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

Ghana Country Report

Department for International Development

Increased Application of Labour-Based Methods Through Appropriate Engineering Standards

Ghana Country Report

ILO

Increased application of labour-based methods through appropriate engineering standards. Ghana country report
Harare, International Labour Office, 2006

ISBN: 92-2-118863-9 and 978-92-2-118863-6 (print)

ISBN : 92-2-118864-7 and 978-92-2-118864-3 (web pdf)

ILO publications can be obtained through major booksellers or ILO local offices in many countries, or direct from ILO Publications, International Labour Office, CH-1211 Geneva 22, Switzerland. Catalogues or lists of new publications are available free of charge from the above address, or by email: pubvente@ilo.org or asist@ilo.org

Visit our website: www.ilo.org/publns and www.ilo.org/asist

Cover design and typeset by Fontline International

Printed in Mauritius by Precigraph

Contents

Tables

Figures

Acknowledgements

The work described in this report was funded by the Department for International Development (DFID), UK. The work was carried out by the International Group of TRL Limited in partnership with the International Labour Organization/Advisory Support, Information Services and Training (ILO/ASIST).

The overall management of the project was undertaken by Mr Tony Greening (TRL project manager) in cooperation with Mr Dejene Sahle from ILO/ASIST. The TRL project team comprised Dr Greg Morosiuk, Mr Kenneth Mukura and Mr Nick Elsworth.

The project team would like to acknowledge the cooperation and assistance of the Ghana Department of Feeder Roads (Director Mr Martin hMensa) and the Ghana Highway Authority (Mr Eric Oduro-Konadu) for providing staff to manage and undertake the field work. The work was carried out by Mr Augustine Kuuire (research engineer) under the leadership of Mr Bonne Acquah (project team leader).

The authors of this report are grateful to Dr John Rolt (TRL), who carried out the quality review and auditing of the report.

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 Ghana. The 24 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 main conclusions of the study were:

- Different rates of gravel loss relating to material plasticity were observed. These were lower on materials with higher values of plasticity (I_p) with an average loss of 15 mm per year for traffic levels of approximately 100 vpd. Materials with low I_p lost 40 mm per year at this level of traffic.
- The average gravel loss for all the sections was 50% higher than the predicted values in HDM and up to 2.5 times the HDM value for materials with low I_p.
- The results indicate that a wider grading envelope could be adopted for the wearing course with Grading Modulus (GM) in the range 1.13 < GM <2.40

- Roughness values were generally 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 Uganda 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 Ghana 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 Ghana. These activities include the selection of test sites that are typical of labour-based roads in Ghana, 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 Uganda 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 Ghana.

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 Ghana.

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 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 Ghana 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

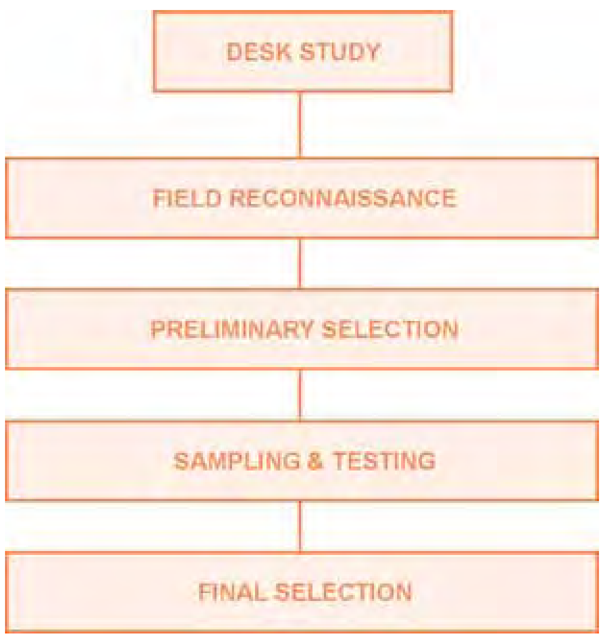


Figure 2.2
Administrative regions

Table 2.1: Length of the trunk road network

Region	Length (km)			
	Surface Treatment	Asphaltic Concrete	Unsealed	Total
Greater Accra	151	182	45	378
Eastern	944	73	346	1363
Central	535	143	220	898
Western	595	252	667	1514
Ashanti	560	350	599	1509
Brong Ahafo	601	239	974	1814
Northern	468	288	1688	2444
Volta	738	0	540	1278
Upper East	125	70	293	488
Upper West	157	0	806	963
Total	**4874**	**1597**	**6178**	**12651**

2.2 Desk study

2.2.1 Administrative regions

Ghana is located on West Africa's Gulf of Guinea, only a few degrees north of the Equator. Figure 2.2 shows the administrative regions of Ghana.

