Increased Application of Labour-Based Methods through Appropriate Engineering Standards

Uganda Country Report

TRL
DFID Department for International Development
ILO

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Abbreviations

ADT	–	Average Daily Traffic
CBR	–	California Bearing Ratio
DFID	–	Department for International Development
DFR	–	Department of Feeder Roads
GL	–	Gravel Loss
GM	–	Grading Modulus
HDM	–	Highway Design Model
ILO	–	International Labour Organization
ILO/ASIST	–	International Labour Organization/Advisory Support, Information Services and Training
IRI	–	International Roughness Index
PP	–	Plasticity Product
TMI	–	Thornthwaite's Moisture Index
TRL	–	Transport Research Laboratory, UK
vpd	–	vehicles per day

Executive Summary

Road access to health centres, schools, jobs, etc. is an important factor in the social and economic development of rural communities in Africa. Most roads providing access to small towns and villages tend to be unsealed and constructed of earth or gravel. Climatic and environmental influences can be dominant factors in the deterioration of these roads and their life-time performance is also influenced by factors such as terrain and construction materials, as well as traffic. Access, through reduced trafficability and passability, is often severely curtailed in the wet season. With unpaved roads typically comprising 70-80% of road networks in Africa, the investment in these roads represent a considerable proportion of the asset value of the total road network.

Construction and maintenance of many low trafficked roads are carried out using local resources supported by light equipment. The use of labour-based methods of work is one such initiative that is widely applied to improve these roads. This approach fulfills two objectives by delivering access through improved road networks and promoting the increased use of local resources, thus contributing to the creation of much needed employment in the process.

Construction costs alone often dominate the appraisal process for the provision of these roads, with items such as haulage distance being an important factor. Many roads are constructed using labour-based technology which can further restrict haulage and access to good road building material. The consequences in qualitative terms, from the use of inferior materials such as ravelling or slipperiness are well known but the impact of their use, together with environmental and other factors, on total costs over the "life" of the road and the implications for investment in these roads is less well known.

In this project, an attempt has been made to quantify the effects of these parameters on rates of deterioration in order to give some guidance on standards and the impact on total costs. The study is also being carried out in other African countries to increase the range of the measured parameters and enable a life-cycle methodology to be developed.

Extensive desk and field studies were carried out to select sites that covered the range of parameters (materials, terrain, climate, etc.) typically found in Uganda. The sites were monitored over a period of three years to determine traffic, gravel loss, changes in road roughness and visually inspected to record any other factors affecting road performance. The results from the research have enabled revised specifications for earth and gravel roads to be recommended and a methodology developed for the estimation of life cycle costs.

The main conclusions of the study were:

- Different rates of gravel loss relating to material plasticity (I_p) and traffic were observed. The lowest rates recorded (approx 10 mm per year) were on materials with the higher values of plasticity (I_p) and traffic less than 20 vpd. Higher values (up to 30 mm per year) were recorded for materials with a lower I_p and carrying 100 vpd.
- The average gravel loss for all the sections was 20% higher than the predicted values in HDM.

- The results indicated that a wider grading envelope could be adopted for the wearing course with Grading Modulus (GM) in the range 1.56 < GM <2.40. This will enable finer materials to be used.
- Roughness values were generally approximately 20% lower than predicted by HDM.

The research has increased the range of materials that are suitable for use in the wearing course of gravel roads, thus making materials more readily available and reducing the difficulty that is increasingly faced by practitioners in finding suitable material locally. It will also reduce costs to government through reduced haulage and increase the length of improved unpaved road network for the same investment. This in turn promotes the use of local resources, increases the application of labour-based methods of work, creating the much-needed employment opportunities to communities in the area.

A life-cycle cost methodology has been developed which will enable the frequency of maintenance interventions to be estimated. Additional results from the research carried out in Zimbabwe and Ghana will be combined in the Regional Report and enable life-cycle costs to be calculated for a wider range of gravel roads than those covered in the Uganda component alone.

1. Introduction

1.1 Background

One of the main factors which affects the performance of all types of road, including very low-volume roads, is the standard to which they have been designed and constructed. For more highly trafficked paved and gravel roads, performance-based deterioration relationships have been derived from research. These models assist in predicting the rates of deterioration for different types of road, help to ensure that roads are designed and built to appropriate standards and that total life-cycle costs are optimised.

Far less quantitative information is available on the engineering performance and modes of deterioration of low-volume earth and gravel roads. These roads are often constructed by labour-based methods using quite different construction techniques and lighter equipment than is used on projects constructed by conventional methods. Deterioration due to environmental and climatic effects on these roads can be greater than the effects of traffic. This is the important difference between these and more highly trafficked roads. Without deterioration relationships for these roads, it is difficult to set appropriate standards or to know the effect of different standards on performance. This means that the expected level of maintenance is also uncertain and whole-life costs and benefits almost impossible to determine.

Therefore, quantitative information on the modes of deterioration is required for different types of very low-volume roads so that appropriate engineering standards can be set, methods to monitor compliance with standards developed and procedures determined that enable total life-cycle costs to be calculated.

1.2 Project objectives

The project goal is to promote sustainable livelihoods and contribute to the socio-economic development of disadvantaged rural populations through the provision of improved road access.

The purpose of the project is to reduce the life-time costs of unpaved rural roads by promoting appropriate engineering standards, planning tools and works procedures for labour-based construction and maintenance.

This project has been carried out in partnership with the International Labour Organization/Advisory Support, Information Services and Training (ILO/ASIST).

1.3 Outputs

The main outputs of the project are:

a) Deterioration relationships established for low-volume unpaved roads.
b) Methodologies developed and documented for determining life-cycle costs of labour-based roads.
c) Appropriate engineering standards developed and guidelines produced for different categories of labour-based roads in different environments.
d) Appropriate methods established and guidelines produced for quality approval of labour-based construction and maintenance works.
e) Results disseminated to training institutions, relevant ministries and small-scale contractors' associations.

The outputs of the project will contribute to increasing awareness by road authorities and other stakeholders, such as policy- and decision-makers, communities, professional bodies, etc. of the potential benefits of using optimised labour-based road technology, and increase the applicability of local resource use.

1.4 Reports

This report covers activities in Uganda. These activities include the selection of test sites that are typical of labour-based roads in Uganda, monitoring and evaluating their performance and estimating their life-cycle costs.

Separate country reports have been produced on similar studies carried out by the TRL/ILO project team in Ghana and Zimbabwe. These two reports focus on the activities in their respective countries.

A Regional Report will be produced by end of 2005 which combines the results from Ghana, Uganda and Zimbabwe.

A report has also been produced giving guidelines on the general methodology used in the selection of test sites and monitoring their performance (see Test Site Selection, Commissioning and Monitoring report). Reference to the guidelines report is made throughout this document, which focuses on the collection and analysis of data from the test sites in Uganda.

Another report has been produced as a field manual which describes the assessment of road works activities associated with labour-based roads (see Guidelines for Quality Assurance Procedures for Road Works Executed Using Labour-Based Methods report). An appendix in the manual includes reference to quality assurance practice in Uganda.

2. Test Sites

2.1 Selection

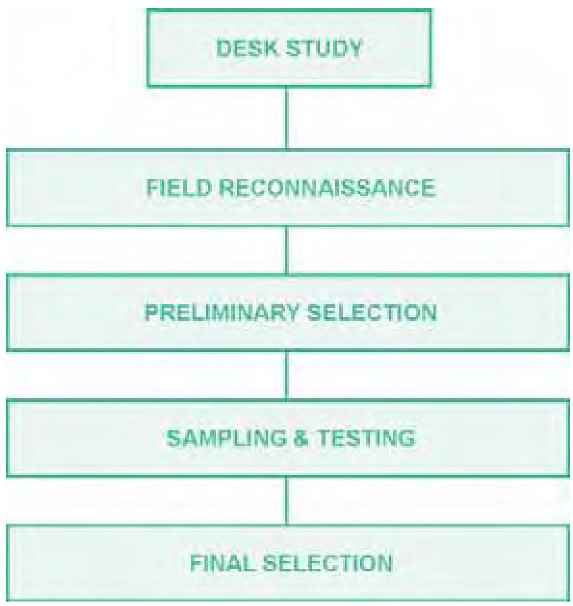
One of the main objectives of this project was to determine the rate of deterioration of gravel roads constructed by labour-based techniques to enable future predictions to be made on the performance of these types of road. In order to monitor the performance of these roads, test sites were selected that covered a wide spectrum of factors, primarily traffic, construction materials, terrain and climate. Site selection was therefore seen to be crucial to enable the study to achieve this aim and the sites were selected to obtain a wide range of data available in the country.

It is recognised that within a country, the ranges of these variables may be limited. Similar studies have been carried out by the TRL/ILO project team in Ghana and Zimbabwe. Combining data from these countries will expand the ranges of the variables. Analysis of the combined data will be reported under the Regional component of the TRL/ILO labour-based suite of projects, with this report focusing only on the Uganda data.

The site selection approach adapted in this study is shown in Figure 2.1.

A more detailed explanation of the processes involved can be found in the Test Site Selection, Commissioning and Monitoring report.

Figure 2.1
Test site selection approach



2.2 Desk study

The desk study included a review of the location of gravel roads that had been constructed by labour-based techniques, and identifying the climate in which they are located.

Uganda's climate is tropical; generally rainy with two dry seasons and is semi-arid in north eastern regions. Thornthwaite's Moisture Index (TMI) was considered a suitable measure to indicate climate in a region. Climatic boundaries in terms of TMI have been classified as shown in Table 2.1.

Table 2.1: Classification based on TMI contours

TMI Range	Climate Classification
> +20	Humid
+20 to 0	Moist sub-humid
0 to -20	Dry sub-humid
-20 to -40	Semi-arid
< -40	Arid

2.3 Field reconnaissance

A series of field reconnaissance visits was undertaken to provide basic information on the material used on road projects, local terrain and climate in the different regions. The aim was to include roads constructed with different materials within each of the climatic or terrain zones. Soil samples were collected from these roads to determine their classification, plasticity and grading.

2.4 Final selection of test sites

The final list of test sites was drawn up based on the desk study, field visits and results from the materials tests. As traffic is also an influential parameter on the performance of roads, the selected sites covered a range of traffic flows. Other information considered during the final site selection process included age of the road and maintenance history. A total of 8 sites were finally selected, as shown in Table 2.2.

For easy referencing, these sites were referred to by a code consisting of four letters, the first and last letter of the two places connected by the road. For example the test site on the **K**isor**O**-**M**uganz**A** road was referred to as KOMA.

