Increased Application of Labour-Based Methods *through* Appropriate Engineering Standards

Zimbabwe Country Report

DANIDA

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ILO

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Abbreviations

ADT	–	Average Daily Traffic
DANIDA	–	Danish International Development Agency
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 raveling 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 Zimbabwe. The sites were monitored over a period of three years to determine gravel loss, changes in road roughness and visually inspected to record any other factors affecting road performance.

The main conclusions of the study were:

- The materials available for road building in Zimbabwe are generally good as is evident from the average gravel loss figure for all the 31 test sites of 10 mm per year. However, for the most heavily trafficked sites (ADT > 100) the gravel loss was double this figure.
- The average gravel loss for the test sections is 65% of the value predicted by HDM.
- Average roughness is similar being 90% of the HDM predicted values.
- There is scope for relaxing the grading specifications for materials and increasing the range of the grading modulus to 1.0 < GM < 2.6.

- ❖ The upper and lower values of the plasticity (I_p) of materials can be changed to give a range of $5 < I_p < 20$ subject to restrictions on the plasticity product (PP) and fineness index.
- ❖ Some non-plastic material also performed well on the very lightly trafficked roads (ADT < 20) and where the coarseness index was less than 30.

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 but insufficient cost information was available to enable recommendations on specific application in Zimbabwe. The methodology will be available in the Regional Report.

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 Zimbabwe. These activities include the selection of test sites that are typical of labour-based roads in Zimbabwe, 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 Uganda. 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 Zimbabwe.

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 Zimbabwe.

2. Test Sites

2.1 Selection

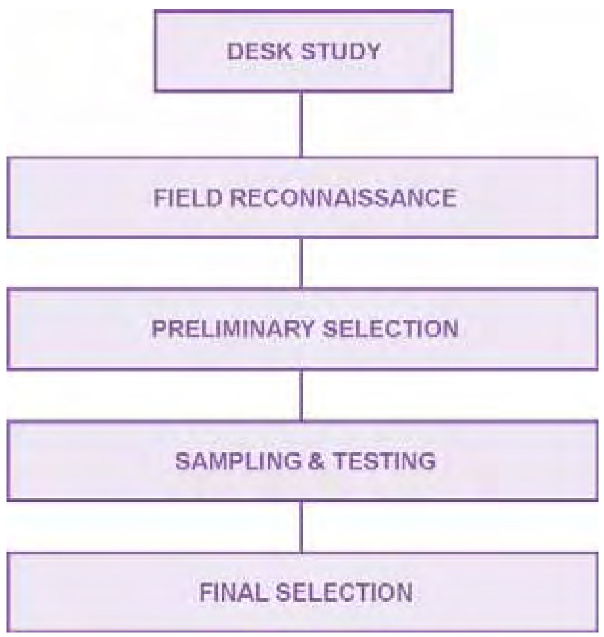
One of the main objectives of this project was to determine the rate of deterioration of gravel roads constructed by labour-based techniques to enable future predictions to be made on the performance of these types of road. In order to monitor the performance of these roads, test sites were selected 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 Uganda and Ghana. 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 Zimbabwe 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

2.2.1 Country background

The length of the total road network in Zimbabwe is about 80,000 km. Of this, approximately 22% are roads for which the Department of Roads is responsible and approximately half of these are paved. The rest are rural and urban roads, which are the responsibility of rural or urban councils. In many areas of Zimbabwe the soils are quartzite in origin and are good road-building materials.

2.2.2 Climate

The climate in Zimbabwe is generally at the drier end of the range of climates in the various African countries participating in the overall programme. The Thornthwaite's Moisture Index (TMI) was used as a measure of climate and the climatic classification in terms of the TMI is 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

Table 2.2: Roads inspected

Province	Road Name	Construction Year	Province	Road Name	Construction Year
Mashonaland	Katarira – Mahuwe	1998	Matebeleland	Tsholotsho – Plumtree	1997
Central	Bullnose – Matowa		North	Tsholotsho – Sihazela	1994
MSC	Mkumbura – Casembi		MTN	Tsholotsho – Lagisa	1992
	Dotito – Bullnose			Tinde – Pashu	2000
	Dotito – St Alberts			Jotsholo – Mzola	1998
	Chaparadza – Chiwaze			Nkayi – Gokwe	
	Nyamasota – Katemere			Gwayi – Binga	1998-
				Lubimbi – Cewali	1998
Manicaland	Headlands – Chikore	1998			
MAN	Nyanga – Ruwangwe		Matebeleland	Filabusi – Avoca	
	Nyanga – Rwenya	1998	South	Kafusi – Manama	1997
	Nyafaru – Katiyo		MTS	St Josephs – Maphisa	
	Nyamaropa – Chisvo	2000		Mpoengs – Maphisa	2000
	Chiriga – Chibunji			Plumtree – Madlambuzi	1996-98
				Plumtree – Somnene	2000
Mashonaland	Mutawatawa – Pfungwe	1998			
East	Suswe – Chitsungo	1995-96	Midlands	Fairfields – Mashava	
MSE	Mutoko – Nyamuzuwe	1991-92	MID	Mberengwa – Mataga	1994-95
	Mutoko – Rwenya	1993-94		Mateta – Manoti	
	Nharira – Mupatsi	1998-2000		Gokwe – Choda	
				Empress – Masoro	1999
Mashonaland	Karoi – Shamrock	1999			
West	Binga – Bumi		Masvingo	Chikuku – Makuwaza	1994-96
MSW	Chegutu – Mubayira		MAS	Mkwasine – Matsvange	1997-98
	Kadoma – Mamina			Chibwedziva – Chilonga	
	Alaska – Copper Queen			Nandi – Boli	
	Karoi – Binga			Bondolfi – Renco	
	Zvimba – Mupfure	2000		Sarahuru – Maranda	1999
	Kutama – Zvimba				

2.3 Field reconnaissance

A field reconnaissance survey was carried out based on roads identified in the desk study and sections were selected to cover the range of factors (climate, traffic, terrain, materials) considered to be influential in the performance of unpaved roads constructed by labour-based techniques. Samples of material were also collected from these sections for classification tests. The roads that were inspected are listed in Table 2.2 and a detailed summary of the visit reports is given in Appendix A. Roads identified for improvement within the study period were not included.

2.4 Final selection of test sites

A final list of 31 test sites was drawn up using information from the reconnaissance survey and the results of tests on materials collected from prospective test sites. The number of sites was limited by the resources available to monitor all those selected within a reasonable time period. Table 2.3

Table 2.3 Test sites selected for monitoring

Road Name	No.	Site Code	Material	Climate
Chikuku – Makuwaza	1	CUMA 1	Quartz	Dry sub-humid
	2	CUMA 2	Quartz	Dry sub-humid
Katarira – Mahuwe	3	KAME 1	Sandstone	Dry sub-humid
	4	KAME 2	Sandstone	Dry sub-humid
	5	KAME 3	Sandstone	Dry sub-humid
Maranda – Mwenezi	6	MAMI 1	Calcrete	Semi-arid
	7	MAMI 2	Calcrete	Semi-arid
Mkwasine – Matsange	8	MEME 1	Laterite	Semi-arid
	9	MEME 2	Laterite	Semi-arid
	10	MEME 3	Laterite	Semi-arid
Mutoko – Nyamuzuwe	11	MONE 1	Quartz	Dry sub-humid
	12	MONE 2	Quartz	Dry sub-humid
Mpoengs – Maphisa	13	MSMA 1	Calcrete	Semi-arid
	14	MSMA 2	Calcrete	Semi-arid
Nyamaropa – Chiso	15	NACO 1	Quartz	Dry sub-humid
	16	NACO 2	Quartz	Dry sub-humid
Nyanga – Rwenya	17	NARA 1	Quartz	Dry sub-humid
	18	NARA 2	Quartz	Dry sub-humid
Nyafaru – Katiyo	19	NUKO 1	Quartz + Feldspar	Moist sub-humid
	20	NUKO 2	Quartz + Feldspar	Moist sub-humid
Plumtree – Somnene	21	PESE 1	Quartz	Dry sub-humid
	22	PESE 2	Quartz	Dry sub-humid
Suswe – Chitsungo	23	SECO 1	Iron oxide, Quartz	Dry sub-humid
	24	SECO 2	Iron oxide, Quartz	Dry sub-humid
Sarahuru – Maranda	25	SUMA 1	Feldspar	Dry sub-humid
	26	SUMA 2	Feldspar	Dry sub-humid
Tinde – Pashu	27	TEPU 1	Quartz + D. Granite	Dry sub-humid
	28	TEPU 2	Quartz	Dry sub-humid
	29	TEPU 3	Quartz + D. Granite	Dry sub-humid
Tsholotsho – Sihazela	30	TOSA 1	Calcrete	Dry sub-humid
	31	TOSA 2	Calcrete	Dry sub-humid

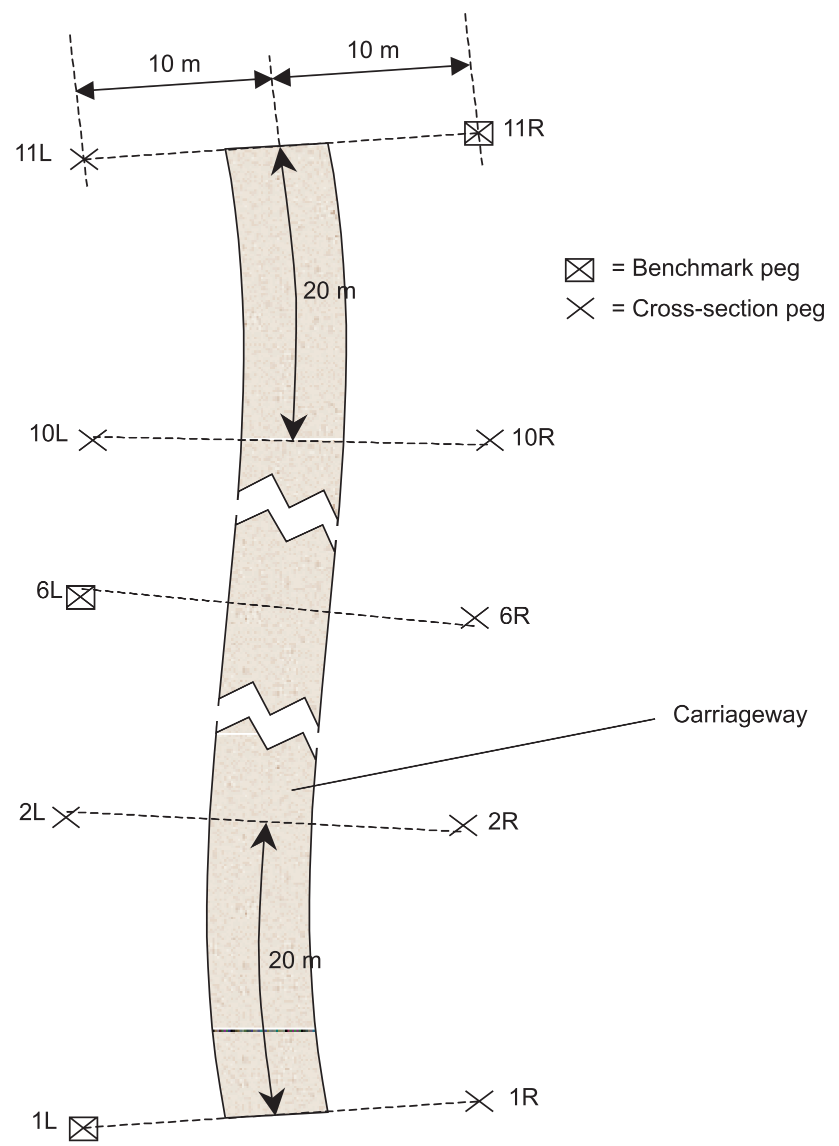
shows the final list of sites selected for the study. For easy referencing these sites are referred to by 4 letters, the first and last letters of the two places connected by the road, e.g. **K**atarir**A** – **M**ahuw**E** (KAME).