2.2.2 Road network

The length of the trunk road network in each region is detailed in Table 2.1 and the length of the feeder road network is given in Table 2.2.

Table 2.2: Length of the feeder road network

Region	Length (km)		
	Paved	Unpaved	Total
Greater Accra	156	678	834
Eastern	311	2839	3150
Central	368	2576	2944
Western	21	4054	4075
Ashanti	129	5821	5950
Brong Ahafo	48	5681	5729
Northern	34	3953	3987
Volta	116	2929	3035
Upper East	7	1011	1018
Upper West	6	1856	1862
Total	**1196**	**31398**	**32594**

In the desk study, lengths of gravel road that had been constructed, or were under construction, by labour-based methods were identified as possible locations for the test sites.

2.2.3 Terrain

Almost half of the country lies less than 152 m (500 ft) above sea level, and the highest point is 883 m (2,900 ft). The 537 km coastline is mostly a low, sandy shore backed by plains and scrub and intersected by several rivers and streams, most of which are navigable only by canoe. A tropical rain forest belt, broken by heavily forested hills and many streams and rivers, extends northward from the shore, near the Côte d'Ivoire frontier. North of this belt, the country varies from 91 to 396 m (300 - 1,300 ft) above sea level and is covered by low bush, park-like savannah, and grassy plains.

2.2.4 Climate

The climate is tropical but temperatures vary with season and elevation. The eastern coastal belt is warm and comparatively dry; the southwest corner, hot and humid; and the north, hot and dry. There are two distinct rainy seasons; in the south May-June and August-September; in the north, the rainy seasons tend to merge. Annual rainfall in the coastal zone averages 830 mm. The Harmattan, a dry desert wind, blows from the northeast from December to March, lowering the humidity and creating hot days and cool nights in the north. In the south the effects of the Harmattan are felt in January. In most areas the highest temperatures occur in March, the lowest in August.

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 2.3.

Table 2.3: 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 were 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

Table 2.4: Roads inspected during field reconnaissance

Region	No. of Roads
Central Region	2
Western Region	24
Brog Ahafo Region	11
Northern Region	12
North Western Region	7
North Eastern Region	5
Volta Region	10
Eastern Region	10
Total	81

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.

The team visited a total of 81 roads from 8 regions, as summarised in Table 2.4. Descriptions of the suitability of each road for inclusion as a monitoring section are detailed in Appendix A. Roads identified for improvement during the study period of two years were not considered.

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

Table 2.5: Test sites selected for monitoring

No.	Road Name	Site Code	Climate	Material
1	Assin Ayaasi – Assin Kruwa	AIAA	Moist sub-humid	Fine lateritic gravel
2	Abrewanko J'tn – Abrewanko	ANAO	Dry sub-humid to semi-arid	Lateritic gravel
3	Ayomso – Awia Awia	AOAA	Dry sub-humid	Brown lateritic gravel
4	Asuotiano – Dorma Akwamu	AODU	Dry sub-humid	Stoney gravel
5	Asawinso – Kojokrom	AOKM	Moist sub-humid	Fine lateritic gravel
6	Bulenga – Chaggu	BACU	Semi-arid	Fine lateritic gravel
7	Bonakye – Potimbo	BEPO	Semi-arid	
8	Bechem – Bremie	BMBE	Dry sub-humid	Brown coarse laterite
9	Bonzain J'tn – Bonzain	BNBN	Moist sub-humid	Fine lateritic gravel
10	Bodi J'tn – Nsawura	BNNA	Humid	Coarse lateritic gravel
11	Dimala – Bongnayili	DABI	Semi-arid	
12	Duayaw Nkwanta – Camposo	DACO	Dry sub-humid	Brown lateritic gravel
13	Dominanse – Wrakese S'tn	DEWN	Moist sub-humid	Fine lateritic gravel
14	Datano – Kokooso	DOKO	Moist sub-humid	Fine lateritic gravel
15	Forifori – Dwamena	FIDA	Semi-arid	
16	Grumani J'tn – Kpachiyili	GNKI	Semi-arid	
17	Linso – Nframakrom	LONM	Sub-humid	Sandy gravel
18	Naha – Loggu	NALU	Semi-arid	Lateritic gravel
19	Nabogu – Sung	NUSG	Semi-arid	
20	Pong Tamale – Yapalsi	PEYI	Semi-arid	
21	Samanhyia – Kyemfre	SAKE	Dry sub-humid to semi-arid	
22	Wiaga – Kpalansa	WAKA	Semi-arid	Lateritic gravel
23	Wamfie – Praprababida	WEPA	Dry sub-humid	Brown laterite gravel
24	Zebilla – Timunde	ZATE	Semi-arid	Lateritic gravel

