. Category IV: Well-Balanced Deep Structure (WBD)

CHAPTER FIVE

CONCLUSIONS AND RECOMMENDATIONS

5.1 Conclusions

The main conclusions drawn from the research are given under the three specific objectives that guided the research.

5.1.1 Assessment of the in-situ moisture and pavement strengths of various sections of LVSRs before improvement

The MDD and OMC for Wamumu – Karaba were 1090kg/m³ and 46.5% while the MDD and the OMC for Kyeni - Karurumu were 1125 kg/m³ and 45.1% respectively. The moisture conditions of 4-days soak results to an average increase of 39% at the various compaction levels. The 0.75-OMC condition gives a CBR of 165 at 98% compaction level giving an average increase of 117%. The existing alignment material exhibit inferior characteristics but when processed and compacted heavily to reduce permeability can be used as subgrade material for LVSRs.

5.1.2 Assessment of the in-situ moisture, pavement layer strength and subgrade strength of various sections of LVSRs during improvement

The road before improvement has consolidated as it requires 4.05 mm/blows to penetrate 150 mm thickness at a CBR of 69. After improvement the road has not fully consolidated as it requires 3 mm/blow to penetrate to 150 mm thickness and at a higher CBR of 102.

It is also clear that during improvement by adding 150 mm base layer with a DN of 3mm/blow gives an increased CBR value of 102 showing that the pavement has achieved the required strength. A correlation before and after adding the 150mm base of 2.7 was derived hence along the road section works can continue based on the DCP and CBR only used for confirmation.

5.1.3 Determination of the pavement strength and moisture after improvement to LVSRs and traffic flow

The CBR achieved is 152% for the 0-150 mm layer and 106% for the 151-300 mm layer. This shows a stronger pavement because it lies on a well trafficked subgrade.

The outer wheel path results show a reduction in strength with CBR values of 70% for 0 -150mm Layer and 45% for layer 151-300mm layer. This is lower than the results of the Center line because the road was widened at the sides to achieve the required carriageway width of 7 m.

It is also clear that after improvement by sealing using bitumen and opening to traffic moisture ingress into the pavement has been reduced and more strength is gained through exposure to traffic. The rutting was at 20 mm showing that the road was performing well after improvement and opening to traffic.

5.2 Recommendation

5.2.1 Recommendations from the study

1. From the findings, in-situ moisture can be used to measure LVSRs pavement strength and performance. It is therefore recommended that final LVSR pavement designs take into account the different in situ soil strengths and moisture conditions; to ensure pavement performance.
2. From the findings, the DCP penetrates up to 800mm deeper into the pavement. It is recommended that pavement strengths be monitored using DSN 800 as it determines depths up to 800mm in layer thicknesses of 150mm and not only the upper50-75mm with the conventional CBR test, taking into account variations of in-situ moisture content while providing data quickly for analysis during construction.
3. From the findings, it is possible to obtain a correlation between the In-situ DCP test and the Laboratory CBR test. After construction and opening to traffic it is recommended that the correlation of CBR and DCP be used to design and construct new pavements. Experience with different materials, particularly the in situ subgrades, from the research sections and subsequent performance monitoring should be compiled into a national materials inventory database. Develop local material performance correlations for possible specifications and research.

5.2.2 Areas for further research

The study monitored the pavement performance for up to 1.5 years of opening the road to traffic and it showed improvement in terms of performance in relationship of the in-situ moisture. It is however not clear how the pavement would perform after the 1.5 years of opening to traffic. Hence, further studies on the performance and continued monitoring of the in-situ moisture regimes and the resulting rutting as the road is exposed to traffic for longer period of time say at least 3 years is recommended.

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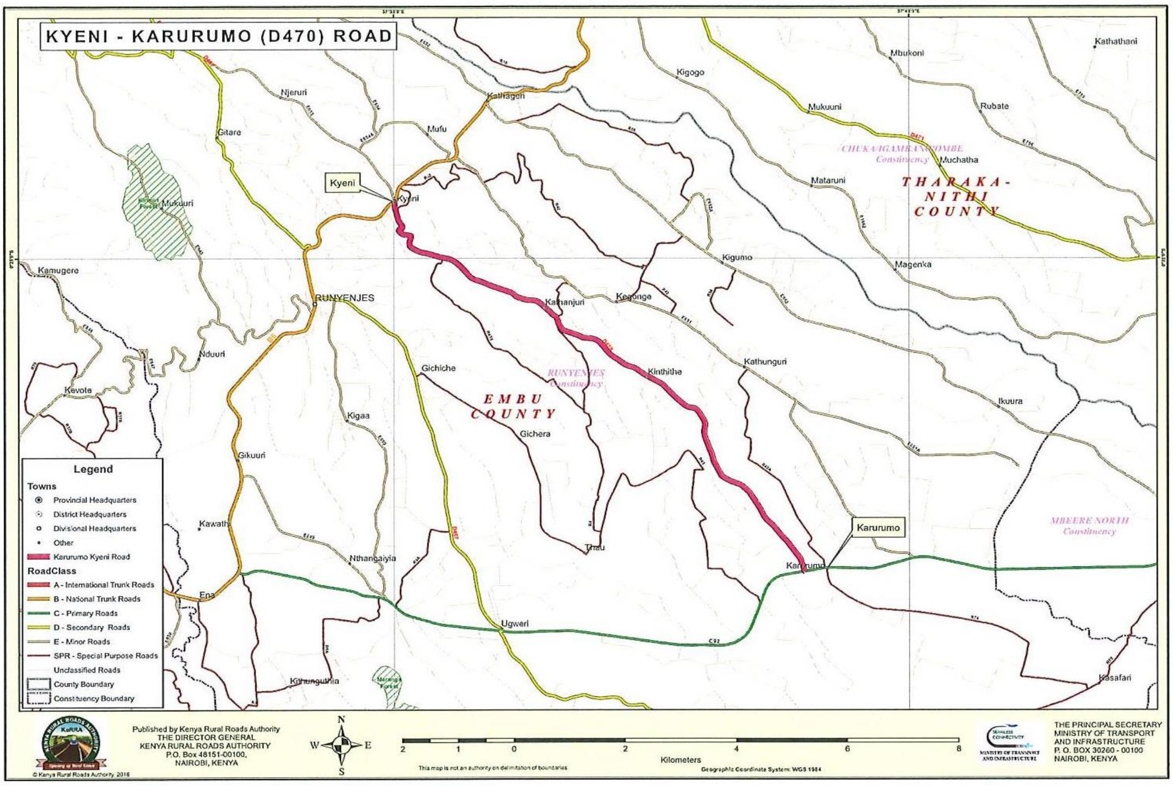
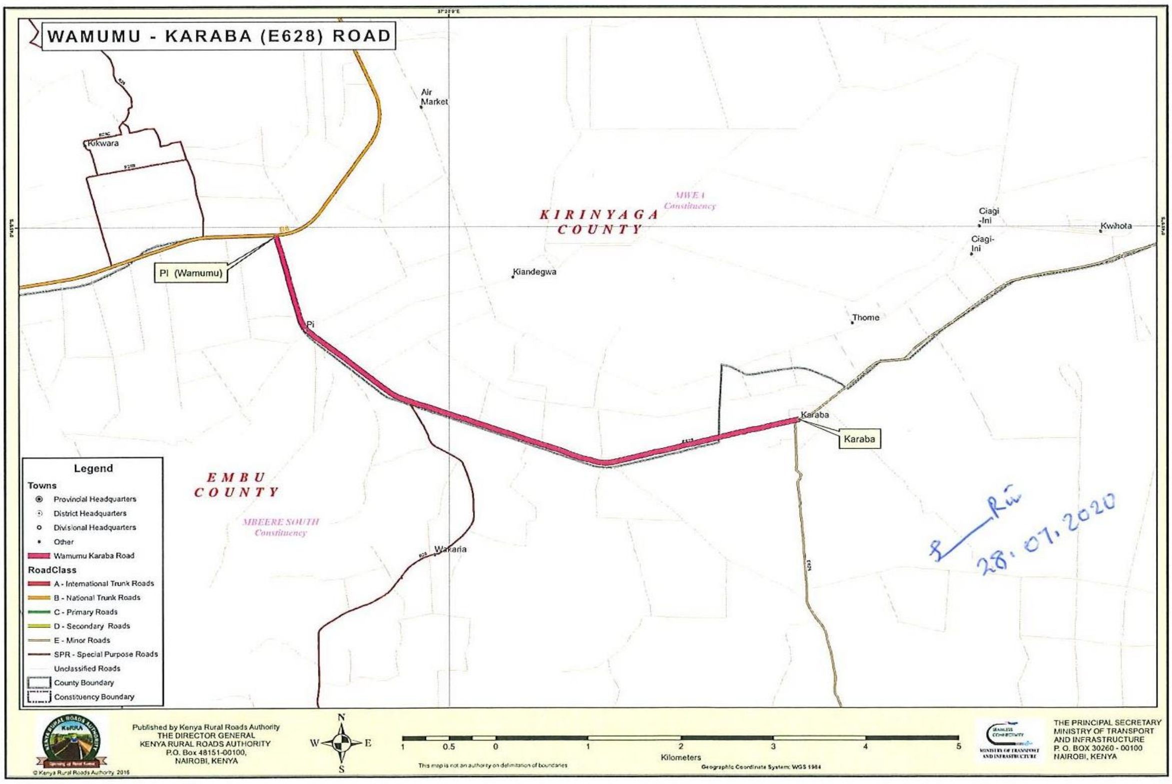
APPENDICES

APPENDIX 1: LOCATION MAP





Appendix 2: CLASSIFICTION OF ALIGNMENT OF SOILS

Location	Specimen reference	Depth	Atterberg Limits					Particle size distribution (%)								Grading Modulus	AASHTO Class	Organic Content	Compaction T 180		CBR		ICL
			LL	PL	PI	LS	PM	Percentage passing sieve size (mm)											MDD	OMC	at 95% MDD	Swell	
			(%)	(%)	(%)	(%)		50	37.5	20	10	5	2	0.425	0.075	GM		(%)	(kg/m^3)	(%)	(%)	(%)	(%)
Before Stabilization																							
SAMPLE 1			NON-PLASTIC					100	89	82	70	58	44	22	9	2.3	A-1-b		1630	14.8	23(30)	0.02	1.0
SAMPLE 2			NON-PLASTIC					100	97	78	67	57	48	31	17	2.0	A-1-b		1600	19.8	35(50)	0.08	1.5
SAMPLE 3			NON-PLASTIC					100	95	78	63	54	40	15	2	2.4	A-1-b		1664	9.6	30	0.6	1.2
SAMPLE 4			NON-PLASTIC					100	92	84	71	46	38	17	3	2.4	A-1-b		1564	23.8	50	0.5	1.1