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.

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.

Figure 2.2
Plan view of peg layout on site



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.

3.2 Traffic

Classified traffic counts were carried out on the test sites. Most of the counts were conducted for periods lasting between 9 and 12 hours, with 24-hour counts carried out on selected sites to provide ratios for the estimation of the Average Daily Traffic (ADT) for all the sites. The 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 140 vpd, with the average ADT for all the sites being 38 vpd.

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 3.1: Road alignment

Site	Terrain	Gradient (m/km)
CUMA 1	Rolling	0.9
CUMA 2	Rolling	15.4
KAME 1	Rolling	28.3
KAME 2	Flat	21.1
KAME 3	Flat	4.4
MAMI 1	Flat	24.6
MAMI 2	Flat	4.8
MEME 1	Flat	1.0
MEME 2	Flat	12.7
MEME 3	Flat	17.9
MONE 1	Mountainous	40.3
MONE 2	Mountainous	24.4
MSMA 1	Flat	1.8
MSMA 2	Flat	1.8
NACO 1	Rolling	58.7
NACO 2	Rolling	41.8
NARA 1	Rolling	31.8
NARA 2	Flat	7.5
NUKO 1	Flat	35.9
NUKO 2	Flat	67.1
PESE 1	Flat	15.2
PESE 2	Flat	15.9
SECO 1	Rolling	1.8
SECO 2	Mountainous	21.8
SUMA 1	Flat	14.4
SUMA 2	Flat	15.1
TEPU 1	Flat	1.0
TEPU 2	Flat	8.1
TEPU 3	Flat	13.5
TOSA 1	Flat	3.7
TOSA 2	Flat	2.4

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

Table 3.2: 24-hour traffic volumes

Site	9-Hour Counts					24-Hour ADT
	Cars	Trucks	Buses	Tractors	Total	
CUMA 1	7	3	1	0	11	60
CUMA 2	16	1	1	0	18	110
KAME 1	4	2	0	0	6	10
KAME 2	19	1	3	0	23	91
KAME 3	4	0	8	1	13	65
MAMI 1	2	5	0	0	7	30
MAMI 2	6	7	0	0	13	45
MEME 1	27	1	3	0	31	44
MEME 2	6	0	1	1	8	50
MEME 3	2	0	0	0	2	20
MONE 1	43	3	0	2	48	140
MONE 2	1	0	0	0	1	8
MSMA 1	8	1	0	0	9	69
MSMA 2	9	2	0	2	13	99
NACO 1	–	–	–	–	–	6
NACO 2	–	–	–	–	–	8
NARA 1	2	0	1	0	3	15
NARA 2	5	1	1	1	8	30
NUKO 1	6	0	7	0	13	33
NUKO 2	9	0	1	1	11	33
PESE 1	2	1	0	0	3	15
PESE 2	1	0	0	0	1	5
SECO 1	4	0	1	0	5	52
SECO 2	4	0	0	0	4	42
SUMA 1	1	1	0	0	2	10
SUMA 2	3	1	0	0	4	20
TEPU 1	5	3	0	0	8	12
TEPU 2	1	1	0	1	3	8
TEPU 3	2	0	0	0	2	4
TOSA 1	5	0	0	0	5	25
TOSA 2	4	0	2	0	6	30

Table 3.3: Rainfall

Sites	Rainfall (mm)												
	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Annual
CUMA 1, 2	185	337	236	65	25	62	33	8	33	81	146	206	1417
KAME 1, 2, 3	264	338	117	9	10	7	0	0	0	9	89	164	1008
MAMI 1, 2	77	116	79	16	4	18	8	0	11	45	80	37	489
MEME 1, 2, 3	97	229	147	18	12	16	19	1	20	48	90	130	829
MONE 1, 2	194	223	139	25	3	3	1	4	1	20	117	139	868
MSMA 1, 2	152	156	33	12	3	16	8	0	4	54	127	77	643
NACO 1, 2	211	229	245	17	5	3	2	11	3	21	152	93	990
NARA 1, 2	275	227	252	97	22	34	30	20	20	35	138	290	1439
NUKO 1, 2	398	435	499	121	36	56	24	28	36	115	215	271	2231
PESE 1, 2	108	135	64	16	6	14	2	2	4	28	130	119	629
SECO 1, 2	282	64	225	0	0	0	0	0	0	0	67	54	692
SUMA 1, 2	77	116	79	16	4	18	8	0	11	45	80	37	489
TEPU 1, 2, 3	104	167	82	13	3	2	2	0	13	21	68	115	589
TOSA 1, 2	79	90	22	71	3	12	1	0	1	42	28	114	462

3.4 Construction details

The year of construction of each road on which the sites were located was gathered from the regional offices. These construction years are listed in Table 3.4.

3.5 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, and dry density at 95% Mod AASHTO.

3.5.1 Gravel wearing course

Grading results obtained for the samples of the gravel wearing course are shown in Table 3.5. A plot of the grading curve from each site is illustrated in Figure 3.1. The grading envelope encompassing the grading curves from all the sites is illustrated in Figure 3.2.

Table 3.4: Year of construction

Road	Year of Construction
CUMA	1994
KAME	1998
MAMI	1997
MEME	1997
MONE	1991
MSMA	2000
NACO	2000
NARA	1998
NUKO	2000
PESE	2000
SECO	1995
SUMA	1999
TEPU	2000
TOSA	1994

The material properties of the wearing course are listed in Table 3.6 and the ranges summarised in Table 3.7.

Table 3.5: Grading of gravel wearing course

Site	Percentage Passing (mm sieve)											
	37.5	26.5	19	9.5	4.75	2.36	1.18	0.6	0.425	0.3	0.15	0.075
CUMA 1	100	100	96	93	90	86	72	61	58	55	46	39
CUMA 2	100	100	100	96	95	87	71	54	50	45	38	33
KAME 1	100	91	83	60	46	41	38	36	35	34	29	24
KAME 2	100	100	100	97	91	83	72	59	53	47	36	30
KAME 3	100	100	99	94	85	74	71	52	47	41	27	19
MAMI 1	100	95	89	88	81	72	63	51	46	41	33	28
MAMI 2	100	96	87	77	65	58	57	46	42	38	28	23
MEME 1	100	100	100	100	93	72	71	55	49	42	27	19
MEME 2	100	100	100	97	95	80	70	55	48	41	28	21
MEME 3	100	96	93	86	78	69	69	41	37	32	23	18
MONE 1	100	100	100	100	100	99	97	61	50	39	26	19
MONE 2	100	100	100	98	97	87	85	53	46	39	28	22
MSMA 1	100	100	100	86	75	70	66	57	48	39	23	15
MSMA 2	100	100	100	87	72	62	62	47	41	35	22	17
NACO 1	100	100	100	100	100	79	62	47	39	31	23	19
NACO 2	100	94	90	80		58	58	39	33	27	16	11
NARA 1	100	96	91	71	60	53	46	36	32	27	13	7
NARA 2	100	97	96	91	85	69	56	44	38	31	19	14
NUKO 1	100	100	100	100	100	96	87	77	70	63	49	43
NUKO 2	100	100	100	98	96	89	88	77	71	65	50	44
PESE 1	100	94	88	71	57	49	41	32	27	22	16	10
PESE 2	100	98	94	83	65	52	51	31	27	22	17	13
SECO 1	100	100	94	81		61	61	43	40	37	31	25
SECO 2	100	89	81	59	47	42	39	36	34	31	24	22
SUMA 1	100	100	100	99	94	85	74	58	50	41	27	20
SUMA 2	100	88	80	69	60	51	45	40	38	35	29	25
TEPU 1	100	97	94	87	79	64	49	36	31	26	19	15
TEPU 2	100	95	93	82	68	50	36	26	24	21	17	14
TEPU 3	100	100	93	81	62	53	52	36	31	26	16	12
TOSA 1	100	100	96	85	67	50	44	40	37	34	23	19
TOSA 2	100	96	95	90	81	63	53	47	44	41	26	22