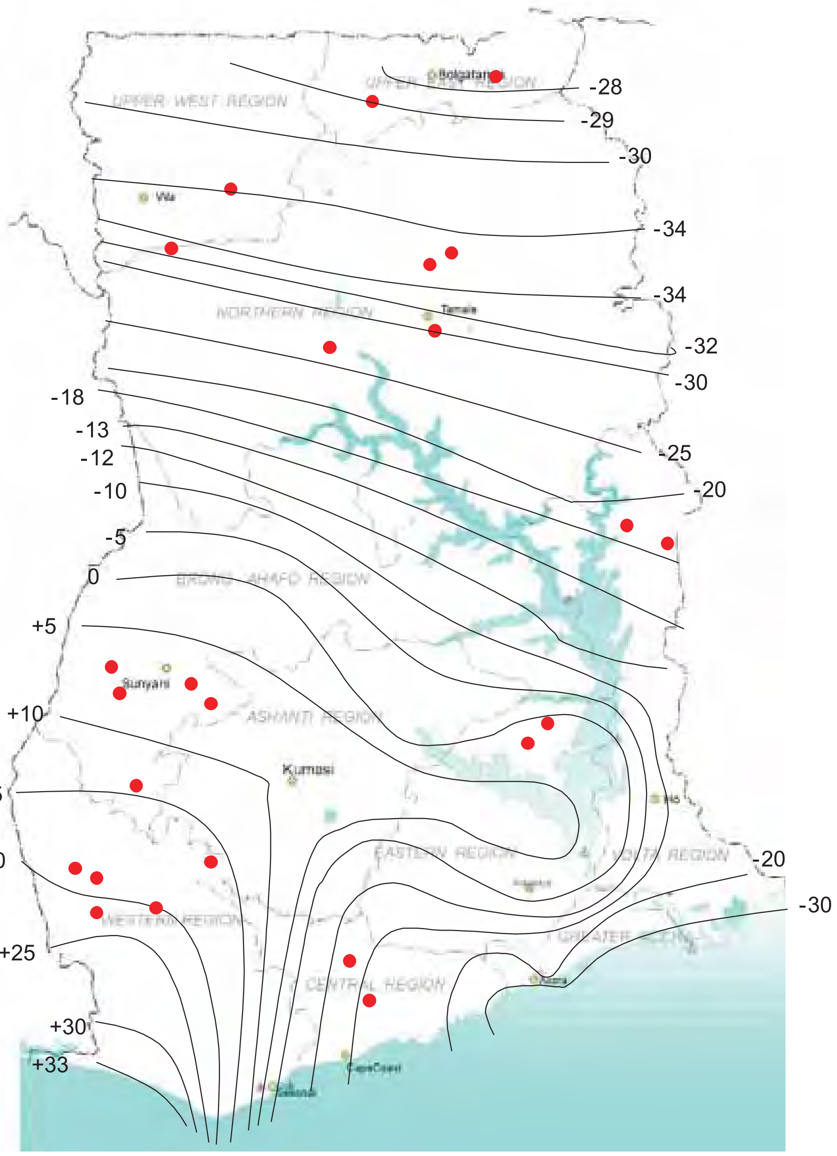
also an influential parameter on the performance of roads, the selected sites covered a range of traffic flows.

Other information considered during the final site selection process included age of the road and maintenance history. The number of sites selected was limited by the resources available to carry out the monitoring. A total of 24 sites were selected as shown in Table 2.5.

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 **N**ah**A**-**L**ogg**U** road was referred to as NALU.

These sites covered a reasonably wide range of conditions to encompass the climatic conditions, material properties and traffic levels encountered in Ghana on labour-based constructed gravel roads. Figure 2.3 shows the location of the 24 test sites across the country. The TMI contours indicate that all the climatic regions were covered by the test sites.