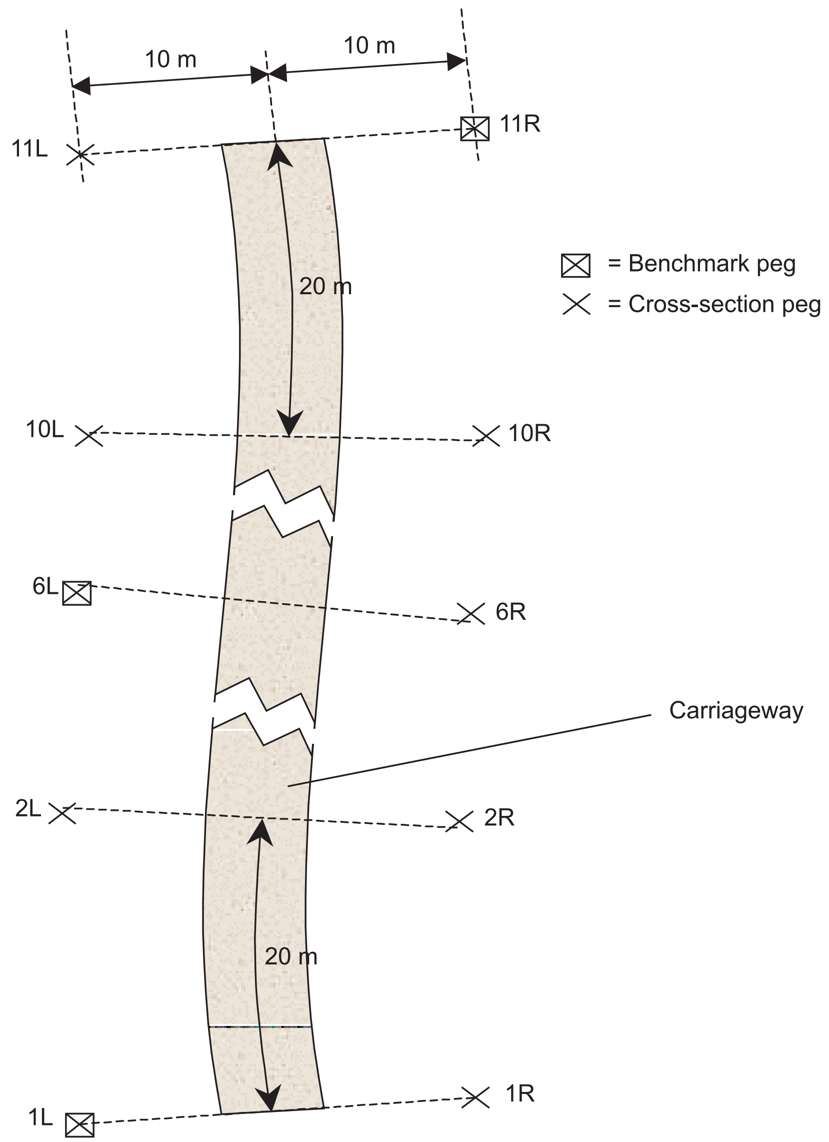
2.5 Site commissioning

The sites were commissioned by establishing steel pegs at 20 m intervals on both sides of the road over the 200 m length of the site. These steel pegs were then used as fixed references for measuring gravel loss. The installation of the steel pegs is described in detail in the Test Site Selection, Commissioning and Monitoring report. Figure 2.2 shows the typical layout of the pegs.

Table 2.2: Test sites selected for monitoring

No.	Road Name	Site Code	Climate	Material
1	Kisanja – Paraa	KAPA	Semi-arid	Brown clayey gravel
2	Kisoro – Muganza	KOMA	Wet humid to sub-tropical	Dark brown sandy clay
3	Muhokya – Mahango	MAMO	Wet humid to sub-tropical	Dark grey volcanic ash
4	Mparo – Kibugubya	MOKA	Semi-arid	Dark grey clayey soil
5	Molo – Kidoko	MOKO	Wet humid to sub-tropical	Brown quarzitic gravel
6	Nakaloke – Kabwangasi	NEKI	Wet humid to sub-tropical	Black cotton soil
7	Nankusi – Bumudu	NIBU	Wet humid to sub-tropical	Dark red clayey gravel
8	Rubona – Kabuzige	RAKE	Wet humid to sub-tropical	Brown sandy clayey gravel

Figure 2.2
Plan view of peg layout on site



After concreting the pegs, they were surveyed with a rod and level to establish their relative positions in relation to the benchmarks. Surveys of the change in the road profile were taken between the benchmarks.

3. Test Site Details

3.1 Road alignment

The gradient of each site was measured using a rod and level and these are listed in Table 3.1. Also listed in Table 3.1 is the terrain in which the sites were located. The terrain refers to the surrounding land in the immediate vicinity of the road and it should be noted that even in mountainous terrain it is possible to have a section of road where the gradient is flat.

Table 3.1: Road alignment

Site	Terrain	Gradient (m/km)
KAPA	Flat	19.2
KOMA	Rolling	20.0
MAMO	Mountainous	47.8
MOKA	Rolling	30.7
MOKO	Rolling	11.9
NEKI	Flat	19.4
NIBU	Mountainous	33.3
RAKE	Mountainous	35.9

Note: *Flat: 0 – 10 five-metre ground contours per kilometre*
Rolling: 11 – 25 five-metre ground contours per kilometre
Mountainous: > 25 five-metre ground contours per kilometre

3.2 Traffic

Classified traffic counts were carried out on the test sites using proformas as shown in Appendix C. Traffic surveys were conducted over 7 consecutive days for a period of 12 hours each day (0600 to 1800 hrs). The 12-hour counts were adjusted to 24-hour counts, based on the estimate that a further 10% of traffic travelled on these rural roads during the period 1800 to 0600 hrs.

The estimated 24-hour traffic volumes on each site are listed in Table 3.2. The traffic volumes on the sites ranged from 4 vehicles per day (vpd) to over 200 vpd, with the Average Daily Traffic (ADT) for all the sites being 57 vpd.

Table 3.2: 24-hour traffic volumes

Test Site	Light Vehicles (ADL)	Heavy Vehicles (ADH)	Total Vehicles (ADT)	Motorbikes
KAPA	73	13	86	114
KOMA	8	3	11	44
MAMO	4	0	4	14
MOKA	10	5	15	66
MOKO	13	9	22	8
NEKI	73	14	87	37
NIBU	188	38	226	33
RAKE	10	3	13	11

Notes: *ADL – Cars, Light Goods, Minibuses*
ADH – Trucks, Buses, Tractors

3.3 Rainfall

Data from the rainfall stations located nearest each test site were collected from the meteorological office and assigned as the rainfall for that site. Rainfall data were collated for the period 1999 to 2003 inclusive, which covered the monitoring period of this project. The average monthly and annual rainfall over this 5-year period are listed in Table 3.3 for each site.

Table 3.3: Rainfall

Month	Rainfall (mm)												
	Jan	Feb	Mar	Apr	May	June	July	Aug	Sep	Oct	Nov	Dec	Annual
KAPA	13	27	127	184	172	57	99	160	128	222	151	56	1396
KOMA	113	85	119	116	77	34	34	57	66	150	135	101	1087
MAMO	14	24	88	149	90	51	35	68	57	116	104	58	854
MOKA	14	4	69	125	105	37	35	80	97	139	140	29	874
MOKO	74	64	105	190	175	94	76	91	105	152	180	108	1415
NIBU	53	26	31	122	90	53	83	42	50	79	77	25	761
NEKI	53	26	31	122	90	53	83	42	50	79	77	25	761
RAKE	80	80	85	131	87	15	20	52	108	144	118	74	994

3.4 Material properties

Samples of the gravel wearing course and the subgrade were taken for material testing from the centre of the carriageway at locations that were immediately adjacent to each of the 200 m sites. Tests carried out on the samples included grading analysis, Atterberg and shrinkage limits.

3.4.1 Gravel wearing course

Grading results obtained for the samples of the gravel wearing course are shown in Table 3.4. A plot of the grading curves from the sites is illustrated in Figure 3.1.

The grading curves are an indication of well-graded wearing courses, with the exception of two sites, KOMA and MOKA, which were gap graded on the fine fraction. The fine fraction is an important parameter which has a significant influence on the performance of the wearing course.

The grading envelope encompassing the grading curves from all the sites is illustrated in Figure 3.2. The grading envelope is much tighter on the coarse fraction and wider on the fine fraction. This grading envelope is generally narrow and may well be a result of the small sample of sites. Material that falls on the lower bound

Table 3.4: Grading of gravel wearing courses

Site	Percentage Passing											
	37.5	26.5	19	13.2	9.5	4.75	2.36	1.18	0.6	0.425	0.15	0.075
KAPA	100	98	95	92	90	69	44	40	34	31	24	23
KOMA	100	100	100	96	89	84	61	36	19	8	5	1
MAMO	92	85	76	72	67	57	43	34	31	30	27	26
MOKA	100	100	100	96	85	58	25	23	21	20	15	13
MOKO	100	100	99	86	78	46	31	28	24	21	14	12
NEKI	100	99	93	89	83	71	61	59	56	54	46	43
NIBU	100	100	98	95	92	84	67	62	55	53	41	37
RAKE	100	100	97	95	88	76	54	44	39	35	31	22

of the grading envelope with nearly 0% passing 0.075 mm is not ideal for a wearing course because internal bonding is compromised due to the absence of clay and fine silt fraction.

The plasticity properties of the wearing course from each site are given in Table 3.5. Other material properties are listed in Table 3.6 and the ranges summarised in Table 3.7.

The formulae used to derive the material properties were as follows:

Coarseness Index = 100 - (% passing 2.36)

Dust Ratio = (% passing 0.075) / (% passing 0.425)

Grading Modulus = [300-(% passing 2.36 + % passing 0.425 + % passing 0.075)] / 100

Grading Coefficient = [(% passing 26.5 - % passing 2.36) x % passing 0.425] / 100

Shrinkage Product = (% passing 0.425) x Linear Shrinkage

Plasticity Modulus = (% passing 0.425) x Plasticity Index

Plasticity Product = (% passing 0.075) x Plasticity Index

Plasticity Factor = (% passing 0.075) x Plastic Limit

Table 3.5: Plasticity properties of the gravel wearing course

Site	Liquid Limit	Plastic Limit	Linear Shrinkage	Plasticity Index
KAPA	55	30	12	25
KOMA	44	0	0	44
MAMO	52	18	15	34
MOKA	50	28	11	22
MOKO	29	11	8	18
NEKI	49	25	11	24
NIBU	45	18	12	27
RAKE	37	20	8	17