Figure 3.1
Particle size distribution of the gravel wearing course

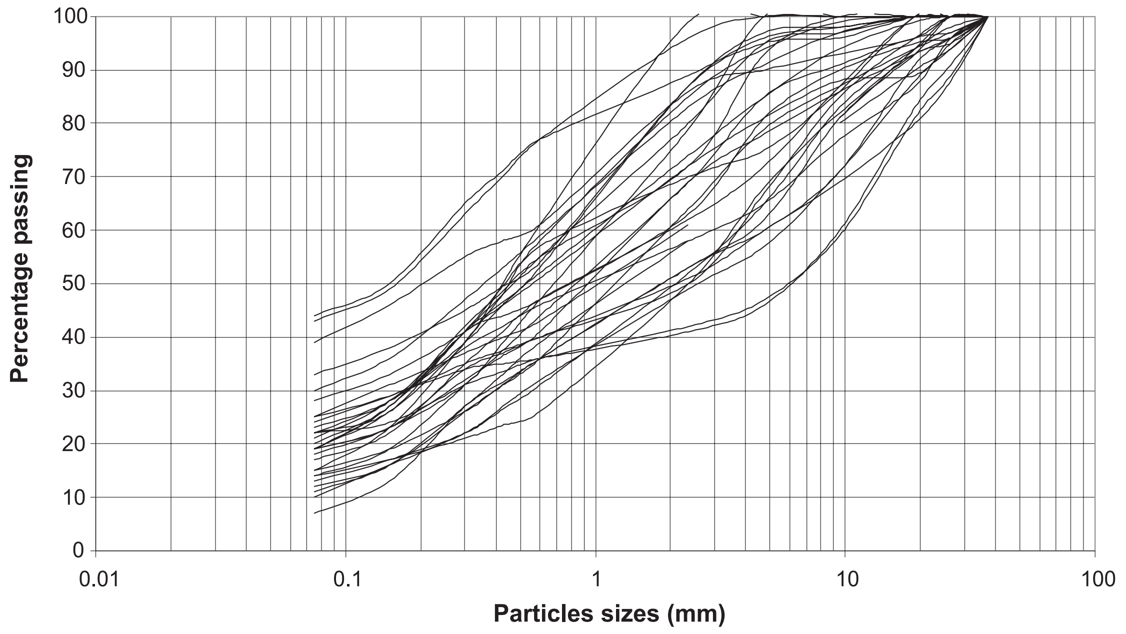


Figure 3.2
Grading envelope for the gravel wearing course

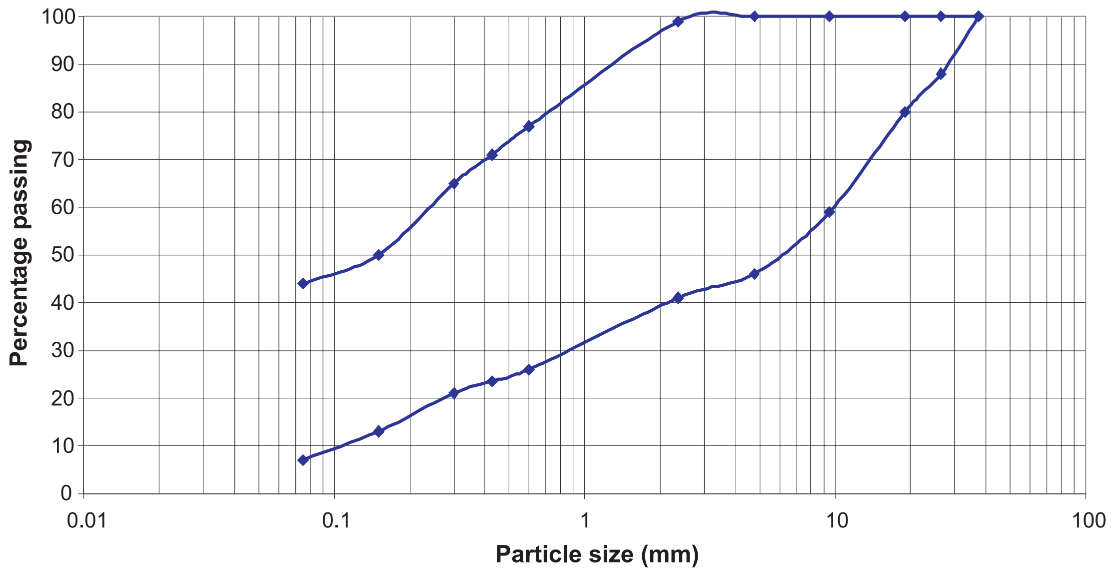


Table 3.6: Material properties of the gravel wearing course

Site	Coarseness Index I_c	Dust Ratio DR	Grading Modulus GM	Grading Coefficient Gc	Plasticity Index I_p	Plasticity Modulus PM	Plasticity Product PP	Plasticity Factor PF
CUMA 1	14	0.67	1.17	8.1	16	986	663	702
CUMA 2	13	0.67	1.31	6.4	14	693	462	528
KAME 1	60	0.69	2.00	17.5	16	560	384	576
KAME 2	17	0.57	1.34	9.0	19	1007	570	570
KAME 3	26	0.41	1.61	12.1	7	326	133	323
MAMI 1	28	0.61	1.54	10.6	27	1242	756	560
MAMI 2	43	0.55	1.77	16.0	15	588	322	690
MEME 1	28	0.39	1.61	13.6	NP	0	0	0
MEME 2	20	0.44	1.51	9.6	NP	0	0	0
MEME 3	31	0.49	1.77	9.9	7	256	126	252
MONE 1	1	0.38	1.32	0.5	NP	0	0	0
MONE 2	13	0.48	1.45	6.0	7	322	154	308
MSMA 1	30	0.31	1.67	14.4	7	288	90	240
MSMA 2	38	0.41	1.80	15.6	7	287	119	289
NACO 1	21	0.49	1.63	8.2	NP	0	0	0
NACO 2	42	0.33	1.98	11.9	NP	0	0	0
NARA 1	47	0.22	2.09	13.5	NP	0	0	0
NARA 2	31	0.37	1.80	10.5	NP	0	0	0
NUKO 1	5	0.61	0.91	2.8	13	910	559	1032
NUKO 2	11	0.62	0.96	7.8	11	781	484	924
PESE 1	51	0.37	2.14	12.2	NP	0	0	0
PESE 2	48	0.49	2.09	12.2	6	159	78	169
SECO 1	39	0.63	1.74	15.6	13	520	325	575
SECO 2	58	0.66	2.03	15.7	8	268	176	330
SUMA 1	15	0.40	1.46	7.4	NP	0	0	0
SUMA 2	49	0.67	1.87	13.9	18	675	450	525
TEPU 1	37	0.48	1.90	10.2	9	248	120	225
TEPU 2	50	0.60	2.13	10.6	7	141	84	210
TEPU 3	47	0.39	2.04	14.6	NP	0	0	0
TOSA 1	50	0.51	1.94	18.5	18	629	323	285
TOSA 2	37	0.50	1.71	14.5	18	396	198	330

The grading curves are typical of good, well-graded wearing course gravel materials. This implies that in terms of grading, the material is generally good. Many of the materials are of quartzite origin and are therefore also likely to be strong, although excessively high plasticity may be indicative of a lower bearing capacity on some sites.

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
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 3.7: Range of wearing course material properties

Parameter	Measured Range
Coarseness Index (I_c)	1 - 60
Grading Modulus (GM)	0.91 – 2.14
Grading Coefficient (Gc)	0.5 – 18.5
Plasticity Index (I_p)	0 – 27
Plasticity Modulus (PM)	0 – 1242
Plasticity Product (PP)	0 – 756
Maximum Dry Density (at 95% mod AASHTO)	1810 - 2280

3.5.2 Subgrade

Grading results were obtained for the samples of the subgrade from 30 of the 31 sites as listed in Table 3.8.

A plot of the grading curves from these sites is illustrated in Figure 3.3 and the grading envelopes encompassing the grading curves from the sites are illustrated in Figure 3.4. As can be seen in Figure 3.3, two of the grading curves were significantly coarser

Figure 3.3
Particle size distribution of the subgrade

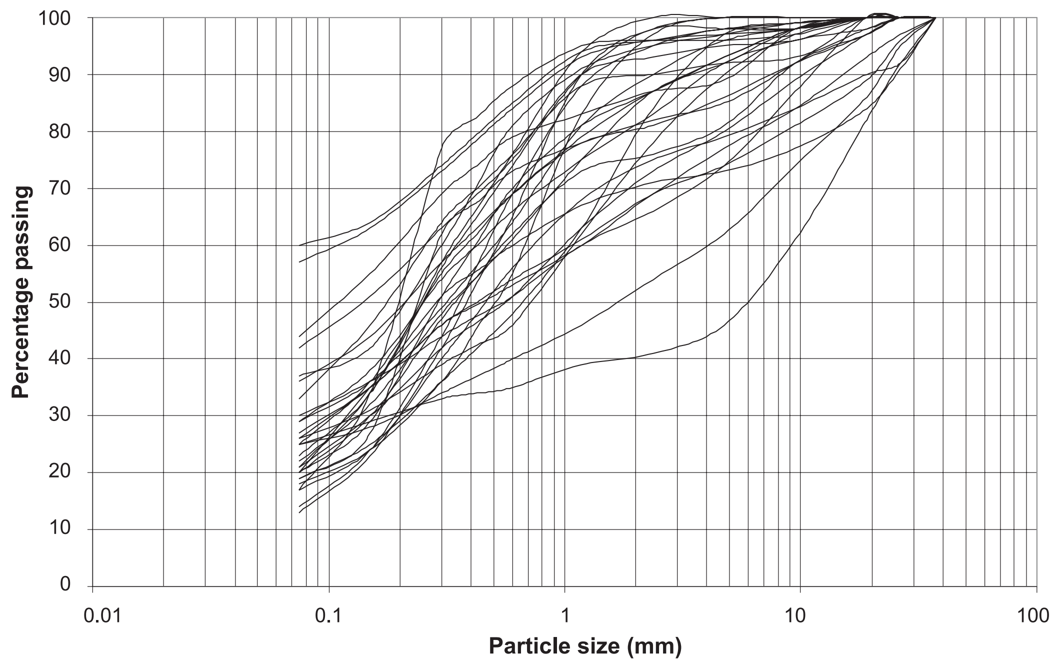


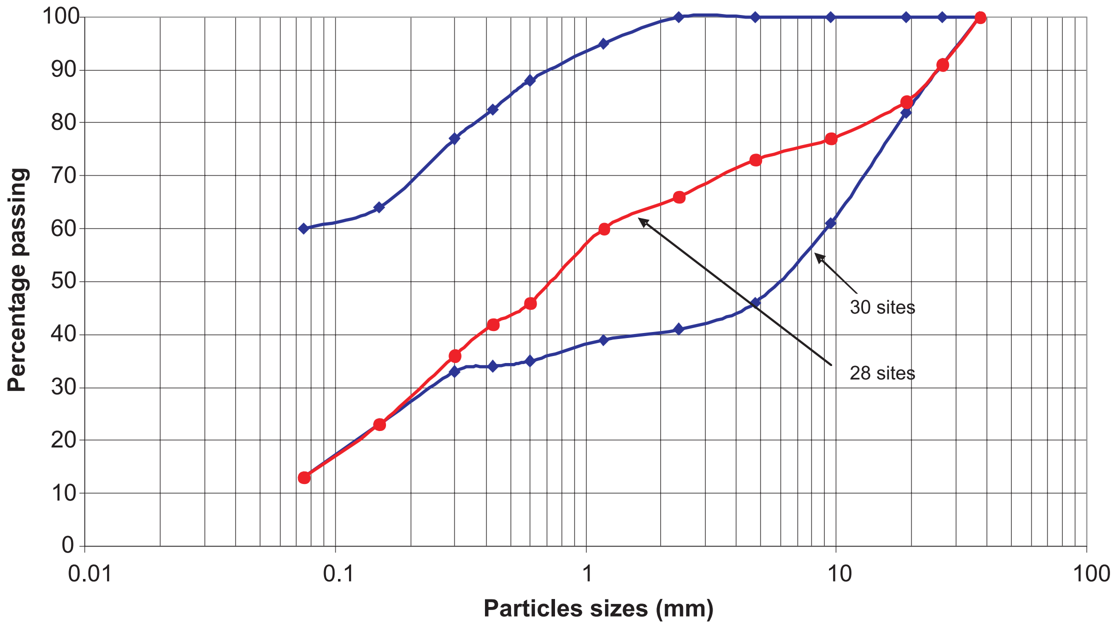
Table 3.8: Grading of the subgrade material