Figure 2.3
Location of test sites and climatic contours

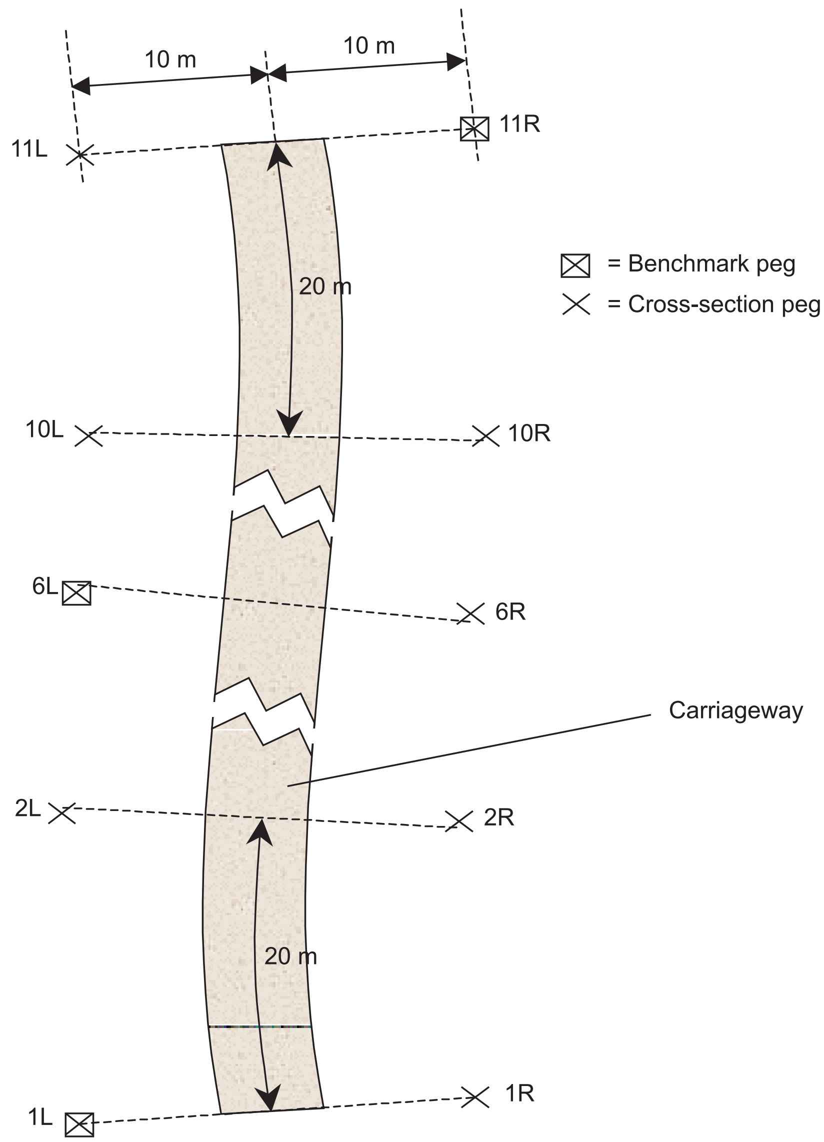


2.5 Site commissioning

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 2.4 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 2.4
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 using proformas as shown in Appendix D. 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.

Traffic volumes ranged from 3 vehicles per day (vpd) to over 400 vpd, with the Average Daily Traffic (ADT) for all the sites being 127 vpd. On 30% of the test sites, traffic flows of over 200 vpd were recorded. Traffic volumes of this level are usually considered to be high flows for unsealed roads.

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. The average rainfall over a 10-year period was used to define the rainfall on each site. The monthly and annual averages on each site are listed in Table 3.3.

Table 3.1: Road alignment

Site	Terrain	Gradient (m/km)
AIAA	Rolling	8.9
ANAO	Rolling	16.2
AOAA	Rolling	23.7
AODU	Rolling	21.8
AOKM	Rolling	21.2
BACU	Flat	19.9
BEPO	Flat	9.0
BMBE	Flat	11.9
BNBN	Rolling	9.7
BNNA	Rolling	11.4
DABI	Flat	5.7
DACO	Rolling	18.1
DEWN	Rolling	15.6
DOKO	Rolling	1.0
FIDA	Flat	9.3
GNKI	Flat	2.7
LONM	Flat	2.3
NALU	Flat	21.2
NUSG	Flat	4.2
PEYI	Flat	1.2
SAKE	Rolling	18.1
WAKA	Flat	17.2
WEPA	Flat	5.2
ZATE	Flat	3.5

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

Test Site	Light Vehicles (ADL)	Heavy Vehicles (ADH)	Total Vehicles (ADT)	Motorbikes	NMT
AIAA	68	8	76	3	16
ANAO	10	5	15	9	140
AOAA	99	135	234	16	57
AODU	277	185	462	56	157
AOKM	139	44	183	13	41
BACU	35	32	67	39	184
BEPO	5	1	6	5	226
BMBE	82	42	124	13	57
BNBN	130	95	225	17	38
BNNA	146	118	264	18	41
DABI	23	3	26	24	299
DACO	60	10	70	7	219
DEWN	81	10	91	6	13
DOKO	105	52	157	11	40
FIDA	14	2	16	5	108
GNKI	28	61	89	83	442
LONM	176	40	216	18	96
NALU	2	1	3	4	37
NUSG	124	142	266	69	426
PEYI	29	37	66	24	325
SAKE	12	5	17	3	58
WAKA	32	35	67	37	518
WEPA	202	94	296	62	476
ZATE	9	5	14	21	356

Notes: ADL – Cars, Light Goods, Minibuses
ADH – Trucks, Buses, Tractors
NMT – Non-Motorised Traffic (animal carts, bicycles, etc.)