RW 27.08.2020

SUMMARY SHEET

PROJECT

KYENI - KARURUMO LVSR

Date

July, 2015

Notes

CBR = 23(30) means

TOP = 23

BOTTOM = 30

Location	Specimen reference	Depth	Atterberg Limits LL (%)	PL (%)	PI (%)	LS (%)	PM	Particle size distribution (%) Percentage passing sieve size (mm) 50	37.5	20	10	5	2	0.425	0.075	Grading Modulus GM	AASHTO Class	Organic Content (%)	Compaction T 180 MDD (kg/m³)	OMC (%)	CBR at 95% MDD (%)	Swell (%)	ICL (%)
Before Stabilization																							
TP 1			60	31	29	14	638			100			32	22	15	2.3	A-2-7						
TP 2			58	30	28	13	728			100			35	26	24	2.2	A-2-7						
TP 3			56	26	30	14	750			100			34	25	20	2.2	A-2-7						
TP 6			81	39	42	20	924			100			30	22	14	2.3	A-2-7						
Mixed sample with			56	29	27	13	675			100			31	25	18	2.3	A-2-7		1794	16.2	20	0.10	
After Stabilization																							
Mixed sample with																					CBR (7DC + 7DS) at 95% MDD (%)	Swell (%)	
Cement Lime																							
3%			NON-PLASTIC																1794	16.2	60	< 0.10	
4%			NON-PLASTIC																		130	< 0.10	
3% 3%			NON-PLASTIC																		136	< 0.10	
4% 3%			NON-PLASTIC																		172	< 0.10	

27.07.2020

SUMMARY SHEET

Stabilized Materials

MATERIAL SITE MS 19

Date

July, 2016

Notes

CBR = 24(47) means
TOP = 24
BOTTOM = 47

Location	Specimen reference	Depth	Atterberg Limits LL (%)	PL (%)	PI (%)	LS (%)	PM	Particle size distribution (%) Percentage passing sieve size (mm) 50	37.5	20	10	5	2	0.425	0.075	Grading Modulus GM	AASHTO Class	Organic Content (%)	Compaction T 180 MDD (kg/m³)	OMC (%)	CBR at 95% MDD (%)	Swell (%)	ICL (%)
Before Stabilization																							
TP 1			43	23	20	10	680			100			48	34	26	1.9	A-2-7						
TP 3			45	25	20	11	740			100			45	37	30	1.9	A-2-7						
TP 5			47	29	18	13	450			100			32	25	23	2.2	A-2-7						
TP 6			53	33	20	10	240			100			23	12	4	2.6	A-2-7						
TP 7			53	33	20	10	840			100			63	42	37	1.6	A-7-5						
TP 8			46	18	28	14	364			100			24	13	4	2.6	A-2-7						
TP 9			51	29	22	11	484			100			46	22	15	2.2	A-2-7						
TP 10			50	26	24	11	720			100			94	30	2	1.7	A-2-7						
TP 11			72	38	34	17	1258			100			63	37	29	1.7	A-2-7						
TP 12																							
TP 13																							
Mixed sample with			51	28	23	12	642			100			49	28	19	2.0	A-2-7		1996	10.2	26	0.10	
After Stabilization																							
Mixed sample with																					CBR (7DC + 7DS) at 95% MDD (%)	Swell (%)	
Cement	**Lime**																						
3%			NON-PLASTIC																1996	10.2	134	< 0.10	
4%			NON-PLASTIC																		198	< 0.10	
3%	3%		NON-PLASTIC																		186	< 0.10	
4%	3%		NON-PLASTIC																		240	< 0.10	

RW
27.07.2020

SUMMARY SHEET

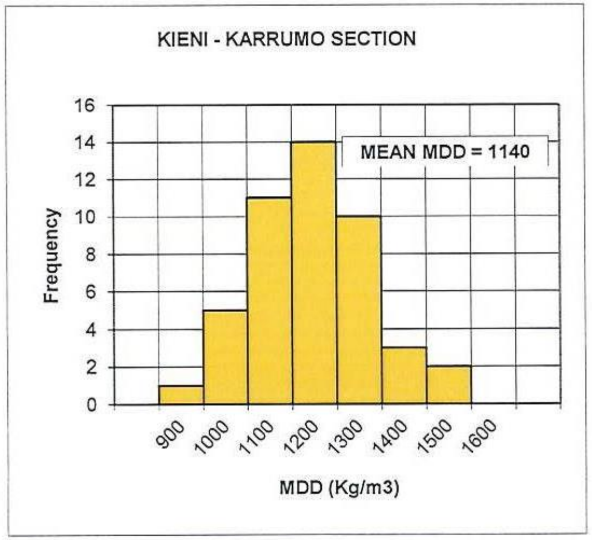
Stabilized Materials

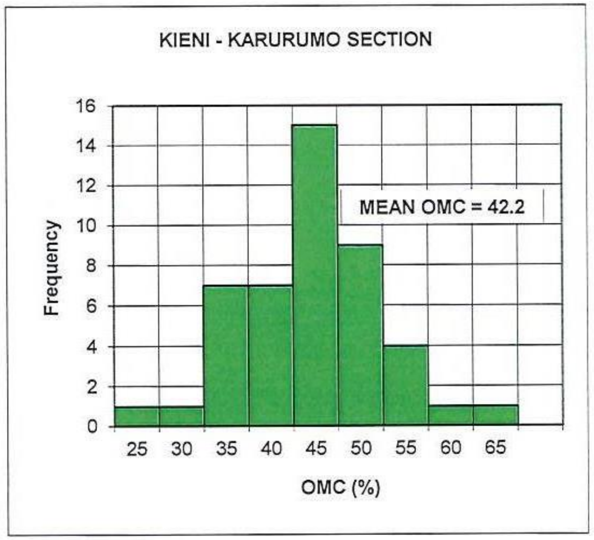
MATERIAL SITE MS 15

Date
July, 2016

Notes
CBR = 24(47) means
TOP = 24
BOTTOM = 47

Bin	Frequency
900	1
1000	5
1100	11
1200	14
1300	10
1400	3
1500	2
1600	0





Bin	Frequency
25	1
30	1
35	7
40	7
45	15
50	9
55	4
60	1
65	1

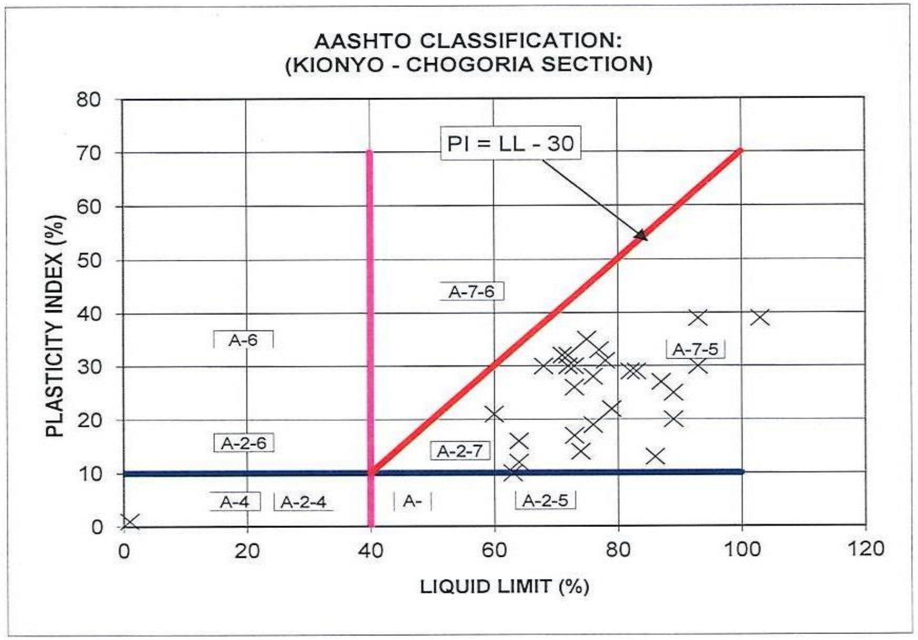
Riv
27.07.2020

0	10
100	10
40	0
40	70
40	10
100	70

CLASSIFICATION

27.07.20

LL	PI	
		1
93	39	2
103	39	3
87	27	4
89	20	5
89	25	6
76	19	7
86	13	8
1	1	9
74	14	10
83	29	11
64	16	12
64	12	13
73	17	14
79	22	15
78	31	16
73	26	17
63	10	18
60	21	19
73	30	20
71	32	21
72	30	22
76	28	23
72	32	24
68	30	25
77	33	



Location	Specimen Reference	Depth	Atterberg Limits					Particle size distribution (%)												AASHTO Class	Compaction T 99		CBR - 4 Days Soak			Description of sample
								Percentage passing sieve size (mm)																		
			LL (%)	PL (%)	PI (%)	LS (%)	PM	50	37.5	20	10	5	2	1.18	0.600	0.425	0.212	0.150	0.075		MDD (kg/m^3)	OMC (%)	Density (kg/m^3)	CBR (%)	Swell (%)	
0+000			93	52	39	20	2613					100	98	89	77	67	46	38	30	A-2-7(2)	1035	45.0	1037	7(10)	0.68	Redish Brown Silty SAND
0+500			103	64	39	20	1872					100	92	77	62	48	28	21	15	A-2-7(0)	990	52.3	1012	14	1.30	Redish Brown Silty SAND
1+000			87	60	27	14	1755					100	92	84	75	65	47	39	32	A-2-7(2)	1128	47.8	1121	11(12)	0.75	Redish Brown Silty SAND
1+500			89	69	20	10	1900						100	98	96	95	91	88	84	A-7-5(29)	920	55.5	913	7	0.20	Dark Brown SILT
2+000			89	64	25	13	1825			100	99	95	86	79	75	73	70	68	66	A-7-5(21)	1010	50.2	992	6(12)	0.46	Light Grey SILT
2+500			76	57	19	10	1596				100	97	92	88	86	84	81	79	72	A-7-5(19)	1023	33.2	1033	7(11)	1.16	Dark Brown SILT
3+000			86	73	13	7	975			100	96	91	84	79	76	75	71	69	64	A-7-5(14)	1005	46.6	1007	10	0.24	Dark Brown Sandy SILT
3+500			N.P				0			100	95	89	85	81	98	77	75	73	68	A-4(0)	1180	34.0	1177	14	0.15	Dark Brown SILT
4+000			74	60	14	7	1344				100	99	98	97	97	96	93	90	83	A-7-5(20)	990	49.0	990	17	0.93	Dark Brown SILT
4+500			83	54	29	14	2407				100	98	95	91	86	83	71	65	57	A-7-5(17)	818	65.0	811	5(10)	0.76	Dark Brown Sandy SILT
5+000			64	48	16	8	1392			100	97	93	91	89	88	87	84	82	78	A-7-5(18)	1188	41.0	1170	10(14)	0.32	Reddish Brown SILT
5+500			64	52	12	6	1068				100	98	94	92	90	89	86	83	75	A-7-5(14)	1016	49.0	1045	6(10)	0.09	Reddish Brown SILT
6+000			73	56	17	8	1292				100	99	95	90	83	76	61	55	47	A-7-5(7)	1028	48.4	1030	13(17)	0.36	Dark Brown Sandy SILT
6+500			79	57	22	10	1232					100	92	81	70	56	35	29	24	A-2-7(0)	1120	46.6	1090	12(16)	1.38	Reddish Brown Silty SAND