Site	Percentage Passing (mm sieve)											
	37.5	26.5	19	9.5	4.75	2.36	1.18	0.6	0.425	0.3	0.15	0.075
CUMA 1	100	100	97	89	80	73	63	51	47	42	30	21
CUMA 2	100	100	100	97	90	78	62	46	43	39	31	26
KAME 1	100	92	82	61	46	41	39	35	34	33	29	25
KAME 2	100	97	93	84	77	69	60	53	50	46	35	27
KAME 3	100	96	89	81	74	69	61	54	50	46	35	26
MAMI 1	100	100	98	96	92	83	75	66	60	54	38	29
MAMI 2	100	100	97	92	81	76	73	62	55	47	30	20
MEME 1	100	100	100	98	93	87	79	69	62	55	36	25
MEME 2	100	100	100	100	100	98	90	74	65	56	34	23
MEME 3	100	100	100	98	92	87	75	61	55	48	32	22
MONE 1	100	100	100	100	100	97	83	55	46	36	24	18
MONE 2	100	100	100	99	97	84	63	48	42	36	25	19
MSMA 1	100	100	97	92	86	81	78	69	63	57	44	36
MSMA 2	100	100	100	98	93	87	81	70	61	51	29	20
NACO 1	100	100	100	96	95	93	90	75	69	63	51	42
NARA 1	100	100	100	98	98	95	88	69	57	44	24	14
NARA 2	100	91	84	77	73	71	67	60	56	51	35	20
NUKO 1	100	100	100	99	98	96	94	85	80	74	64	60
NUKO 2	100	100	100	98	96	90	81	64	57	49	36	29
PESE 1	100	100	100	98	98	98	90	65	54	42	23	13
PESE 2	100	100	100	100	100	100	94	76	67	57	34	21
SECO 1	100	100	100	98	89	87	83	79	74	69	55	44
SECO 2	100	92	90	84	79	75	68	56	48	40	24	17
SUMA 1	100	100	98	93	92	90	88	76	70	64	46	33
SUMA 2	100	100	100	99	97	95	91	81	71	60	42	37
TEPU 1	100	95	86	74	62	54	46	40	37	34	28	25
TEPU 2	100	100	100	98	97	96	93	84	79	73	63	57
TEPU 3	100	100	100	87	74	66	60	51	48	44	36	30
TOSA 1	100	100	100	92	85	82	78	73	68	62	27	19
TOSA 2	100	100	100	97	96	96	95	88	83	77	34	17

Table 3.9: Material properties of the gravel wearing course

Site	Coarseness Index I_c	Dust Ratio DR	Grading Modulus GM	Grading Coefficient Gc	Shrinkage Product SP	Plastic Modulus PM	Plastic Product PP	Plastic Factor PF
CUMA 1	27	0.45	1.6	13	47	0	0	0
CUMA 2	22	0.61	1.5	9	213	340	208	390
KAME 1	59	0.74	2.0	17	306	306	225	725
KAME 2	31	0.55	1.5	14	248	347	189	486
KAME 3	31	0.52	1.6	14	50	250	130	520
MAMI 1	17	0.48	1.3	10	360	480	232	493
MAMI 2	24	0.37	1.5	13	109	0	0	0
MEME 1	13	0.40	1.3	8	62	0	0	0
MEME 2	2	0.35	1.1	1	65	0	0	0
MEME 3	13	0.40	1.4	7	0	0	0	0
MONE 1	3	0.40	1.4	1	0	0	0	0
MONE 2	16	0.45	1.6	7	42	0	0	0
MSMA 1	19	0.57	1.2	12	504	945	540	576
MSMA 2	13	0.33	1.3	8	61	0	0	0
NACO 1	7	0.61	1.0	5	0	621	378	882
NARA 1	5	0.25	1.3	3	0	0	0	0
NARA 2	29	0.36	1.5	11	56	0	0	0
NUKO 1		0.75	0.6	3	875	1590	1200	1680
NUKO 2	10	0.51	1.2	6	170	452	232	522
PESE 1	2	0.24	1.4	1	0	0	0	0
PESE 2	0	0.32	1.1	0	0	0	0	0
SECO 1	13	0.59	1.0	10	296	740	440	704
SECO 2	25	0.35	1.6	8	0	0	0	0
SUMA 1	10	0.47	1.1	7	210	0	0	0
SUMA 2	5	0.52	1.0	4	71	0	0	0
TEPU 1	46	0.68	1.8	15	222	370	250	325
TEPU 2	4	0.73	0.7	3	1099	1492	1083	1140
TEPU 3	34	0.63	1.6	16	285	380	240	420
TOSA 1	18	0.28	1.3	12	203	270	76	266
TOSA 2	4	0.21	1.0	3	83	1320	272	0

Figure 3.4
Grading envelope for the subgrade



than the rest. Therefore in Figure 3.4 a grading envelope encompassing all the results and another omitting the two course curves have been plotted.

The grading of subgrade samples is characteristic of generally fine materials. An analysis of the individual grading curves showed that about two thirds of the samples met the specifications of wearing course material. This indicates that the in-situ material could have been used as wearing course and that it may not have been necessary to import possibly inferior wearing course material and incur additional haulage costs in places where the existing subgrade is of an adequate quality for use as wearing course material.

Other material properties of the subgrade are listed in Table 3.9 and their ranges are summarised in Table 3.10.

No subgrade-related failures were noted during the study period. Thus it can be assumed that the subgrade on the test sections actually performed its functions satisfactorily as a sound foundation for the road structure.

Table 3.10: Range of the subgrade material properties

Parameter	Measured Range
Reject Index (I_R)	0
Coarseness Index (I_c)	2 – 59
Grading Modulus (GM)	0.65 – 2.0
Grading Coefficient (Gc)	0 – 17
Liquid Limit (W_L)	16 – 48
Plastic Limit (P_L)	13 – 29
Plasticity Index (I_p)	4 – 20
Linear Shrinkage (LS)	0 – 14
Shrinkage Product (SP)	0 – 1099
Plasticity Product (PP)	0 – 1200
Plasticity Modulus (PM)	0 – 1590

The values shown in Table 3.10 indicate the range of subgrade material properties in the country. The plasticity indices are within reasonable ranges and this, combined with good grading and high strength, is typical of most subgrade soils in Zimbabwe.

4. Monitoring

4.1 Schedule

The sites were monitored for a period of two years from late 2001, with each site being monitored at least three 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 C.

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.

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

4.3 Roughness

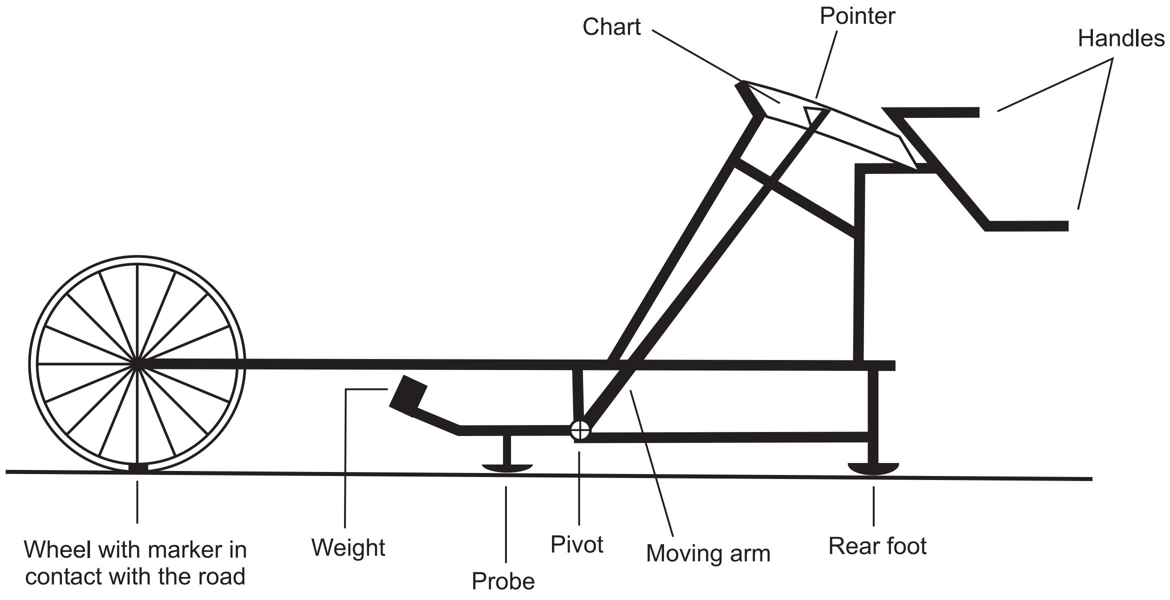
Roughness is a measure of the riding quality of the surface and can be measured using a variety of instruments. Whichever instrument is used, it is important that the measurements are standardised in the universally accepted units of International Roughness Index (IRI). A relatively inexpensive roughness measuring device is the Merlin (see Figure 4.1) and 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

Table 4.1: Monitoring Dates

Site	Survey			
	First	Second	Third	Fourth
CUMA 1	October 01	November 02	November 03	–
CUMA 2	October 01	November 02	November 03	–
KAME 1	November 01	January 03	December 03	–
KAME 2	November 01	January 03	December 03	–
KAME 3	November 01	December 02	December 03	–
MAMI 1	October 01	November 02	November 03	–
MAMI 2	October 01	November 02	November 03	–
MEME 1	October 01	November 02	November 03	–
MEME 2	October 01	November 02	November 03	–
MEME 3	October 01	November 02	November 03	–
MONE 1	November 01	November 02	October 03	–
MONE 2	November 01	November 02	October 03	–
MSMA 1	September 01	December 02	November 03	–
MSMA 2	September 01	December 02	November 03	–
NACO 1	September 01	November 02	June 03	October 03
NACO 2	September 01	November 02	June 03	October 03
NARA 1	September 01	November 02	June 03	October 03
NARA 2	September 01	November 02	June 03	October 03
NUKO 1	September 01	November 02	June 03	December 03
NUKO 2	September 01	November 02	June 03	December 03
PESE 1	September 01	December 02	November 03	–
PESE 2	September 01	December 02	November 03	–
SECO 1	November 01	November 02	October 03	–
SECO 2	November 01	November 02	October 03	–
SUMA 1	October 01	November 02	November 03	–
SUMA 2	October 01	November 02	November 03	–
TEPU 1	October 01	December 02	December 03	–
TEPU 2	October 01	December 02	December 03	–
TEPU 3	October 01	December 02	December 03	–
TOSA 1	September 01	December 02	December 03	–
TOSA 2	September 01	December 02	December 03	–

Figure 4.1
MERLIN roughness measuring device



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.

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 B, 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.

