REDESIGNING A SECTION OF HAMU-MUKASA ROAD FROM SIR ALBERT COOK ROAD, MENGO MARKET JUNCTION TO CANON APOLLO ROAD

FINAL YEAR PROJECT REPORT SUBMITTED TO KAMPALA INTERNATIONAL UNIVERSITY IN PARTIAL FULLFIILLMENT OF THE REQUIREMENT FOR THE AWARD OF DEGREE

OF

Bachelor of Science in Civil Engineering

BY

NAME	**REG. NO**	**SIGNATURE**
ALITUBERA ESTHER	BSCE/44987/143/DU	………………………
MBONEKO MIRIA LILIAN	BSCE/44831/143/DU	………………………

DEPARTMENT OF CIVIL ENGINEERING

SCHOOL OF ENGINEERING AND APPLIED SCIENCES

CERTIFICATION

We the undersign, do here by certify that we have read and forwarded for acceptance to Kampala International University, school of Engineering and Applied science, department of civil and Mechanical Engineering a project report entitled **"redesign of a section of hamu-mukasa road from sir albert cook road, Mengo market junction to canon Apollo road**" in partial fulfillment of the requirement for the award of degree of Bachelor of science in Civil Engineering.

MUSIIME ENOS BAHATI

Signed ………………………………….

Date …………………………………….
(Supervisor)

Declaration

We the undersigned, do hereby declare that the work there in this report is original and shows the knowledge learnt, skills got and the activities done during the period of doing the project. It has never been submitted to any University or Higher Institution of learning for the award of a degree in any field for any academic purposes.

ALITUBERA ESTER BSCE/44987/143/DU …………………

MBONEKO MIRIA LILLIAN BSCE/44831/143/DU ……………………

ACKNOWLEDGEMENTS

First of all, we thank the Almighty God for his provision, protection and guidance from the conception stages of this project, its execution and to the writing of the report. We also acknowledge the help of our parents/guardians;

1 Contents

LIST OF FIGURES

LIST OF TABLES

ABSTRACT

Hamu - Mukasa road, just like most other roads in Kampala city has a number of defects such as pot holes, cracks, ravels aggregate polishing and many more accompanied by lack of drainage which in turn facilitate the high traffic congestion and delays especially at peak demand periods in addition to uncomfortablility and unsaftyness of the road users. Due to the strategic location and importance of the road as it provides access to institutions of learning such as Muteesa 1 Royal University, Buganda Royal Institute and sir Apolo Kaggwa Primary school. This has been a basis for the redesign.

This report is a presentation of the entire design processes and activities we as a team engaged in during our final year project for the award of a bachelor's degree in civil engineering. The report contains five chapters with the first chapter all about the introduction and the project background. In this, the location of the project road was defined together with the problem statement and the specific objectives to be achieved in order to overcome the problems.

The second chapter is of the literature review. This describes the various types and classification of roads in Uganda, and the different methodologies adopted during the design of this project plus recognizing works done by different researchers and writers towards highway design development. (Geometrical and structural). These methodologies mainly discussed in this chapter are the AASHTO 2001 and AASHTO 1993 for the geometrical and structural design respectively.

The third chapter is all about the methodology i.e. the various methods used to meet the set specific objectives in chapter one. The fourth chapter is about the results from the methods in the third chapter together with their discussion and engineering judgments as far as pavement design is concerned and then the design of the road is done here.

The final chapter is the fifth that includes success incurred during the entire process, challenges together with the recommendations made where applicable. This chapter is closed by various references from where our research was based and followed by the appendices. In the appendices, are various tables mainly of results from laboratory test that were being used in the calculation of engineering parameters for the design models being used.

1.0 CHAPTER ONE: INTRODUCTION

1.1 PROJECT BACKGROUND

A project section of Hamu-Mukasa road is located in Mengo, Rubaga Division. It provides a connection between Sir Albert Cook road and Canon Apollo road. It is the major access road to Muteesa 1 Royal University Kakeeka Campus

Its functionality further extends to providing access to Buganda royal institute, Super FM radio, Rubaga Cathedral, Rubaga hospital, Centenary bank Rubaga branch, Sir Apollo Kaggwa primary school, Mengo market, and so many other facilities in addition to providing a traffic diversion during peak traffic hours on Sir Albert Cook Road.

This road has a number of defects (as shown in Figures; 1-1, 1-2 and 1-3) which in turn have accelerated a deterioration in its performance and hence causing difficulty in transportation especially during certain weather seasons. Since the university is steadily growing and this is one of its major linkup roads and according to the views from the locals, its structural layers, the lane width, walkways and zebra crossing have to be redesigned so as to suit the current demand hence the purpose of the project.

allegator cracking and ravelling of the road

Figure 1: Allegator cracking and ravelling on the road

poor current drainage condition and pot holes in the road

Figure 2: poor current drainage condition and potholes in the road.

excessive ravelling and of the road

Figure 3: excessive ravelling of the road

1.2 PROBLEM STATEMENT

Like the most roads in Kampala, Hamu-Mukasa road is also in a very poor state with a number of defects such as pot holes, and roughness which are of a mechanical disadvantage to tyres and even cause delays in service delivery, poor drainage leading to silting and area flooding which leads to an early pavement deterioration that results into regular and high maintenance costs. Also the narrow lane width and a small turning radius of the existing road give difficulty to drivers especially those of bigger vehicles when turning to or off the road. Lack of walkways, signals and lightings make it difficult for pedestrians (mostly MRU students) to safely use the road. In reference to the anticipated future traffic growth as a result of increasing number of students each academic year and settlements around Rubaga division, the road is likely to worsen due to depressions that are caused by traffic delays and congestions. This therefore has been the basis of the study.

1.3 Main Objective

To redesign a section of Hamu-Mukasa road (structurally, geometrically).

1.4 Specific Objectives

- To carry out traffic assessment
- To determine the subgrade strength
- To carry out topographic survey
- To carry out geotechnical investigations

1.5 Justification

According to the millennium development goals which emphasizes a good transport network, implementation of this project will lead to an efficient service delivery, convenient access to a number of facilities in the community including schools, hospitals, institutions, and so many others and also improve on the aesthetics of the surrounding locality in addition to providing a traffic diversion to and from Kampala city center especially during peak traffic hours.

1.6 Scope of the Project

This project is limited to redesigning the road structurally and geometrically.

1.7 Project Location

Hamu-Mukasa road is located in Mengo, Rubaga division in Kampala District. It starts from Kabaka Anjagala road and cuts through Canon Apollo Kivebulaya road, Sir Albert Cook road and finally connects to Muteesa 1 road.

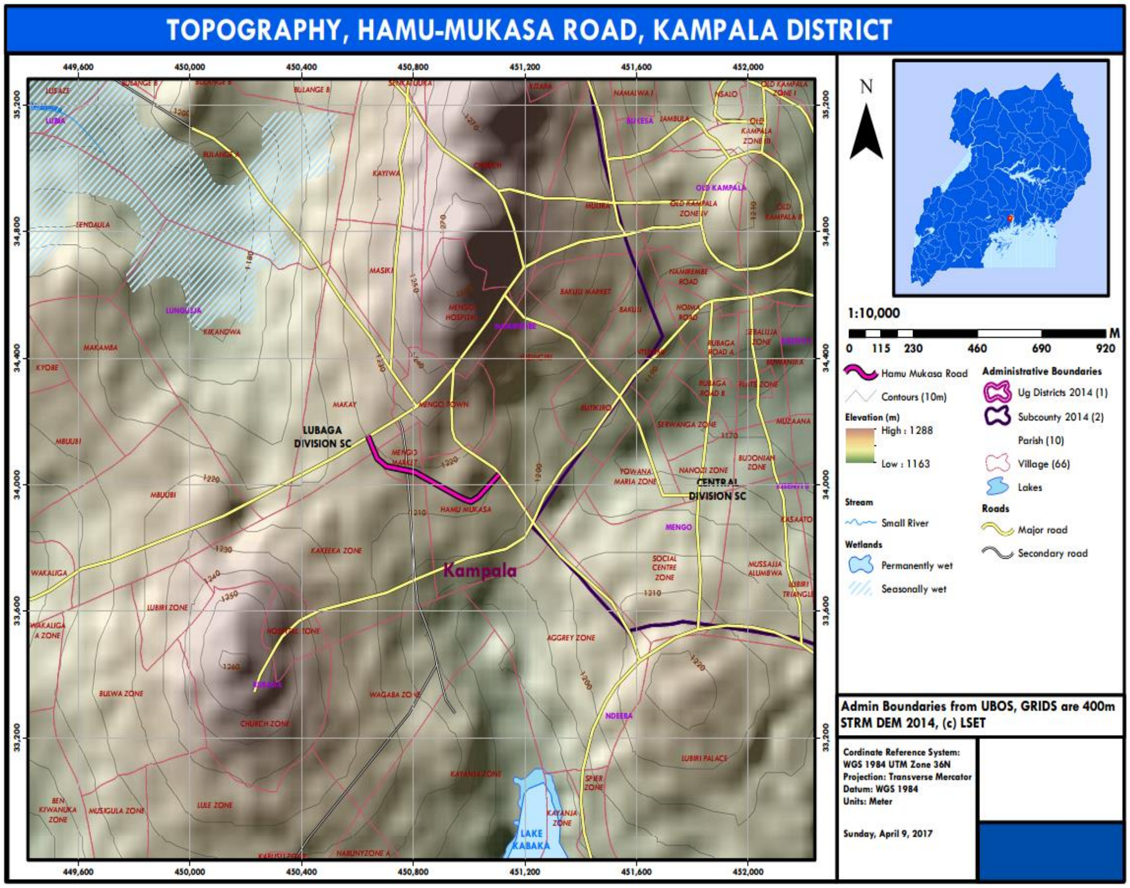


Figure 4: Location of HAMU-MUKASA road.

2 CHAPTER TWO: LITERATURE REVIEW

2.0 INTRODUCTION

This chapter describes the various types and classification of roads in Uganda, and the different methodologies adopted during the design of this project. (Geometrical and structural). These methodologies for the design are further discussed in, AASHTO 2001 and AASHTO 1993 for the geometrical and structural design respectively.

However, some important design decisions were taken also basing on the on other internationally recognized design manuals such as Overseas Road Note (ORN) and our own Uganda road design manual for Ministry of Works and Transport 2010.

2.1 Roads

One of the strongest indicators of the society's level of development is its road system or lack of one (Sponholtz, 2017). A road can be defined as a wide way leading from one place to another (traffic act 1988). For over time, road system has been developed through invasions of many contributors up to the high technology modern road system at the present.

2.2 Types and classification of roads in Uganda

There are a number of types of roads in Uganda basing on their functionality and location, and in these roads include; national roads, district roads, village roads and foot paths. All these have one major importance of providing access to the respective intended users.

According to the Ministry of Works and Transport design manual volume 1 2010, Road function determines the level of access control needed. Roads of higher classes have their major function to provide mobility, while the function of lower classes is to provide access. Motorways should always have full control of access. For all purpose roads the following general guidelines are given for the level of access control in relation to the functional road classification in table below

Table 1 **functional classification of roads in Uganda (source: Vol 1 MoWT 2001)**

Functional Class	Level of Access Control	
	Desirable	Reduced
A	Full	Partial
B	Full or Partial	Partial
C	Partial or Unrestricted	Partial
D	Partial	Unrestricted
E	Partial or Unrestricted	Unrestricted

There are six Design Classes of roads in Uganda (Vol 1 MoWT pavement design manual). Design Class road I, II, & III are bitumen surfaced. Design Class A, B, & C are gravel surfaced. Design class I is further divided into two. Ia is four lanes and Ib two lane. The division into Road Design Class is governed by the design speed and design traffic. There are many factors that affect the capacity of a road and these include design speed, width, lateral clearance, grade, alignment, weaving sections, ramp Terminals, traffic composition, type of surface, and level of service. It is therefore not possible to stipulate precise design volumes for each class of road. The values given in the table 2 below should be used as a guide. Each road must be assessed individually during the feasibility and preliminary design stage.

Table 2: **Guide for the selection of a design class of a road in Uganda (source: Vol 1 MoWT pavement design manual 2010)**

Design Class	Capacity [pcu x 1,000/day]	Road-way width[m]	Maximum Design speed Kph			Functional Classification				
			Level	Rollin g	Mountaino us	A	B	C	D	E
Ia Paved	12 - 20	20.80-24.60	120	100	80	√				
Ib Paved	6 – 10	11.0	110	100	80	√	√			
II Paved	4 – 8	10.0	90	70	60	√	√	√		
III Paved	2 – 6	8.6	80	70	50	√	√	√		
A Gravel	4 – 8	10.0	90	80	70		√	√	√	
B Gravel	2 – 6	8.6	80	60	50				√	√
C Gravel		6.4	60	50	40					√

2.3 METHODOLOGIES USED IN THE DESIGN

A complete analysis and design of urban roads requires use of a more complex model that addresses all of the many variables affecting intersection operations as well as some of the more intricate interactions among component flows (Roess, et al., 2004). The most frequently used models in the road designing internationally are AASHTO 2001 for the geometric design and AASHTO 1993 for the structural design and both methods are discussed as follows;

2.4 THE AASHTO 2001 METHODOLOGY FOR GEOMETRIC DESIGN

AASHTO (2001) specifies that the local highways consisting of two lanes should be designed to accommodate the highest practical criteria compatible with traffic and topography. Among the fundamental concepts considered for the geometric design include;

- Design traffic volume
- Design speed
- Sight distances
- Alignments.

2.4.1 Design traffic

Here, the road should be designed for a specific traffic volume and a specified level of service. The Average Daily Traffic (ADT) volume which may be either current or projected for some future design. The design year is always about 15 to 20 years from the date of completion of construction but may vary depending on the nature of improvements and reliability of the input data.

2.4.2 Design speed

Design speed is the selected speed used to determine the various design features of the road way (AASHTO 2001). Therefore, geometric design features should be consistent with a specified design speed selected as appropriate for the environmental and terrain conditions. During design, always speeds equal to or greater than the minimum values shown in table below are selected. The selection of design speeds is further discussed in AASHTO 2001, chapter two.

But as per this specific project, the design speed was selected basing on the MoWT pavement design manual volume iii, section 4, page 45.

2.4.3 Sight distances

A sight distance is a minimum distance required for a driver driving at a given design speed to make a maneuver on seeing an obstacle. The criteria for measuring sight distances both vertical and horizontal; for stopping sight distance, the height of the driver's eye is 1050mm and the height of the object is 600mm and the Full Overtaking Sight Distance (FOSD), the object height is 1050, sight distances are further discussed in AASHTO 2001, chapter 3. The sight distances were calculated from equations 1 and 2 respectively.

2.4.4 Alignment

Here, the alignments between control points are designed to be as favorable as possible (AASHTO 2001) and in agreement with the environmental impact, topography, terrain, design traffic volume and the different radii between long tangents and sharp curves.