Rw
27.07.2020

SUMMARY SHEET

Alignment Soil Tests

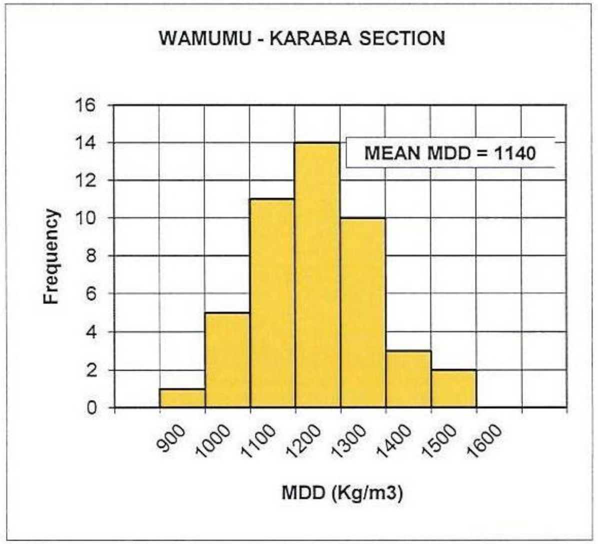
PROJECT

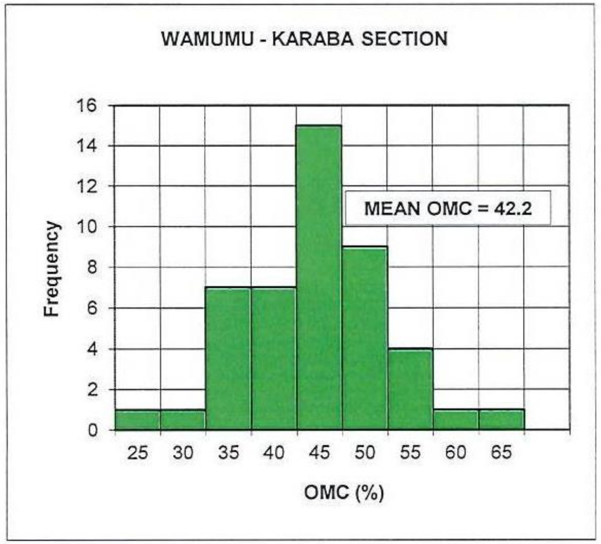
WAMUMU - KARABA

Date:
May, 2015

Sheet 1 of 3

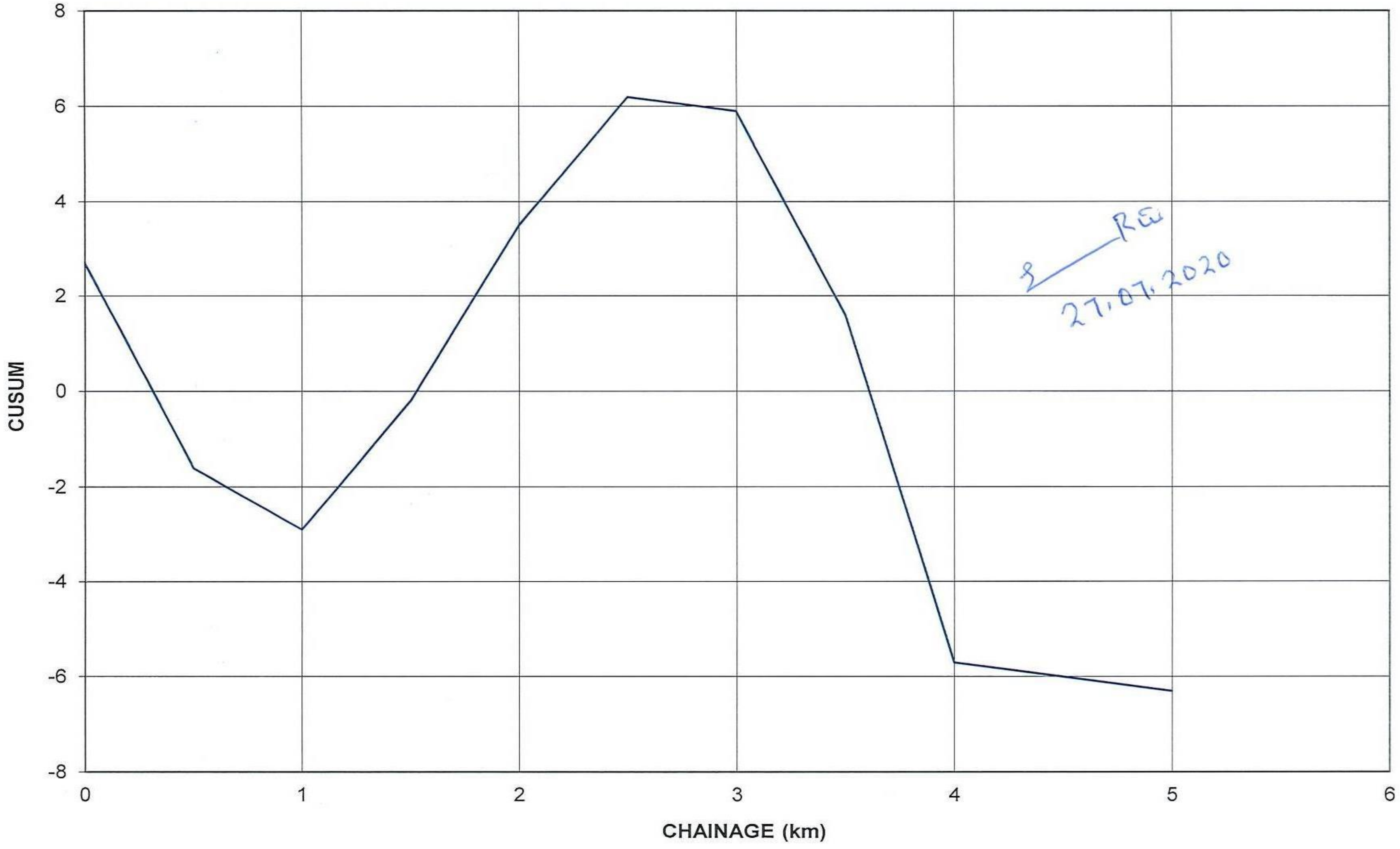
Bin	Frequency
900	1
1000	5
1100	11
1200	14
1300	10
1400	3
1500	2
1600	0





Bin	Frequency
25	1
30	1
35	7
40	7
45	15
50	9
55	4
60	1
65	1

27.07.2020



8
6
4
2
0
-2
-4
-6
-8
CUSUM
0
1
2
3
4
5
6
CHAINAGE (km)
27.07.2020

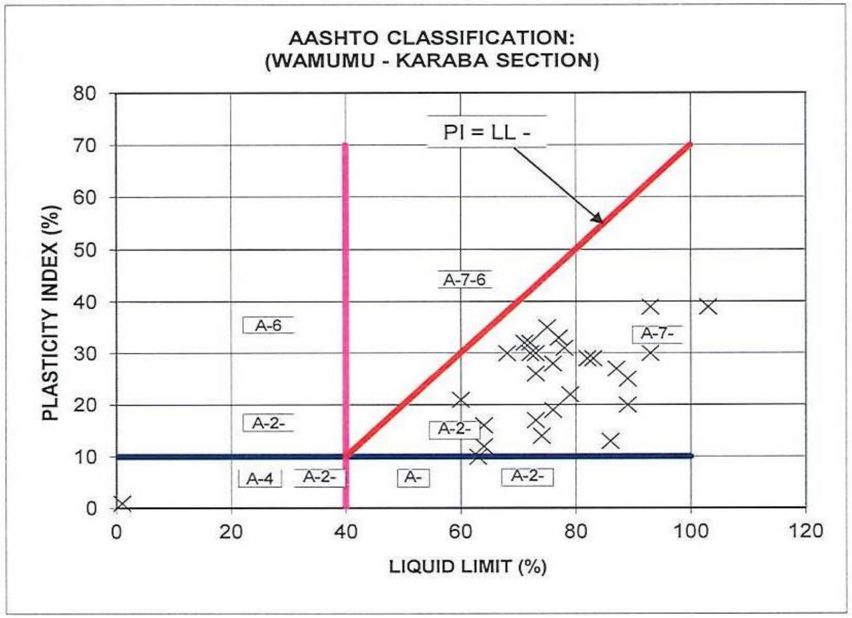
0	10
100	10
40	0
40	70
40	10
100	70

Rw
27.07.2020

LL	PI
93	39
103	39
87	27
89	20
89	25
76	19
86	13
1	1
74	14
83	29
64	16
64	12
73	17
79	22
78	31
73	26
63	10
60	21
73	30
71	32
72	30
76	28
72	32
68	30
77	33
93	30
75	35
82	29