Figure 4.2
Merlin probe assembly

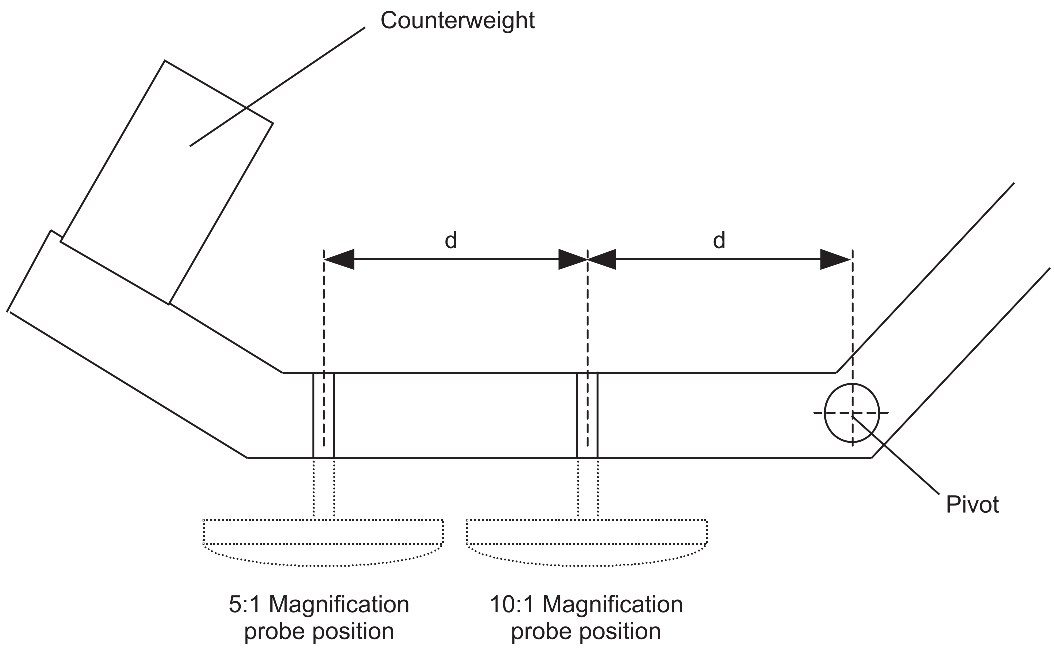


Table 4.2: Visual condition codes

	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

The deterioration of unsealed roads is governed by the behaviour of the surfacing material and the roadbed under the combined actions of traffic and the environment. As the surfacing comprises a natural material, it is usually permeable and thus material properties, rainfall and surface drainage influence the performance of the surfacing under traffic.

For unsealed roads with generally adequate material specifications and pavement thickness, the principal modes of distress are roughness and gravel loss. Roughness increases over time under the actions of traffic and environment and is defined in units of a standard roughness scale such as IRI. Gravel loss from the surfacing occurs under the actions of traffic (through whip-off of stones and dust loss) and through erosion by water and wind, and is defined by the change in average thickness of the surfacing material over time

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 100,000 profile heights from the 3 to 4 surveys conducted on the 31 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 G for all the cross-sections on each site. These plots enabled the locations of the carriageway, shoulders, 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 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 over the carriageway between surveys on all the sites are summarised in Table 5.1.

Table 5.1: Gravel loss between surveys

Site	Carriageway Gravel Loss in mm			
	First to Second	Second to Third	Third to Fourth	Average
CUMA 1	9.7	6.6		8.1
CUMA 2	18.3	10.2		14.2
KAME 1	3.0	11.3		7.2
KAME 2	2.0	4.7		3.3
KAME 3	3.2	2.3		2.7
MAMI 1	5.5	11.1		8.3
MAMI 2	6.5	12.7		9.6
MEME 1	6.2	13.9		10.1
MEME 2	2.7	5.6		4.1
MEME 3	8.6	4.4		6.5
MONE 1	19.9	14.4		17.2
MONE 2	13.2	11.2		12.2
MSMA 1	13.2	-6.8		3.2
MSMA 2	10.1	-1.1		4.5
NACO 1	9.8	-19.6		-4.9
NACO 2	0.7	7.0	1.8	3.2
NARA 1	6.1	7.8	1.3	5.1
NARA 2	6.1	10.6	8.5	8.4
NUKO 1	16.8	9.2	2.6	9.5
NUKO 2	11.6	15.5	14.1	13.7
PESE 1	13.1	9.9		11.5
PESE 2	3.6	2.4		3.0
SECO 1	9.7	8.5		9.1
SECO 2	11.0	7.5		9.2
SUMA 1	8.5	12.0		10.2
SUMA 2	12.9	10.1		11.5
TEPU 1	12.2	5.6		8.9
TEPU 2	10.0	3.6		6.8
TEPU 3	1.6	5.8		3.7
TOSA 1	2.3	7.9		5.1
TOSA 2	2.7	3.7		3.2

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 average gravel loss over the carriageway observed for all the sites was 8 mm/year.

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 typical rates of gravel loss have been plotted against traffic in Figure 5.1. Although there appears to be a small rise in the rate of gravel loss with increasing levels of traffic, the large scatter around the trend indicates that it is rather insignificant for the level of traffic observed on most of the sites (ADT < 100).

The sites were then split into two groups according to their plasticity; one group with high plasticity (average PP = 480) and the other group with low plasticity (average PP = 65). These two groups of sites have been distinguished in Figure 5.2. This plot illustrates that there appears to be an insignificant relationship between the rates of gravel loss and traffic or plasticity.

For material types such as calcretes, laterites, sand stone and quartz with iron oxide, plasticity is less important than for other material types because of their inherent cementitious properties. The apparent insignificance of plasticity on the performance of the roads could be due to the fact that the wearing courses on a substantial number of the sites had these material types (see Table 5.3). Therefore to determine the influence of the material properties in more detail, the performance of each site was examined separately.

Figure 5.1
Rates of gravel loss vs ADT

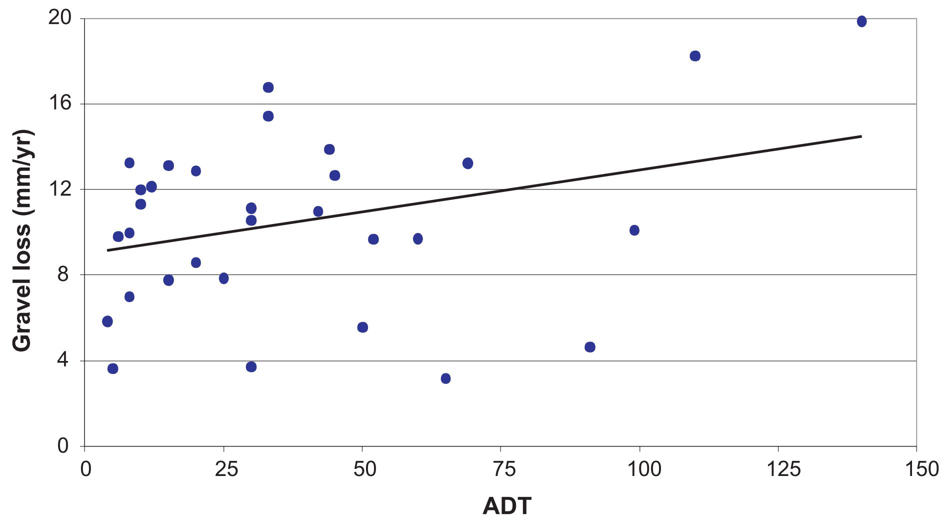
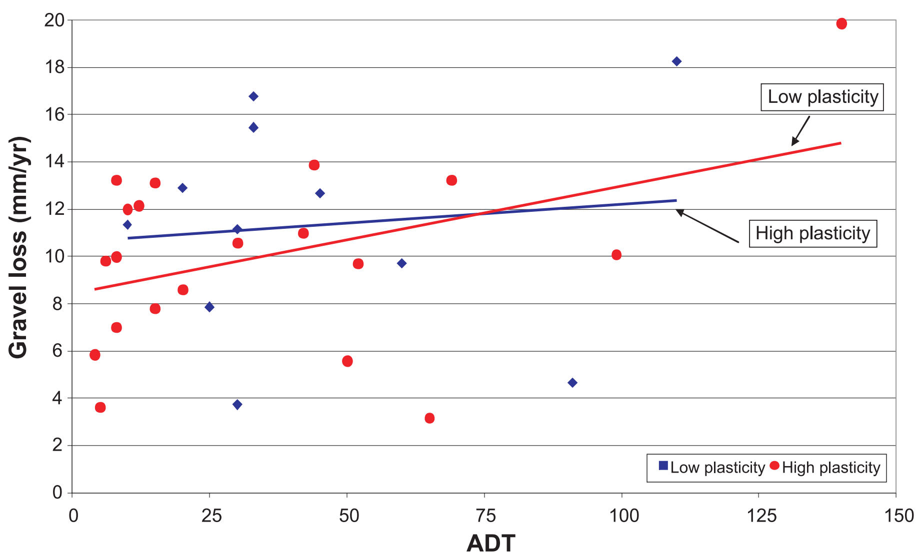


Figure 5.2
Rates of gravel loss by plasticity groups



The next stage of the analysis involved standardising the traffic on each site in order to examine the effect of the material properties on the rates of gravel loss. The typical rates of gravel loss for each site (as illustrated in Figure 5.1) 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, 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 be taken into account. The observations on the sites indicated that 3 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 5.2.

Table 5.2: Performance criteria

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

Based on these performance criteria, 14 sites were classified as good, 9 sites were classified as moderate and 8 sites as poor. The details are given in Table 5.3.

The following observations have been made on the performance of the sites.

Material type

Most of the sites that performed poorly consisted of quartzitic wearing course material, which has high strength but often lacks the cohesive properties essential for binding the material together. Thus the strength benefits, which reduce abrasion and gravel loss, are often offset by ravelling and increased roughness, especially if oversize material is present.

Sites with calcrete, quartz with iron oxide, laterite and sandstone performed reasonably well. This good performance could be attributed to the inherent cohesive properties of these materials. Lateritic materials and some quartzitic materials, depending on their minerality, can undergo physiological changes on compaction, which result in a dense and hard macro-structure. This cohesive structure is much more resistant to abrasion, hence lower rates of gravel loss. However, any road section constructed with poorly compacted material, whether cohesive or not, can be expected to deteriorate more rapidly than a material compacted to the required density.

Material properties

The wearing course consists of a wide range of material properties. The properties of the wearing course on each site were examined to determine how they influenced the performance of the site in terms of gravel loss.

CUMA 1 and 2 were situated in a region with a relatively wet climate and had quartz wearing courses which showed good performance. The plasticity was high with an I_p of 16 and 14, and PP equal to 663 and 462 respectively. The grading parameters show a fine material with the coarse fraction constituting 14% and 13% respectively. The good performances were likely due to the combination of these material properties. There was little evidence of erosion on the relatively low gradient.

KAME 1, 2 and 3 consisted of sandstone wearing courses. KAME 1 showed a poor performance while KAME 2 and 3 performed well. The wearing course on KAME 1 was very coarse (I_c = 60%) though the plasticity was high. The coarseness of the material seems to have been the main factor for the poor performance on KAME 1, possibly resulting in inadequate compaction with the light rollers available during construction. Finer materials of this type might be expected to perform better.

MAMI 1 and 2 had calcrete wearing courses and showed moderate and good performances respectively. The calcrete on MAMI 1 was very plastic but also very fine, and the site was situated in a dry climate. Under such circumstances the wearing course could be expected to degrade under traffic, hence the moderate performance. MAMI 2 on the contrary was well-graded with slightly more fines.