Figure 3.1
Particle size distribution of the gravel wearing course

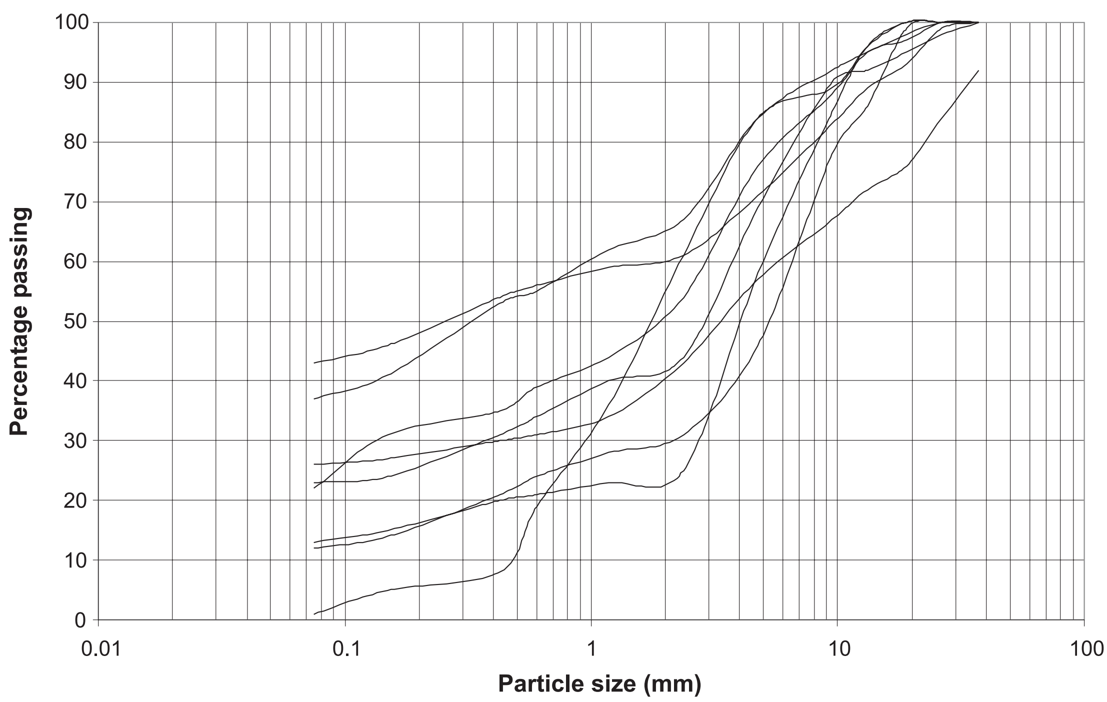


Figure 3.2
Grading envelope for the gravel wearing course

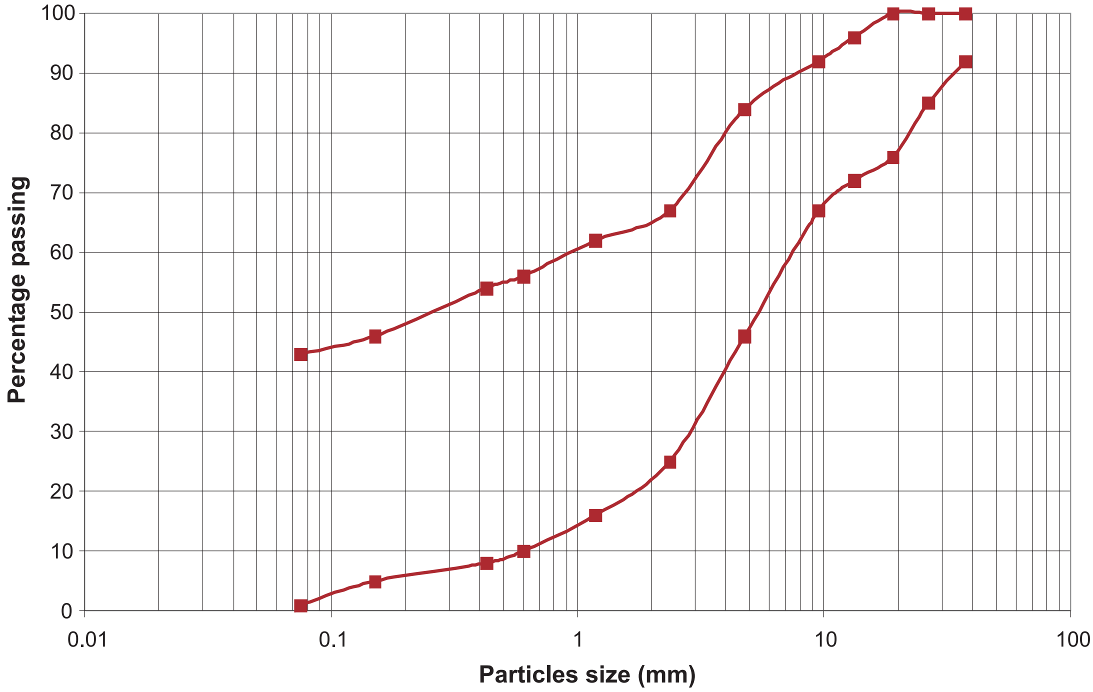


Table 3.6: Material properties of the gravel wearing course

Site	Coarseness Index I_c	Dust Ratio DR	Grading Modulus GM	Grading Coefficient Gc	Shrinkage Product SP	Plasticity Modulus PM	Plasticity Product PP	Plasticity Factor PF
KAPA	56	0.74	2.02	16.7	372	775	575	690
KOMA	39	0.13	2.30	3.1	0	352	44	0
MAMO	57	0.87	2.01	12.6	450	1020	884	468
MOKA	75	0.65	2.42	15.0	228	440	286	364
MOKO	69	0.57	2.36	14.5	168	378	216	132
NEKI	39	0.80	1.42	20.5	594	1296	1032	1075
NIBU	33	0.70	1.43	17.5	636	1431	999	666
RAKE	46	0.63	1.89	16.1	280	595	374	440

Table 3.7: Range of wearing course material properties

Parameter	Measured Range
Reject Index (IR)	0 – 8
Coarseness Index (I_c)	33 – 75
Grading Modulus (GM)	1.42 – 2.42
Grading Coefficient (Gc)	3.1 – 20.5
Liquid Limit (WL)	29 – 55
Plastic Limit (PL)	0 – 30
Plasticity Index (I_p)	17 – 44
Linear Shrinkage (LS)	0 – 15
Shrinkage Product (SP)	0 – 636
Plasticity Product (PP)	44 – 1032
Plasticity Modulus (PM)	352 – 1431

The important parameters to note are the coarseness index, the grading modulus, the plasticity index and the plasticity product, as well as the material type. The coarseness index ranges from medium to high and the grading modulus is indicative of well-graded but slightly coarse material. The plasticity was on the high side, with a maximum of 44 for I_p, which is extremely high. A large range of values of plasticity product from 44 to 1032 were measured.

No deformation resulting from poor strength was noted during the course of the study and it can be assumed that the upper limits of plasticity did not significantly affect performance of the sites, despite being on the high side and generally indicative of many of the roadbuilding materials in Uganda.

3.4.2 Subgrade

Grading results for the subgrade samples are listed in Table 3.8. The range of the other material properties of the subgrade are summarised in Table 3.9. A plot of the grading curves from these sites is illustrated in Figure 3.3.

The samples exhibited a wide range of particle size distribution. The majority were very fine graded materials which is typical of the predominantly clay soils in Uganda. The fineness of the materials depicted by the percentage by weight passing the 0.075 mm sieve is a clear indication of a high content of silt or clay or both.

The material on one site, RAKE, was significantly coarser than the material on the other sites. The grading envelope excluding RAKE is much narrower than the envelope including this site. Both envelopes are shown in Figure 3.4.

Table 3.8: Grading of the subgrade material

Site	Percentage Passing (mm sieve)											
	37.5	26.5	19	13.2	9.5	4.75	2.36	1.18	0.6	0.425	0.15	0.075
KAPA	98	94	93	90	87	82	75	71	68	66	61	58
KOMA	100	93	92	88	86	83	81	79	77	76	74	72
MAMO	100	100	100	100	100	100	100	99	97	94	82	76
MOKA	100	100	100	100	99	95	86	81	77	73	52	46
MOKO	100	100	100	100	99	98	97	94	86	78	51	43
NEKI	100	100	100	99	98	92	87	86	84	81	61	54
NIBU	100	100	100	99	97	95	79	64	58	55	46	44
RAKE	100	100	86	84	71	61	45	31	24	20	17	10