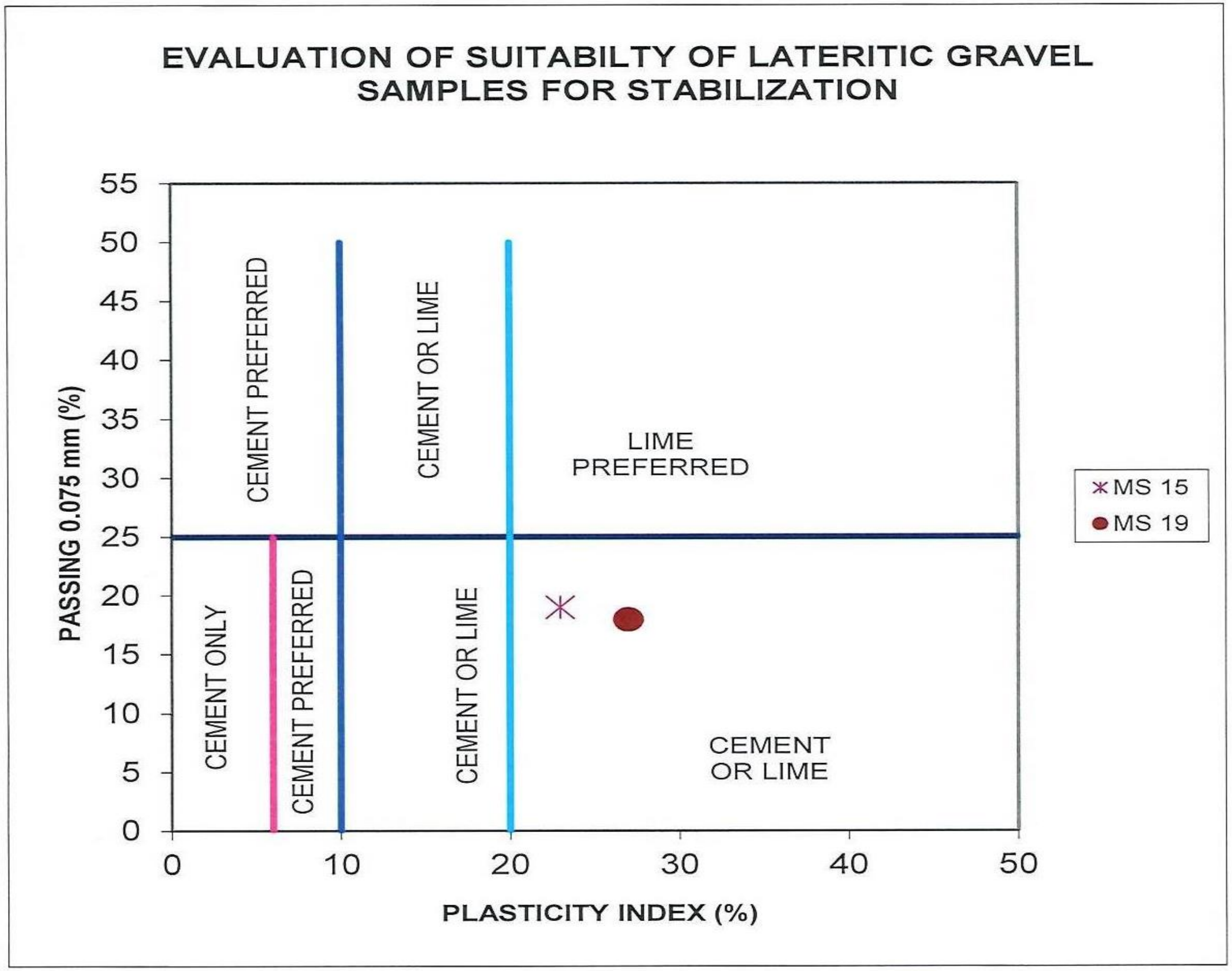
1
2
3
4
5
6
7
8
9
10
11
12

1
2
3
4
5
6
7
8
9
10
11
12



Location	Specimen Reference	Depth	Atterberg Limits					Particle size distribution (%)												AASHTO Class	Compaction T 99		CBR - 4 Days Soak			Description of sample
			LL (%)	PL (%)	PI (%)	LS (%)	PM	Percentage passing sieve size (mm)													MDD (kg/m^3)	OMC (%)	Density (kg/m^3)	CBR (%)	Swell (%)	
								50	37.5	20	10	5	2	1.18	0.600	0.425	0.212	0.150	0.075							
0+000			93	52	39	20	2613					100	98	89	77	67	46	38	30	A-2-7(2)	1035	45.0	1037	7(10)	0.68	Redish Brown Silty SAND
0+500			103	64	39	20	1872					100	92	77	62	48	28	21	15	A-2-7(0)	990	52.3	1012	14	1.30	Redish Brown Silty SAND
1+000			87	60	27	14	1755					100	92	84	75	65	47	39	32	A-2-7(2)	1128	47.8	1121	11(12)	0.75	Redish Brown Silty SAND
1+500			89	69	20	10	1900						100	98	96	95	91	88	84	A-7-5(29)	920	55.5	913	7	0.20	Dark Brown SILT
2+000			89	64	25	13	1825			100	99	95	86	79	75	73	70	68	66	A-7-5(21)	1010	50.2	992	6(12)	0.46	Light Grey SILT
2+500			76	57	19	10	1596				100	97	92	88	86	84	81	79	72	A-7-5(19)	1023	33.2	1033	7(11)	1.16	Dark Brown SILT
3+000			86	73	13	7	975			100	96	91	84	79	76	75	71	69	64	A-7-5(14)	1005	46.6	1007	10	0.24	Dark Brown Sandy SILT
3+500			N.P				0			100	95	89	85	81	98	77	75	73	68	A-4(0)	1180	34.0	1177	14	0.15	Dark Brown SILT
4+000			74	60	14	7	1344				100	99	98	97	97	96	93	90	83	A-7-5(20)	990	49.0	990	17	0.93	Dark Brown SILT
4+500			83	54	29	14	2407				100	98	95	91	86	83	71	65	57	A-7-5(17)	818	65.0	811	5(10)	0.76	Dark Brown Sandy SILT
5+000			64	48	16	8	1392			100	97	93	91	89	88	87	84	82	78	A-7-5(18)	1188	41.0	1170	10(14)	0.32	Reddish Brown SILT

RW
27.07.2020


EVALUATION OF SUITABILTY OF LATERITIC GRAVEL SAMPLES FOR STABILIZATION
PASSING 0.075 mm (%)
55
50
45
40
35
30
25
20
15
10
5
0
0
10
20
30
40
50
PLASTICITY INDEX (%)
CEMENT PREFERRED
CEMENT OR LIME
LIME PREFERRED
CEMENT ONLY
CEMENT PREFERRED
CEMENT OR LIME
CEMENT OR LIME
MS 15
MS 19

27.07.20

Location	Specimen reference	Depth	Atterberg Limits					Particle size distribution (%)								Grading Modulus	AASHTO Class	Organic Content	Compaction T 180		CBR		ICL
			LL	PL	PI	LS	PM	Percentage passing sieve size (mm)											MDD	OMC	at 95% MDD	Swell	
			(%)	(%)	(%)	(%)		50	37.5	20	10	5	2	0.425	0.075	GM		(%)	(kg/m^3)	(%)	(%)	(%)	(%)
Before Stabilization																							
SAMPLE 1			NON-PLASTIC					100	89	82	70	58	44	22	9	2.3	A-1-b		1630	14.8	23(30)	0.02	1.0
SAMPLE 2			NON-PLASTIC					100	97	78	67	57	48	31	17	2.0	A-1-b		1600	19.8	35(50)	0.08	1.5
SAMPLE 3			NON-PLASTIC					100	95	78	63	54	40	15	2	2.4	A-1-b		1664	9.6	30	0.6	1.2
SAMPLE 4			NON-PLASTIC					100	92	84	71	46	38	17	3	2.4	A-1-b		1564	23.8	50	0.5	1.1

RW
27.07.2020

SUMMARY SHEET

WAMUMU MATERIAL SITE

Date

July, 2016

Notes

CBR = 23(30) means
TOP = 23
BOTTOM = 30

Appendix 3: MOISTURE CONTENT AND COMPACTION TEST

Test Certificate for Soil samples
(Testing as per BS 1377:1990)

JOB NAME EMBU (E628 ROAD)

Sample No	Layer	Reference	GRADING							ATTERBERG LIMITS					Compaction T180		FMC	Moisture Condition	CBR at various compaction levels			
			% passing Sieve							LL	PL	PI	LS	PM	MDD	OMC						
				20 mm	10 mm	5 mm	2mm	425 µm	75 µm	(%)	(%)	(%)	(%)		(Kg/m^3)	(%)			93%	95%	98%	
560/S/17	Gravel	0+000		100	86	69	48	34	20	37	26	11	6	374	1895	13.4	18	4-Dsoak	15	17	20	22
																		OMC				
																		.75 OMC				
561/S/17	Subgrade	0+100		100	98	93	89	65	32	44	26	18	9	1170	1230	35.2	57	4-Dsoak	24	26	28	30
																		OMC				
																		.75 OMC				
562/S/17	Gravel	0+450		100	88	66	44	25	10	48	25	23	11	575	1880	14.8	17	4-Dsoak	15	18	24	26
																		OMC				
																		.75 OMC				
563/S/17	Subgrade	0+700		100	98	95	92	68	37	50	42	8	4	544	1405	25.8	44	4-Dsoak	17	20	26	28
																		OMC				
																		.75 OMC				
564/S/17	Subgrade	0+900		100	97	95	90	64	30	53	40	13	7	832	1365	28.0	37	4-Dsoak	15	19	24	25
																		OMC				
																		.75 OMC				
																		4-Dsoak				
																		OMC				
																		.75 OMC				

24.07.2020

Test Certificate for Soil samples
(Testing as per BS 1377:1990)

JOB NAME:-Embu (D470 ROAD)

Sample No	Layer	Reference		GRADING						ATTERBERG LIMITS					Compaction T180		FMC	Moisture Condition	CBR at various compaction levels			
				% passing Sieve						LL	PL	PI	LS	PM	MDD	OMC						
				20 mm	10 mm	5 mm	2mm	425 µm	75 µm	(%)	(%)	(%)	(%)		(Kg/m^3)	(%)			93%	95%	98%	100%
554/S/2017	Subgrade	0+000		100	100	96	90	75	57	50	28	22	11	1650	1658	14.4	22	4-Dsoak	3	4	5	6
																		OMC				
																		.75 OMC				
555/S/2017	Gravel	0+000		100	92	75	53	33	24	44	22	22	11	726	1910	14.0	16	4-Dsoak	20	30	40	42
																		OMC				
																		.75 OMC				
556/S/2017	Subgrade	0+200		100	100	96	90	78	59	45	20	25	12	1950	1700	18.6	23	4-Dsoak	10	12	15	16
																		OMC				
																		.75 OMC				
557/S/2017	Gravel	0+200		100	94	78	52	34	23	40	22	18	9	612	1910	14.0	11	4-Dsoak	22	27	34	35
																		OMC				
																		.75 OMC				
558/S/2017	Subgrade	0+350		100	98	92	82	63	45	46	21	25	12	1575	1700	18.6	16	4-Dsoak	9	12	15	17
																		OMC				
																		.75 OMC				
559/S/2017	Gravel	0+350		100	96	86	66	45	31	42	23	19	9	855	1865	13.2	15	4-Dsoak	5	7	11	12
																		OMC				
																		.75 OMC				

Rw 27.07.2020

Test Certificate for Soil samples
(Testing as per BS 1377:1990)

JOB NAME:- EMBU (D470 ROAD)