For horizontal alignments, considerations are much focused on the horizontal curve radii and the super elevation (Fwa, 2004). A horizontal curve provides the directional transition on the horizontal plane, between two straight sections of the highway running in different directions. Horizontal curves are expressed as circular curves with constant radii, or successive curves with different radii. Calculation of the curve radii is associated with selection of various input parameters such as the design speed (V), super elevation (e), and coefficient of friction (μ).

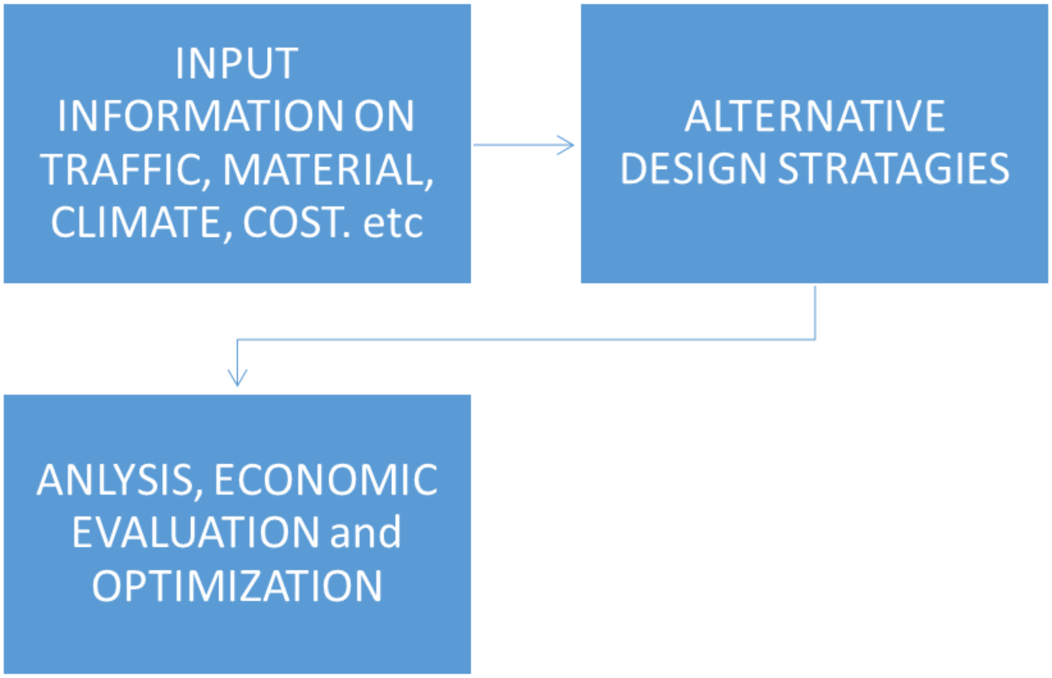
- The selection of super elevation e was based the Vol iii MoWT pavement design manual, that limits a super elevation e of 4% for urban roads.
- The selection of coefficient of friction μ was as according to the AASHTO 2001, TABLE 4.6.

For vertical alignments, a vertical curve provides a smooth transition between two tangent grades. There are two types of vertical (Fwa, 2004), that is, crest vertical curves and sag vertical curves that are defined by their grades. The grades (G) depend majorly on the terrain of the road location.

For this particular project, the selection of the grade was done basing on the MoWT design manual (2004) table 4.7, considering a selected design speed of 50 km/h.

2.5 THE AASHTO 1993

The AASHTO 1993 methodology of highway structural design is summarized in the in figure below.



The fundamental concepts considered by the AASHTO model for pavement structural design has the main requirement of determining the thicknesses of various pavement layers to satisfy the design objectives. Assuming that the pavement section consists of surface, base and sub base, three thicknesses: D1, D2 and D3 are required for the three layers, respectively. The design procedure can be divided into 12 steps as presented below. These steps have been incorporated in several computer programs to facilitate the design procedure (AASHTO, 1986).

Step 1 — Reliability

A reliability level (R) is selected depending on the functional classification of the road and whether the road is in urban or rural area. The reliability is the chance that pavement will last for the design period without failure. A larger reliability value will ensure better performance, but it will require larger layer thicknesses. The table below shows reliability levels suggested by the 1993 AASHTO design guide. The reliability levels shown below have a wide range to accommodate different field conditions. Different agencies typically select reliability values from the table that match their local conditions.

Table 3: **Typically selected reliability values (source: AASHTOO 1993)**

Functional Classification	Recommended Level of Reliability	
	Urban	Rural
Interstate and other freeways	85–99.9	80–99.9
Principal arterials	80–99	75–95
Collectors	80–95	75–95
Local	50–80	50–80

Step 2 — Overall Standard Deviation

The overall standard deviation (So) takes into consideration the variability of all input data. The AASHTO 1993 design guide recommends an approximate range of 0.4 to 0.5 for flexible pavements. An overall standard deviation value (So) is selected by the designer within this range.

Step 3 — Cumulative Equivalent Single Axle Load

In this step, the designer assumes a design life, typically in the range of 10 to 20 years. The cumulative expected 18-kip (80-kN) ESAL during the design life in the design lane is then determined as discussed earlier. If the cumulative two-directional 18-kip ESAL is known, the designer must factor the design traffic by directions by multiplying by the directional distribution factor (D) to get the ESAL in the predominate direction. For example, if the traffic split during the peak hour is 70 – 30%, D is taken as 0.7. To get the ESAL in the design (right) lane, the design traffic in the predominant direction is multiplied by the lane distribution factor.

Step 4 — Effective Roadbed Soil Resilient Modulus

Determine the resilient modulus (M_R) of the roadbed soil in the laboratory according to AASHTO T307 method (AASHTO, 2004). Since the resilient modulus of the soil depends on the moisture content, different resilient moduli will be obtained in different seasons depending on the amount of rain or snow in each season.

Step 5 — Resilient Moduli of Pavement Layers

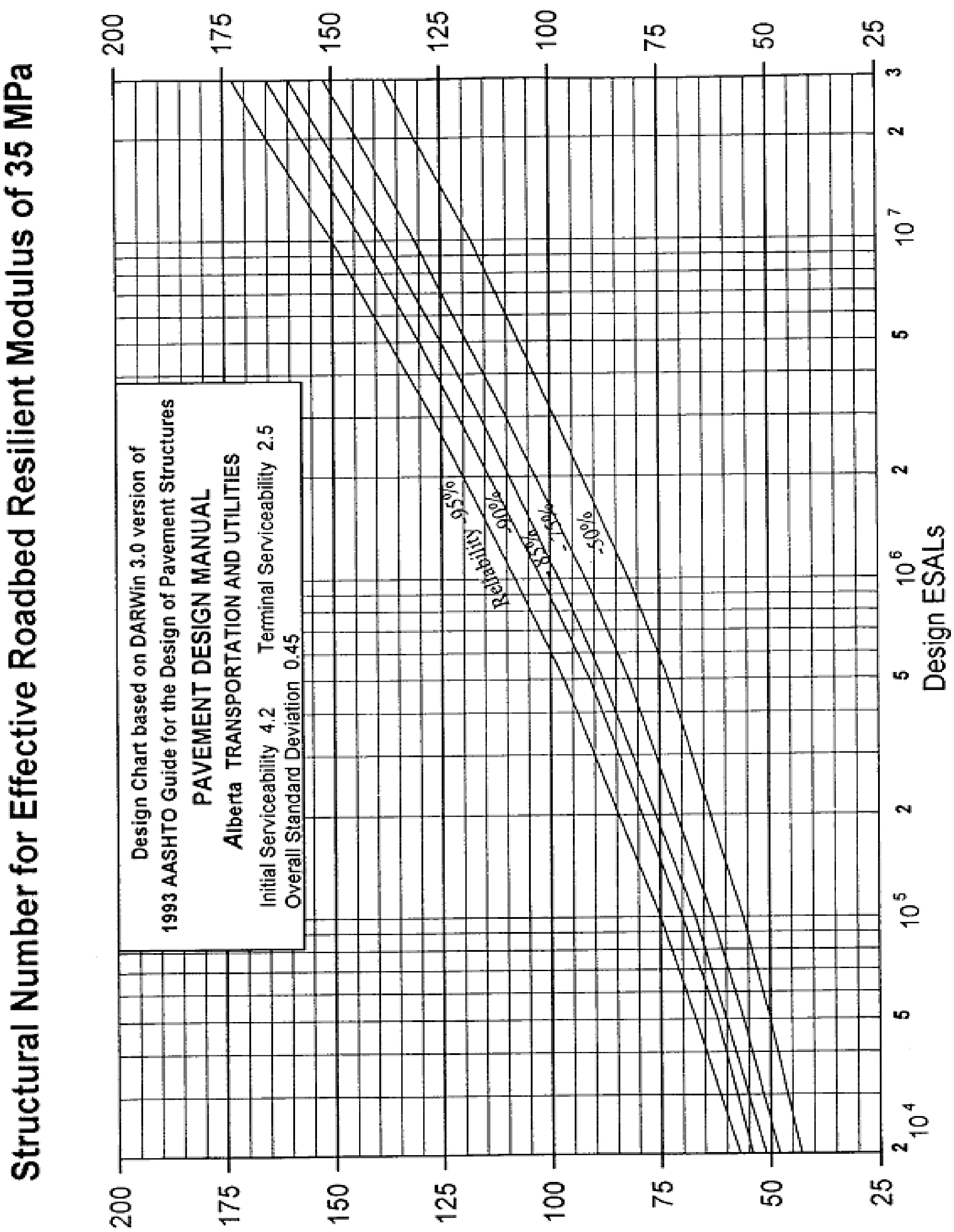
The resilient moduli (M_R) of the surface, base, and sub base layers are either determined using laboratory testing or estimated using previously developed correlations.

Step 6 — Serviceability Loss

The serviceability loss is the difference between the initial serviceability index (po) and the terminal serviceability index (pt). The typical Po value for a new pavement is 4.6 or 4.5. The recommended values of pt are 3.0, 2.5 or 2.0 for major roads, intermediate roads and secondary roads, respectively.

Step 7 — Structural Numbers

The structural number (SN) is an index value that combines layer thicknesses, structural layer coefficients, and drainage coefficients.



Step 8 — Structural Layer Coefficients

The structural layer coefficient is a measure of the relative ability of a unit thickness of a given material to function as a structural component of the pavement. Three structural layer coefficients (a1, a2 and a3) are required for the surface, base and sub base, respectively. These coefficients can be determined from road tests, as was done in the AASHTO Road Test. It is recommended that the structural layer coefficients be based on the resilient modulus, which is a more fundamental material property. A typical a1 value for the dense-graded HMA is 0.44, which corresponds to a resilient modulus of 450,000 ps

Step 9 — Layer Thicknesses

Using the structural numbers required above the base, subbase and the subgrade (SN1, SN2 and SN3) obtained in Step 7, the layer thicknesses of the surface, base and subbase (D1, D2 and D3) can be obtained from Equations below.

$$SN_1 \leq a_1 D_1$$

$$SN_2 \leq a_1 D_1 + a_2 D_2 m_2$$

$$SN_3 \leq a_1 D_1 + a_2 D_2 m_2 + a_3 D_3 m_3$$

The values of D1, D2 and D3 have to meet certain minimum practical thicknesses as shown below;

Traffic ESAL's	Asphalt Concrete	Aggregate Base
Less than 50,000	1.0 (or surface treatment)	4
50,001 – 150,000	2.0	4
150,001 – 500,000	2.5	4
500,001 – 2,000,000	3.0	6
2,000,001 – 7,000,000	3.5	6
Greater than 7,000,000	4.0	6

Step 10— Freeze or Thaw and Swelling

If the pavement is located in an area where freeze or thaw and soil swelling exist, the AASHTO (1993) design guide recommends additional procedure to estimate the reduction in the service life due to this environmental effect (AASHTO, 1993).

Step 11 — Life-Cycle Cost

In step 3, a pavement design period is assumed, which may not produce the least life-cycle cost. In this step, the designer assumes a few other design periods and repeats the design process for each design period. A life cycle cost analysis is then performed, to obtain the most economic design strategy. In this analysis all the costs included in the analysis period are considered such as the costs of initial construction, maintenance, rehabilitation, and the salvage value of the pavement section at the end of the analysis period.

This chapter represents the Environmental Impact Assessment Report, Maintenance criteria and explains the different procedures that were used in the redesigning of a section of Hamu - Mukasa road.

2.6 ENVIRONMENTAL IMPACT ASSESSMENT

This is a systematic study that is always carried out to analyze the effect of the road project investment on the environment and recommend appropriate solutions to forestall any disagreeable effects resulting from the improvement of the road.

The study on this specific project assesses the impacts of the road project and also gives the description of the environmental impacts in the developmental plans at local and national levels, from construction, maintenance, and traffic use of the road.

Some of the negative impacts that are likely to be brought about during the project implementation included; destruction of the vegetative cover for example trees since it requires expansion of the lane width, displacement of electric poles, and local residents, noise and air pollution during the execution of works, interference with traffic which may cause delay.

As an EIA team, we were able to propose different mitigation measures for all the adverse impacts and also providing alternative solutions as explained below;

2.6.1 Displacement of people.

Since the project requires expansion of the lane widths to the required standards, this means people that reside along the road are to be displaced to other places and this can easily be done through compensating them for their property loss that is to say their plots of land.

2.6.2 Traffic interference

The redesigning of Hamu- Mukasa road will lead to interference with the all the traffic that is using that section of the road since the road requires grading and a lot of road works and this can lead to traffic delays especially during the peak hours hence slowing down the construction works. However, the solution to curb this is diverting the traffic using the road section for the whole construction period and this can be achieved through employment of traffic diversion sign posts which therefore means that all the traffic that originally uses the section to town all diverted to sir Albert cook road during the construction period.

2.6.3 Air and noise pollution

This is another adverse effect to the people surrounding the project during the implementation of the project since machinery mostly graders produce a lot of noise and also release of dust to the surrounding air and this can lead to health problems to the people around and the only way to curb air pollution is ensuring regular watering of the road surface during the entire construction period through employment of enough machinery for example water browsers to do the work.

Secondly, ensuring good construction and maintenance practices such as the provision of well graded materials and adequate drainage. Mechanical stabilization where natural materials are used to enhance the properties of the material to meet the requirements of a high quality wearing surface.

2.7 HIGHWAY MAINTENANCE CRITERIA.

Highway maintenance can broadly be defined as actions taken to retain all the highway elements in a safe and usable condition. Road maintenance has been defined by Permanent International Association of Road Congress, (PIARC, 1982a) as suitable routine, periodic and urgent activities to keep pavement, shoulder slopes, drainage facilities and all other structures and property within the road margins as near as possible to their as constructed or renewed condition. Maintenance includes major repairs and improvements to eliminate cause of defects and avoid excessive repetitions of maintenance efforts.

AASHTO defines periodic maintenance as a planned strategy of cost effective treatments that preserves and maintains or improves a roadway system and its appurtenances and retards deterioration, but without substantially increasing structural capacity.

Highway maintenance is classified as follows;

Preventative maintenance. This deals with restoring the condition of the highway, reducing the rate of deterioration and increasing the life of the pavement.