Figure 3.3
Particle size distribution of the subgrade

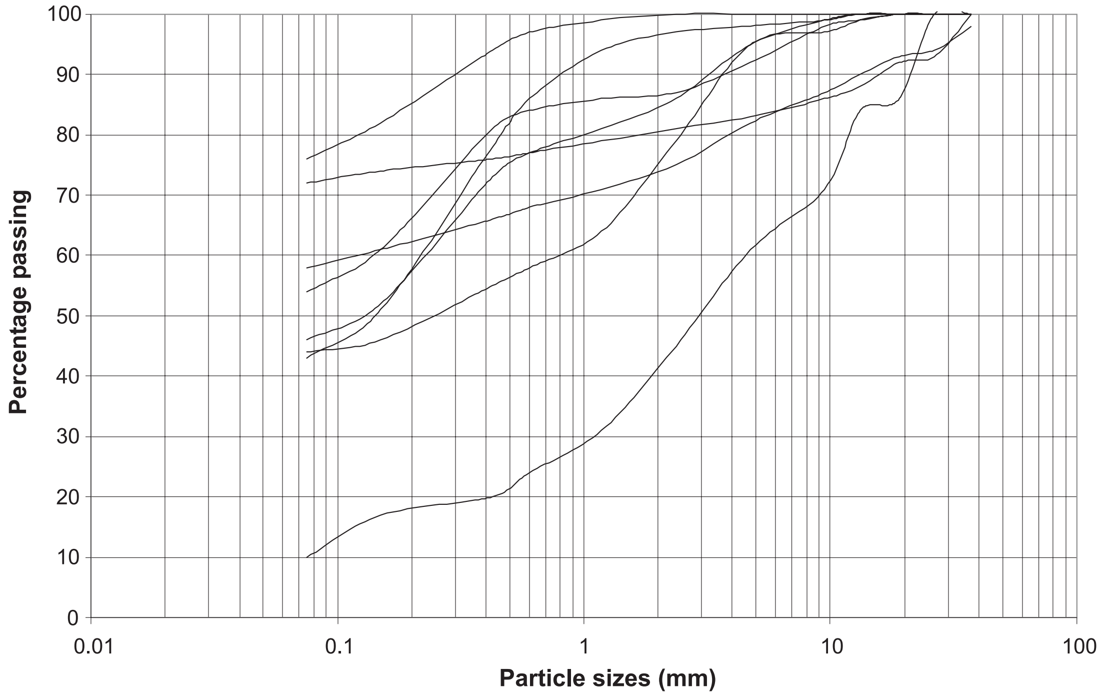


Figure 3.4
Grading envelope for the subgrade

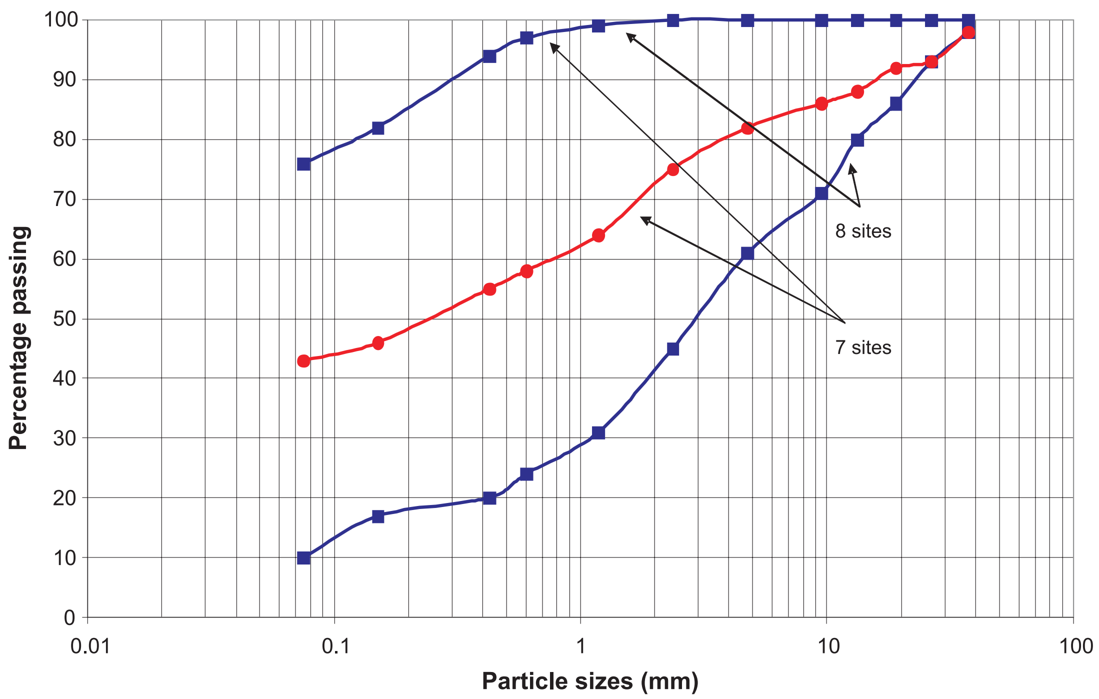


Table 3.9: Material properties of the subgrade

Parameter	Measured Range
Reject Index (I_R)	0 – 2
Coarseness Index (I_c)	0 – 55
Grading Modulus (GM)	0.3 – 2.25
Grading Coefficient (Gc)	0 – 12.5
Liquid Limit (W_L)	40 – 84
Plastic Limit (P_L)	19 – 51
Plasticity Index (I_p)	16 – 33
Linear Shrinkage (LS)	9 – 17
Shrinkage Product (SP)	186 – 1277
Plasticity Product (PP)	160 – 2376
Plasticity Modulus (PM)	320 – 2508

The grading envelopes give an indication of the range of grading properties of the subgrade material. The upper limits may consist of deleterious materials but based on information gathered during the study, it was concluded that the subgrade performed well as no site showed distress related to subgrade failures. It can therefore be concluded that unless traffic is significantly higher than that observed in the study there is little risk in using material that is within the grading envelope.

The important subgrade material properties to note are I_c, GM, I_p and PP. Several samples exhibited extremely high plasticity. The minimum I_p of the samples was 19 which is relatively high and appears to be characteristic of subgrade soils in Uganda.

The high I_p values are an indication that the subgrade materials could be expected to have low in-situ CBRs, perhaps less than 5% when wet. However, the fact that no subgrade related failures were noticed in the study period indicates that the in-service CBRs were sufficient to provide adequate subgrade support. This may be due to the good drainage regime of the sites or to the characteristic of the type of material present or both.

4. Monitoring

4.1 Schedule

Most parts of Uganda experience at least two annual rainfall seasons and in order to assess the seasonal effects, the sites were monitored several times a year. The sites were monitored for a period of 2½ years from April 2002, with most sites being monitored seven times, as shown in Table 4.1.

The following surveys were conducted during each site visit:

a. Gravel loss measurements.
b. Roughness measurements.
c. Visual condition survey.

4.2 Gravel loss

Gravel loss was estimated by monitoring cross-section profiles of the road between each pair of pegs, i.e. every 20 m along the test site. At each cross-section, the spot height was measured at 20 cm intervals (called offsets) across the carriageway using a rod and level. The 20 cm intervals were identified using a measuring tape held tightly across the carriageway between a pair of pegs. The spot heights were then referenced to the benchmark readings. A form for recording the cross-section profile measurements at 20 cm intervals is given in Appendix B.

Before measuring the cross-section profiles, it was important to check whether the pegs had moved, as movement of the pegs would significantly affect the profile and estimated gravel thickness/loss. The height of each peg was therefore checked against the original survey records at the start of each survey and any movement taken into account when comparing the reduced levels between surveys.

The width of the carriageway was determined at each cross-section on a test site and the average of the reduced levels across the defined width was used to estimate the height of the gravel wearing-course at each cross-section. The same defined width at a cross-section was used throughout the monitoring period. The change in the average height of the carriageway between surveys was used as the indicator of the change in gravel loss.

Table 4.1: Monitoring dates

Site	1st	2nd	3rd	4th	5th	6th	7th
KAPA	Aug 02	Oct 02	Apr 03	Sep 03	Apr 04	Jul 04	Nov 04
KOMA	Aug 02	Feb 03	Jul 03	Mar 04	Jul 04	Oct 04	–
MAMO	Aug 02	Dec 02	May 03	Sep 03	Mar 04	Jun 04	Oct 04
MOKA	Aug 02	Oct 02	Apr 03	Sep 03	Apr 04	Jul 04	Nov 04
MOKO	May 02	Sep 02	Mar 03	Aug 03	Apr 04	Jul 04	Nov 04
NEKI	Sep 02	MAR 03	Aug 03	Dec 03	Apr 04	Jul 04	Nov 04
NIBU	May 02	Sep 02	Mar 03	Aug 03	Apr 04	Jul 04	Nov 04
RAKE	Apr 02	Aug 02	Feb 03	Jul 03	Mar 04	Jul 04	Oct 04

The cross-section profiles for each site have been plotted in Appendix E. From these profiles, the carriageway, the invert of the drains, etc. can be readily identified.

4.3 Roughness

Roughness is a measure of the riding quality of the surface and can be measured using a variety of instruments. Whichever instrument is used, it is important that the measurements are standardised in the universally accepted units of International Roughness Index (IRI). A relatively inexpensive roughness measuring device is the Merlin (see Figure 4.1) and it was used to measure roughness on the test sites. The measurements from the Merlin can be standardised to IRI units. The Merlin's operation is detailed in the Test Site Selection, Commissioning and Monitoring report.

The Merlin can be operated in one of two different modes based on the location of the measuring foot shown in Figure 4.2. By changing the position of the foot the magnification factor can be set to either 5:1 (for rough surfaces) or 10:1 (for smooth surfaces), indicating how far the chart pointer moves compared to the measurement probe. For the unsealed labour-based sites, a magnification of 5:1 was used. Prior to use, the Merlin has to be calibrated to correct any discrepancy in the magnification between the probe and the chart pointer.

The number of Merlin measurements along the site (in each wheelpath) should be approximately 200 to ensure that the data are representative of the site. The measurement interval is usually determined by the circumference of the Merlin wheel, i.e. the distance along the ground travelled by one rotation of the wheel, which is approximately 2.1 m. Hence for the 200 m long test sites, it was necessary for a reading to be made every half revolution of the Merlin wheel, which meant that approximately 190 readings were made in each wheelpath.

Figure 4.1
MERLIN roughness measuring device

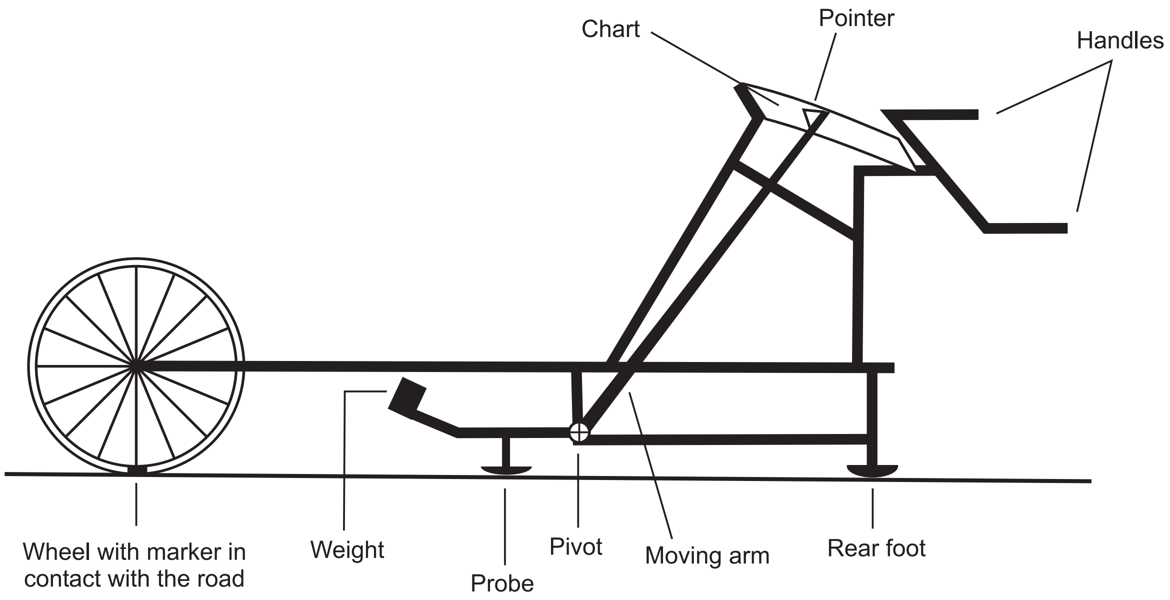
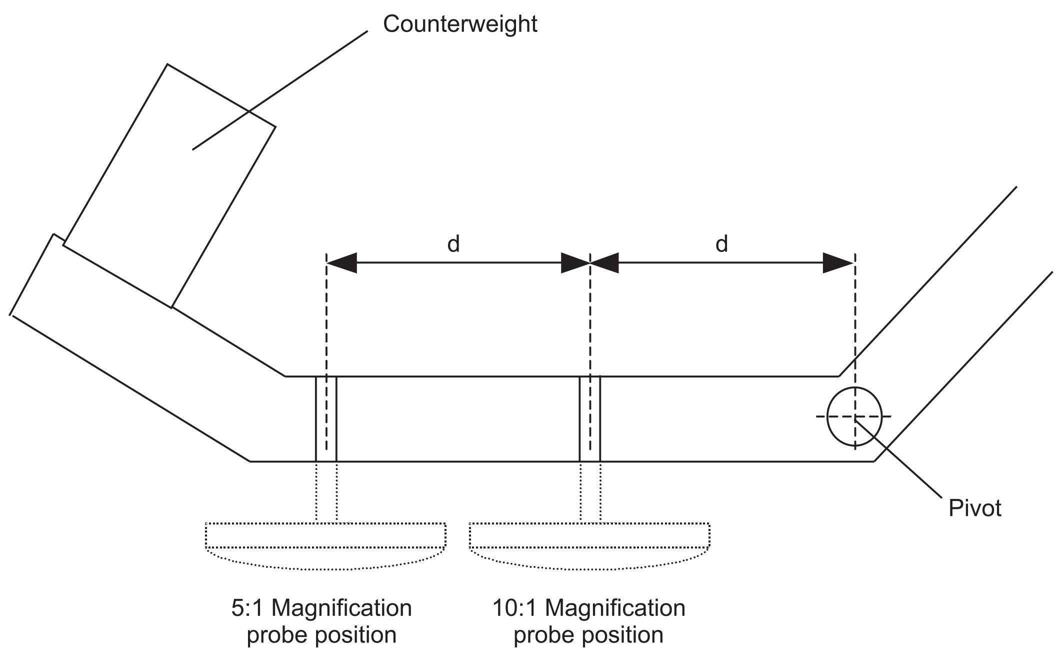


Figure 4.2
Merlin probe assembly



The measure of spread of 90% of the Merlin readings (i.e. 5% of readings from either end of the distribution are ignored) is referred to as 'D'. The roughness, in terms of IRI units, was then evaluated using the relationship:

$$IRI = 0.593 + (0.0471 \times D)$$

4.4 Visual condition survey

Each 200 m test site was divided into 20 m sub-sections with the pegs forming the boundaries. For each 20 m sub-section, the surface condition was recorded on a data sheet, as shown in Appendix A, by a surveyor/technician who walked along the road. The parameters that were recorded are listed in Table 4.2, with the drain and shoulder information collected separately for both the left and right side of the road.