Sample No	Layer	Reference	GRADING: % passing Sieve 20 mm	10 mm	5 mm	2mm	425 µm	75 µm	ATTERBERG LIMITS: LL (%)	PL (%)	PI (%)	LS (%)	PM	Compaction T180: MDD (Kg/m³)	OMC (%)	FMC	Moisture Condition	CBR at various compaction levels: 93%	95%	98%	100%
680/S/17	Subgrade	0+000	100	100	93	80	69	50	45	25	20	10	1380	1625	18.8	24.1	4-Dsoak	8.8	9.6	10.8	11
																	OMC				
																	.75 OMC				
681/S/17	Subgrade	0+000	100	100	92	84	65	48	55	33	22	11	1430	1560	22.6	30.2	4-Dsoak	8	9	10	11
																	OMC				
																	.75 OMC				
682/S/17	Gravel	0+000	100	95	84	62	47	34	42	23	19	9	893	1625	17.0	38.6	4-Dsoak	12	16.5	22	24
																	OMC				
																	.75 OMC				
683/S/17	Subgrade	0+200	100	99	88	73	68	52	44	24	20	10	1360	1425	22.0	34.2	4-Dsoak	11	12.5	14.5	15
																	OMC				
																	.75 OMC				
684/S/17	Subgrade	0+450	100	98	90	82	70	55	45	26	19	10	1330	1495	22.4	29.3	4-Dsoak	24	25	27	28
																	OMC				
																	.75 OMC				

KW
22.07.2020

Test Certificate for Soil samples
(Testing as per BS 1377:1990)

JOB NAME:- EMBU (D470 ROAD)

Sample No	Layer	Reference		GRADING						ATTERBERG LIMITS					Compaction T180		FMC	Moisture Condition	CBR at various compaction levels			
				% passing Sieve						LL	PL	PI	LS	PM	MDD	OMC						
				20 mm	10 mm	5 mm	2mm	425 µm	75 µm	(%)	(%)	(%)	(%)		(Kg/m³)	(%)			93%	95%	98%	100%
62/S/2017	Subgrade	0+500		100	94	91	84	67	44	46	26	20	10	1340	1595	16	16.4	4-Dsoak	0.6	1.2	2	2.6
																		OMC	36	42	50	56
																		.75 OMC	70	81	85	105
63/S/2017	Gravel	0+500		100	86	77	74	52	40	39	21	17	9	884	1630	15.4	13.6	4-Dsoak	5	5.3	5.8	6.1
																		OMC	60	75	90	100
																		.75 OMC	74	83	96	105
64/S/2017	Subgrade	0+600		100	90	84	78	65	48	48	30	18	10	1170	1590	17.0	17.8	4-Dsoak	2.3	2.8	3.5	4.1
																		OMC	50	53	57	61
																		.75 OMC	65	70	76	80
65/S/2017	Subgrade	0+700		100	99	96	92	82	67	47	23	24	11	1968	1600	17.0	19	4-Dsoak	2	2.3	2.8	3.2
																		OMC	25	28	33	38
																		.75 OMC	47	49	52	55
66/S/2017	Subgrade	0+800		100	100	99	96	90	86	63	26	37	16	3330	1365	25.8	30.4	4-Dsoak	1.1	1.3	1.7	2
																		OMC	26	29	35	39
																		.75 OMC	34	39	46	55

RW
27.07.2020

Test Certificate for Soil samples
(Testing as per BS 1377:1990)

JOB NAME:- EMBU9 (E628 ROAD)

Sample No	Layer	Reference	GRADING % passing Sieve 20 mm	10 mm	5 mm	2mm	425 µm	75 µm	ATTERBERG LIMITS LL (%)	PL (%)	PI (%)	LS (%)	PM	Compaction T180 MDD (Kg/m³)	OMC (%)	FMC	Moisture Condition	CBR at various compaction levels 93%	95%	98%	100%
58/S/2017	Subgrade	0+000	100	100	100	98	86	70	64	36	28	14	2408	1355	29.4	26	4-Dsoak	2.3	2.6	2.9	3.2
																	OMC	62	72	84	94
																	.75 OMC	63	69	78	84
59/S/2017	Subgrade	0+200	100	98	97	93	72	52	54	35	19	11	1368	1450	21.6	20	4-Dsoak	3.3	3.6	4	4.2
																	OMC	55	60	63	65
																	.75 OMC	65	70	80	85
60/S/2017	Subgrade	0+400	100	93	82	75	64	48	48	24	24	13	1536	1720	19.4	20	4-Dsoak	1.8	2.4	3.3	4
																	OMC	36	38	43	48
																	.75 OMC	44	48	54	58
61/S/2017	Subgrade	0+600	100	97	92	88	76	62	54	29	25	14	1900	1475	21.4	20	4-Dsoak	2.9	3	3.2	3.4
																	OMC	54	64	80	91
																	.75 OMC	82	94	110	121

Rü
27.07.2020

Appendix 4: DCP TESTS

DCP TEST.

E628 WAMUMU -KARABA

DATE 05/06/2017 LAYER: SUBGRADE **KM 0+250 -KM 0+400 LHS BENCH**

CHAINAGE	NO. OF BLOWS	PENETRATION	CHAINAGE	NO OF BLOWS	PENETRATION
0+250LHS	0	85	0+400 LHS	0	90
	5	113		5	129
	5	155		5	172
	5	210		5	230
	5	290		5	300
0+280 LHS	0	88			
	5	115			
	5	157			
	5	220			
	5	295			
0+310 LHS	0	82			
	5	112			
	5	135			
	5	170			
	5	215			
	5	275			
0+340 LHS	0	83			
	5	105			
	5	133			
	5	166			
	5	208			
	5	258			
	5	315			
0+370 LHS	0	89			
	5	114			
	5	140			
	5	174			
	5	215			
	5	285			

27.07.2020

DCP TEST.

WAMUMU - KARABA

DATE 05/06/2017 LAYER: **SUBGRADE** **KM 0+030 -KM 0+240 RHS BENCH**

CHAINAGE	NO. OF BLOWS	PENETRATION	CHAINAGE	NO OF BLOWS	PENETRATION
0+030 RHS	0	82	0+180 RHS	0	88
	5	100		5	118
	5	135		5	160
	5	175		5	230
	5	248		5	330
	5	335			
			0+210 RHS	0	90
0+060 RHS	0	87		5	100
	5	103		5	120
	5	123		5	140
	5	144		5	165
	5	175		5	190
	5	220		5	225
	5	288		5	255
			0+240 RHS	0	82
0+090 RHS	0	90		5	100
	5	104		5	122
	5	125		5	157
	5	150		5	190
	5	188		5	228
	5	268		5	265
0+120 RHS	0	95			
	5	116			
	5	148			
	5	205			
	5	300			
0+150 RHS	0	90			
	5	110			
	5	130			
	5	155			
	5	190			
	5	250			
	5	320			

27.07.2020

DCP TEST.

D470 KIENI - KARURUMO

DATE 07/06/2017 LAYER: SUBGRADE **KM 0+000 - KM 0+050 BENCH**

CHAINAGE	NO. OF BLOWS	PENETRATION	CHAINAGE	NO OF BLOWS	PENETRATION
0+010 LHS	0	86			
	5	100			
	5	115			
	5	131			
	5	153			
	5	188			
	5	232			
	5	298			
0+040 LHS	0	85			
	5	113			
	5	142			
	5	180			
	5	228			
	5	290			
0+010 RHS	0	86			
	5	100			
	5	113			
	5	140			
	5	170			
	5	200			
	5	229			
	5	238			
	5	252			
	5	268			

RWS
27.07.2020

DCP TEST.

D470 KYIENI - KARURUMO

DATE 07/06/2017 LAYER: **SUBGRADE** **KM 0+010 RHS BENCH**

CHAINAGE	NO. OF BLOWS	PENETRATION	CHAINAGE	NO OF BLOWS	PENETRATION
0+010 RHS	0	86			
	5	100			
	5	113			
	5	140			
	5	170			
	5	200			
	5	229			
	5	238			
	5	252			
	5	268			

27.07.2020

DCP TEST.

D470 AS BUILT

DATE 31.05.2017 LAYER: SUBGRADE **KM 0+070 -KM 0+240 LHS BENCH**

CHAINAGE	NO. OF BLOWS	PENETRATION	CHAINAGE	NO OF BLOWS	PENETRATION
0+070 LHS	0	70	0+220 LHS	0	82
	5	100		5	110
	5	136		5	145
	5	192		5	195
	5	229		5	235
	5	256		5	274
0+100 LHS	0	75			
	5	105			
	5	150			
	5	225			
	5	275			
0+130 LHS	0	75			
	5	110			
	5	168			
	5	228			
	5	P			
0+160 LHS	0	75			
	5	86			
	5	110			
	5	145			
	5	185			
	5	230			
	5	292			
0+190 LHS	0	78			
	5	100			
	5	138			
	5	190			
	5	250			
	5	280			

27.07.2020

DCP TEST.

E628 AS BUILT

DATE 31.05.2017 LAYER: **SUBGRADE** **KM 0+280 -KM 0+400 RHS BENCH**

CHAINAGE	NO. OF BLOWS	PENETRATION	CHAINAGE	NO OF BLOWS	PENETRATION
0+280 RHS	0	85			
	5	130			
	5	190			
	5	245			
	5	288			
0+310 RHS	0	75			
	5	115			
	5	182			
	5	235			
	5	281			
0+340 RHS	0	85			
	5	132			
	5	170			
	5	208			
	5	280			
0+370 RHS	0	82			
	5	106			
	5	148			
	5	201			
	5	243			
	5	286			
0+400 RHS	0	84			
	5	108			
	5	152			
	5	215			
	5	270			