Remedial maintenance. This refers to actions associated with the rectification of defects on the carriageway or road reserve.

Emergency maintenance refers to activities associated with the urgent repair of defects caused by natural disasters or accidents.

However, on this specific project, since we are dealing with a flexible pavement, we shall deal with majorly preventative and remedial actions during the maintenance of the road.

Below are some of the pavement defects under which maintenance actions must be taken during the course of using the road;

2.7.1 Cracking

This is classified into three types and these include; surfacing cracks, fatigue cracks, longitudinal and transverse cracking.

I. **Surfacing cracks.**

These cracks are associated with aging and deterioration of the surface bituminous layers due to shrinking and hardening of the bituminous binder with a loss of volatiles and such cracking is not load related.

II. Fatigue cracking

These are called alligator cracks and are a series of interconnected cracks in a chicken wire pattern. These are caused by traffic loading, and they occur only in wheel paths and often associated with deformation. Others include longitudinal and transverse cracks, block and stabilization cracks etc.

III. Potholes

These are bowl-shaped holes of various sizes on the pavement surface. The repair of potholes entails patching which is the removal of the defective layers and the replacement with normally, a bituminous mixture.

IV. Rutting

Is the longitudinal surface depression in the wheel path caused by compaction or shear deformation of the pavement layers through traffic loading.

Wide spread rutting is normally an indication of deformation of the lower pavement layers or subgrade and narrower or more sharply defined rutting of deformation in upper layers.

V. shoving

This refers to the longitudinal displacement of localized areas of the pavement caused by shear forces, induced by traffic loading. Shoving is the most evident where vehicles stop and start. The repair criterion is similar to that of rutting.

VI. **Raveling**.

This refers to gravel loss and describes the process where the aggregate particles are dislodged and weathering where the asphalt binder is removed. This is caused by the abrasive action of traffic. It is extensive on surface dressings when the binder content is too low, chippings contaminated or bituminous binder too cold to effectively adhere to the chippings during construction.

Table4: proposed maintenance action.

Defect	Appropriate maintenance action	
	Preventative	remedial
Surface cracking	Rejuvenation, resurfacing	Mill and replace (if severe)
Fatigue cracking	Resurfacing	Mill and replace
Longitudinal/transverse cracks		Crack sealing
Block stabilization cracks	Resurfacing	Mill and replace crack
Potholes		Patch sealing
Rutting and shoving	Resurfacing	Rut filling mill and replace
Polished aggregate raveling and weathering.	Resurfacing	Mill and replace
Poor binder condition	Rejuvenation, resurfacing	

3 PROCEDURES INVOLVED IN THE DESIGN ACTIVITIES

The following are the steps involved;

- Collection of the survey data
- Traffic counting
- Conducting geotechnical investigation
- Geometric design
- Structural layer design

3.1 Collection of the survey data

The necessary survey information for this project will be obtained through the following:

- Performing a reconnaissance survey
- Topographic surveys

3.2 Reconnaissance survey

In this exercise, the project section was visited with an aim of collecting information concerning the current condition of the site, the population’s perspective about the proposed road section, identifying major possible challenges that are most likely to be encountered and backing them up with photographs for further desk studies.

The information of interest involved defining temporary control points for example bench marks, Identifying locations of utility services such; Water supplies and electricity. Information regarding this was obtained from the area maps at the local council offices and Google map and taking notes of the surface and drainage conditions of the existing road.

Tools and equipment used.

- Camera
- Stationery
- Reflecting jackets

3.3 Topographic surveys.

This was done to determine the positions and shape of the natural and manmade features together with measurement of horizontal angle and distance using engineering soft wares which also provided levels of the existing ground surface. This exercise was in two steps i.e.; establishing survey control points and a detailed topographic survey.

3.3.1 Establishing Survey Control Points

Due to time and financial constraints that prohibited the use of professional surveyors to come up with survey control points, a series of software that imported data from Google Earth Maps and processed the data to come up with survey rough points that were based on in the project design. Among the Engineering softwares used in this process include;

- The google earth pro
- TCX converter
- UTM converter
- Auto CAD Civil3D

3.3.2 The Google earth pro

This is a software that enables the user to access each and every part of the world when connected on the internet. While online, the location of Hamu Mukasa road was searched and the area through which the survey points and topographic information is required was selected by clicking on the select icon and then shading the entire area using the cursor and the shaded path was saved as a KML file.

3.3.3 The TCX converter

The information from Google earth can be interpreted by the TCX converter, therefore the KLM file from Google earth pro was opened with the TCX converter which displayed the various survey points with in the selected area in terms of latitudes, longitudes, altitudes and distances and were saved as CSV files that could be opened in Microsoft excel. After opening the CSV file in Excel, the latitude and longitude columns were copied and pasted in their respective places with in the UTM converter.

3.3.4 The UTM converter

This converted the latitudes and longitudes into Eastings and Northings, and were copied and pasted in the in an Excel sheet. Also the altitude column was copied from the CSV file and also pasted with the Eastings and Northings in the Excel sheet and saved as an Excel file. At this stage, the data in the excel file could be understood by AutoCAD civil3D as survey points of the selected area.

3.3.5 AutoCAD civil3D

This is an engineering modeling software that is a core tool in the transport and road designing industry. The data from the UTM converter was imported into AutoCAD civil3d which facilitated the creation of the surface where the road alignment was drawn together with the profiles.

3.3.6 Detailed Topographical survey

Detailed topographical survey was carried out for the full length of the road. The survey covered the existing right of way, all topographical details like existing roads, tracks, drainage structures, buildings, services/utilities (electric, telephone and water lines), existing road furniture, were surveyed basing on AASHTO Guide for Design of Pavement Structures, 1993 and 1998 Supplement.

3.4 Traffic data.

This involved manual counting of traffic along the road. Traffic counts were conducted to determine the number and classifications of roadway vehicles and cyclists at section between 0+000 and 0+595 of Hamu - Mukasa road for a period of five days, 3 days of 12 hours each day This data was used in coming up with the equivalent single axle load, identifying critical flow time periods, determine the influence of large vehicles or pedestrians on vehicular traffic flow and also to document traffic volume trends. This included determination of vehicle classification, turning movements, direction of travel, pedestrian movements, and vehicle occupancy.

3.4.1 Key steps used in manual traffic count.

- Performing necessary office preparations.
- Selecting proper observer location.
- Labeling data sheets and record observations.

i. Performing Necessary Office Preparations

Office preparations started with a review of the purpose of the manual count. This type of information was of help in determining the type of equipment to use, the field procedures to follow, and the number of observers required.

ii. **Recording of the results**

The traffic count data was recorded on a tally sheet according to AASHTO Policy on Geometric Design of Highways and Streets, Fourth Edition, 2001 (Green Book) and a typical tally sheet used in the traffic counting of this specific project is shown in appendix 1

The information was processed to get the AADT on selecting only large commercial vehicles of two and more axles for the design, this was used in the calculation of the equivalent single axle load (ESAL) from the equations below.

$$ESAL = AADT * EALF * G * 365$$

Where;

$$\text{EALF} = \left(\frac{Axle\ load\ (kN)}{80\ (KN)}\right)^{4.5} \quad \text{(i)}$$

$$\textbf{GROWTH FACTOR G} = \frac{(1+r)^n - 1}{n.r} \quad \text{..........(ii)}$$

Where;

r is the predicted traffic growth rate for the design period

n is the design period

G is the growth factor

3.5 Conducting geotechnical investigations

The geotechnical investigations were carried out to assess the geotechnical strength and classification parameters of the soils of the existing subgrade of the project section of Hamu Mukasa road. This was achieved through collecting samples from the existing subgrade and carrying out the laboratory tests to achieve the laboratory CBR, and laboratory soil classification tests (PSD and Atterberg's limits.)

3.5.1 Determination of Moisture Content

- This test was done to determine the amount of water present in a soil sample and it will be expressed as a percentage of the total mass of the dry soil; this was termed as water c ontent.
- This test was done according to **(**BS 1377: Part 2: 1990)

Test Equipment

- A drying oven with temperature of 105 to 110 degrees
- A balance readable to 0.1g
- non corrosive moisture tins
- A scoop
- Moisture tins

Test Procedures

The clean dry container was weighed and its mass recorded in the data sheet (M_1)A representative moist sample of about 300g of the specimen sample was scooped and loosely placed in each container, and immediately the container weighed and its masses recorded in the data sheet (M_2).Then the representative sample was taken to the drying oven for 24 hours at a temperature of 105-110°C.After oven drying, the moisture tin with the dry sample was weighed and its mass recorded in the data sheet as (M_3).

Calculation for Moisture Content

The calculation of the moisture content, w, as a percentage of the oven dry mass of the soil was done in the following equation;

$W = \frac{M_2 - M_1}{M_3 - M_1} \times 100$……………………………………………………………………………… (iii)

Where;

M_1 is the mass of the moisture tin,

M_2 is the mass of the moisture tin + wet soil

$M_3$is the mass of dry soil + moisture tin.

3.5.2 Particle Size Distribution Test

The Particle Size Distribution test is very necessary in the classification of the soil since it presents the relative portions of different particle sizes especially for course soils. This makes it possible to determine whether the soil is predominantly gravel, sand, silt or clay. All these size ranges greatly control the engineering properties and behavior of the soil.

This test was done according to **(**BS 1377: Part 2:1990)

Test equipment

- Test sieves;75mm, 63mm, 50mm, 37.5mm, 28mm, 20mm, 14mm, 10mm, 6.3mm, 5mm, 2mm, 1.18mm, 0.600mm, 0.425mm, 0.212mm, 150mm, 0.075mm
- Lid and received/pan
- A balance readable up to 0.5g
- Riffle box
- Drying oven
- Metal tray
- Scoop
- Sieve brash.
- Moisture content apparatus

Sample Preparation

After the sample that was obtained from the road, it was air dried for 24 hours (Figure 3-2). Then it was reduced into a small sample of about 2.5kg by quartering and after riffling.

Test Procedure

A representative riffled sample to be tested was taken for moisture content determination. The remaining sample was placed on a metal tray of know mass and then weighed and the mass recorded in the data sheet (m_1).

Then water was added into the sample and allowed to soak for 24 hours to allow disintegration of cohesive particles. After 24 hours of soaking, the sample was washed on a 0.075mm sieve allowing all the material passing through the sieve run as waste. The washing was continued until the water running as waste was clear. The material that was retained on the 0.075mm sieve was transferred to the tray and then placed in the drying oven at 110℃ for 24 hours.After drying, the sample was left to cool. The test sieves were assembled in the descending from the top with the receiver/pan.

Then the oven dry washed sample was placed on to the upper coarsest sieve and then a lid was paced to cover the sample top prevent it from pouring. Then the dry washed sample was sieved manually from the 75.00mm down to 0.075mm sieve and then to the pan. Particles that were retained on each sieve were weighed and their corresponding masses recorded in the data sheet.

Calculations

The proportion of the material that was retained on each sieve was calculated and recorded in the data sheet as a percentage of the unwashed air dry sample. (M_1)

For example;

Percentage retained by sieve 14mm = $\frac{M_{14}}{M_1}$ x 100………………………………………………. (iv)

Where $M_{14}$the mass was retained on a 14mm sieve and M_1 was the original mass of the sample.

The cumulative percentage (C %) was calculated from the equation below;

C% that sieve= (% passing the previous sieve) – (% retained on that sieve)

Then percentage finer was calculated from subtracting the cumulative percentage from 100%

A typical data sheet for the Particle Size Distribution test is shown in appendix

3.5.3 Atteberg's limits

These limits will be used to provide information regarding the effect of water content on the mechanical properties of soil, thereby addressing the effect of water content on volume change and soil consistency of the existing subgrade soils. The Atterberg's limits comprise of three tests that's; the Liquid Limit test, Plastic Limit test and the Linear Shrinkage test whose results will be used in the calculation of the Plasticity Index.

3.5.3.1 **Liquid Limit Test**

The Liquid Limit is the moisture content at which a soil passes from solid state to the plastic state (Craig 2001). The Liquid Limit will be used to provide a means of identifying and classifying fine grained cohesive soils

This test will be done to (BS 1377: Part 2: 1990)

Test equipment

- 425 mm test sieves
- An airtight container
- A flat glass plate
- Two palette knives
- A penetrometer
- A metal cup 55mm in diameter and 40mm deep with the rim parallel to the flat base
- A damp cloth
- A wash bottle containing clean water
- A metal straight edge
- A stop watch

A sample of sufficient size was taken to give a test specimen weighing about 400 g which passes the 425 mm sieve. The soil was transferred to a glass plate. Water was added and then mixed thoroughly with two palette knives until the mass could become a thick homogeneous paste. The paste was then placed in an airtight container and then allowed to stand for 24 hours to enable the water to permeate through the soil

Test Procedure

The sample was placed on a glass plate and mixed for about 10 minutes using the two palette knives (Figure 3-5). Distilled water when necessary was added so that the first cone penetrometer reading was about 15mm (in the range 14.5- 15.4)

A portion of the mixed soil sample was pushed into the metal cup using a palette knife, taking care not to trap air by gently tapping the cup against a firm surface if necessary. Excess soil was struck off using a straight edge to give a smooth level surface. With the penetration cone locked in the raised position, the cone was lowered so that it just touches the surface of the soil. When the cone was in the correct position, a slight movement of the cup could just mark the soil surface. The dial gauge was then lowered to contact the cone shaft and the reading of the dial gauge was recorded.

The cone was then released for 5 seconds . After locking the cone in position, the dial gauge was lowered to contact the cone shaft and the reading of the dial gauge was also recorded. The difference between the readings was recorded as the cone penetration

Then the moisture content sample of about 20 g was taken from the area penetrated by the cone and the moisture content was determined, the cone was then carefully lifted out and cleaned.

Figure 5: **sample penetration by the penetrometer**

The remaining sample in the cap after removal of representative sample for moisture content was removed from the cap made wet by adding water using wash bottles and then placed back to the cap and the penetration procedure repeated for the second point.

When the difference between the first and the second penetration readings was less than 0.5 mm, the average of the two penetrations was recorded. If the second penetration was more than 0.5 mm and less than 1 mm different from the first, a third test was carried out. If the overall range was more than 1 mm, the soil was removed from the cup, remixed and the test was repeated until consistent results were obtained and the penetration repeated for the next three test points.

A relationship between the moisture content and cone penetration was plotted and the best line fitting the points was drawn.

The Liquid Limit of the soil sample was the moisture content corresponding to a cone penetration of 20 mm and it was expressed to the nearest whole number.

A representative sample passing 0.425mm test sieve will be homogeneously mixed with water and then molded within the palm to form balls. The balls rolled with fingers on the glass plate a long side of a 3mm steel rod to act as a reference. The 3mm threads will be left to stand until they just crumbled, then moisture content will be determined and the results will be recorded in the data sheet. (BS 1377: Part 2: 1990)

3.5.3.2 **Plasticity Index (PI)**

The plasticity index will be got as a difference between the Liquid Limit and Plastic Limit.