Table 4.2: Visual condition codes

	Parameter	Ranges
Drain	Drainage	Very Good, Good, Average, Poor, Very Poor
	Drain existence	Exists, Not required, Required
	Scouring	None, Slight, Severe
	Blockage	None, Slight, Severe
Shoulder	Side slope condition	No damage, Moderate, Badly Damaged
	Side slope damaged	Area damaged in square metres
	Shoulder condition	No damage, Moderate damage, Severe damage
	Shoulder level	Level or Low, High
Carriageway	Shape	Very Good, Good, Average, Poor, Very Poor, Failed
	Effective width	Length where width has receded by greater than 1 m
	Crown height	As built > 300 mm, 150-300 mm, < 300 mm
	Surface condition	Very Good, Good, Average, Poor, Very Poor
	Ruts	None, < 15 mm, 15 – 30 mm, > 50 mm
	Corrugations	None, < 15 mm, 15 – 30 mm, > 50 mm
	Potholes	None, 1-5, 5-10, >10 per 20 m sub-section
	Loose material	None, < 15 mm, 15 – 30 mm, > 50 mm
	Oversize materials	None, Yes (if 5% of the material > 50 mm)

5. Performance of the Labour-Based Roads

5.1 Gravel loss

5.1.1 Data collation

The gravel loss constituted by far the largest data set. For a typical 200 m site, profile heights were taken at 20 cm intervals over a 20 m cross-sectional width at intervals of 20 m along a site. This equated to over 1100 readings on a site during each survey, which totalled over 60,000 profile heights from the 7 surveys conducted on the 8 sites.

It is inevitable that errors will occur with this quantity of data in either recording of the field measurements, input of data into computer spreadsheets, manipulation of the data to reduced levels for each site, accounting for any peg movements between surveys, etc. It was therefore essential to ascertain which data were appropriate to use in the analysis prior to commencement of any analysis.

In order to 'quality assure' the data, the cross-sectional profiles were plotted for each cross-section on each site. This visual display of the profiles enabled discrepancies and errors to be quickly identified. In many cases, errors could be corrected once the field sheets had been re-examined. Common errors included data being input incorrectly into spreadsheets or field data being recorded in an obviously incorrect manner – usually by increasing a value by 0.1 m rather than decreasing by the same amount, or vice versa.

Plots of the accepted cross-sectional profiles have been illustrated in Appendix E for all the cross-sections on each site. These plots enabled the locations of the carriageway, shoulder, drains, etc. to be clearly identified, enabling gravel loss to be deduced for different widths of the road.

5.1.2 Rates of gravel loss

The height of the road at each cross-section was estimated by taking the average of the readings over the carriageway width at the cross-section. The average height of the site was then determined by taking the average

Table 5.1: Gravel loss between surveys

Site	Carriageway Gravel Loss in mm						
	1st – 2nd Surveys	2nd – 3rd Surveys	3rd – 4th Surveys	4th – 5th Surveys	5th – 6th Surveys	6th – 7th Surveys	Average
KAPA	24.6	42.9	23.5	10.0	33.9	17.2	25.4
KOMA	3.7	7.0	20.2	53.1	-19.2		12.9
MAMO	23.8	23.3	-11.3	18.9	5.2	33.4	15.5
MOKA	53.2	16.3	8.3	-0.4	35.3	14.9	15.7
MOKO	53.2	16.3	8.3	-0.4	35.3	14.9	21.3
NEKI	17.2	25.8	-5.1	12.5	31.1	-6.0	12.6
NIBU	12.0	35.8	8.6	15.3	-3.2	19.0	14.6
RAKE	-4.2	16.1	9.7	3.0	21.4	13.8	10.0

of the 11 cross-sectional heights. The rates of gravel loss on each site were then determined by comparing the average height of the site from each survey. The rates of gravel loss between surveys on all the sites are summarised in Table 5.1.

In Table 5.1, several of the values are negative. The negative values indicate an increase in the height of the road. This is usually caused by maintenance activities such as the grader bringing back material from the shoulders and/or drains on to the carriageway, or by new material being placed on the carriageway during spot improvements.

The gravel loss over other cross-sectional widths, such as between the drain inverts, were also examined in a similar manner. This enabled typical gravel loss rates to be determined for each site. The average gravel loss observed for all the sites was 16 mm/year.

A more detailed examination of the gravel loss rates was conducted to determine the influence of variables such as traffic and material properties. The typical rates of gravel loss have been plotted against ADT, as illustrated in Figure 5.1.

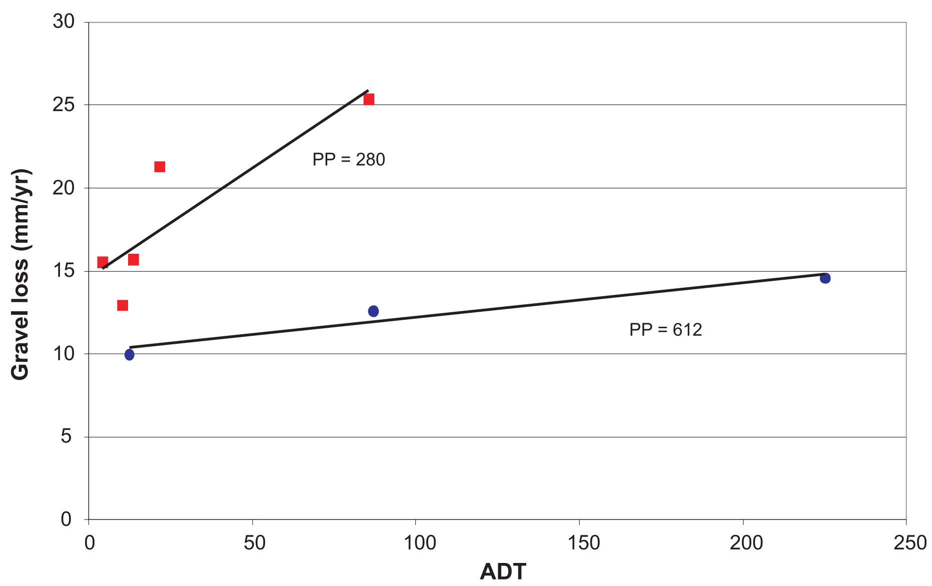
Table 5.2: Rates of gravel loss for sites categorised by plasticity

ADT	Gravel Loss (mm/year)	
	Low Plasticity	High Plasticity
20	17	11
50	21	11
100	28	12
250	48	15

Two distinct groups were evident from this plot. The average plasticity of one group of sites was low, average plasticity product (PP) of 280, with the average PP of the other group being 612. The rates of gravel loss from the 'low plasticity' sites were significantly higher than the rates from the 'high plasticity' sites as shown in Table 5.2, being approximately double for traffic levels of 50 vpd.

In order to assess the influence of the material properties in more detail, the traffic was standardised. The rates of gravel loss for each site were adjusted to a standard ADT of 100 vpd. In other words if the observed rate of gravel loss (GL) was 10 mm/year on a site with an ADT of 50,

Figure 5.1
Rates of gravel loss vs traffic and plasticity



then this rate was adjusted to 20 mm/year for the standard ADT of 100. However, prior to this adjustment, the gravel loss due to the environment (GL_E) needs to taken into account. The observations on the sites indicated that 6 mm/year of gravel were lost due to the environment. Thus the adjusted gravel loss on each site was calculated using the following formula.

$$\text{Adjusted GL} = (GL - GL_E)(100/ADT) + GL_E$$

The performance of the sites were then ranked as 'Good', 'Moderate' or 'Poor' according to their adjusted rates of gravel loss using the thresholds given in Table 5.3.

Table 5.3: Performance criteria

Performance	Adjusted Gravel Loss (mm/year/100 vpd)
Good	≤ 25
Moderate	25 – 60
Poor	> 60

Based on these performance criteria, 2 sites were classified as good, 2 sites were classified as moderate and 4 sites as poor. The details are given in Table 5.4.

The sites were examined individually to determine possible causes for their level of performance.

KAPA was situated in a high rainfall area and the performance of the wearing course was ranked as moderate on the basis of gravel loss. The coarseness index was high (I_c = 56) and the plasticity was also high (I_p = 25 and PP = 575 respectively). This section could have been expected to have performed better and the gravel loss my have been influenced by the high coarseness.

KOMA was situated in a high rainfall area and performed poorly. The coarseness was moderate (I_c = 39) with very little fines (percentage passing 0.075 mm was 1%). The plasticity was generally low (PP = 44). The wearing course consisted of volcanic ash hence the very low PP. The poor performance exhibited by the wearing course may be attributed to the combination of low plasticity and moderate coarseness.

MAMO was situated in a region with moderate rainfall. The wearing course consisted of quartzitic gravel with very high plasticity (I_p = 34 & PP = 884) and with a high coarseness (I_c = 57). The larger size fraction was significant with 15% > 26.5 mm and 8% reject material, i.e. > 37.5 mm. This is likely to have been the cause of the poor performance of the wearing course, which may have performed better but for the high coarseness.

Table 5.4: Performance of the test sites

Site	Perf	Adj GL mm/yr/ 100 ADT	ADT	Annual Rainfall mm/yr	Gradient (m/km)	I_p	I_c	GM	PP
KAPA	M	29	86	1396	19.2	25	56	2.02	575
KOMA	P	73	11	1087	20.0	44	39	2.30	44
MAMO	P	231	4	854	47.8	34	57	2.01	884
MOKA	P	77	15	874	30.7	22	75	2.42	286
MOKO	P	76	22	1415	11.9	18	69	2.36	216
NEKI	G	14	87	731	19.4	24	39	1.42	1032
NIBU	G	10	226	731	33.3	27	33	1.43	999
RAKE	M	38	13	994	35.9	17	46	1.89	374

Note: Performance (Perf) denoted as: G – Good, M – Moderate, P – Poor

MOKA was situated in a region with moderate rainfall. The coarseness of the wearing course was very high (I_c = 75). I_p was high (22) and PP was moderate (286). The poor performance was probably caused by the high coarseness.

MOKO wearing course consisted of sandy gravel. Poor performance of the wearing course was observed. The coarseness was very high (I_c = 69). The I_p was high (18) and the PP was low (216). A combination of low plasticity and very high coarseness may have contributed to the poor performance of the wearing course.