27.07.2020

Appendix 5: DCP REPORTS AND ANALYSIS

	A	B	C	D	E	F	G	H	I	J	K	L	M	N	O
64		0	110.00	31.00		111.00	944.00	373.00							
65		0	120.00	35.00		116.00	983.00	388.00							
66		0	130.00	40.00		145.00	1196.00	468.00							
67		0	140.00	43.00		105.00	902.00	358.00							
68		0	150.00	46.00		103.00	889.00	353.00							
69		0	160.00	50.00		102.00	877.00	348.00							
70		0	170.00	53.00		100.00	867.00	344.00							
71		0	180.00	56.00		101.00	868.00	345.00							
72		0	190.00	60.00		99.00	852.00	339.00							
73		0	200.00	63.00		99.00	852.00	339.00							
74		0	210.00	66.00		98.00	846.00	336.00							
75		0	220.00	69.00		86.00	759.00	304.00							
76		0	230.00	72.00		79.00	704.00	283.00							
77		0	240.00	74.00		80.00	711.00	285.00							
78		0	250.00	77.00		74.00	663.00	267.00							
79		0	260.00	79.00		68.00	612.00	248.00							
80		0	270.00	82.00		64.00	582.00	236.00							
81		0	280.00	84.00		64.00	586.00	237.00							
82		0	290.00	86.00		62.00	563.00	229.00							
83		0	300.00	88.00		56.00	516.00	210.00							
84		0	310.00	90.00		49.00	458.00	188.00							
85		0	320.00	92.00		48.00	455.00	187.00							
86		0	330.00	94.00		47.00	444.00	182.00							
87		0	340.00	96.00		46.00	432.00	178.00							
88		0	350.00	97.00		40.00	388.00	161.00							
89		0	360.00	99.00		38.00	364.00	151.00							
90		0	370.00	100.00		34.00	332.00	138.00							
91		0	380.00	102.00		30.00	300.00	126.00							
92		0	390.00	103.00		30.00	296.00	124.00							
93		0	400.00	104.00		28.00	280.00	118.00							
94		0	410.00	105.00		24.00	248.00	105.00							
95		0	420.00	106.00		24.00	248.00	105.00							
96		0	430.00	107.00		23.00	237.00	101.00							
97		0	440.00	108.00		21.00	221.00	94.00							
98		0	450.00	109.00		20.00	213.00	91.00							
99		0	460.00	110.00		20.00	208.00	89.00							
100		0	470.00	111.00		19.00	201.00	86.00							
101		0	480.00	112.00		18.00	195.00	84.00							
102		0	490.00	113.00		17.00	182.00	78.00							
103		0	500.00	113.00		16.00	169.00	73.00							
104		0	510.00	114.00		15.00	167.00	72.00							
105		0	520.00	115.00		15.00	164.00	71.00							
106		0	530.00	116.00		14.00	155.00	67.00							
107		0	540.00	116.00		14.00	154.00	67.00							
108		0	550.00	117.00		14.00	156.00	68.00							
109		0	560.00	118.00		14.00	151.00	65.00							
110		0	570.00	118.00		14.00	151.00	65.00							
111		0	580.00	119.00		13.00	146.00	64.00							
112		0	590.00	120.00		14.00	149.00	65.00							
113		0	600.00	120.00		14.00	154.00	67.00							
114		0	610.00	121.00		14.00	154.00	67.00							
115		0	620.00	122.00		15.00	161.00	70.00							
116		0	630.00	123.00		14.00	153.00	66.00							
117		0	640.00	123.00		13.00	148.00	64.00							
118		0	650.00	124.00		13.00	148.00	64.00							
119		0	660.00	125.00		13.00	148.00	64.00							
120		0	670.00	125.00		13.00	147.00	64.00							
121		0	680.00	126.00		13.00	139.00	61.00							
122		0	690.00	127.00		13.00	145.00	63.00							
123		0	700.00	127.00		13.00	145.00	63.00							
124		0	710.00	128.00		13.00	144.00	63.00							
125		0	720.00	129.00		13.00	143.00	62.00							
126		0	730.00	129.00		11.00	126.00	55.00							
127		0	740.00	130.00		10.00	113.00	50.00							
128		0	750.00	130.00		10.00	114.00	50.00							
129		0	760.00	131.00		10.00	117.00	51.00							
130		0	770.00	131.00		10.00	117.00	51.00							
131		0	780.00	132.00		10.00	117.00	51.00							
132		0	790.00	132.00		10.00	116.00	51.00							
133		0	800.00	133.00		10.00	114.00	50.00							
134															

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	A	B	C	D	E	F	G	H	I	J	K	L	M	N	O
1															
2		*DCP Report - Average analysis*													
3		Region:	Embu												
4		Road number:	E628												
5		Project date:	30 January, 2017												
6		Selected Design tra	Light traffic												
7		Road Category	C												
8		Base type	Granular												
9		Moisture condition o	Dry												
10															
11		*Measurements included in analysis:*													
12		Number	Measurement Name	Date	F	Distance	Condition	Rutting	Pumping	Long Crack	Croc. Crack	Deforomation	Other		
13		1	Measurement 1	30-01-17		0.05	Sound	No	No	No	No	No	No		
14		2	Measurement 2	30-01-17		0.05	Sound	No	No	No	No	No	No		
15		3	Measurement 3	30-01-17		0.15	Sound	No	No	No	No	No	No		
16		4	Measurement 4	30-01-17		0.15	Sound	No	No	No	No	No	No		
17		5	Measurement 5	30-01-17		0.25	Sound	No	No	No	No	No	No		
18		6	Measurement 6	30-01-17		0.25	Sound	No	No	No	No	No	No		
19		7	Measurement 7	30-01-17		0.35	Sound	No	No	No	No	No	No		
20															
21		*Results*													
22		Design Structure Nu	133												
23		BN100 of data	19.9												
24		BN100 of SPBC	32.0												
25		Rut Limit	20mm												
26		Structural capacity (	1.7												
27		SPBC (Standard Pa	A=2122, B=28												
28															
29		*Average equivalent strength (Existing pavement structure)*													
30		Depth (mm)	Weighted average pene	Blows	S	80P (mm/blow)	CBR (%)	UCS (kPa)	Ave. E-Moduli (	E-Moduli Range 20P - 80P Mpa					
31		0	4.12	50.00		5.30	68.00	613.00	248.00	102 - 659					
32		0	4.40	43.00		5.20	62.00	571.00	231.00	103 - 537					
33		0	10.21	20.00		12.20	21.00	223.00	95.00	42 - 224					
34		0	15.99	11.00		16.80	12.00	135.00	59.00	30 - 117					
35		0	19.31	10.00		20.90	10.00	109.00	48.00	23 - 100					
36															
37		*0*													
38		0	0.00	0.00		0.00	0.00	0.00	0.00	E-Moduli Range 20P - 80P Mpa					
39		0	5.04	24.00		6.00	53.00	490.00	200.00	88 - 479					
40		0	4.37	75.00		5.50	63.00	575.00	233.00	97 - 606					
41		0	14.39	27.00		16.60	14.00	152.00	66.00	30 - 148					
42		0	20.07	7.00		21.40	9.00	105.00	46.00	23 - 94					
43															
44		*0*													
45		0	0.00												
46		0	-1582.00												
47		0	532.00												
48		0	0.00												
49		0	8.00												
50		0	2123.00												
51															
52		*0*													
53		0	0.00	0.00		0.00	0.00	0.00							
54		0	10.00	2.00		47.00	445.00	183.00							
55		0	20.00	4.00		48.00	457.00	187.00							
56		0	30.00	6.00		66.00	597.00	242.00							
57		0	40.00	9.00		90.00	788.00	315.00							
58		0	50.00	12.00		86.00	758.00	303.00							
59		0	60.00	15.00		86.00	753.00	301.00							
60		0	70.00	18.00		95.00	826.00	329.00							
61		0	80.00	21.00		100.00	865.00	344.00							
62		0	90.00	25.00		97.00	844.00	336.00							
63		0	100.00	28.00		98.00	847.00	337.00							

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	1	2	3	4	5	6	7	8	9	10	11	12	13
110		0	550.00	124.00	13.32	15.00	165.00	71.00					
111		0	560.00	125.00	14.26	14.00	153.00	66.00					
112		0	570.00	126.00	14.37	14.00	152.00	66.00					
113		0	580.00	126.00	14.46	14.00	151.00	65.00					
114		0	590.00	127.00	14.65	14.00	149.00	65.00					
115		0	600.00	128.00	15.17	13.00	143.00	62.00					
116		0	610.00	128.00	15.57	13.00	139.00	61.00					
117		0	620.00	129.00	15.57	13.00	139.00	61.00					
118		0	630.00	129.00	15.57	13.00	139.00	61.00					
119		0	640.00	130.00	15.80	12.00	137.00	60.00					
120		0	650.00	131.00	15.97	12.00	135.00	59.00					
121		0	660.00	131.00	16.30	12.00	132.00	58.00					
122		0	670.00	132.00	16.61	12.00	129.00	57.00					
123		0	680.00	133.00	17.09	11.00	125.00	55.00					
124		0	690.00	133.00	17.60	11.00	121.00	53.00					
125		0	700.00	134.00	17.60	11.00	121.00	53.00					
126		0	710.00	134.00	17.64	11.00	121.00	53.00					
127		0	720.00	135.00	17.81	11.00	120.00	52.00					
128		0	730.00	135.00	17.69	11.00	120.00	53.00					
129		0	740.00	136.00	17.69	11.00	120.00	53.00					
130		0	750.00	137.00	17.69	11.00	120.00	53.00					
131		0	760.00	137.00	17.69	11.00	120.00	53.00					
132		0	770.00	138.00	17.69	11.00	120.00	53.00					
133		0	780.00	138.00	16.42	12.00	131.00	57.00					
134		0	790.00	139.00	15.66	12.00	138.00	60.00					
135		0	800.00	140.00	15.66	12.00	138.00	60.00					

	1	2	3	4	5	6	7	8	9	10	11	12	13
65		0	100.00	42.00	3.64	79.00	704.00	283.00					
66		0	110.00	45.00	3.61	80.00	712.00	286.00					
67		0	120.00	48.00	3.13	96.00	835.00	332.00					
68		0	130.00	51.00	3.29	90.00	789.00	315.00					
69		0	140.00	54.00	3.28	91.00	793.00	316.00					
70		0	150.00	57.00	3.36	88.00	772.00	309.00					
71		0	160.00	60.00	3.35	88.00	774.00	309.00					
72		0	170.00	63.00	3.62	80.00	709.00	284.00					
73		0	180.00	66.00	3.82	75.00	669.00	269.00					
74		0	190.00	68.00	3.84	74.00	665.00	268.00					
75		0	200.00	71.00	3.72	77.00	687.00	276.00					
76		0	210.00	74.00	3.36	88.00	770.00	308.00					
77		0	220.00	77.00	3.71	77.00	690.00	277.00					
78		0	230.00	79.00	4.02	70.00	631.00	255.00					
79		0	240.00	81.00	4.09	69.00	619.00	250.00					
80		0	250.00	84.00	3.90	73.00	653.00	263.00					
81		0	260.00	86.00	4.40	63.00	571.00	232.00					
82		0	270.00	89.00	4.28	65.00	588.00	238.00					
83		0	280.00	91.00	4.45	62.00	563.00	229.00					
84		0	290.00	93.00	4.68	58.00	532.00	217.00					
85		0	300.00	95.00	4.84	55.00	513.00	209.00					
86		0	310.00	97.00	5.08	52.00	486.00	199.00					
87		0	320.00	99.00	5.04	53.00	490.00	200.00					
88		0	330.00	101.00	6.12	41.00	395.00	163.00					
89		0	340.00	102.00	6.13	41.00	394.00	163.00					
90		0	350.00	104.00	6.48	38.00	370.00	154.00					
91		0	360.00	105.00	6.95	35.00	342.00	143.00					
92		0	370.00	107.00	7.68	31.00	306.00	128.00					
93		0	380.00	108.00	8.05	29.00	290.00	122.00					
94		0	390.00	109.00	8.36	28.00	278.00	117.00					
95		0	400.00	110.00	8.63	27.00	269.00	113.00					
96		0	410.00	111.00	8.81	26.00	263.00	111.00					
97		0	420.00	112.00	9.16	25.00	252.00	106.00					
98		0	430.00	113.00	9.16	25.00	251.00	106.00					
99		0	440.00	115.00	9.05	25.00	255.00	108.00					
100		0	450.00	116.00	9.72	23.00	235.00	100.00					
101		0	460.00	117.00	9.92	22.00	230.00	98.00					
102		0	470.00	118.00	10.33	21.00	220.00	94.00					
103		0	480.00	119.00	10.78	20.00	210.00	89.00					
104		0	490.00	119.00	11.12	19.00	202.00	87.00					
105		0	500.00	120.00	11.54	18.00	194.00	83.00					
106		0	510.00	121.00	12.58	16.00	176.00	76.00					
107		0	520.00	122.00	12.92	16.00	171.00	74.00					
108		0	530.00	123.00	13.27	15.00	166.00	72.00					
109		0	540.00	123.00	13.27	15.00	166.00	72.00					