The Linear Shrinkage Test

The linear shrinkage test will be carried out to determine the shrinking rate of the existing subgrade soils when dried. In determination of linear shrinkage, a portion of the sample will be removed at the third point of the liquid limit test and it will be cast into the linear shrinkage mold of a standard length of 140mm.

Then the specimen sample will be left to stand for about two hours and then will be transferred to the drying oven for 24 hours where after drying, there will be a decrease in length of the sample in the mold. The percentage decrease will be taken as the linear shrinkage and it will be used as a design parameter.

3.5.4 The Proctor Compaction Test

This test was carried out to obtain a relationship between compacted dry densities of the soil and its moisture content using the magnitude of mechanical compactive efforts. The results from this test are used when providing a guide for specification on field compaction and also come up with the optimum moisture content that was to be used to carry out the CBR test. The dry densities of the soils which were achieved depended on a degree of compaction applied and moisture content. The moisture content which gave the Maximum Dry Density was referred to as the Optimum Moisture Content for a given degree of compaction. The method used for the moisture density relations was the 4.5kg rammer (BS modified heavy compaction).

This test was done to (BS 1377: Part4: 1990)

Tools and Equipment Used

- A cylindrical compaction mold with internal diameter of115mm and a volume of 1000cm^3, the mold was fitted with a detachable base plate and a removable extension collar.
- A 4.5kg metal rammer having a 50mm diameter circular face having a drop height of 450mm
- A weighing balance readable up to 1g
- A steel straightedge
- A 20mm test sieve.
- Mixing trays
- A graduated measuring cylinder
- Riffle box
- A rubber mallet
- Apparatus for moisture content

Sample Preparation

After the material to be tested was got from the field, it was first spread on hard clean floor and air dried for 24 hours then the sample was sieved through a 20mm test sieve and the particles that were retained were discarded as waste. The remaining sample was quartered to divide the sample into five specimen samples of about 3 kg.

The large soil; lumps were broken down by a rubber mallet, and each sample was mixed with a different amount of water using a measuring cylinder to give a suitable moisture. The moisture range was that at least two points lay on the either sides of the optimum water content. Then the samples were each kept in a moisture tight polythene bag waiting for compaction in the mold.

Test Procedures

The mold with the base plate attached was weighed on weighing balance and its mass recorded in the data sheet as M_1.The extension collar was attached to the mold and the whole mold assembly placed on a flat concrete surface

Figure 6: AASHTO mould assembly

Moist soil was placed in the mold in that when it is compacted, it occupies 1/5 of the volume of the mold extension collar assembly. Then the 4.5kg rammer was placed on top of the soil surface in the, then the soil was given 56 blows being evenly distributed all over the soil surface and being controlled by the guide tube to form the first layer.

The above process was repeated for four more soil layers while applying 56 blows to form the second, third fourth and fifth layers. The last layer was compacted in that it's about 6mm above the mold collar joint after compaction. After, the extension layer was removed and the excess soil was removed and the mold surface leveled using a straightedge. The mold, compacted soil and base plate were weighed and their masses recorded in the data sheet as M_2.

The soil was then extruded from the mold and representative samples were removed for the determination of moisture content and the remaining sample discarded and the procedure repeated for the next four portions of the sample and the corresponding results were recorded in the data sheet.

Calculations

The results got from the test were used in the calculation of bulky density and dry density in the equations below;

Bulky density (ρ_b) =$\frac{M_2 - M_1}{v}$kg/$\boldsymbol{m^3}$…………………………………………………………. (v)

Where,

M_1 is the mass of the mold and base plate

M_2 is the mass of mold, compacted soil and base plate.

Dry density (ρ_d) = $\frac{\rho_b}{(100+w)} \times 100$kg/$m^3$……………………………………………………….(vi)

Where;

w is the water content of a given compaction effort.

A typical density moisture relations data sheet is shown in the Appendices (Table3)

A graph of dry density against moisture content was plotted and a curve of best fit was drawn from which the Optimum Moisture Content (OMC) and Maximum Dry Density (MDD) were obtained.

The optimum moisture content was read from the highest value of dry density (MDD).

3.5.5 CALIFORNIA BEARING RATIO TEST

Test Introduction

The strength of the soils is the main factor in the determination of the required thickness of a flexible pavement roads or air fields. The strength of the sub grade, sub base and base course materials are expressed in terms of their CBR value, which is a percentage in strength of a certain standard rock. The CBR value is a great requirement for the design of the pavement of natural gravel material.

This test was done to (ASTM: D 1883-99)

Tools and Equipment Used

- Riffle box/ sample divider
- Straight edge
- Sieve 20mm diameter
- Measuring cylinder
- A cylindrical corrosion-resistant metal mold that was, the compaction /proctor mold.
- Spacer disc
- A metal rammer, having a 50 ± 0.5 mm diameter circular face, and weighing 4.5 kg
- A rammer with a suitable arrangement for controlling the height of drop to 450 mm
- Balance readable to 1g
- A CBR testing machine (Figure 3-9)
- Extruder
- Trays
- Moisture content apparatus

Figure 7: The CBR testing machine.

Preparation of Test Samples

The sample to be tested was poured and spread on metallic trays and air dried for 24 hours and after was then sieved through 20mm and to remove very large sized particles. Then using the quartering, a representative sample was obtained that weighed 18kg.

The material was then reduced to test samples by riffling using the riffle box. 18kg of the sample were then taken off and stored in an air tight container for the test. The material was then mixed with water to its Optimum Moisture Content and left to cure for about 15 minutes.

Test Procedure

The mold was fixed on the base plate and the mass of the empty mold and the base plate was weighed using a balance readable up to one gram and recorded in the test data sheet. Then the spacer disk was placed in the mold with a filter paper disk on top. A lubricant was spread in the internal wall of the molds and placed on a solid concrete floor ready to receive the specimen.

Figure 8: Sample to be tested.

$\frac{1}{5}$ Of the mold was filled with the moist soil and given 62 blows by a 4.5kg rammer dropped freely from a 450mm height to compact the sample in the mold where the blows were evenly distributed throughout sample layer surface; this was repeated for the other four layers

After compaction of the fifth layer, the collar was removed from the mold and the excess soil was trimmed off to the mold opening level by use of a steel straight edge, after the base plate was replaced with the perforated base plate with a filter paper in the mold seat and then mass of the mold, perforated base plate and specimen sample was weighed and recorded as (M_2).

The procedure was repeated for the 10 and 30 blows. The collars were fixed back on the moulds and filter disks were placed on the samples, and perforated swelling plates with two surcharge weights placed around the stem of the perforated swelling plates which were on top of the filter disk, then the whole setups containing the samples were submerged in the soaking tank, the soaking process took four days.

Figure 9: penetration of the sample for the CBR test.

After the molds containing sample had been soaked for 4 days, they were removed from the soaking tank and drained for 15 minutes by inclining them. And after the collar was removed from the molds and they were taken to the CBR testing machine for penetration. The base plate was placed on the CBR testing machine and the top surface of the sample exposed. The annular surcharge discs were placed on top of the sample and the cylindrical plunger was set to be on the surface of the sample and the force gauge and the penetration dial gauge were set at zero.

Figure 10: CBR testing machine.

Then the CBR testing machine was started and the plunger begun penetrating the sample at a rate of 1.27mm/min. Readings on the force gauge were taken and noted down every after an interval of 0.5mm penetration.

Different values of the dial reading were recorded in the data sheet up to a penetration of 7.5mm and the testing machine was stopped. The sample was then removed from the machine and the position of the plunger was filled with sand.

Then the mold was inverted upside down on the base plate and the penetration procedure repeated on the bottom part of the sample and the results recorded. Then the results were transferred to the CBR data sheet. The penetration procedure was repeated for the next two samples and the results were recorded in the data sheet.

A typical data sheet for the CBR test is shown the appendices graph of force reading against penetration was plotted on which the force reading on the 2.5 and 5.0mm penetration were read and noted.

3.6 Geometric Design

The geometric design criteria were based on the Ministry of Works and transport (MoWT) Road Design Manual Volume 1 (Road Geometric Design) and requirements of AASHTO Policy on Geometric Design of Highways and Streets, Fourth Edition, 2001 (Green Book) and Rural Road Access Road standards.

To meet the objective of fitting the highway to site topography and yet satisfy the safety, service and performance standards, the following considerations were properly addressed in the design process which included; Design speed, Design traffic volume, Number of lanes, Level of Service (LOS), Sight distance, Alignment, super-elevation and grade

3.6.1 Design speed

This is the speed determined for the design and correlation of the physical features of the highway that influence vehicle operation.

According to ministry of works and transport Uganda design manual,

3.6.2 Level of service (LOS)

According to AASHTO 2001, LOS = type D since it is a local road in an urban center

3.6.3 SIGHT DISTANCES

A sight distance is a minimum distance required for a driver driving at a given design speed to make a maneuver on seeing an obstacle.

a. Stopping sight distance

This was calculated using the formula below

$$\frac{V^2}{\left[254\left(\frac{a}{9.81}\right)\pm G\right]} \text{ ....(vii)}$$

Where

a is the constant deceleration rate

G is the longitudinal grade in %

V is the design speed

b. Full overtaking sight distance (FOSD)

This was calculated from the equation below;

$$\boldsymbol{FOSD = 0.5Vt} \text{ ....(viii)}$$

Where;

V is the design speed in km/hr

t is the time taken to complete the entire maneuver and it is generally taken as 10 seconds

3.6.4 HORIZONTAL ALIGNMENT

According to table 4.6 from AASHTO design manual, the coefficient of friction μ=0.16

The design speed **V**=50km/hr According to the ministry of works and transport Uganda design manual 2010 and the super elevation **e** is limited to 8% b according to the Ugandan standards.

Since we have a low design speed a super elevation of 4% is considered.

$\boldsymbol{Radius\ of\ the\ curve\ R = V^2/127(e+\mu)}$**……………………………………………(ix)**

Where;

V is the design speed in km/hr

e is the super elevation which in this case 4% is considered

μ is the coefficient of friction which in this case is taken as 0.16

3.6.4.1 Transition curve

This is the curve in which the radius changes continuously along its length and is used for the purpose of connecting a straight with a curve or two circular curves of different radii

They provide convenient sections over which super elevation or pavement widening may be applied and they improve the appearance of the road by avoiding sharp discontinuities in alignment at the end and beginning of circular curves.

$\boldsymbol{L = \left(\frac{V}{3.6}\right)^3 * \frac{1}{CR}}$ **……………………………………………………………………(x)**

Where;

L = the length of the transition curve

V = the design speed in km/hr

C = the rate of change of centrifugal acceleration which is taken as $0.3m/s^3$

R = the radius of the circular curve

3.6.4.2 Vertical alignment

This refers to the arrangement of tangents and curves which compose the profile of the road. The major aim of vertical alignment is to ensure that a continuously unfolding stretch of the road is presented to motorists so that their anticipation of directional change and future action is instantaneous and correct.

From the ministry of works and transport road design manual (2004) and considering a design speed of 50 km/hr and a flat terrain, a gradient of 6% was considered

- The cross fall from one edge to the other of 2.5% was selected
- Vertical crest curve design and sight distance requirements

First assume that S ≤ L

$$L_{min} = \frac{AS^2}{200(\sqrt{h_1}+\sqrt{h_2})^2} \text{ ....}\textbf{(xi)}$$

Where;

L_{min} is the minimum length of the vertical crest curve

h_1 is the driver eye height = 1.05 m

h_2 is the objective height = 0.26 m

S is the required stopping sight distance = 35 m

A is the grade = 5% for a flat terrain according to the ministry of works and transport road design manual

Assume that S > L

$$L_{min} = 2S - \frac{200(\sqrt{h_1}+\sqrt{h_2})^2}{A} \text{ ....}\textbf{(xii)}$$

- **Vertical sag curve design and sight distance requirements Based on clearance from structures during day time**

When S ≤ L

$$L_{min} = \frac{AS^2}{\left[8D-8\frac{(h_1-h_2)}{2}\right]} \text{ ....}\textbf{(xiii)}$$

When S > L

$$L_{min} = \frac{\left[2S-8(\frac{h_1+h_2}{2})\right]}{A} \text{ ....}\textbf{(xiv)}$$

Where;

h_1 is the driver's eye height = 1.05 m

h_2 is the object height =0.26 m

L_{min} is the minimum length of the sag curve

A is the algebraic difference in grades expressed as a decimal

D is the vertical clearance to the critical edge of the bridge ideally taken as 5.7 m

- **Based on night time conditions**

When S ≤ L

$$\boldsymbol{L_{night} = \frac{AS^2}{200(h_3 + S\ tan\alpha)}}$$ **…………………………………………………………….(xv)**

When S > L

$$\boldsymbol{L_{night} = 2S - \frac{[200(h_3 + S\ tan\alpha)]}{A}}$$ **………………………………………………………….(xvi)**

Where;

h_3 is the headlight height which is taken as 0.6 m above the carriageway

α is the angle of upward divergence of light beam taken as 1.0 degrees

L is the minimum length of the curve

A is the algebraic difference in grades

S is the required stopping sight distance

- **Based on motorist comfort**

$$\boldsymbol{Minimum\ length\ of\ the\ vertical\ sag\ curve\ L_{min} = \frac{AV^2}{13a}}$$ **……………………….(xvii)**

Where;

V is the design speed in km/hr

A is the algebraic difference in grade

a is the radial acceleration taken as 0.3 m/s^2 for a comfortable design.

3.7 Pavement Design

The pavement shall be designed in accordance with the Ministry of Works and transport (MoWT) Road Design Manual Volume 111 (Pavement Design Manual) and the AASHTO
Include;

a) **RELIABILITY (R)**

This was selected from the table (AASHTO 1993) and it was selected basing on the function (local).

A design life of 15 years was selected basing on the AASHTO 1993 design manual A distribution factor of 100% was selected since it is a single carriage way with two lane road

b) **SELECT MATERIAL PROPERTIES**

The design on this project is based on the nominal material strength classifications given in Table5.1 of the MoWT flexible pavement design manual 2010. For structural purposes, this provides a guide to the probable performance, assuming that no unexpected deterioration (for example, due to water ingress) takes place. The full specifications, given elsewhere, include a number of other indicatory properties to assure that such deterioration ought not take place during the life of the road.