NEKI was situated in an area with high rainfall. The wearing course exhibited a good performance. The coarseness was moderate (I_c = 39) and plasticity was high (I_p = 24 and PP = 1032). The good performance can be attributed to the combination of moderate coarseness and high plasticity..

NIBU wearing consisted of quartzitic clayey gravel and was situated in a low rainfall area. The coarseness was moderate (I_c = 33) and plasticity was high (I_p = 27 and PP = 999). The good performance of the wearing course may have been as a result of the combination of moderate coarseness and high plasticity.

RAKE wearing course performed moderately. The coarseness was moderate (I_c = 46) and plasticity was high (I_p = 17 and PP = 374). This combination of material properties is likely to have contributed to the moderate performance observed on this site.

5.2 Roughness

The roughness measured on each site has been plotted in Appendix F and summarised in Table 5.5.

The average roughness of all the sites over the monitoring period was evaluated as 6.0 IRI, which indicates that the labour-based gravel roads were in a relatively good condition. Somewhat surprisingly, the two sites (KOMA and MAMO) with the highest levels of roughness were the two sites with the lowest levels of traffic. This may be a reflection of the lack of maintenance on very low trafficked roads.

Table 5.5: Observed roughness during each survey

Site	Observed IRI (mm/km)							
	1st	2nd	3rd	4th	5th	6th	7th	Average
KAPA	3.8	5.6	3.6	5.4	6.1	5.8	5.0	5.0
KOMA	8.3	9.2	8.5	7.5	8.2	8.6	8.6	8.4
MAMO	10.4	9.6	11.3	11.4	10.1	9.6	9.4	10.3
MOKA	3.9	3.8	4.0	3.5	3.8	3.6	3.6	3.7
MOKO	5.5	5.3	5.3	4.6	4.0	4.8	4.8	4.9
NEKI	4.8	4.4	4.1	5.1	4.9	5.0	5.6	4.9
NIBU	6.2	7.1	7.6	6.3	5.0	5.8	6.1	6.3
RAKE	4.4	5.0	4.8	4.8	5.6	5.5	5.1	5.0

6. Comparison with HDM-4 Models

One of the objectives of this project was to compare the observed rates of deterioration on the test sites with those predicted by HDM-4. For unsealed roads, HDM-4 predicts the rate of gravel loss and the rate of roughness progression. A comparison between these predicted rates and those observed on the test sites is described below.

6.1 Gravel loss

Regravelling is the major maintenance operation on unsealed roads, analogous in importance to the overlaying of a paved road, so the frequency required is an important planning decision. Gravel loss is defined as the change in gravel thickness over a period of time and is used to estimate when the thickness of the gravel wearing course has decreased to a level where regravelling is necessary.

The HDM-4 relationship for predicting the annual quantity of gravel loss is a function of monthly rainfall, traffic volume, road geometry and characteristics of the gravel and is given below.

$$GL = K_{gl}\ 3.65\ [3.46 + 0.246(MMP/1000)(RF) + (KT)(AADT)]$$

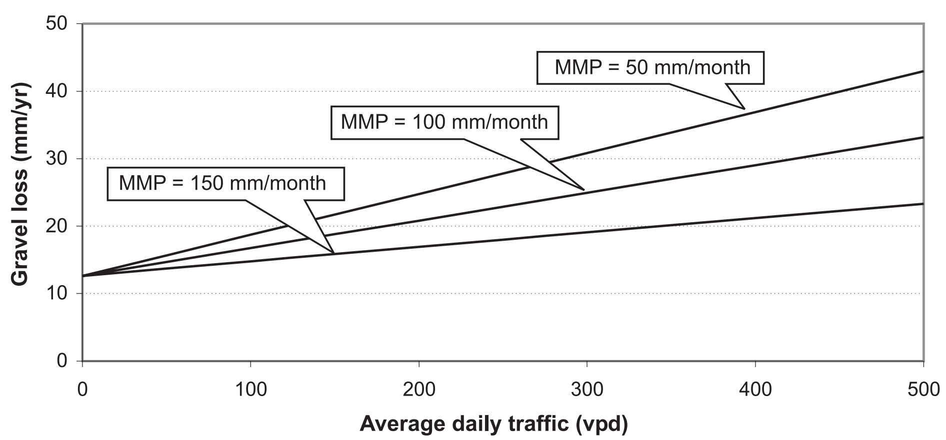
where

$$KT = K_{kt}\ \max\ [0,\ 0.022 + 0.969(HC/57300) + 0.00342(MMP/1000)(P075) - 0.0092(MMP/1000)(PI) - 0.101(MMP/1000)]$$

and

GL	=	annual material loss, in mm/year
KT	=	traffic-induced material whip-off coefficient
AADT	=	annual average daily traffic, in vpd
MMP	=	mean monthly precipitation, in mm/month
RF	=	average rise plus fall of the road, in m/km

Figure 6.1
HDM-4 predicted rates of gravel loss



HC = average horizontal curvature of the road, in deg/km

P075 = amount of material passing the 0.075 mm sieve, in % by mass

PI = plasticity index of the material, in %

K_{gl} = calibration factor for material loss

K_{kt} = calibration factor for traffic-induced material whip-off coefficient

Table 6.1: Observed and HDM-4 predicted rates of gravel loss

Site	Observed Gravel Loss (mm/yr)	Default HDM-4 Predicted Gravel Loss (mm/yr)	Calibration Factor K_{gl}
KAPA	25.4	12.8	1.98
KOMA	12.9	13.3	0.97
MAMO	15.5	12.9	1.20
MOKA	15.7	13.0	1.21
MOKO	21.3	12.8	1.67
NEKI	12.6	13.1	0.96
NIBU	14.6	21.0	0.70
RAKE	10.0	13.2	0.75
Average	**16.0**	**14.0**	**1.2**

The rates of material loss predicted by the above relationship have been illustrated in Figure 6.1 for a range of traffic levels and rainfall for an unsealed road in flat terrain.

The HDM-4 predicted rates of gravel loss for the sites were compared with the typical rates of gravel loss observed on the sites. The HDM-4 model was then calibrated so that the predicted rate matched the observed rate on each site.

The observed rates of gravel loss on each site are listed in Table 6.1 together with the HDM-4 default predicted rates. Also listed in Table 6.1 are the values for the HDM-4 calibration factor K_{gl} used to adjust the predicted rates to match the observed rates of gravel loss for each site.

The results listed in Table 6.1 show that the average value of the gravel loss calibration factor K_{gl} was 1.2, which indicates that on average the amount of gravel lost on these labour-based roads was 20% more than the amount predicted by HDM-4.

However, as illustrated in Figure 5.1, two distinct rates of gravel loss were observed; one for sites with low plasticity and one for sites with high plasticity. The average value for the calibration factor K_{gl} for the low plasticity sites was evaluated as 1.4 and the average value for the high plasticity sites was 0.8. These findings are summarised in Table 6.2.

Table 6.2: Calibration factors for gravel loss

Sites	K_{gl}
Low plasticity	1.4
High plasticity	0.8
All	1.2

6.2 Roughness

In HDM-4, the roughness progression relationship constrains the roughness to a high upper limit, or maximum roughness (RI_{max}), by a convex function in which the rate of progression decreases linearly with roughness to zero at RI_{max} as illustrated in Figure 6.2.

The maximum roughness is a function of material properties and road geometry. The rate of roughness progression is a function of the roughness, maximum roughness, time, light and heavy vehicle passes and material properties. The roughness progression relationship is given by:

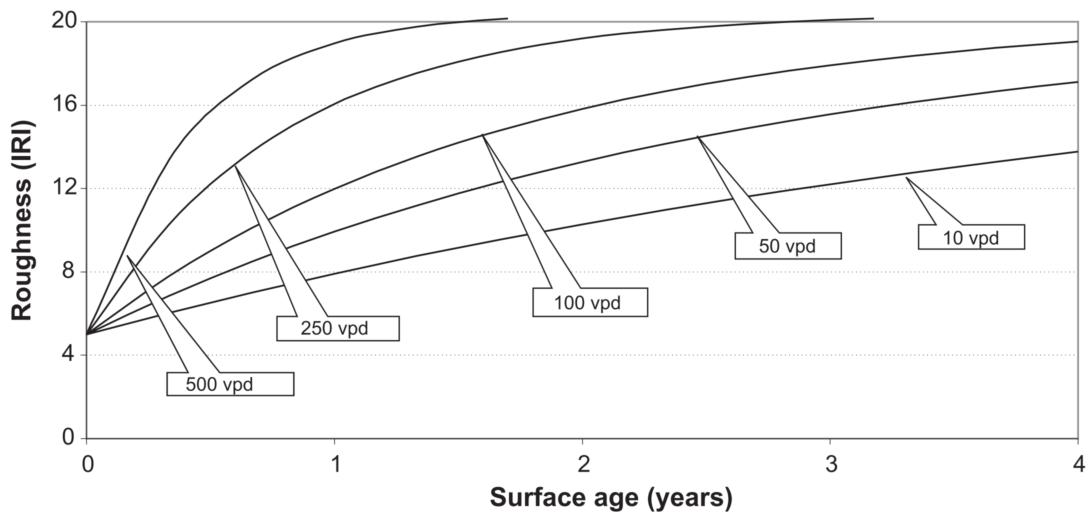
$$RI_{TG2} = RI_{max} - b\,[RI_{max} - RI_{TG1}]$$

where

$$RI_{max} = \max\{[21.5 - 32.4(0.5 - MGD)^2 + 0.017(HC) - 0.764(RF)(MMP/1000)], 11.5\}$$

$$b = \exp\,[c(TG2 - TG1)] \qquad \text{where } 0 < b < 1$$

Figure 6.2
Roughness progressions on unsealed roads with no maintenance



$c = -0.001\ K_c\ [0.461 + 0.0174\ (ADL) + 0.0114(ADH) - 0.0287(ADT)(MMP/1000)]$

and

RI_{TG1} = roughness at time TG_1, in m/km IRI

RI_{TG2} = roughness at time TG_2, in m/km IRI

RI_{max} = maximum allowable roughness for specified material, in m/km IRI

TG_1, TG_2 = time elapsed since latest grading, in days

ADL = average daily light traffic (GVW < 3500kg) in both directions, in vpd

ADH = average daily heavy traffic (GVW ≥ 3500kg) in both directions, in vpd

ADT = average daily vehicular traffic in both directions, in vpd

MMP = mean monthly precipitation, in mm/month

HC = average horizontal curvature of the road, in deg/km

RF = average rise plus fall of the road, in m/km

MGD = material gradation dust ratio
= P075 / P425 if P425 > 0
= 1 if P425 = 0

P425 = amount of material passing the 0.425 mm sieve, in % by mass

P075 = amount of material passing the 0.075 mm sieve, in % by mass

K_c = calibration factor for roughness progression

The roughness progression relationship given above was derived using observations from roads under repeated grading cycles with no special compaction. The rates of roughness progression after construction or rehabilitation with full mechanical shaping and compaction were observed to be much slower than given by this model.