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	1	2	3	4	5	6	7	8	9	10	11	12	13
46													
47		*0*											
48		0	0.00										
49		0	-539.00										
50		0	410.00										
51		0	-23.00										
52		0	972.00										
53													
54		*0*											
55		0	0.00	0.00	0.00	0.00	0.00	0.00					
56		0	10.00	5.00	2.09	161.00	1314.00	511.00					
57		0	20.00	11.00	1.74	191.00	1526.00	620.00					
58		0	30.00	16.00	1.73	192.00	1532.00	624.00					
59		0	40.00	22.00	1.90	177.00	1426.00	565.00					
60		0	50.00	26.00	2.20	150.00	1235.00	482.00					
61		0	60.00	30.00	2.85	108.00	926.00	367.00					
62		0	70.00	33.00	2.92	105.00	903.00	358.00					
63		0	80.00	36.00	2.92	105.00	901.00	357.00					
64		0	90.00	39.00	3.40	87.00	762.00	305.00					

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	1	2	3	4	5	6	7	8	9	10	11	12	13
1													
2		*DCP Report - Average analysis*											
3		Region:	Embu										
4		Road number:	E628										
5		Project date:	30 January, 2017										
6		Selected Design tra	Light traffic										
7		Road Category	C										
8		Base type	Granular										
9		Moisture condition c	Optimum										
10													
11		*Measurements included in analysis:*											
12		Number	Measurement Name	Date	Position	Distance	Condition	Rutting	Pumping	Long Crack	Croc. Crack	Deforomation	Other
13		1	Measurement 1	05-07-12		0.02	Sound	No	No	No	No	No	No
14		2	Measurement 2	05-07-12		0.07	Sound	No	No	No	No	No	No
15		3	Measurement 3	05-07-12		0.12	Sound	No	No	No	No	No	No
16		4	Measurement 4	05-07-12		0.17	Sound	No	No	No	No	No	No
17		5	Measurement 5	05-07-12		0.22	Sound	No	No	No	No	No	No
18		6	Measurement 6	05-07-12		0.27	Sound	No	No	No	No	No	No
19		7	Measurement 7	05-07-12		0.32	Sound	No	No	No	No	No	No
20		8	Measurement 8	05-07-12		0.37	Sound	No	No	No	No	No	No
21		9	Measurement 9	05-07-12		0.42	Sound	No	No	No	No	No	No
22													
23		*Results*											
24		Design Structure Nu	140										
25		BN100 of data	21.7								Rw		
26		BN100 of SPBC	34.0										
27		Rut Limit	20mm								27.07.2020		
28		Structural capacity (	1.0										
29		SPBC (Standard Pa	A=972, B=30										
30													
31		*Average equivalent strength (Existing pavement structure)*											
32		Depth (mm)	Weighted average penetration rate (%)	Blows	SD (mm/blow)	80P (mm/blow)	CBR (%)	UCS (kPa)	Ave. E-Moduli (	E-Moduli Range 20P - 80P Mpa			
33		0	3.44	60.00	0.60	3.90	85.00	750.00	300.00	138 - 668			
34		0	5.05	39.00	1.00	5.90	52.00	489.00	200.00	90 - 459			
35		0	11.14	20.00	1.40	12.30	19.00	202.00	86.00	41 - 183			
36		0	16.61	12.00	1.70	18.00	12.00	129.00	56.00	27 - 117			
37		0	20.38	9.00	0.40	20.70	9.00	103.00	45.00	24 - 87			
38													
39		*0*											
40		0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	E-Moduli Range 20P - 80P Mpa			
41		0	2.61	13.00	0.10	2.70	121.00	1024.00	403.00	206 - 789			
42		0	2.46	4.00	0.10	2.50	131.00	1095.00	430.00	224 - 827			
43		0	3.53	33.00	0.40	3.90	83.00	729.00	292.00	139 - 620			
44		0	5.64	55.00	1.90	7.30	46.00	432.00	178.00	72 - 478			
45		0	16.66	34.00	3.60	19.70	12.00	129.00	56.00	25 - 131			

	A	B	C	D	E	F	G	H	I	J	K	L	M	N	O
1						WinDCP 5									
2															
3	Region	Embu													
4	Road No	D470													
5	Project date	07/11/2017													
6	Design curves	3	(1=Heavy traffic, 2=Medium traffic, 3=Light traffic)												
7	Category	C	(A,B,C)												
8	Base	granular	(granular, cemented)												
9	Moisture	optimum	(wet, moist, optimum, dry)												
10															
11	Theoretical														
12	Pavement		Site Positions:		Road Conditions:		Distress conditions:								
13	Depth		1 - SHL		1 = Severe		R = Rutting								
14	100		2 -		2 = Warning		P = Pumping								
15	250		3 -		3 = Sound		L = Long Cracks								
16	400		4 -				C = Crocadile Cracks								
17	550		5 - MID				D = Deformation								
18	850		6 -				O = Other								
19			7 -												
20			8 -												
21			9 - SHR												
22															
23	Name	Test 1	Test 2	Test 3	Test 4	Test 5	Test 6	Test 7	Test 8	Test 9					
24	Distance	0.02	0.07	0.12	0.17	0.22	0.27	0.32	0.37	0.42					
25	Position	5	2	8	5	2	8	5	2	8					
26	Road condition	3	3	3	3	3	3	3	3	3					
27	Distress conditions														
28	Date	05/07/2017	05/07/2017	05/07/2017	05/07/2017	05/07/2017	05/07/2017	05/07/2017	05/07/2017	05/07/2017					
29	Blows/point	5	5	5	5	5	5	5	5	5					
30	0	85	85	85	86	88	85	85	85	85					
31	1	98	110	100	92	96	98	90	105	100					
32	2	105	125	120	99	103	103	95	120	122					
33	3	115	145	142	102	108	106	102	143	145					
34	4	123	165	165	108	115	110	110	170	170					
35	5	133	19	185	110	123	116	120	197	19					
36	6	145	220	215	115	130	122	130	230	205					
37	7	158	250	245	119	145	130	140	270	223					
38	8	172	270	270	123	155	135	152	300	240					
39	9	185	290	300	129	165	150	165	325	255					
40	10	198	320	330	133	180	160	179	359	270					
41	11	208	350	370	140	198	180	200	400	290					
42	12	215	390	430	148	205	195	215	450	305					
43	13	225	420	520	158	220	215	225	515	320					
44	14	235	445	650	165	240	230	235	605	337					
45	15	245	480	815	175	260	245	245	715	357					
46	16	255	525		190	280	270	260	870	385					

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	A	B	C	D	E	F	G	H	I	J	K	L	M
1													
2		*DCP Report - Average analysis*											
3		Region:	Embu										
4		Road number:	D379										
5		Project date:	30 January, 2017										
6		Selected Design tra	Light traffic										
7		Road Category	C										
8		Base type	Granular										
9		Moisture condition	Dry										
10													
11		*Results*											
12		Design Structure N	133										
13		BN100 of data	19.9										
14		BN100 of SPBC	32.0										
15		Rut Limit	20mm										
16		Structural capacity	1.7										
17		SPBC (Standard P	A=2122, B=28										
18													
19		*Average equivalent strength (Existing pavement structure)*											
20		Depth (mm)	Weighted average penetration rate (%)	Blows	SD (mm/blow)	80P (mm/blow)	CBR (%)	UCS (kPa)	Ave. E-Moduli	E-Moduli Range 20P - 80P Mpa			
21		0	4.12	50.00	1.40	5.30	68.00	613.00	248.00	102 - 659			
22		0	4.40	43.00	0.90	5.20	62.00	571.00	231.00	103 - 537			
23		0	10.21	20.00	2.40	12.20	21.00	223.00	95.00	42 - 224			
24		0	15.99	11.00	0.90	16.80	12.00	135.00	59.00	30 - 117			
25		0	19.31	10.00	1.90	20.90	10.00	109.00	48.00	23 - 100			
26													
27		*0*											
28		0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	E-Moduli Range 20P - 80P Mpa			
29		0	5.04	24.00	1.20	6.00	53.00	490.00	200.00	88 - 479			
30		0	4.37	75.00	1.40	5.50	63.00	575.00	233.00	97 - 606			
31		0	14.39	27.00	2.60	16.60	14.00	152.00	66.00	30 - 148			
32		0	20.07	7.00	1.60	21.40	9.00	105.00	46.00	23 - 94			
33													
34		*0*											
35		0	0.00										
36		0	-1582.00										
37		0	532.00										
38		0	0.00										
39		0	8.00										
40		0	2123.00										
41													
42		*0*											
43		0	0.00	0.00	0.00	0.00	0.00	0.00					
44		0	10.00	2.00	5.49	47.00	445.00	183.00					
45		0	20.00	4.00	5.37	48.00	457.00	187.00					
46		0	30.00	6.00	4.22	66.00	597.00	242.00					
47		0	40.00	9.00	3.30	90.00	788.00	315.00					
48		0	50.00	12.00	3.41	86.00	758.00	303.00					
49		0	60.00	15.00	3.43	86.00	753.00	301.00					
50		0	70.00	18.00	3.16	95.00	826.00	329.00					
51		0	80.00	21.00	3.03	100.00	865.00	344.00					
52		0	90.00	25.00	3.10	97.00	844.00	336.00					
53		0	100.00	28.00	3.09	98.00	847.00	337.00					
54		0	110.00	31.00	2.80	111.00	944.00	373.00					
55		0	120.00	35.00	2.70	116.00	983.00	388.00					
56		0	130.00	40.00	2.27	145.00	1196.00	468.00					
57		0	140.00	43.00	2.92	105.00	902.00	358.00					
58		0	150.00	46.00	2.96	103.00	889.00	353.00					