Table 5: **nominal strength classification of materials used for the design, (source; MoWT manual 2010)**

Layer	Material	Nominal Strength
Base	Granular	Soaked CBR > 80% @ 98% mod. AASHTO density
	Cemented	7 day UCS*1.5 - 3.0 MPa @ 100% mod. AASHTO density (or 1.0 - 1.5 MPa @ 97% if modified test is followed)
	Bituminous	See specification
Subbase	Granular	Sokaed CBR > 30% @ 95% mod. AASHTO desnity
	Cemented	7 day UCS*0.75 - 1.5MPA @ 100% mod. AASHTO density (or 0.5 - 0.75 MPa @ 97% if modified test is followed)
Capping/selected	Granular	Soaked CBR > 15% @ 93% mod. AASHTO density
* 7 day uncofined compressive strength		

Table 6: **layer coefficients used in the design (source; MoWT manual 2010)**

Layer/Material	Layer Coefficient
Surfacing	
Surface dressing	$a_1 = 0.20$
Asphalt concrete	$a_1 = 0.35$
Base	
Bitumen Macadam	$a_2 = 0.20$
Natural or crushed gravel	$a_2 = 0.12$
Crushed stone	
On natural gravel subbase	$a_2 = 0.14$
On stabilised subbase	$a_2 = 0.18$
Cement treated gravel (4)	
Type A, $3.5 \leq$ UCS (Mpa) < 5.0	$a_2 = 0.18$
Type B, $2.0 \leq$ UCS (Mpa) < 3.5	$a_2 = 0.14$
Subbase	
Natural gravel, CBR $\geq$ 30%	$a_3 = 0.11$
Cement treated material	
Type B, $2.0 \leq$ UCS (Mpa)	$a_3 = 0.16$
Type C, $0.7 \leq$ UCS (Mpa) < 2.0	$a_3 = 0.12$

c) STRUCTURAL NUMBER

The structural number was read from the 1993 AASHTO tables (Table 5) that was referring to the resilient modulus and the design EASL (AASHTO 1993.

d) Structural Layer Coefficients

The structural layer coefficients were selected in accordance with Table 8.4.

4 CHAPTER FOUR: RESULTS AND DISCUSSIONS

This chapter discusses the analysis of the obtained data and the redesign of a section of Hamu-Mukasa road

4.0 Selection of the design life.

A design life of 15 years was selected basing on Volume 3 part I of the MoWT pavement design manual 2010, table 2.1. This means that basing on the reliability of data sources for the design, (low), and level of service, (high), and the road is expected to serve for 15 years in good shape with minimum major rehabilitation works.

Table 7: **pavement life selection table (source: MoWT flexible pavement design 2010)**

Design Data Reliability	Importance/Level of Service	
	Low	High
Low	10-15 years	15 years
High	10-20 years	15-20 years

4.1 Traffic data

Table 4 traffic data for the calculation of the ESAL

$$EALF = (\frac{Axle\ load\ (kN)}{80\ (KN)})^{4.5}$$

$$GROWTH\ FACTOR\ G = \frac{(1+r)^n - 1}{r}$$

$$EASL = 365 * G * EALF * AADT$$

$$\Sigma EASL = 30\ million\ EASL's$$

NO OF AXLES	AADT (VEH/DAY)	AXLE LOADS (KN)	EALF	NO OF REPETITIONS	GROWTH FACTOR (G)	EASL (MILLIONS)
2 Axles	85	177	23.96	365	19	7
3 Axles	120	235	24.46	365	19	10
≥4 Axles	20	294	182.4	365	19	13
					TOTAL	30

4.2 Pavement Design Procedure

4.2.1 Reliability (R)

This is intended to account largely for chance variations in traffic prediction and performance prediction, and therefore provides a predetermined level of assurance (R) that pavement sections will survive the period for which they are being designed.

This was selected from the table (AASHTO 1993) and it was selected basing on the calculated ESALS of the road.

Table 8: **Recommended reliability levels for different values of ESALS (source: table 2.2 of the AASHTO GUIDE 1993)**

Design ESALS ($x10^6$)	Reliability %
<0.1	75
0.1 to 5.0	85
5.0 to 10.0	90
>10.0	95

Therefore basing on table 7, reliability levels $R = 95\%$

- **This indicates that pavement being designed will survive by 95% the period of 15 years without major repairs or rehabilitation works.**

4.3 GEOTECHINICAL INVESTIGATIONS

4.3.1 Particle Size Distribution Test results.

The results after the sieve analysis were tabulated in the table below, and a graph of % finer against sieve number was plotted as in figure below

Mass of dry sample (gm), m_1	1099.2		
BS Sieve size (mm)	**Mass retained (grams)**	**% retained**	**% passing (finer)**
75	0.0	0.0	100.0
50	0.0	0.0	100.0
37.5	0.0	0.0	100.0
28	0.0	0.0	100.0
20	0.0	0.0	100.0
14	76.7	7.0	93.0
10	96.4	8.8	84.3
6.35	195.0	17.7	66.5
5	71.6	6.5	60.0
2.36	141.6	12.9	47.1
1.18	83.6	7.6	39.5
0.600	107.9	9.8	29.7
0.425	74.7	6.8	22.9
0.300	111.7	10.2	12.7
0.150	121.8	11.1	1.7
0.075	17.5	1.6	0.1
Pan	0.7		0.1
Pan+C	1037.5		

weight of wet unwashed sample = 2248.5

weight of dry washed sample = 1099.2

water content = 5.0

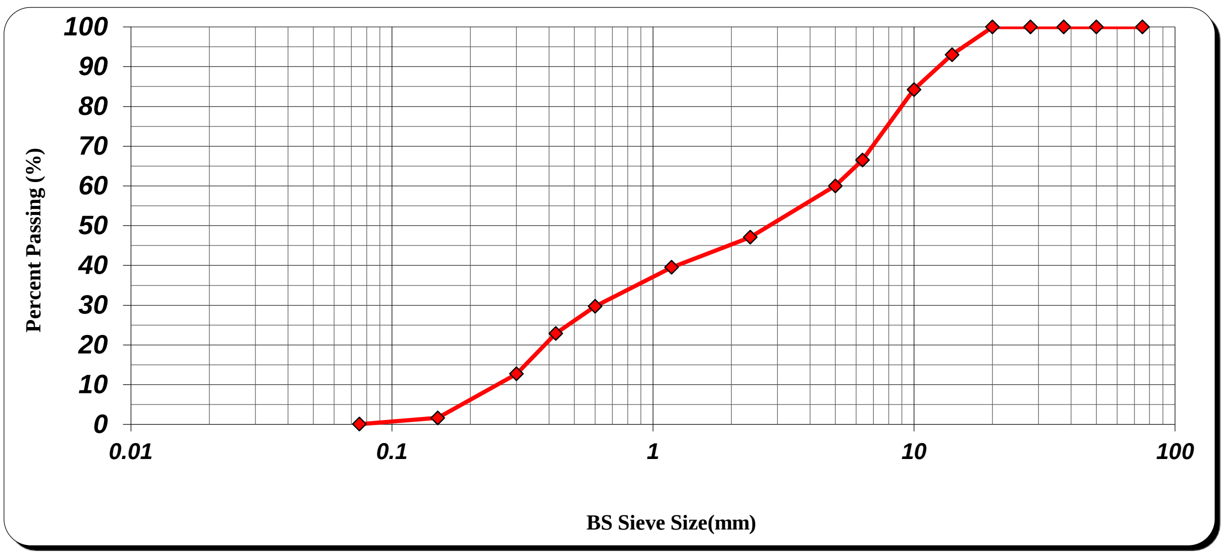
weight of water = 112.4

weight of washed away fines C = (2248.4-1099.2-112.4) = 1036.8

459.5
439.7
43.5
5.0

Total weight of dry sample = (1099.2 + 1036.8) = 2136g

- **Therefore; % of fines = $\frac{1036.8}{2136}$ = 48%**



From the graph, particle sizes corresponding to percentage finer of; 60,30 and 10 were read.

% passing	Particle size (mm)
60	5
30	0.6
10	0.25

Then the coefficient of uniformity C_u , and coefficient of curvature C_Z were calculated from the equations below;

$C_u = \frac{D_{60}}{D_{10}}$ …………………………………………………………………………… (xxvii)

$C_Z = \frac{(D_{30})2}{D_{60}*D_{10}}$ ……………………………………………………………………………(xxviii)

From the equations,

$where$;

D_{60} = the particle size corresponding to % passing 60

D_{30} = the particle size corresponding to % passing 30

D_{10} = the particle size corresponding to % passing 10

C_u	20
C_Z	0.29

- *The higher the value of the coefficient of uniformity the larger the range of particle sizes in the soil. A well-graded soil has a coefficient of curvature between 1 and 3 (Craig, 2004) there for, the subgrade soils, with coefficient of curvature 0.29, it implies its poorly graded, with dominant particles being 48% fines and 52% coarse of which as observed from the graph, 50% is gravel and 2% sand.*

- *On reference to the graph in figure 3-1, the dominant part is gravel implying that the soils are course. According to AASHTO 1993, the larger the soil particles, the better the drainage characteristics it possesses. And in reference to the PSD results, the will poses a good drainage which is a preferred characteristic for road subgrade.*

4.3.2 Atteberg's Limits test results

An empirical boundary called the 'A' line, with a slope expressed by the equation plasticity index=0-73 (liquid limit--20), separates inorganic clays from inorganic silts and organic soils on the plasticity chart (Casagrande, 1948). Further vertical subdivisions of the chart are made to distinguish differences in engineering properties such as compressibility, permeability and toughness. Soils located within each area of the chart are assigned a definitive code in the classification and would be expected to behave similarly in road constructional engineering.

For a good road subgrade, it should exhibit a high level of permeability, toughness together with a low level of compressibility under working loading.

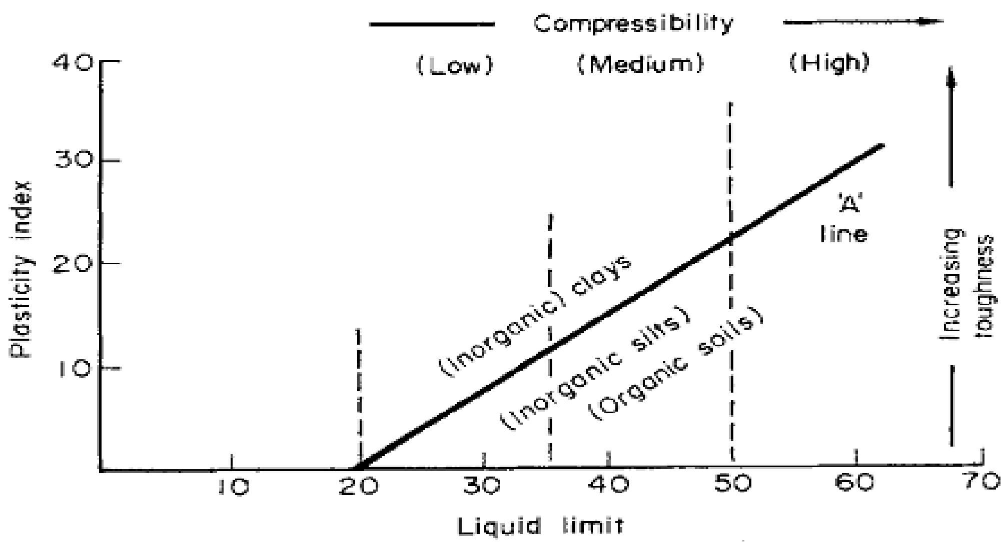


Figure 11: plasticity chart (After casagrande, 1948)

4.3.2.1 Liquid Limit test results

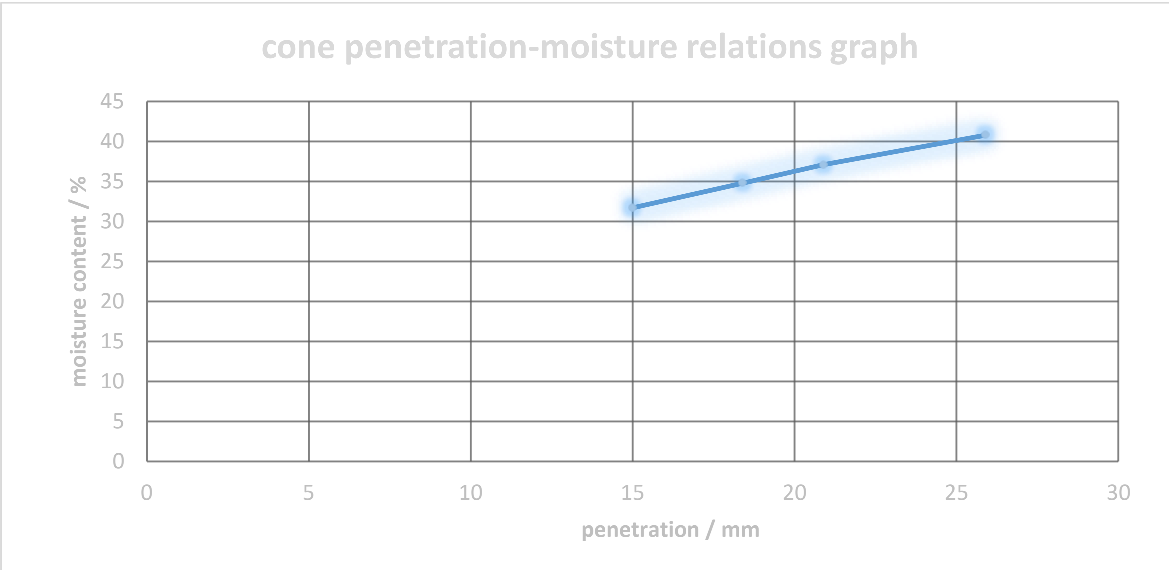
In this test, different moisture contents were corresponding with their con penetration were tested and recorded in the data sheet shown below.

Table 9: **Liquid limit test results**

	LIQUID LIMIT							
TEST NO.	**1**		**2**		**3**		**4**	
Initial dial gauge reading mm	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Final dial gauge reading mm	14.95	14.95	18.40	18.40	20.90	20.90	25.92	25.92
Average penetration mm	15.0		18.4		20.9		25.9	
Container No.	**Gc**	**g9**	**M3**	**8D**	**11X**	**gw**	**10.0**	**Gz**
Mass of wet soil + container (a)	53.84	51.53	65.66	63.18	59.01	56.73	60.34	64.82
Mass of dry soil + container (b)	46.73	45.02	55.00	53.26	49.66	48.04	49.97	53.16
Mass of container (c)	24.32	24.52	24.51	24.70	24.53	24.52	24.50	24.61
Mass of moisture (d = a-b)	7.11	6.51	10.66	9.92	9.35	8.69	10.37	11.66
Mass of dry soil (e = b-c)	22.41	20.50	30.49	28.56	25.13	23.52	25.47	28.55
Moisture content (w =100X(d)/(e)	31.73	31.76	34.96	34.73	37.21	36.95	40.71	40.84
Average Moisture content	31.7		34.8		37.1		40.8	

A graph of moisture content against cone penetration was plotted and the liquid limit was read at the 20mm penetration as shown below.

Table 10: **Moisture content cone penetration relation.**



From table, the liquid limit was read as 36% moisture content.