Table 5.3: Performance of the test sites

Site	Material	Perf	Adj GL mm/yr/ 100 ADT	ADT	Annual Rainfall mm/yr	Gradient (m/km)	I_p	If	I_c	GM	PP
CUMA 1	Quartz	G	14	60	1417	0.9	16	39	14	1.17	663
CUMA 2	Quartz	G	17	110	1417	15.4	14	33	13	1.31	462
KAME 1	Sandstone	P	86	10	1008	28.3	16	24	60	2.00	384
KAME 2	Sandstone	G	5	91	1008	21.1	19	30	17	1.34	570
KAME 3	Sandstone	G	3	65	1008	4.4	7	19	26	1.61	133
MAMI 1	Calcrete	M	30	30	489	24.6	27	28	28	1.54	756
MAMI 2	Calcrete	G	24	45	489	4.8	15	23	43	1.77	322
MEME 1	Laterite	M	28	44	829	1.0	NP	19	28	1.61	0
MEME 2	Laterite	G	8	50	829	12.7	NP	21	20	1.51	0
MEME 3	Laterite	M	31	20	829	17.9	7	18	31	1.77	126
MONE 1	Quartz	G	15	140	868	40.3	NP	19	1	1.32	0
MONE 2	Quartz	P	131	8	868	24.4	7	22	13	1.45	154
MSMA 1	Calcrete	G	18	69	643	1.8	7	15	30	1.67	90
MSMA 2	Calcrete	G	10	99	643	1.8	7	17	38	1.80	119
NACO 1	Quartz	P	117	6	990	58.7	NP	19	21	1.63	0
NACO 2	Quartz	M	53	8	990	41.8	NP	11	42	1.98	0
NARA 1	Quartz	M	35	15	1439	31.8	NP	7	47	2.09	0
NARA 2	Quartz	M	28	30	1439	7.5	NP	14	31	1.80	0
NUKO 1	Quartz+ Feldspar	M	45	33	2231	35.9	13	43	5	0.91	559
NUKO 2	Quartz+ Feldspar	M	41	33	2231	67.1	11	44	11	0.96	484
PESE 1	Quartz	P	71	15	629	15.2	NP	10	51	2.14	0
PESE 2	Quartz	G	16	5	629	15.9	6	13	48	2.09	78
SECO 1	Iron oxide, Quartz	G	16	52	689	1.8	13	25	39	1.74	325
SECO 2	Iron oxide, Quartz	G	22	42	690	21.8	8	22	58	2.03	176
SUMA 1	Feldspar	P	93	10	489	14.4	NP	20	15	1.46	0
SUMA 2	Feldspar	M	52	20	489	15.1	18	25	49	1.87	450
TEPU 1	Quartz+DGr	P	79	12	589	1.0	9	15	37	1.90	120
TEPU 2	Quartz	P	90	8	589	8.1	7	14	50	2.13	84
TEPU 3	Quartz+DGr	P	74	4	589	13.5	NP	12	47	2.04	0
TOSA 1	Calcrete	G	22	25	462	3.7	18	19	50	1.94	323
TOSA 2	Calcrete	G	5	30	462	2.4	18	22	37	1.71	198

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

MEME 1, 2 and 3 had laterite wearing courses with very low plasticity (I_p of 0, 0, & 7 respectively). Under normal circumstances poor performance would be expected but the sections performed reasonably well. Lateritic materials generally provide good wearing courses and the fine grading with Ifs of 19, 21 and 18 respectively probably assisted the binding process.

MONE 1 and 2 had quartzitic wearing courses and performed well and poorly respectively. MONE 1 wearing course was very fine with only 1% coarser than 2.36 mm. This means that the wearing course was a silty sand material. Even with a relatively small amount of clay present, silty sand can be compacted to produce a reasonable wearing course. MONE 2 performed poorly but it was also fairly fine with 13% coarse fraction and slightly plastic. There are no obvious reasons for the poor performance of this section which had less traffic than MONE 1.

MSMA 1 and 2 wearing courses were calcrete that were fine (I_c = 30 & 38) and slightly plastic (both I_p = 7). Calcrete materials are notoriously variable in performance as gavel wearing courses depending on the mode of formation and properties of the calcrete. However, the material on these sections performed well.

NACO 1 and 2, NARA 1 and 2, NUKO 1 and 2 and PESE 1 and 2 wearing courses were quartzitic gravel which were very fine to slightly coarse and non plastic to slightly plastic. NACO 1 and PESE 1 performed poorly, while PESE 2 performed well and the rest exhibited moderate performances. The poor performance of PESE 1 could have been caused by a combination of lack of plasticity and relatively high coarseness. There are no obvious reasons why NACO 1 performed poorly but the important factor to note is that 100% passed 4.75 mm and about 80% passed 2.36 mm sieves. So the material was basically sand with about 20% passing the 0.075 mm sieve. However, the fines appeared to lack the required bonding properties. NUKO 1 and 2 were equally fine but slightly plastic. The plasticity possibly improved the performance on this site. The NUKO sites were in a very wet climate (rainfall > 2000 mm/year) and the environmental contribution to gravel loss could explain the reason for the poor performance on these sections.

SECO 1 and 2 wearing courses consisted of fine to well-graded quartz containing iron oxide. The materials were also slightly plastic (I_p = 13 & 8 respectively) and fine with a reasonable content of fines (If = 25 & 22 respectively). SECO 1 performed better than SECO 2 possibly because the latter was relatively coarse.

SUMA 1 and 2 had feldspar wearing courses and this material is similar to quartz. SUMA 1 wearing course was fine and non-plastic and performed poorly, while SUMA 2 was well-graded and coarser with high plasticity (I_p = 18 and PP = 450) and performed moderately.

TEPU 1, 2 and 3 wearing courses consisted of quartz with decomposed granite and they all performed poorly. The wearing course was relatively coarse (I_c = 37, 50 & 47 respectively) and non-plastic to slightly plastic (I_p = 9, 7 & 0 and PP = 120, 84 & 0 respectively). Decomposed granite particles tend to be weak and crumble under load. This appears to be the reason for the performance of these sites.

TOSA 1 and 2 wearing courses consisted of well-graded calcrete with high plasticity (both I_p = 18, PP = 323 & 198 and I_c = 50% & 37% respectively). The good performance of these two sites could be as a result of high plasticity, despite the material being coarse in the case of TOSA 1.

The assessment of the performance of each site indicated that the performance of the wearing course was influenced by the material type, plasticity and grading. Materials with good bonding properties such as laterite and some sandstones and quartz gravels containing compounds such as ferric and alumino oxides perform better than materials such as pure quartz or silica.

Table 5.4: Observed roughness during each survey

Site	First Survey	Second Survey	Third Survey	Fourth Survey
CUMA 1	8.4	10.5	11.0	–
CUMA 2	10.7	13.5	13.8	–
KAME 1	13.6	6.9	7.6	–
KAME 2	10.3	9.7	5.6	–
KAME 3	12.1	14.5	11.6	–
MAMI 1	8.6	7.1	6.7	–
MAMI 2	8.4	8.9	9.0	–
MEME1	8.3	11.7	10.1	–
MEME 2	12.4	9.7	10.3	–
MEME 3	11.9	12.0	8.8	–
MONE 1	10.0	8.9	10.5	–
MONE 2	7.7	6.6	8.7	–
MSMA 1	10.2	9.8	7.9	–
MSMA 2	8.1	7.3	6.2	–
NACO 1	6.5	9.1	8.4	7.1
NACO 2	8.1	6.8	8.5	9.2
NARA 1	8.6	8.4	6.1	9.0
NARA 2	11.7	10.9	11.8	11.9
NUKO 1	12.0	14.0	13.1	13.2
NUKO 2	8.3	12.3	10.9	11.5
PESE 1	8.8	10.3	13.1	–
PESE 2	8.8	8.0	8.6	–
SECO 1	9.1	9.1	11.5	–
SECO 2	12.3	12.6	8.7	–
SUMA 1	7.4	7.1	7.4	–
SUMA 2	9.5	9.3	10.4	–
TEPU 1	7.1	8.7	10.3	–
TEPU 2	11.6	11.8	12.5	–
TEPU 3	10.5	10.2	9.6	–
TOSA 1	7.9	8.6	7.3	–
TOSA 2	7.5	6.9	12.1	–

In general, high plasticity in wearing course material increases bonding and reduces the rate of gravel loss, although if too high it can cause slippery conditions in wet weather. However, the effects of plasticity on performance differ in different materials.

High coarseness negatively affects the performance of the wearing course and therefore the grading properties is important in the selection of suitable material for wearing course.

5.2 Roughness

The roughness measured on each site has been plotted in Appendix G and summarised in Table 5.4.

The average roughness of all the sites over the monitoring period was evaluated as 9.7 IRI, which indicates that the labour-based gravel roads were in a relatively poor condition in terms of roughness. The sites were grouped according to their traffic levels. The average roughness for each group of sites indicated that the roughness was relatively constant between all the groups as shown in Table 5.5.

Table 5.5: Roughness vs ADT

No. of Sites	ADT	IRI
13	≤ 20	9.1
9	21 – 50	10.0
5	51 – 75	10.5
4	> 75	9.5

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

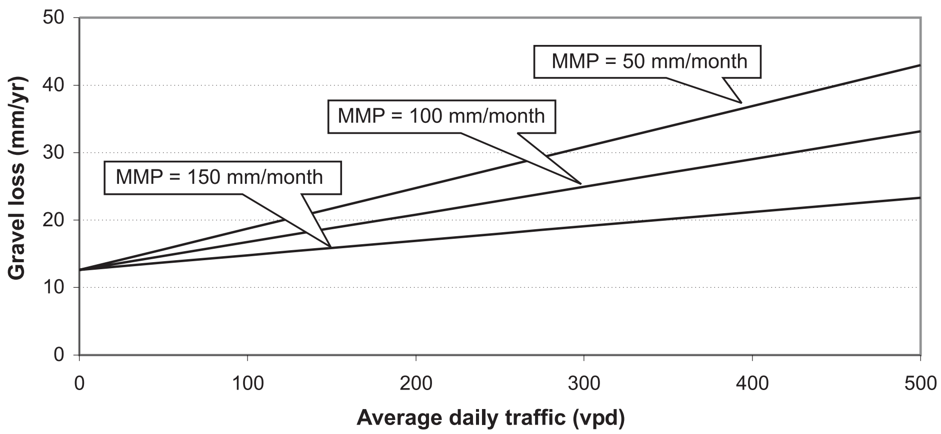


Table 6.1: Observed and HDM-4 predicted rates of gravel loss

Site	Observed Gravel Loss (mm/yr)	Default HDM-4 Predicted Gravel Loss (mm/yr)	Calibration Factor K_{gl}
CUMA 1	9.7	14.6	0.67
CUMA 2	18.3	17.6	1.03
KAME 1	11.3	15.1	0.75
KAME 2	4.7	16.8	0.28
KAME 3	3.2	16.2	0.20
MAMI 1	11.1	14.8	0.75
MAMI 2	12.7	15.4	0.83
MEME1	13.9	15.9	0.87
MEME 2	5.6	17.1	0.33
MEME 3	8.6	14.8	0.58
MONE 1	19.9	25.2	0.78
MONE 2	13.2	14.7	0.90
MSMA 1	13.2	16.7	0.79
MSMA 2	10.1	18.6	0.55
NACO 1	9.8	17.4	0.56
NACO 2	7.0	16.2	0.44
NARA 1	7.8	16.8	0.47
NARA 2	10.6	15.2	0.70
NUKO 1	16.8	19.7	0.85
NUKO 2	15.5	25.3	0.61
PESE 1	13.1	14.4	0.91
PESE 2	3.6	13.7	0.26
SECO 1	9.7	15.4	0.62
SECO 2	11.0	16.3	0.68
SUMA 1	12.0	13.9	0.87
SUMA 2	12.9	14.3	0.91
TEPU 1	12.2	13.4	0.92
TEPU 2	10.0	13.5	0.75
TEPU 3	5.8	13.5	0.43
TOSA 1	7.9	14.1	0.56
TOSA 2	3.7	14.3	0.26
Average	**10.5**	**16.2**	**0.65**

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

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.