Thus if "mechanical compaction" is specified in the model inputs, the coefficient c is reduced, initially to one quarter of its predicted value and rising to the full predicted value after a few grading cycles, but in a period not exceeding 4 years, as follows:

$$c' = c\ \{\min\ [1,\ 0.25(t)\ \max\ (1,\ n^{0.33})]\}$$

where

t = time since regravelling or construction with mechanical compaction, in years

n = frequency of grading, in cycles/year

and

$$b' = \exp[365(c'/n)]$$

When mechanical compaction is specified, then b′ and c′ are used in place of b and c respectively in the roughness progression relationship.

Maintenance, in the form of grading, on unsealed roads tends to reduce the level of roughness. The HDM-4 relationship for predicting this reduction in roughness is a function of the roughness before grading, the material properties and the minimum roughness (RI_{min}). The minimum roughness, below which grading cannot reduce roughness, increases as the maximum particle size increases and the gradation of the surfacing material worsens.

The HDM-4 relationship for predicting the roughness after grading is expressed as a linear function of the roughness before grading, dust ratio and the minimum roughness, as follows:

RI_{ag} = RI_{min} + a [RI_{bg} - RI_{min}]

where

a = K_a max{0.5, min [GRAD [0.553 + 0.23(MGD)], 1]}

RI_{min} = max {0.8, min [7.7, 0.36(D95) (1 - 2.78MG)]}

and

RI_{ag} = roughness after grading, in m/km IRI

RI_{bg} = roughness before grading, in m/km IRI

RI_{min} = minimum allowable roughness after grading, in m/km IRI

D95 = maximum particle size of the material, defined as the equivalent sieve size through which 95% of the material passes, in mm

MG = slope of mean material gradation

MGD = material gradation dust ratio

GRAD = 1.4 for non-motorised grading, bush or tyre dragging

= 1.0 for light motorised grading, little or no water and no roller compaction

= 0.7 for heavy motorised grading, with water and light roller compaction

K_a = calibration factor for the effect of grading

The slope of mean material gradation is calculated as follows:

MG = min [MGM, (1 - MGM), 0.36]

where

MGM = (MG075 + MG425) + MG02) / 3

MG075 = $\log_e$(P075/95) / $\log_e$(0.075/D95)

MG425 = $\log_e$(P425/95) / $\log_e$(0.425/D95)

MG02 = $\log_e$(P02/95) / $\log_e$(2.0/D95)

The HDM-4 predicted rates of roughness for the sites were compared with the roughness observed on the sites. It was assumed that light motorised grading with little or no water and no roller compaction was used on an annual basis (i.e. GRAD = 1.0). The HDM-4 roughness model was then calibrated so that the predicted roughness matched the average roughness observed on the site during the two-year monitoring period.

The average roughness values observed on the sites are listed in Table 6.3, together with the values for the HDM-4 calibration factor K_c used to adjust the predicted roughness to match the observed roughness on each site.

The roughness on the MAMO site was consistently high, averaging 10.3 IRI over the 2-year monitoring period, with a value of 10.4 IRI being recorded during the 1st survey in August 2002 (see Table 5.5). Information gathered for this site indicated that it was constructed in 2001. It is extremely unusual for a site to reach roughness levels of 10 IRI one year after construction. Alternatively, the construction year of 2001 is incorrect. Therefore the MAMO site was excluded from the calculation of the average value of the calibration factor K_c.

The average value of K_c for the remaining seven sites was 0.8. This indicates that the rates of roughness progression observed on the sites were, on average, lower than that

Table 6.3: HDM-4 roughness calibration factors

Site	Construction Year	ADT	Observed Roughness IRI	Calibration Factor K_c
KAPA	2001	86	5.0	0.35
KOMA	2000	11	8.4	3.0
MAMO	2001	4	10.3	5.5
MOKA	2001	15	3.7	0.55
MOKO	2000	22	4.9	0.55
NEKI	1998	87	4.9	0.01
NIBU	2002	226	6.3	0.35
RAKE	2000	13	5.0	0.65
Average				**0.8**[1]

Note: [1] – excludes MAMO

predicted by HDM-4. It is also evident that the higher trafficked sites (KAPA, NEKI & NIBU) had the lowest values of K_c. This indicates that the effect of increased traffic levels in HDM-4 is much higher than observed on the sites in Uganda.

7. Life-Cycle Cost Methodology

As mentioned in Section 1.4, this study in Uganda is one of several that have been carried out in Africa on the performance of labour-based roads. The results from these studies will be combined and used to estimate life-cycle costs for roads constructed using labour-based techniques, and will be reported in the Regional Report. The methodology that is proposed to estimate these life-cycle costs is described below.

The performance of each site has been assessed as described in Section 5. This assessment indicated that the material properties, primarily grading and plasticity, were important factors in the performance of the sites, with the rates of gravel loss generally lower on sites that had a gravel wearing course that was fine with high plasticity.

The quality of the gravel wearing course can be assigned to one of four 'material quality zones', as illustrated in Figure 7.1. The higher quality materials are represented by Zone A where PP > 300 and GM < 1.9. Sites with this material quality would be expected to perform well. The poorest material quality is represented by Zone D where PP < 300 and GM > 1.9. Sites with this quality material would be expected to perform poorly, with Zones B and C representing material of moderate quality.

The sites from all the studies in the region (i.e. Ghana, Uganda and Zimbabwe) will be assigned to one of the four 'material quality

Figure 7.1
Material quality zones

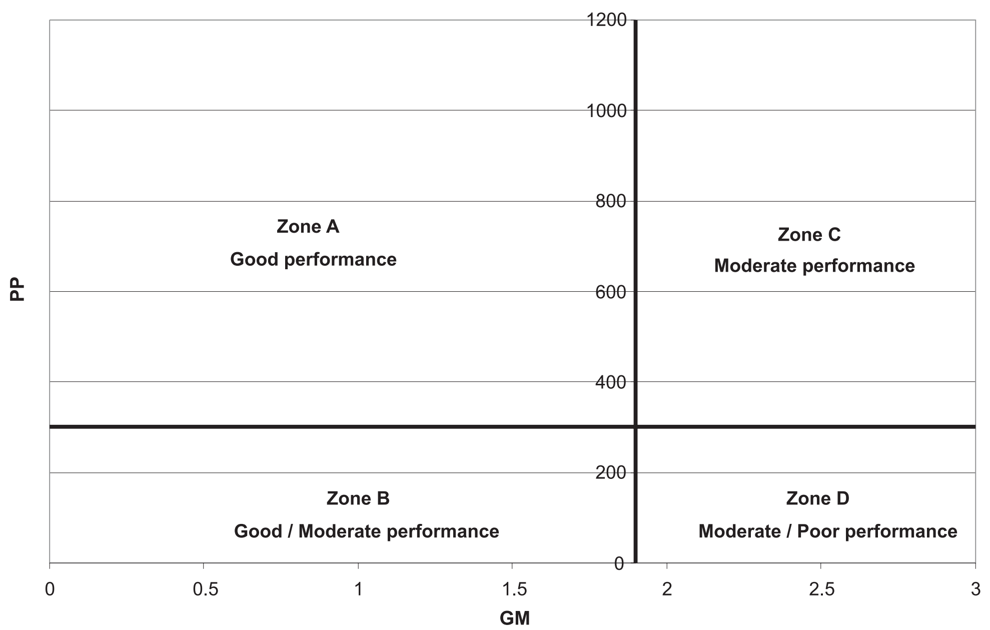


Figure 7.2
Regravelling frequency

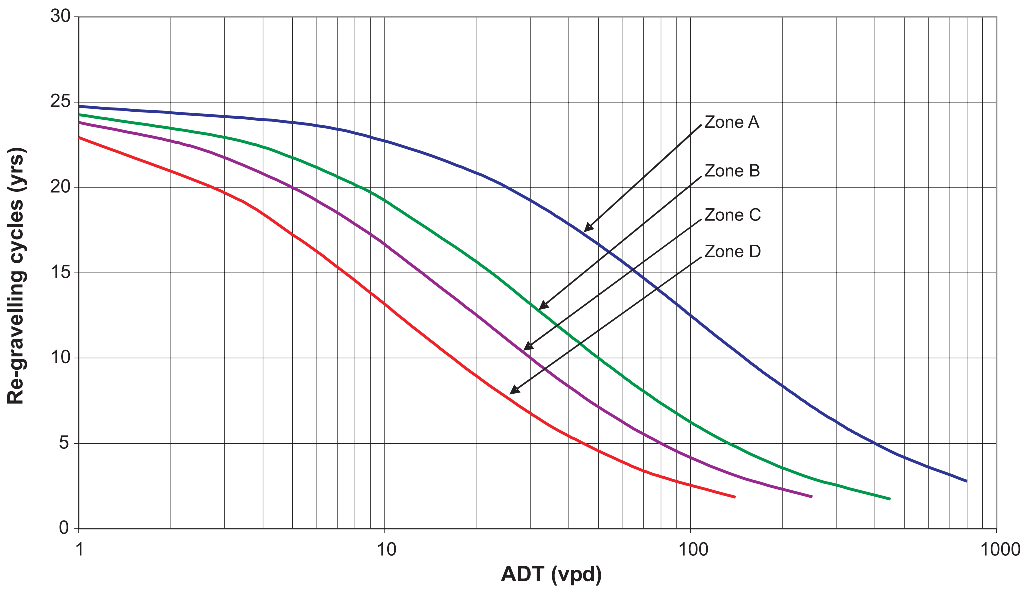
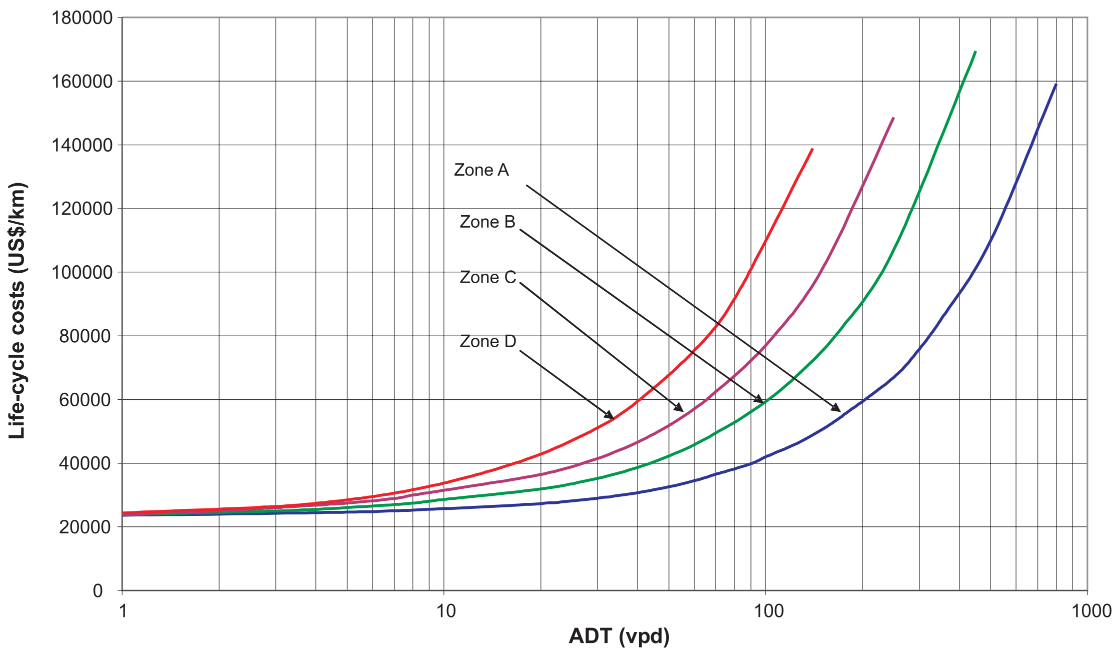


Figure 7.3
Example of life-cycle costs



zones' based on the properties of their gravel wearing course. The performance of the sites, in terms of gravel loss, will be assessed and average rates of gravel loss evaluated for the sites in each zone. These average rates of gravel loss for each zone will indicate the frequency that sites with particular material properties need to be regravelled, depending on the thickness of the wearing course and traffic volumes. An example of regravelling frequencies for a gravel wearing thickness of 150 mm, is illustrated in Figure 7.2.