Table11: ***Plastic Limit test results***

PLASTIC LIMIT TEST		
Container No.	BL	BE
Mass of wet soil + container (a)	17.96	19.3
Mass of dry soil + container (b)	17.1	18.3
Mass of container (c)	13.26	13.4
Mass of moisture (d = a-b)	0.86	1.06
Mass of dry soil (e = b-c)	3.84	4.82
Moisture content (w =100X(d)/(e)	22.39583	22
Average Moisture content	22.19376729%	

- ***Then the test results for the plastic limit was 22% water content.***
- ***The plasticity index was calculated from the test results of liquid limit and plastic limit from the equation below.***

$$\boldsymbol{plasticity\ index\ (PI) = Liquid\ Limit\ (LL) - Plastic\ Limt\ (PL)}$$

= (36 – 22) %

PI = 14%

- Here, a judgment of engineering properties that’s, compressibility, permeability, and toughness were taken basing on casagrande’s plasticity chart and the classification of the subgrade soil as below;
  - **I.** By the unified classification system, the subgrade soil of the existing subgrade lays with in the region of inorganic clays,
  - **II.** Therefore, it can be concluded that the subgrade soli is a Very Clayey Sandy Gravel, with lager portions of Gravel and Clay but with almost no sand and silt, making it a Gap graded soil (Craig, 2004)
  - III. When the chemical stabilization or modification of subgrade soils is considered as the most economical or feasible alternate, the following criteria should be

considered for chemical selection based on index properties of the soils (Atterberg's limits)

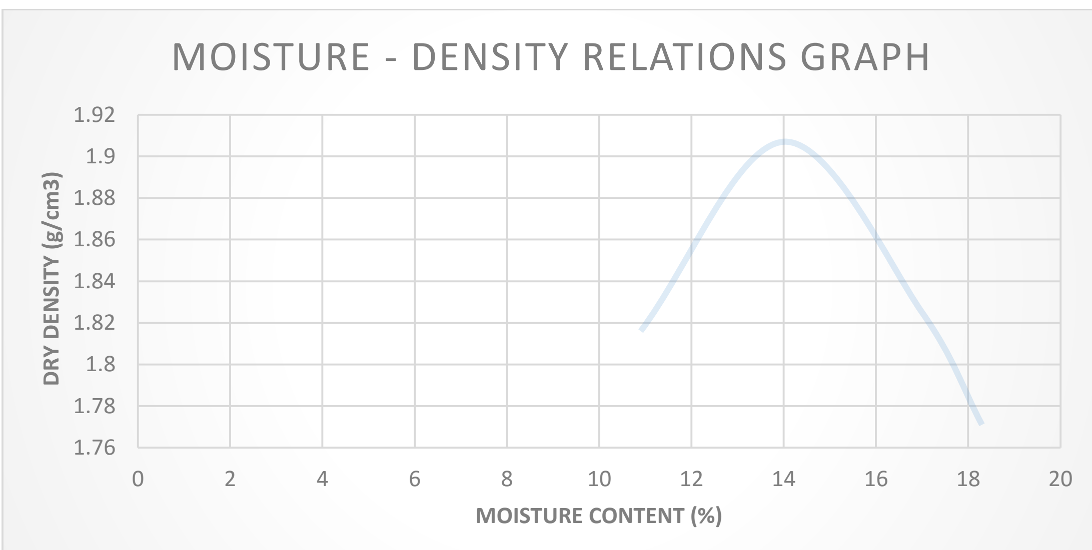
Chemical Selection for Stabilization according to MoWT pavement design manual is recommended as below;

- ***Lime: If PI > 10 and clay content (2μ) > 10%.***
- ***Cement: If PI ≤10 and < 20% passing No. 200.***

The high portion of fines and high value of PI makes the subgrade soils have poor engineering properties and hence modification/ stabilization is recommended as per (BS 1924: PART 1)

4.3.3 COMPACTION TEST

The moisture density relations test results were summarized in the table of the appendices and a graph of moisture against dry density was plotted to read off the OMC and MDD as shown below;



MDD = 19.1
OMC = 14

- **The MDD and the OMC results were used in the casting process of the CBR test**

4.3.3.1 CBR TEST RESULTS

The CBR test results shown in table below, then the ring factor of 0.002 was multiplied to achieve the equivalent force in kilo newton's in table below. A typical data sheet for the all CBR test is in appendices.

Table 12: **Dial gauge reading and their corresponding penetrations**

Penetration (mm)		Dial Gauge Reading					
		LIGHT		INTRM.		HEAVY	
		Top	Bottom	Top	Bottom	Top	Bottom
0.0		0.0	0.0	0.0	0.0	0.0	0.00
0.5		4	2	15	12	12	23
1.0		5	3	24	19	37	39
1.5		6	3	30	22	60	47
2.0		7	3	33	24	84	55
2.5		7	3	37	27	108	64
3.0		7	3	40	30	124	72
3.5		7	4	44	31	135	78
4.0		7	4	46	32	144	84
4.5		7	4	49	34	152	89
5.0		7	5	51	35	159	92
5.5		7	5	52	36	169	99
6.0		7	5	53	38	181	103
6.5		8	5	55	40	187	108
7.0		9	6	59	41	195	110
7.5		9	6	60	41	202	113
8.0		9	6	61	42	208	115

Table 13: **Equivalent force in kN**

Equivalent Force in Kn					
LIGHT		INTRM.		HEAVY	
Top	Bottom	Top	Bottom	Top	Bottom
0.00	0.00	0.00	0.00	0.00	0.00
0.08	0.04	0.30	0.24	0.24	0.46
0.10	0.06	0.48	0.38	0.74	0.78
0.12	0.06	0.60	0.44	1.21	0.94
0.14	0.06	0.66	0.48	1.69	1.11
0.14	0.06	0.74	0.54	2.17	1.29
0.14	0.06	0.80	0.60	2.49	1.45
0.14	0.08	0.88	0.62	2.71	1.57
0.14	0.08	0.92	0.64	2.89	1.69
0.14	0.08	0.98	0.68	3.06	1.79
0.14	0.10	1.03	0.70	3.20	1.85
0.14	0.10	1.05	0.72	3.40	1.99
0.14	0.10	1.07	0.76	3.64	2.07
0.16	0.10	1.11	0.80	3.76	2.17
0.18	0.12	1.19	0.82	3.92	2.21
0.18	0.12	1.21	0.82	4.06	2.27
0.18	0.12	1.23	0.84	4.18	2.31

- ***The CBR value was calculated by dividing the 2.5 and 5.0mm force reading by 13.24 and 19.96 respectively. taking averages of the top and bottom penetration, the highest of the three compaction efforts results, that is; light, interim and heavy compaction was confirmed the CBR value for the existing subgrade soils.***

Table 14: **Test results from the CBR test**

CALCULATION OF CBR %						
CBR at 2.5 mm penetration	1.1	0.5	5.6	4.1	16.4	9.7
CBR at 5.0 mm penetration	0.7	0.5	5.1	3.5	16.0	9.3
Retained CBR	0.8		4.9		13.1	

The CBR = 13.1%

- ***Considering the MoWT, flexible pavement design manual 2010. Table 3.1, (Table), the subgrade is suitable for road construction as it exhibits a convincing strength level of S4 when compacted at 105% laboratory tested MDD (Appendices).***

- ***This implies that according to table in appendices, the compaction layer thickness during processing of the subgrade should not go beyond 550mm.***

- ***Also according to the MoWT pavement design manual 2010, also the level of S4 is strong enough to go unsterilized during the subgrade processing.***

Subgrade classification (source : MoWT flexible pavement design manual 2010)

Subgrade Class Designation						
Subgrade CBR ranges (%)	S1	S2	S3	S4	S5	S6
	2	3-4	5-7	8-14	15-29	30+

4.3.4 Effective Resilient Moduli (M_R)

The characteristic material property of subgrade soils used for pavement design is the resilient modulus (M_R). The resilient modulus is defined as being a measure of the elastic property of a soil recognizing selected non-linear characteristics. Methods for the determination of M_R are described in AASHTO T294-92 test method. For many years, standard California Bearing Ratio (CBR) tests were utilized to measure the subgrade strength parameter as a design input from the following equation;

$\boldsymbol{M_R = 17.6(CBR)^{0.64}}$ ....(xxviii)

From the test in the laboratory, $\boldsymbol{CBR = 13.1\%}$

$$M_R = 17.6(13.1)^{0.64}$$

$$\boldsymbol{M_R = 91.32\ Mpa}$$

- This relationship is dependent upon the particular seasons of conditions similar to the laboratory test conditions of the CBR test.

parameters assumed and therefore should be considered as climatic/geographic specific. It is suggested that this be considered a regional adjustment factor, Combining the adjustment factor (C) with this regional adjustment factor, the Effective Roadbed Resilient Modulus for design purposes can be determined by the following equation: Design $\boldsymbol{M_R}$**= 0.36 x (back calculated** $\boldsymbol{M_R}$**)** (Fwa, 2005)

- therefore; Design resilient modulus

$$\boldsymbol{M_R = 0.36 * 91.32 = 33Mpa}$$

4.3.5 STRUCTURAL NUMBER

- $\boldsymbol{EASL = 3 * 10^7}$
- $\boldsymbol{R = 95\%}$

Rounding off M_R = 35Mpa

From chart of resilient modulus (35Mpa) and considering a design EASL of 30 million EASL's

$$Structural\ number, SN = 152mm$$

$$SN = a_1 D_1 m_1 + a_2 D_2 m_2 + a_3 D_3 m_3$$

$$a_1 = 0.42$$

$$a_2 = 0.14$$

$$a_3 = 0.08$$

The layer thicknesses in coloration with the structural number (SN) were calculated from the equation above by try and error method

TRIAL 1

Taking asphalt concrete thickness (D_1) as 180mm, and then calculating for the course base thickness (D_2) from equation as follows;

$$SN_1 = 0.42 * 180$$

= 76

$$SN_2 = SN - SN_1$$

= 152 – 76

= 76

Then $D_2 = \frac{SN_2}{a_2 m_2}$

$$= \frac{76}{0.14 * 1.2} = 452mm$$

Calculated $SN_2 = 452 * 0.41 * 1.2$

= 76

Therefore; provided SN = 76 + 76 = 152 (OK).

Other trials were made to come up with various layer thicknesses that may be favorable to meet construction demands and material availability and are summarized in table below;

	STRUCTURAL LAYERS	Ai	Di	Mi	SNi	sum SNi
1	AC	0.42	180	1.2	76	152
	GBC	0.14	452	1.2	76	
2	AC	0.42	180	1.2	76	152
	GBC	0.14	350	1.2	59	
	GSBC	0.08	177	1.2	17	
3	AC	0.42	180	1.2	76	152
	GSBC	0.08	792	1.2	76	

Where;

- AC is the Asphalt Concrete layer
- GBC is the Granular Base Course layer
- GSBC is the Granular Subbase Course layer

Considering trial 2, the layers are summarized as below;

	180 mm asphalt concrete + a seal coat whose thickness is to be determined by the engineer
	350 mm base course layer
	Modified compacted subgrade to 105% laboratory MDD to achieve a CBR of 13.1% whose layer thickness will be determined by the engineer on site but should not aceed 550mm.

4.4 GEOMETRIC DESIGN

4.4.1 Design speed

This is the speed determined for the design and correlation of the physical features of the highway that influence vehicle operation.

According to ministry of works and transport Uganda design manual,

$$design\ speed = \frac{50km}{hr}$$

$$design\ load = 30million\ ESAL's$$

Number of lanes =2

Level of service (LOS)

According to AASHTO 2001, LOS = type D since it is a local road in an urban center

4.4.2 Sight Distances

Stopping sight distance was calculated using the formula below

$$\frac{V^2}{\left[254\left(\frac{a}{9.81}\right) \pm G\right]}$$

$$\frac{50^2}{\left[254\left(\frac{3.4}{9.81}\right) \pm 0.05\right]}$$

$$stopping\ sight\ distance = 33m$$

Design stopping sight distance =35m

Full overtaking sight distance (FOSD)

$$FOSD = 0.5Vt$$

FOSD =0.5*50*9.1

FOSD =227.5m

4.4.3 Horizontal Alignment

According to table 4.6 from AASHTO design manual, the coefficient of friction μ=0.16

The design speed V=50km/hr

According to the ministry of works and transport Uganda, the super elevation e is limited to 8%

Since we have a low design speed a super elevation of 4% is considered.

$Radius\ of\ the\ curve\ R = V^2/127(e + \mu)$

R=$50^2/127(0.04 + 0.16)$

R=98.4m calculated

Design curve radius=100m

Transition curve

$$L = \left(\frac{V}{3.6}\right)^3 * \frac{1}{CR}$$

$$L = (\frac{50}{3.6})^3 * 1/(0.3 * 100)$$

L=89.3m calculated

L=90m design

Vertical alignment

Vertical crest curve design and sight distance requirements

First assume that S ≤ L

$$L_{min} = \frac{AS^2}{200\left(\sqrt{h_1} + \sqrt{h_2}\right)^2}$$

$$L = 5 * 35^2/200(\sqrt{1.05} + \sqrt{0.26})^2$$

L=13m

S is the required stopping sight distance = 35 m

A is the grade = 5% for a flat terrain according to the ministry of works and transport road design manual

Assume that S > L

$$L_{min} = \frac{2s - 8(\frac{h_1 + h_2}{2})}{A}$$

$$l = \frac{(2 * 35) - 8(\frac{1.05 + 0.26)}{2}}{5}$$

L=13m

4.4.4 Vertical sag curve design and sight distance requirements

Based on clearance from structures during day time

When S ≤ L

$$L_{min} = \frac{AS^2}{\left[8D - 8\frac{(h_1 - h_2)}{2}\right]}$$

$$lmin = \frac{5 * 35^2}{8 * 5.7 - 8(\frac{1.05 - 0.26)}{2}}$$

Lmin=144m

When S > L

$$L_{min} = \frac{\left[2S - 8(\frac{h_1 + h_2}{2})\right]}{A}$$

L=13m

Based on night time conditions

When S ≤ L

$$L_{night} = \frac{AS^2}{200(h_3 + S\,tan\alpha)}$$

$$L_{night} = \frac{5 * 35^2}{200(0.6 + 35\tan 1.0)}$$

$$L = 25m$$

When S > L

$$L_{night} = 2S - \frac{[200(h_3 + S\,tan\tan\alpha)]}{A}$$

$$L_{night} = 2 * 35 - \frac{200(0.6 + 35\tan 1)}{5}$$

L night=22m

Based on motorist comfort

$$Minimum\ length\ of\ the\ vertical\ sag\ curve\ L_{min} = \frac{AV^2}{390}$$

$$L_{min=\frac{5*50^2}{390}}$$

$$lmin = 32m$$

4.5 DRAWING

4.5.1 DRAWINGS

The following are the AUTOCAD Civil3d drawings for the newly designed Hamu Mukasa road.