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	A	B	C	D	E	F	G	H	I	J	K	L	M
59		0	160.00	50.00	2.99	102.00	877.00	348.00					
60		0	170.00	53.00	3.03	100.00	867.00	344.00					
61		0	180.00	56.00	3.02	101.00	868.00	345.00					
62		0	190.00	60.00	3.07	99.00	852.00	339.00					
63		0	200.00	63.00	3.07	99.00	852.00	339.00					
64		0	210.00	66.00	3.09	98.00	846.00	336.00					
65		0	220.00	69.00	3.41	86.00	759.00	304.00					
66		0	230.00	72.00	3.65	79.00	704.00	283.00					
67		0	240.00	74.00	3.61	80.00	711.00	285.00					
68		0	250.00	77.00	3.85	74.00	663.00	267.00					
69		0	260.00	79.00	4.13	68.00	612.00	248.00					
70		0	270.00	82.00	4.32	64.00	582.00	236.00					
71		0	280.00	84.00	4.29	64.00	586.00	237.00					
72		0	290.00	86.00	4.45	62.00	563.00	229.00					
73		0	300.00	88.00	4.81	56.00	516.00	210.00					
74		0	310.00	90.00	5.35	49.00	458.00	188.00					
75		0	320.00	92.00	5.39	48.00	455.00	187.00					
76		0	330.00	94.00	5.51	47.00	444.00	182.00					
77		0	340.00	96.00	5.65	46.00	432.00	178.00					
78		0	350.00	97.00	6.21	40.00	388.00	161.00					
79		0	360.00	99.00	6.57	38.00	364.00	151.00					
80		0	370.00	100.00	7.15	34.00	332.00	138.00					
81		0	380.00	102.00	7.83	30.00	300.00	126.00					
82		0	390.00	103.00	7.91	30.00	296.00	124.00					
83		0	400.00	104.00	8.31	28.00	280.00	118.00					
84		0	410.00	105.00	9.28	24.00	248.00	105.00					
85		0	420.00	106.00	9.28	24.00	248.00	105.00					
86		0	430.00	107.00	9.65	23.00	237.00	101.00					
87		0	440.00	108.00	10.29	21.00	221.00	94.00					
88		0	450.00	109.00	10.61	20.00	213.00	91.00					
89		0	460.00	110.00	10.87	20.00	208.00	89.00					
90		0	470.00	111.00	11.20	19.00	201.00	86.00					
91		0	480.00	112.00	11.48	18.00	195.00	84.00					
92		0	490.00	113.00	12.25	17.00	182.00	78.00					
93		0	500.00	113.00	13.03	16.00	169.00	73.00					
94		0	510.00	114.00	13.21	15.00	167.00	72.00					
95		0	520.00	115.00	13.39	15.00	164.00	71.00					
96		0	530.00	116.00	14.15	14.00	155.00	67.00					
97		0	540.00	116.00	14.24	14.00	154.00	67.00					
98		0	550.00	117.00	14.04	14.00	156.00	68.00					
99		0	560.00	118.00	14.48	14.00	151.00	65.00					
100		0	570.00	118.00	14.48	14.00	151.00	65.00					
101		0	580.00	119.00	14.87	13.00	146.00	64.00					
102		0	590.00	120.00	14.64	14.00	149.00	65.00					
103		0	600.00	120.00	14.18	14.00	154.00	67.00					
104		0	610.00	121.00	14.22	14.00	154.00	67.00					
105		0	620.00	122.00	13.64	15.00	161.00	70.00					
106		0	630.00	123.00	14.30	14.00	153.00	66.00					
107		0	640.00	123.00	14.71	13.00	148.00	64.00					
108		0	650.00	124.00	14.71	13.00	148.00	64.00					
109		0	660.00	125.00	14.71	13.00	148.00	64.00					
110		0	670.00	125.00	14.84	13.00	147.00	64.00					
111		0	680.00	126.00	15.56	13.00	139.00	61.00					
112		0	690.00	127.00	15.03	13.00	145.00	63.00					
113		0	700.00	127.00	15.03	13.00	145.00	63.00					
114		0	710.00	128.00	15.05	13.00	144.00	63.00					
115		0	720.00	129.00	15.20	13.00	143.00	62.00					
116		0	730.00	129.00	16.95	11.00	126.00	55.00					
117		0	740.00	130.00	18.73	10.00	113.00	50.00					
118		0	750.00	130.00	18.62	10.00	114.00	50.00					
119		0	760.00	131.00	18.23	10.00	117.00	51.00					
120		0	770.00	131.00	18.23	10.00	117.00	51.00					
121		0	780.00	132.00	18.23	10.00	117.00	51.00					
122		0	790.00	132.00	18.31	10.00	116.00	51.00					
123		0	800.00	133.00	18.66	10.00	114.00	50.00					
124													

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	A	B	C	D	E	F	G	H	I	J	K	L	M
1													
2		***DCP Report - Single analysis***											
3		Region:	Embu										
4		Road number:	D470										
5		Project date:	11 July, 2017										
6		Selected Design	Light traffic										
7		Road Category	B										
8		Base type	Granular										
9		Moisture conditio	Optimum										
10													
11		***Measurements included in analysis:***											
12		Number	Measurement Name	Date	Position	Distance	Condition	Rutting	Pumping	Long Crack	Croc. Crack	Deforomatio	Other
13		1	Measurement 1	05-07-12		0.02	Sound	No	No	No	No	No	No
14													
15		***Results***											
16		Design Structure	181										
17		BN100 of data	24.8										
18		BN100 of SPBC	33.0										
19		Rut Limit	20mm										
20		Structural capac	2.4										
21		SPBC (Standard	A=1304, B=29										
22													
23		***Average equivalent strength (Existing pavement structure)***											
24		Depth (mm)	Weighted average penetration ra	Blows	SD (mm/blow	80P (mm/blov	CBR (%)	UCS (kPa)	Ave. E-Moduli (Mpa)	E-Moduli Range 20P - 80P Mpa			
25		0	2.24	70.00	0.40	2.80	147.00	1215.00	475.00	165 - 1460			
26		0	3.05	51.00	0.60	3.80	99.00	859.00	341.00	118 - 1059			
27		0	5.63	27.00	0.90	6.80	46.00	433.00	178.00	64 - 514			
28		0	9.10	17.00	1.40	10.90	25.00	253.00	107.00	39 - 307			
29		0	12.83	16.00	1.60	14.90	16.00	172.00	74.00	28 - 204			
30													
31		*0*											
32		0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	E-Moduli Range 20P - 80P Mpa			
33		0	2.33	47.00	0.40	2.90	140.00	1160.00	454.00	159 - 1376			
34		0	7.66	135.00	4.10	13.00	31.00	307.00	129.00	32 - 1024			
35													
36		*0*											
37		0	0.00										
38		0	-1011.00										
39		0	294.00										
40		0	1305.00										
41													
42		*0*											
43		0	0.00	0.00	0.00	0.00	0.00	0.00					
44		0	13.00	5.00	2.60	122.00	1027.00	405.00					
45		0	20.00	10.00	1.40	232.00	1810.00	781.00					
46		0	30.00	15.00	2.00	170.00	1377.00	535.00					
47		0	38.00	20.00	1.60	206.00	1630.00	677.00					
48		0	48.00	25.00	2.00	170.00	1377.00	535.00					
49		0	60.00	30.00	2.40	135.00	1123.00	440.00					
50		0	73.00	35.00	2.60	122.00	1027.00	405.00					
51		0	87.00	40.00	2.80	111.00	945.00	374.00					
52		0	100.00	45.00	2.60	122.00	1027.00	405.00					
53		0	113.00	50.00	2.60	122.00	1027.00	405.00					
54		0	123.00	55.00	2.00	170.00	1377.00	535.00					
55		0	130.00	60.00	1.40	232.00	1810.00	781.00					
56		0	140.00	65.00	2.00	170.00	1377.00	535.00					
57		0	150.00	70.00	2.00	170.00	1377.00	535.00					
58		0	160.00	75.00	2.00	170.00	1377.00	535.00					
59		0	170.00	80.00	2.00	170.00	1377.00	535.00					

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	A	B	C	D	E	F	G	H	I	J	K	L	M
60		0	185.00	85.00	3.00	102.00	875.00	348.00					
61		0	200.00	90.00	3.00	102.00	875.00	348.00					
62		0	213.00	95.00	2.60	122.00	1027.00	405.00					
63		0	227.00	100.00	2.80	111.00	945.00	374.00					
64		0	245.00	105.00	3.60	81.00	714.00	286.00					
65		0	260.00	110.00	3.00	102.00	875.00	348.00					
66		0	280.00	115.00	4.00	70.00	635.00	256.00					
67		0	295.00	120.00	3.00	102.00	875.00	348.00					
68		0	315.00	125.00	4.00	70.00	635.00	256.00					
69		0	340.00	130.00	5.00	53.00	495.00	202.00					
70		0	365.00	135.00	5.00	53.00	495.00	202.00					
71		0	395.00	140.00	6.00	42.00	403.00	167.00					
72		0	425.00	145.00	6.00	42.00	403.00	167.00					
73		0	460.00	150.00	7.00	35.00	340.00	141.00					
74		0	495.00	155.00	7.00	35.00	340.00	141.00					
75		0	545.00	160.00	10.00	22.00	228.00	97.00					
76		0	595.00	165.00	10.00	22.00	228.00	97.00					
77		0	645.00	170.00	10.00	22.00	228.00	97.00					
78		0	710.00	175.00	13.00	16.00	170.00	73.00					
79		0	780.00	180.00	14.00	14.00	156.00	68.00					
80		0	853.00	185.00	14.60	14.00	149.00	65.00					
81													

RW
27-07-2020

JOMO KENYATTA UNIVERSITY
OF
AGRICULTURE AND TECHNOLOGY
DEPARTMENT OF CIVIL, CONSTRUCTION & ENVIRONMENTAL ENGINEERING

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E-mail: civil@eng.jkuat.ac.ke

REF: JKU/2/31/65/VOL.1 10th June, 2014

TO WHOM IT MAY CONCERN

SUBJECT: PETER O. ODERO
REGISTRATION NO. EN351-1614/2013

The above named is a bona fide student of JKUAT undertaking a Masters degree in Civil Engineering, **Transportation Engineering Option.**

We are writing to kindly request you to assist him with any information he may require to finalize on his project.

JOMO KENYATTA UNIVERSITY OF AGRICULTURE AND TECHNOLOGY CIVIL CONSTRUCTION AND ENVIRONMENTAL ENGINEERING (CCEE)

P. M. KIBETU
CHAIR, DEPT.OF CIVIL, CONSTRUCTION & ENVIRONMENTAL ENGINEERING

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Setting trends in Higher Education, Research and Innovation