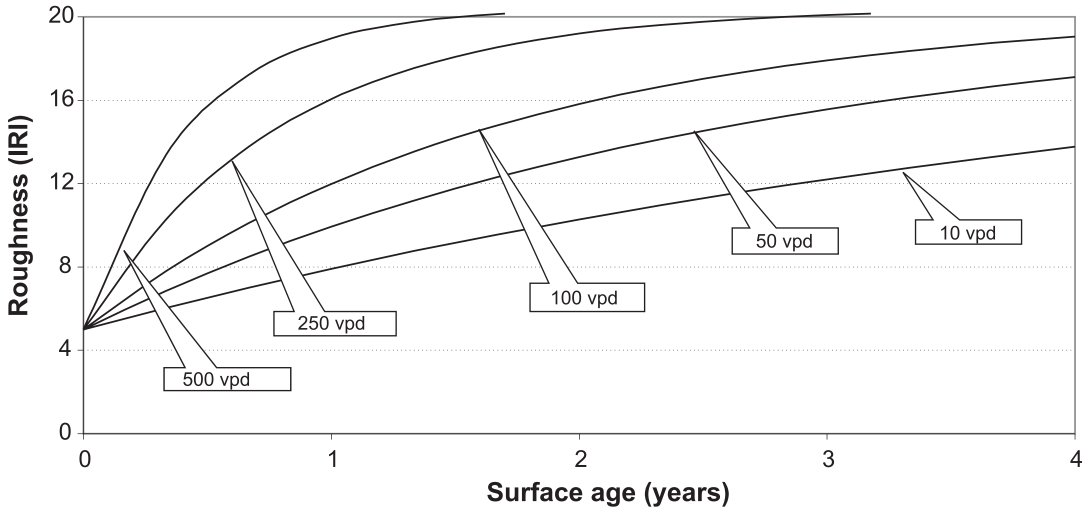
The observed rates of gravel loss on each site are listed in Table 6.1 together with the HDM-4 default predicted rates. Also listed in Table 21 are the values for the HDM-4 calibration factor K_{gl} used to adjust the predicted rates to match the observed rates of gravel loss for each site.

The results in Table 6.1 show that HDM-4 consistently predicted rates of gravel loss that were higher than those observed on the sites. The average value of the gravel loss calibration factor K_{gl} was 0.65, which indicates that on average the amount of gravel lost on these labour-based roads was 65% of the amount predicted by HDM-4.

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.

Figure 6.2
Roughness progressions on unsealed roads with no maintenance



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)] where $0 < b < 1$

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 6.2, 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.

Table 6.2: HDM-4 roughness calibration factors

Site	Construction Year	ADT	Observed Roughness IRI	Calibration Factor K_c
CUMA 1	1994	60	9.9	0.25
CUMA 2	1994	110	12.7	0.6
KAME 1	1998	10	9.3	1.0
KAME 2	1998	91	8.5	0.25
KAME 3	1998	65	12.7	1.0
MAMI 1	1997	30	7.7	0.05
MAMI 2	1997	45	8.8	0.25
MEME1	1997	44	10.1	0.9
MEME 2	1997	50	10.8	0.7
MEME 3	1997	20	10.9	0.75
MONE 1	1991	140	9.8	0.45
MONE 2	1991	8	7.7	0.6
MSMA 1	2000	69	9.3	0.75
MSMA 2	2000	99	7.2	0.35
NACO 1	2000	6	7.7	2.0
NACO 2	2000	8	8.2	1.4
NARA 1	1998	15	8.0	1.5
NARA 2	1998	30	11.6	0.8
NUKO 1	2000	33	13.1	6.0
NUKO 2	2000	33	10.8	6.0
PESE 1	2000	15	10.7	2.1
PESE 2	2000	5	8.5	1.4
SECO 1	1995	52	9.9	0.45
SECO 2	1995	42	11.2	0.7
SUMA 1	1999	10	7.3	0.7
SUMA 2	1999	20	9.7	0.75
TEPU 1	2000	12	8.7	1.0
TEPU 2	2000	8	12.0	2.6
TEPU 3	2000	4	10.1	2.5
TOSA 1	1994	25	7.9	0.45
TOSA 2	1994	30	8.8	0.2
Average				**0.8**[1]

Note: [1] – excludes NUKO 1 and NUKO 2

The two NUKO sites belong to the group of sites that were constructed in 2000 (i.e. 2 years prior to the start of the monitoring), yet their roughness levels over the monitoring period were unusually high (13 and 11 IRI) for roads of this age. Alternatively, the information gathered regarding the construction year was incorrect. The very high roughness progression over a short period of time resulted in values of K_c that were more than double those for any of the other sites. Therefore the K_c values for these two sites were not considered in the average.

The average value of K_c for the remaining 29 sites was 0.9. This indicates that the rates of roughness progression observed on the sites were, on average, slightly lower than that predicted by HDM-4. It is also evident that the lowest trafficked sites (ADT < 20) had the highest values of K_c, which indicates that as traffic increases the progression of roughness needs to be reduced by the use of smaller values of K_c. This implies that the effect of increased traffic levels in HDM-4 is much higher than observed on the sites in Zimbabwe.

7. Life-Cycle Cost Methodology

As mentioned in Section 1.4, this study in Zimbabwe 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 performances of the sites in these studies have 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 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.

Figure 7.1
Material quality zones

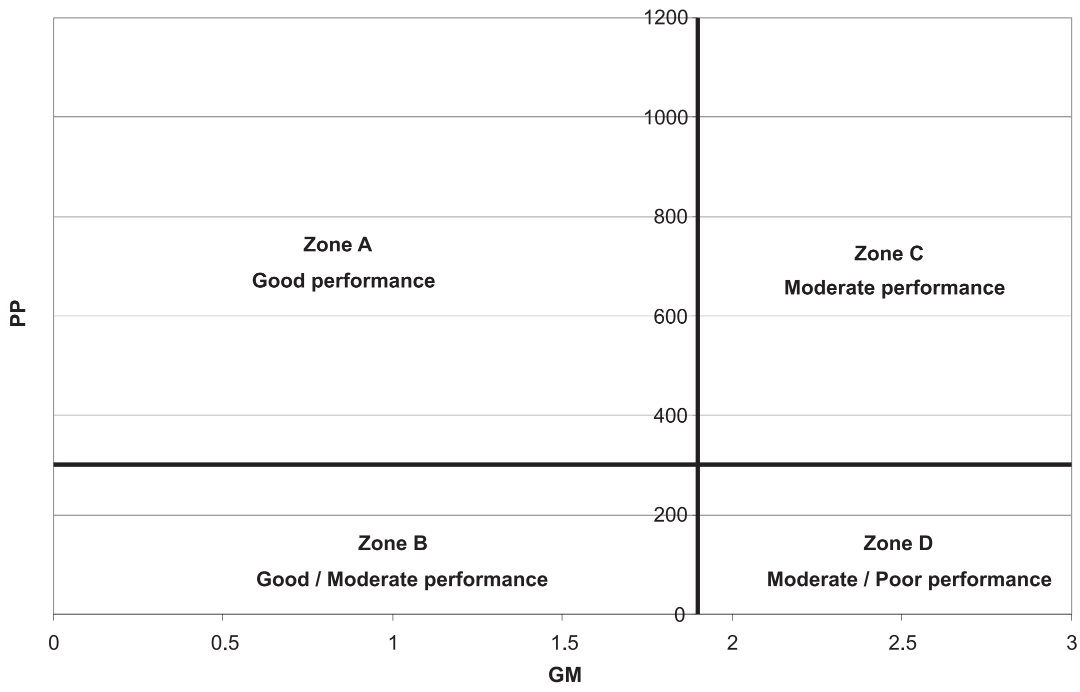


Figure 7.2
Regravelling frequency

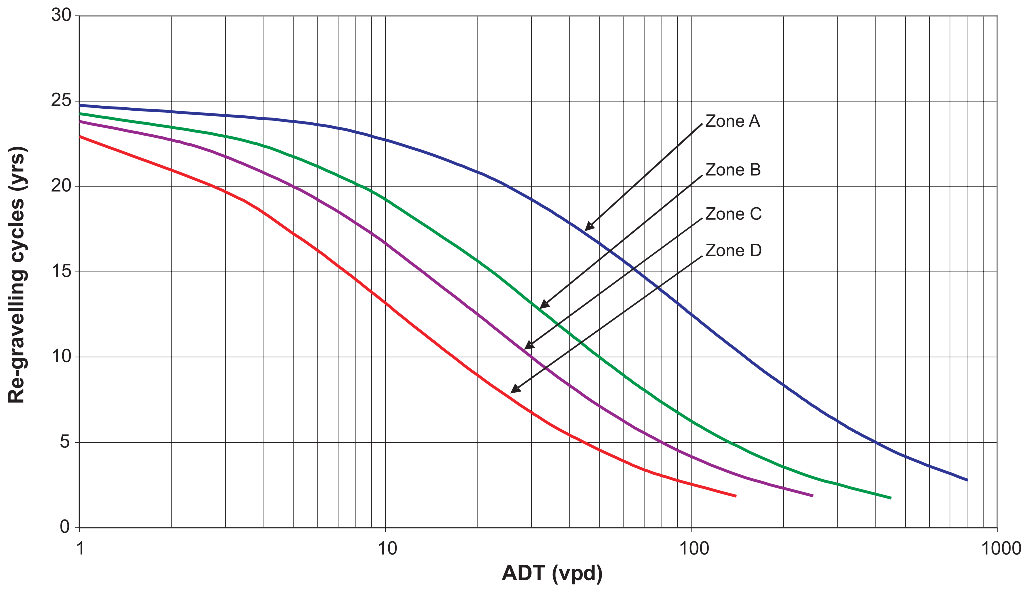
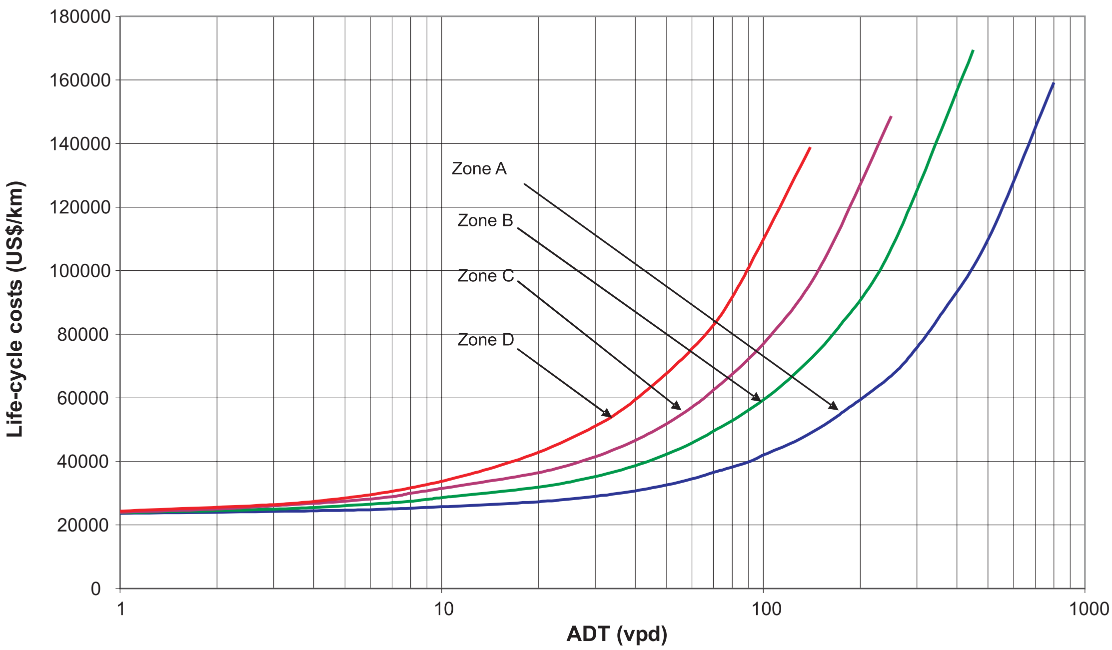