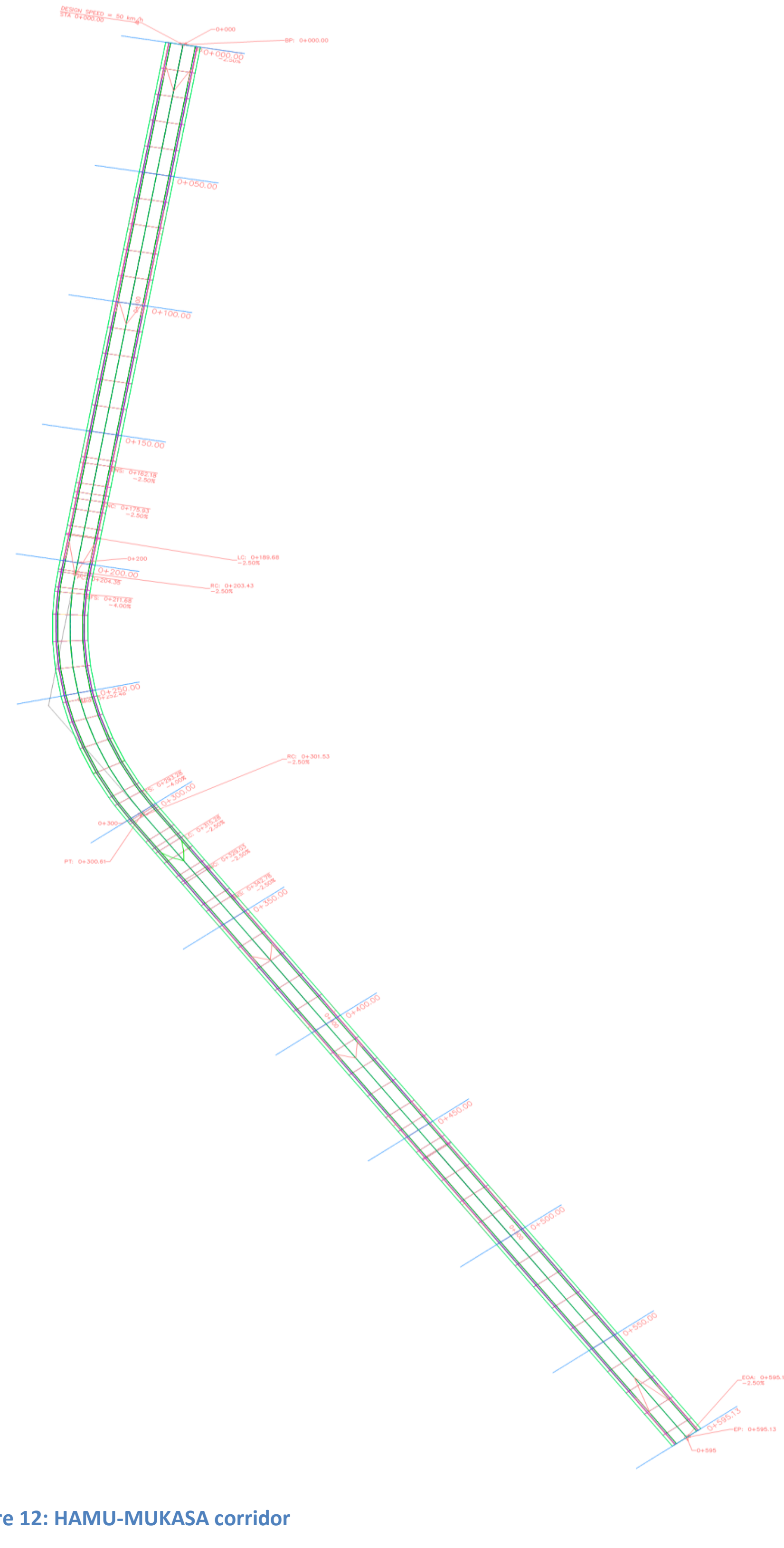


Figure 12: HAMU-MUKASA corridor

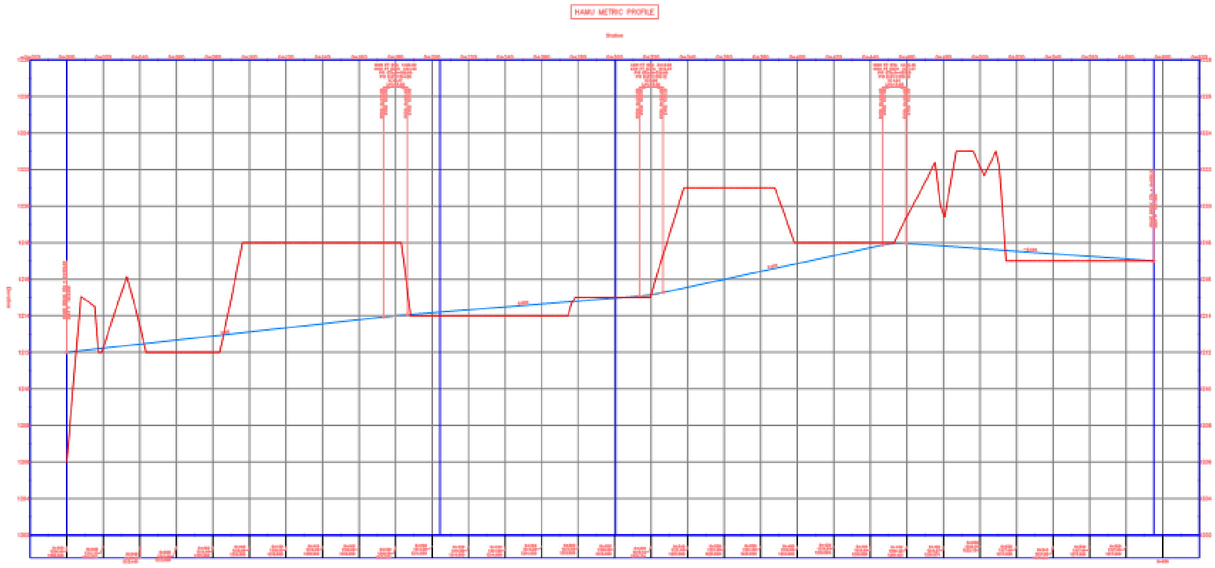


Figure 13: Profile of HAMU- MUKASA road.

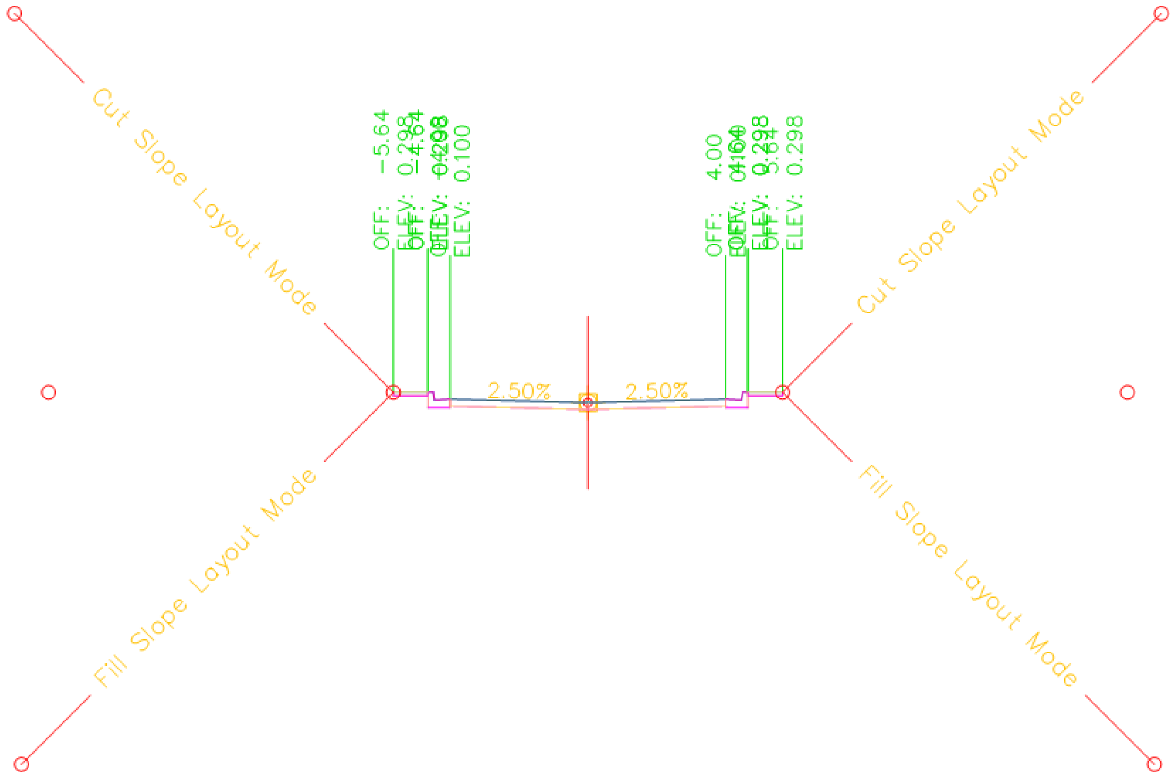


Figure 14: Road assembly

The tabulations of the mass haul values for the cats and fills at each station of the designed Hamu Mukasa road are provided in the appendices

5 CHAPTER FOUR: CHALLENGES CONCLUSIONS AND RECOMMENDATIONS

5.0 INTRODUCTION

In the following chapter, the key design procedures are highlighted, and the conclusions and recommendations are derived based on them. The challenges are discussed first then the conclusions are discussed next. The recommendations are discussed last. The conclusions have been derived in response to the main and specific objectives, and in trying to answer the key research questions that the project was aimed at answering.

5.1 CHALLENGES

There were several challenges experienced during the execution of the project, these are discussed below;

The time frame designated for the entire project which is less than four months, and the data required for the precise design process require much time for analysis, combining the limited time with other school demands, it was really tiresome during the project period

Data analysis process involved expenses as it required transportation of the material to the laboratory in addition to paying some of the laboratory assistants for their help towards us.

During the traffic counts, it required to wake up very early in the morning and count multitudes of cars all day long while standing in sunshine, this made the work more laborious for the design team.

The methods adopted for the topographic studies i.e. the use of google earth to get survey points required extra efforts since it was not familiar.

5.2 CONCLUSIONS

Basing on the fact there is no designated right of way for the proposed road, lane widening for the redesigned road is a constraint in a way that as it will require land acquisition that is expensive and economically not viable basing on the use or service of the road in relation to displacing people, the best way to go is to do away with lane widening and utilize the available road area.
The entire project has been more than educative to the team members as a lot has been learnt as far as pavement design is concerned.

RECOMENDATIONS

Despite the fact that the minimum design speed of an urban area according the Ugandan road design manual 2010 being 50 km/hr, we recommend a consideration of not more than 30 km/hr due to the functionality of the road i.e. it serves the university, schools and institutes, it's located in a very active business area and it has a sharp corner.

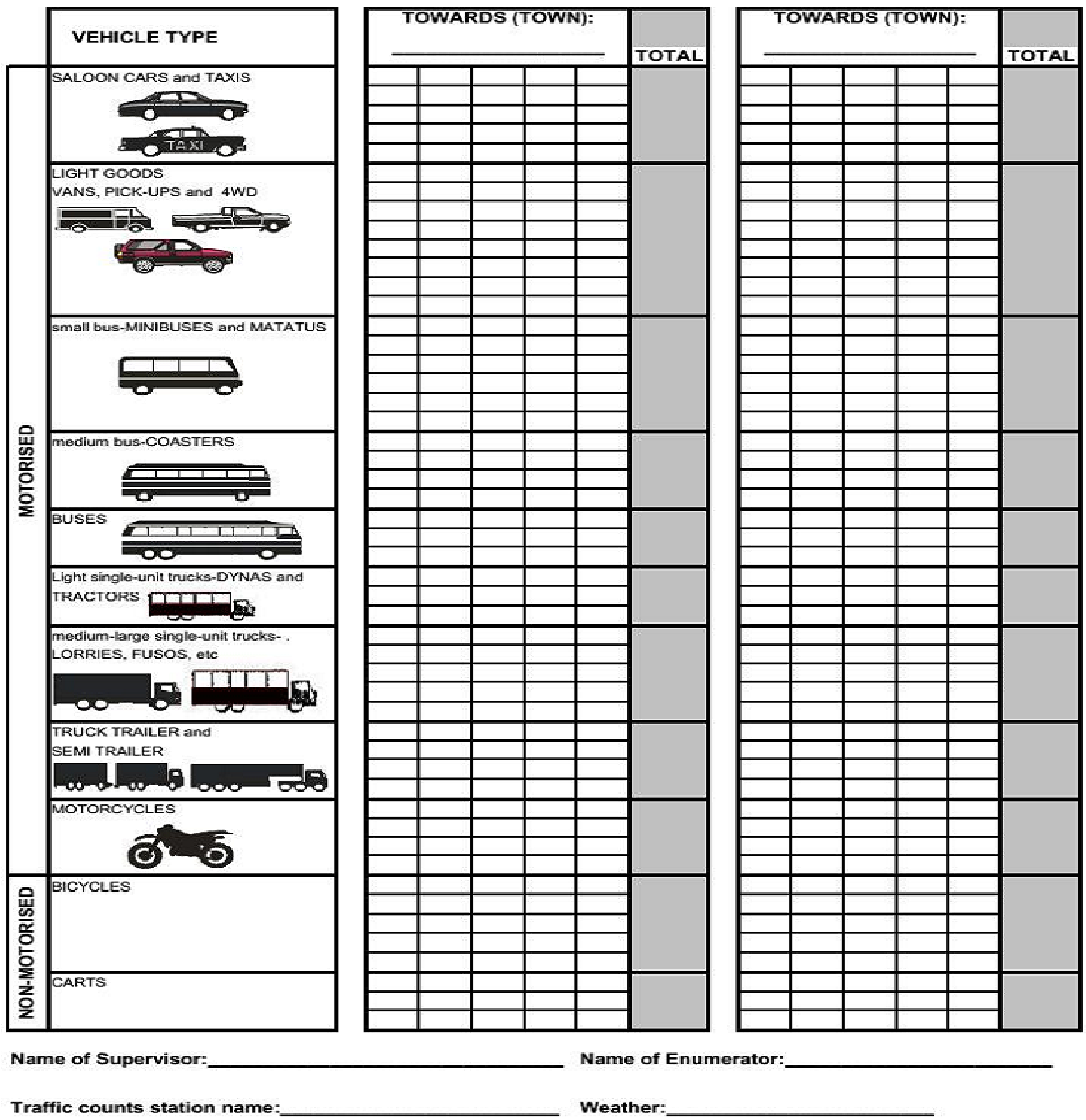
Also basing on the calculated full overtaking sight distance (250 m), we recommend that no overtaking should be done at any section since there is no tangent that reaches such a distance.
A drainage system should be designed to cater for the surface water that may cause early failure of the pavement structure before the design life.

APPENDIX 1 : TRAFFIC COUNT SHEET

TITLE: REDESIGNING OF A SECTION (0+00 TO 0+120) OF HAMU-MUKASA ROAD FROM SIR ALBERT COOK ROAD, MENGO MARKET JUNCTION TO CANON APOLLO KIVEBULAYA ROAD.