Using graphs such as that illustrated in Figure 7.2, the number of times a road will need to be regravelled over its life can be estimated, knowing the quality of the gravel wearing course and the traffic volume. The cost of regravelling over the life of the road can then be estimated.

In addition to regravelling costs, life-cycle costs also include initial construction or rehabilitation costs and regular routine maintenance costs. Routine maintenance includes grading and other activities such as spot regravelling, vegetation control, etc. The frequency of these routine maintenance activities will depend on perceived acceptable conditions of roads for various levels of traffic.

A spreadsheet-based program will be developed for computing life-cycle costs for various levels of traffic and for the different material quality zones, as illustrated in the fictitious example in Figure 7.3. This example was developed using fictitious unit costs for the construction, regravelling and routine maintenance activities. These unit costs, as well as other parameters such as frequency of routine maintenance activities, will need to be adjusted in the spreadsheet program with country-specific data.

8. Conclusions

8.1 Performance of the roads

An analysis of the performance of the labour-based roads indicated that the dominant factors that affected the rates of gravel loss were traffic and plasticity of the gravel wearing course. Two distinct trends were observed. For sites with low plasticity wearing course, the rate of gravel loss was high, ranging from approximately 15 mm/year for low trafficked roads (ADT < 20) to 30 mm/year for higher trafficked roads (ADT = 100). For sites with high plasticity wearing course, the rates of gravel loss ranged between 10 mm/year for low trafficked roads to 15 mm/year for high trafficked roads.

The findings of this analysis can be summarised as follows:

i) Durability of the wearing course is largely dependent on both the volume of traffic and the plasticity of the wearing material.

ii) For the same volume of traffic wearing course with low plasticity is less durable and therefore less economical than wearing course with relatively high plasticity.

iii) The performance in terms of gravel loss of the high plasticity and low plasticity wearing courses tend to converge with reduction in ADT. This indicates that for very low volume roads, materials of low plasticity may be used without significantly increasing the maintenance costs. However, when traffic volume exceeds 20 vehicles per day, the cost of maintenance increases significantly for the low plasticity wearing course, which in turn increases the whole-life costs, hence compromising viability and sustainability of the road structure.

iv) High plasticity in a wearing course greatly minimises the impact of increases in traffic volumes and it also results in substantial reductions in maintenance costs and hence the whole-life costs. It is noted, however, that the strength of the gravel layer is sensitive to the plasticity of the material. The minimum soaked CBR required for wearing course material should be adhered to during prospecting, selection of the wearing materials. Excessive plasticity may result in premature failure of the road due to deformation and loss of traction by vehicles.

v) The trends established during this analysis can be used to estimate the rate of gravel loss for a range of traffic and plasticity products. The results can be used to estimate the cost of re-gravelling through the life-cycle of the road.

vi) This relationship can be used as a design tool for gravel roads which incorporates traffic, materials, performance, economics and future maintenance, in addition to the structural strength and geometry as design parameters.

vii) The plasticity product of the material should be > 300, otherwise the rate of gravel loss will be high.

A comparison of the observed rates of gravel loss with the rates predicted by HDM-4 indicated that, on average, the observed rates were 20% higher than the HDM-4 predicted rates, giving an average value of 1.2 for the gravel loss calibration factor K_{gl}. However, the average value for K_{gl} for the

Table 8.1: Grading specifications for gravel wearing courses

Sieve Size (mm)	Percentage Passing (by weight)	
	ADT < 150	ADT > 150
40	100	100
28	95 – 100	100
20	85 – 100	95 – 100
14	65 – 100	80 – 100
10	55 – 100	65 – 100
5	35 – 92	45 – 85
2	23 – 77	30 – 68
1	18 – 62	25 – 56
0.425	14 – 50	18 – 44
0.075	10 - 40	12 – 32

Table 8.2: Plasticity and strength specifications

Parameter	Wet Zone	Dry Zone
Liquid Limit, LL	≥ 35	≥ 35
I_p	5 – 15	10 – 30
Plasticity Modulus, PM	200 – 1200	200 – 1200
Soaked CBR (%)	≥ 25	≥ 25
DCP (mm/blow)	≥ 9	≥ 9

low plasticity sites was evaluated as 1.4 and the average value for the high plasticity sites was 0.8. This indicates that on wearing courses with low plasticity, the rates of gravel loss are approximately 40% higher than those predicted by HDM-4, whereas for high plasticity wearing courses the rates are approximately 20% less than those predicted by HDM-4.

A comparison of the observed roughness levels on the sites with the rates predicted by HDM-4 indicated that, on average, the observed roughness levels were lower than the levels predicted by HDM-4, with an average value of 0.8 for the roughness calibration factor K_c. The values of the K_c were lower for the higher trafficked sites, indicating that the effect of increasing traffic in HDM-4 is much higher than observed in Uganda.

8.2 Material specifications

The current grading specifications for gravel wearing course are listed in Table 8.1.

The GM, limits are:

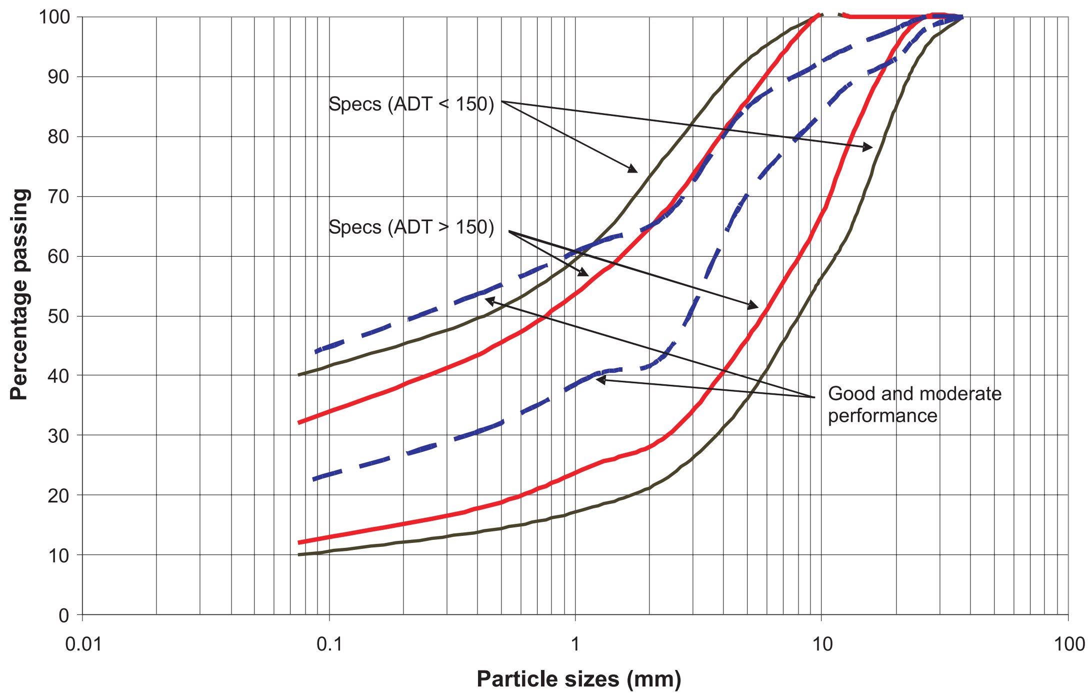
$1.33 \leq GM \leq 2.53$ (for ADT < 150)

$1.56 \leq GM \leq 2.40$ (for ADT > 150)

Table 8.3: Grading envelopes of monitored sites

Sieve (mm)	Percentage Passing (by weight)		
	Current Specifications		Good and Moderate Performance
	ADT < 150	ADT > 150	
40	100	100	100
28	95 – 100	100	98 – 100
20	85 – 100	95 – 100	93 – 98
14	65 – 100	80 – 100	89 – 95
10	55 – 100	65 – 100	83 – 92
5	35 – 92	45 – 85	69 – 84
2	23 – 77	30 – 68	44 – 67
1	18 – 62	25 – 56	40 – 62
0.425	14 – 50	18 – 44	31 – 54
0.075	10 - 40	12 – 32	22 – 43

Figure 8.1
Comparison of grading envelopes



The specifications for the plasticity and strength of the wearing course are as follows:

The analysis of the performance of the test sites, as reported in Section 5.1.2, can be used to modify these material specifications. As shown in this section, sites were categorised according to their performance as either 'good', 'moderate' or 'poor'. The grading envelopes for the sites that exhibited either a good or moderate performance are listed in Table 8.3 together with the current specifications.

The grading envelopes listed in Table 8.3 have been plotted in Figure 8.1. It is evident from these plots that the grading envelope derived from the samples of wearing course that performed either well or moderately is much narrower than the currently specified envelopes. This narrow envelope is basically a result of the small sample used to derive the envelope.

For good performance there is need for bias towards finer materials. It is therefore recommended that the specifications for the wearing course are amended as follows:

- No distinction is made between ADT < 150 and ADT > 150
- The lower limit of the grading envelope for ADT > 150 is used as the lower (i.e. coarse) limit
- The upper limit of the grading envelope for ADT < 150 is used as the upper (i.e. fine) limit for particle sizes > 1 mm
- The upper limit of the grading envelope for 'good or moderate' performances is used as the upper (i.e. fine) limit for particle sizes < 1 mm

The recommended grading envelope for wearing course is given in Table 8.4.

From this new grading envelope, the recommended grading modulus limits are:

$$1.56 \leq GM \leq 2.40$$

No modifications to the existing plasticity specifications are recommended because of the small range of plasticity of the samples.

Table 8.4: Recommended new grading for gravel wearing courses

Sieve Size (mm)	Percentage Passing (by weight)
37.5	100
26.5	98 – 100
20	93 – 100
13.2	80 – 100
9.5	65 – 100
4.75	45 – 92
2.36	30 – 77
1.18	25 – 62
0.425	18 – 54
0.075	12 – 43

8.3 Life-Cycle costs

A methodology for estimating life-cycle costs has been developed as outlined in Section 7. Data from other regional studies (Ghana and Zimbabwe) will be combined with the results from this study to derive life-cycle costs for gravel roads constructed using labour-based techniques, and will be reported in the Regional Report.

Figure 8.2
Recommended new grading envelope