Figure 7.3
Example of life-cycle costs



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 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

The average rate of gravel loss observed on the test sites was 10 mm/year. The influence of traffic appeared to be minor for the range of traffic observed on most of the sites (ADT < 100). However, the rates of gravel loss on the two highest trafficked sites (ADT > 100) were approximately double that of the average for the remainder of the sites. The effect of the plasticity of the gravel wearing course appeared to be insignificant when considered as the sole material properties factor.

The performance of each site was categorised as either 'good', 'mediocre' or 'poor' and each site's performance was examined in detail. Sites with wearing course materials constructed from calcrete, laterite, sandstone and quartz, which sometimes contain minerals that assist the bonding process, performed better than materials such as pure quartz or silica for which plasticity appears to be a more important factor on performance in terms of gravel loss.

High coarseness negatively affects the performance of the wearing course, indicating that the grading of the wearing course is also important.

A comparison of the observed rates of gravel loss with the rates predicted by HDM-4 indicated that, on average, the amount of gravel lost on these labour-based roads was 65% of the amount predicted by HDM-4, giving an average value of 0.65 for the gravel loss calibration factor K_{gl}.

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 slightly lower than the levels predicted by HDM-4, with an average value of 0.9 for the roughness calibration factor K_c. The highest values of K_c were assigned to the low trafficked sites (ADT < 20). This indicates that the effect of increasing traffic in HDM-4 is much higher than observed in Zimbabwe.

8.2 Material specifications

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

Table 8.1: Grading specifications for gravel wearing courses

Sieve Size (mm)	Percentage Passing
37.5	100
26.5	100
19	73 – 100
9.5	50 – 80
4.75	34 – 65
2.36	23 – 52
0.6	16 – 33
0.425	15 – 30
0.3	14 – 27
0.15	10 – 23
0.075	4 – 15

The grading and plasticity specifications are:

Grading Modulus	$1.5 \leq GM \leq 2.5$
Plasticity	$10 \leq I_p \leq 15$

Table 8.2: Grading envelopes of monitored sites

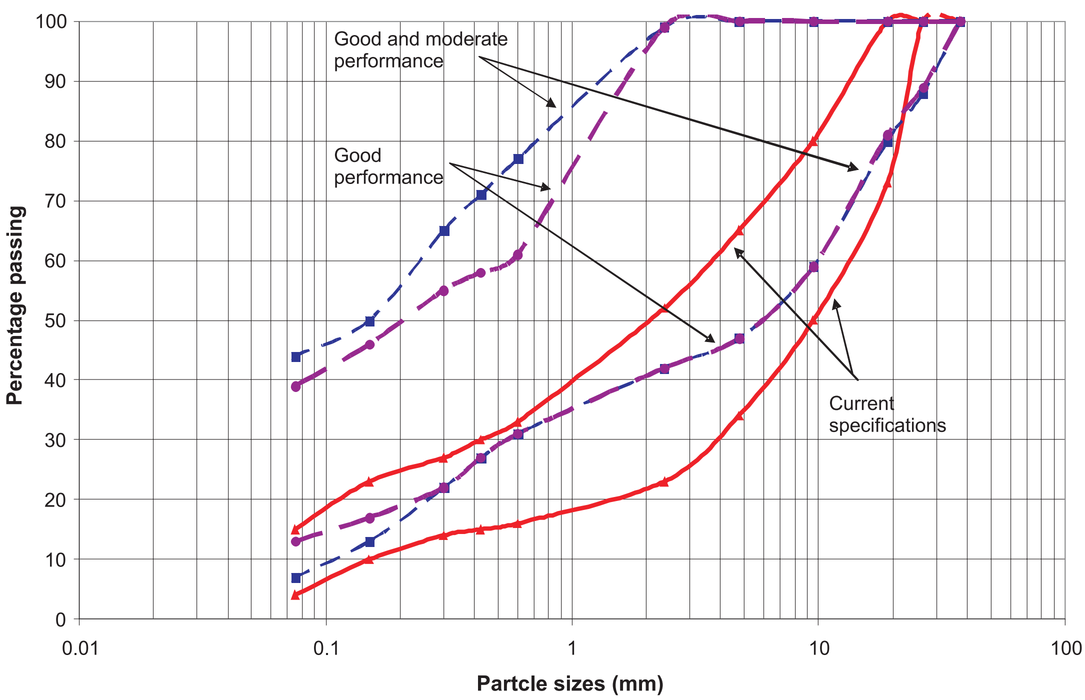
Sieve Size (mm)	Percentage Passing (by weight)	
	Good and Moderate Performance	Good Performance
37.5	100	100
26.5	88 – 100	89 – 100
19	80 – 100	81 – 100
9.5	59 – 100	59 – 100
4.75	47 – 100	47 – 100
2.36	42 – 99	42 – 99
1.18	31 – 77	31 – 61
0.6	27 – 71	27 – 58
0.425	22 – 65	22 – 55
0.15	13 – 50	46 – 17
0.075	7 – 44	13 – 39

The analysis of the performance of the test sites, as reported in Section 5.1.2, can be used to modify these material specifications. Sites were categorised according to their performance as either 'good', 'moderate' or 'poor'. The grading envelopes for the sites that exhibited a good performance and those that exhibited either a good or moderate performance are listed in Table 8.2.

The grading envelopes listed in Table 8.2 and the current specifications listed in Table 8.1 have been plotted in Figure 8.1.

The plot of the grading envelopes indicates that there is scope for widening the existing grading specifications in order to encompass finer materials. In order to increase the level of confidence in the widened specification limits, only materials that were classified as having a 'good' performance were considered. Combining

Figure 8.1
Comparison of grading envelopes



the 'good' grading envelope with the existing specifications produced a new specification for the wearing course grading which is recommended for Zimbabwe. The recommended grading envelope is given in Table 8.3 and plotted in Figure 8.2.

Table 8.3: Recommended new grading for gravel wearing courses

Sieve Size (mm)	Percentage Passing
37.5	100
26.5	88 – 100
19	73 – 100
9.5	50 – 100
4.75	34 – 100
2.36	23 – 99
0.6	16 – 61
0.425	15 – 58
0.3	14 – 55
0.15	10 – 46
0.075	4 – 39

This means that the Grading Modulus (GM) limits can be relaxed to:

Grading Modulus $1.0 \le GM \le 2.6$

Similarly the plasticity specifications can be relaxed based on the examination of the performance of the test sites. The results show that the I_p of the wearing course can be increased from 15 to 20. The upper limit can be further increased from 20 to 27 but on condition that the PP does not exceed 800. The lower limits can also be reduced from the current specification of 10 to 5 on condition that the coarseness index is ≤ 30. This means that the fine fraction, i.e. percentage passing the 2.36 mm sieve, should be ≤ 70%.

The recommended new specifications for the plasticity of gravel wearing course are given in Table 8.4. The acceptable limits are based on the analysis of the sites that had a 'good' performance. The conditional acceptable limits are for specific conditions as noted in Table 8.4.

Figure 8.2
Recommended new grading envelope

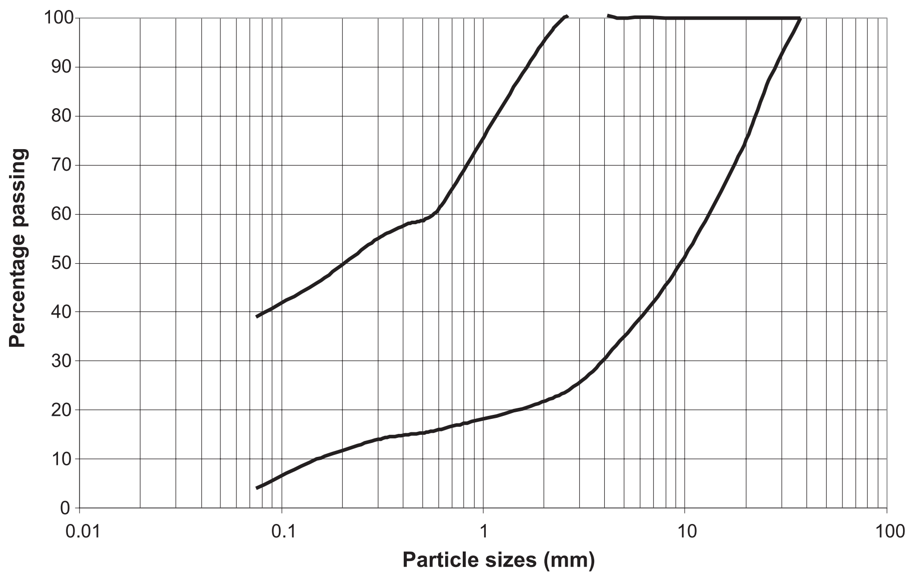


Table 8.4: Recommended specifications for plasticity of gravel wearing course

Parameter	Acceptable	Conditionally Acceptable	Reject
I_p	10 – 20	5 – 9[1] 20 – 27[2]	< 5 > 27
PP	300 – 800	80 – 300[1] 800 – 1000[3]	< 80 >1000

Notes: [1] - $I_p \leq 30$ and ADT < 20
[2] - PP < 700 and rainfall < 700 mm/year
[3] - Rainfall < 700 mm/year

Several sites with wearing course that was non-plastic (I_p = 0) performed well. However, it is not appropriate at this moment to recommend the use of non-plastic material because factors such as the mineralogical composition of the constituents of the wearing course become predominant and this is likely to be difficult to determine on site. It is recommended that non-plastic or slightly plastic material is used as wearing course only on condition that there are no better materials available, the road is lowly trafficked (ADT < 20) and $I_c \leq 30$.

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 Uganda) 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.