TRAFFIC COUNT SHEET

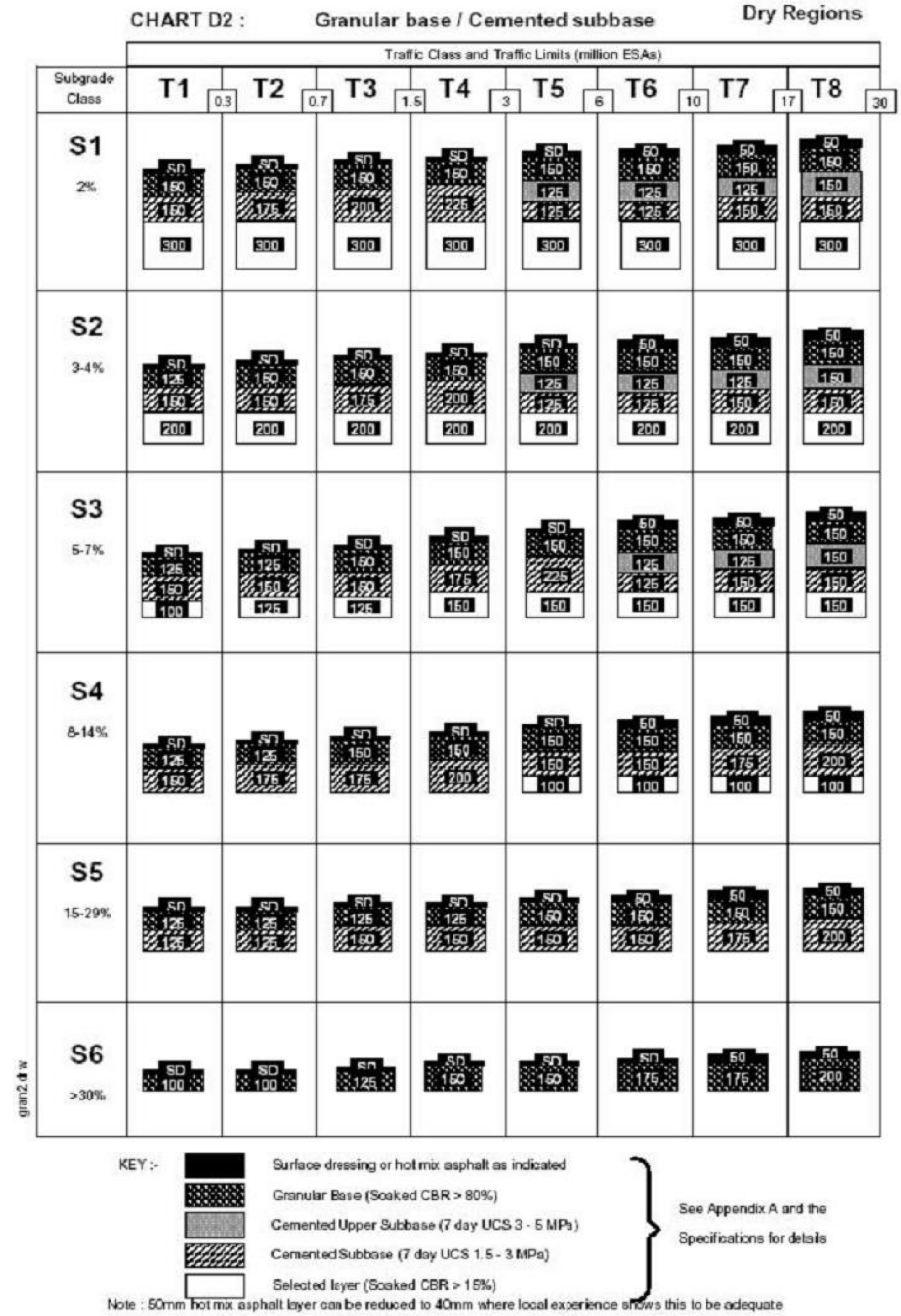
TIME: (FROM…………… TO………………..)



	VEHICLE TYPE	TOWARDS (TOWN): ________	TOTAL	TOWARDS (TOWN): ________	TOTAL
MOTORISED	SALOON CARS and TAXIS				
	LIGHT GOODS VANS, PICK-UPS and 4WD				
	small bus-MINIBUSES and MATATUS				
	medium bus-COASTERS				
	BUSES				
	Light single-unit trucks-DYNAS and TRACTORS				
	medium-large single-unit trucks- , LORRIES, FUSOS, etc				
	TRUCK TRAILER and SEMI TRAILER				
	MOTORCYCLES				
NON-MOTORISED	BICYCLES				
	CARTS				

Name of Supervisor:____________________ Name of Enumerator:____________________

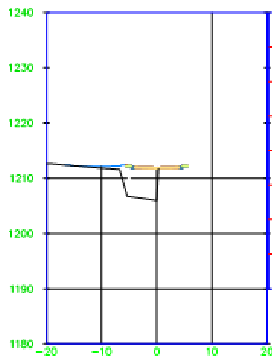
Traffic counts station name:____________________ Weather:____________________



SA TCC Code of Practice for the Design of Road Pavements/C2

0APPENDIX 6 : MASS HAUL VOLUMES FOR CUTS AND FILLS

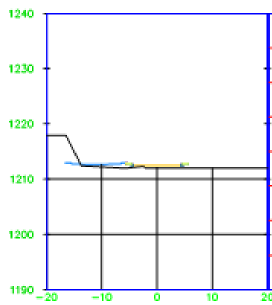
0+000.00



Total Volume at Station 0+000.00	
Cut Area	0.07
Fill Area	0.00
Cut Vol	0.00
Fill Vol	0.00
Cum Cut Vol	0.00
Cum Fill Vol	0.00
Net Vol	0.00

Material(s) at Station 0+000.00			
Material Name	Area	Volume	Cumulative Volume
Pavement	1.60	0.00	0.00
Base	0.38	0.00	0.00
SubBase	0.20	0.00	0.00
Binder	1.60	0.00	0.00

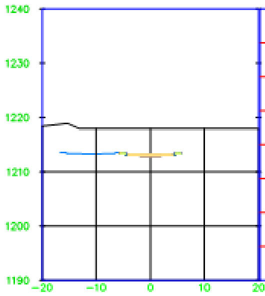
0+050.00



Total Volume at Station 0+050.00	
Cut Area	0.07
Fill Area	0.00
Cut Vol	3.56
Fill Vol	0.00
Cum Cut Vol	3.56
Cum Fill Vol	0.00
Net Vol	3.56

Material(s) at Station 0+050.00			
Material Name	Area	Volume	Cumulative Volume
Pavement	1.60	80.00	80.00
Base	0.38	18.87	18.87
SubBase	0.20	9.99	9.99
Binder	1.60	80.00	80.00

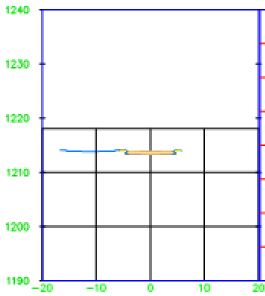
0+100.00



Total Volume at Station 0+100.00	
Cut Area	0.07
Fill Area	0.00
Cut Vol	3.56
Fill Vol	0.00
Cum Cut Vol	7.12
Cum Fill Vol	0.00
Net Vol	7.12

Material(s) at Station 0+100.00			
Material Name	Area	Volume	Cumulative Volume
Pavement	1.60	80.00	160.00
Base	0.38	18.87	37.73
SubBase	0.20	9.99	19.98
Binder	1.60	80.00	160.00

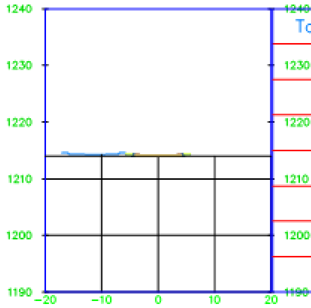
0+150.00



Total Volume at Station 0+150.00	
Cut Area	0.07
Fill Area	0.00
Cut Vol	3.56
Fill Vol	0.00
Cum Cut Vol	10.68
Cum Fill Vol	0.00
Net Vol	10.68

Material(s) at Station 0+150.00			
Material Name	Area	Volume	Cumulative Volume
Pavement	1.60	80.00	240.00
Base	0.38	18.87	56.60
SubBase	0.20	9.99	29.97
Binder	1.60	80.00	240.00

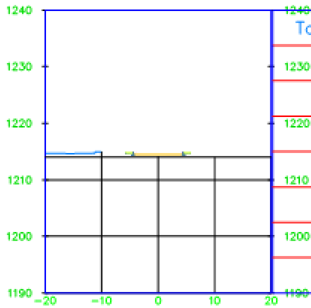
0+200.00



Total Volume at Station 0+200.00	
Cut Area	0.00
Fill Area	0.00
Cut Vol	1.78
Fill Vol	0.00
Cum Cut Vol	12.46
Cum Fill Vol	0.00
Net Vol	12.46

Material(s) at Station 0+200.00			
Material Name	Area	Volume	Cumulative Volume
Pavement	1.60	80.00	320.00
Base	0.38	18.87	75.47
SubBase	0.20	9.99	39.96
Binder	1.60	80.00	320.00

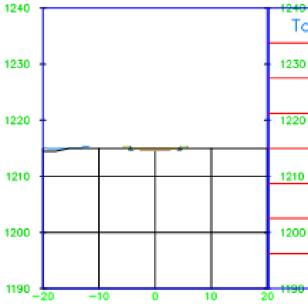
0+250.00



Total Volume at Station 0+250.00	
Cut Area	0.00
Fill Area	0.00
Cut Vol	0.00
Fill Vol	0.00
Cum Cut Vol	12.46
Cum Fill Vol	0.00
Net Vol	12.46

Material(s) at Station 0+250.00			
Material Name	Area	Volume	Cumulative Volume
Pavement	1.60	80.00	400.00
Base	0.38	18.87	94.34
SubBase	0.20	9.99	49.95
Binder	1.60	80.00	400.00

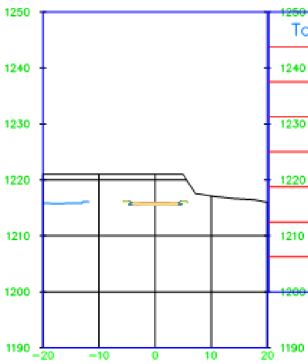
0+300.00



Total Volume at Station 0+300.00	
Cut Area	0.00
Fill Area	0.00
Cut Vol	0.00
Fill Vol	0.00
Cum Cut Vol	12.46
Cum Fill Vol	0.00
Net Vol	12.46

Material(s) at Station 0+300.00			
Material Name	Area	Volume	Cumulative Volume
Pavement	1.60	80.00	480.00
Base	0.38	18.87	113.20
SubBase	0.20	9.99	59.94
Binder	1.60	80.00	480.00

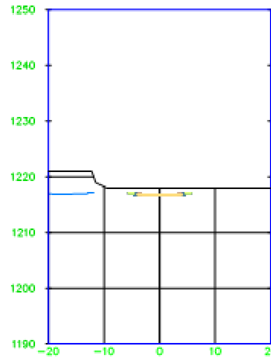
0+350.00



Total Volume at Station 0+350.00	
Cut Area	0.00
Fill Area	0.00
Cut Vol	0.00
Fill Vol	0.00
Cum Cut Vol	12.46
Cum Fill Vol	0.00
Net Vol	12.46

Material(s) at Station 0+350.00			
Material Name	Area	Volume	Cumulative Volume
Pavement	1.60	80.00	560.00
Base	0.38	18.87	132.07
SubBase	0.20	9.99	69.93
Binder	1.60	80.00	560.00

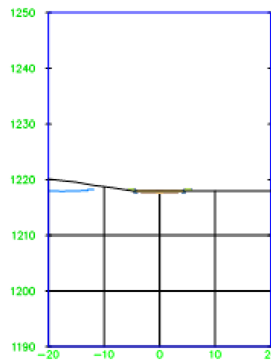
0+400.00



Total Volume at Station 0+400.00	
Cut Area	0.00
Fill Area	0.00
Cut Vol	0.00
Fill Vol	0.00
Cum Cut Vol	12.46
Cum Fill Vol	0.00
Net Vol	12.46

Material(s) at Station 0+400.00			
Material Name	Area	Volume	Cumulative Volume
Pavement	1.60	80.00	640.00
Base	0.38	18.87	150.94
SubBase	0.20	9.99	79.92
Binder	1.60	80.00	640.00

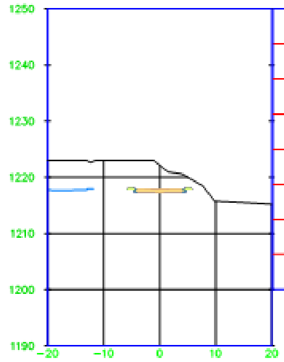
0+450.00



Total Volume at Station 0+450.00	
Cut Area	0.00
Fill Area	0.00
Cut Vol	0.00
Fill Vol	0.00
Cum Cut Vol	12.46
Cum Fill Vol	0.00
Net Vol	12.46

Material(s) at Station 0+450.00			
Material Name	Area	Volume	Cumulative Volume
Pavement	1.60	80.00	720.00
Base	0.38	18.87	169.81
SubBase	0.20	9.99	89.91
Binder	1.60	80.00	720.00

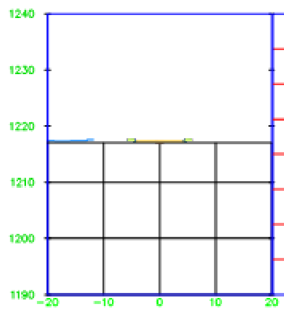
0+500.00



Total Volume at Station 0+500.00	
Cut Area	0.00
Fill Area	0.00
Cut Vol	0.00
Fill Vol	0.00
Cum Cut Vol	12.46
Cum Fill Vol	0.00
Net Vol	12.46

Material(s) at Station 0+500.00			
Material Name	Area	Volume	Cumulative Volume
Pavement	1.60	80.00	800.00
Base	0.38	18.87	188.67
SubBase	0.20	9.99	99.90
Binder	1.60	80.00	800.00

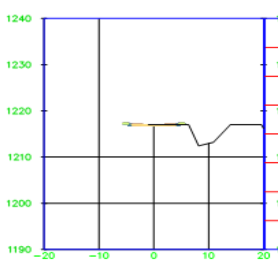
0+550.00



Total Volume at Station 0+550.00	
Cut Area	0.00
Fill Area	0.00
Cut Vol	0.00
Fill Vol	0.00
Cum Cut Vol	12.46
Cum Fill Vol	0.00
Net Vol	12.46

Material(s) at Station 0+550.00			
Material Name	Area	Volume	Cumulative Volume
Pavement	1.60	80.00	880.00
Base	0.38	18.87	207.54
SubBase	0.20	9.99	109.89
Binder	1.60	80.00	880.00

0+595.13



Total Volume at Station 0+595.13	
Cut Area	0.00
Fill Area	0.00
Cut Vol	0.00
Fill Vol	0.00
Cum Cut Vol	12.46
Cum Fill Vol	0.00
Net Vol	12.46

Material(s) at Station 0+595.13			
Material Name	Area	Volume	Cumulative Volume
Pavement	1.60	72.21	952.21
Base	0.38	17.03	224.57
SubBase	0.20	9.02	118.91
Binder	1.60	72.21	952.21