Table 3.3: Rainfall

Site	Rainfall (mm)												
	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Annual
AIAA	9	9	25	58	154	98	53	14	38	52	20	9	539
ANAO	9	13	44	112	135	193	232	236	250	184	9	3	1415
AOAA	11	47	114	192	213	200	118	62	137	155	75	34	1357
AODU	2	32	105	115	126	164	107	77	164	150	41	21	1106
AOKM	17	57	138	186	185	222	132	112	123	171	88	54	1485
BACU	7	9	18	78	128	172	154	221	205	95	5	1	1094
BEPO	9	13	44	112	135	193	232	236	250	184	9	3	1415
BMBE	6	36	92	151	161	178	105	72	134	145	43	15	1138
BNBN	17	57	138	186	185	222	132	112	123	171	88	54	1485
BNNA	17	57	138	186	185	222	132	112	123	171	88	54	1485
DABI	6	10	35	77	145	148	163	189	210	126	7	3	1115
DACO	6	36	92	151	161	178	105	72	134	145	43	15	1138
DEWN	9	9	25	58	154	98	53	14	38	52	20	9	539
DOKO	17	57	138	186	185	222	132	112	123	171	88	54	1485
FIDA	13	27	90	85	109	208	155	118	174	104	15	22	1119
GNKI	6	10	35	77	145	148	163	189	210	126	7	3	1115
LONM	9	46	125	128	153	227	122	72	149	162	67	23	1285
NALU	7	9	18	78	128	172	154	221	205	95	5	1	1094
NUSG	6	10	35	77	145	148	163	189	210	126	7	3	1115
PEYI	6	10	35	77	145	148	163	189	210	126	7	3	1115
SAKE	11	17	50	46	60	115	72	38	74	61	10	3	556
WAKA	0	3	9	34	119	138	172	291	152	57	0	0	974
WEPA	2	32	105	115	126	164	107	77	164	150	41	21	1106
ZATE	0	0	4	55	121	141	164	227	188	72	1	0	973

Table 3.4: Construction details

Test Site	Location (Region, District)	Construction Year	Length of Road (km)	Construction Cost of Road GHC million	Cost per km GHC million (normalised to 2004)
AIAA	Central, Assin Fosu	2001	13.3	187.9	20.5
ANAO	Volta, Nkwanta	1998		n/a	
AOAA	Brong Ahafo, Asunafo	2001	14.0	240.1	25.0
AODU	Brong Ahafo, Dormaa	1996	7.0	207.4	124.6
AOKM	Western, Sefwi Wiawso	1990	12.2	102.4	164.7
BACU	Upper West, Wa	2002	3.6	70.0	24.4
BEPO	Volta, Nkwanta	2002		n/a	
BMBE	Brong Ahafo, Tano	2001	9.3	478.7	74.8
BNBN	Western, Sefwi Wiawso	1990	5.0	n/a	
BNNA	Western, Sefwi Wiawso	1988	24.4	n/a	
DABI	Northern, Tamale	2002	8.6	435.5	63.8
DACO	Brong Ahafo, Tano	2001	9.3	478.7	74.8
DEWN	Central, Assin Fosu	2001	8.4	157.5	27.3
DOKO	Western, Sefwi Wiawso	1987	25.4	n/a	
FIDA	Eastern, Afram Plains	2000*		n/a	
GNKI	Northern, Tolon/Kumbungu	2000		n/a	
LONM	Western, Sefwi Wiawso	1994	10.8	184.4	133.0
NALU	Upper West, Wa	2002	11.5	278.0	30.5
NUSG	Northern, Gushiegu/Karaga	2002	22.2	522.7	29.6
PEYI	Northern, Savelugu/Nanton	1998	38.1	368.7	26.5
SAKE	Eastern, Afram Plain	2001		n/a	
WAKA	Upper East, Bulisa	2002	10	1638.0	206.4
WEPA	Brong Ahafo, Dormaa	2000	17.2	721.8	81.4
ZATE	Upper East, Bawku West	2002	13.2	482.2	46.0

Notes: n/a – information not available
** – estimated*

3.4 Construction details

Construction details for the roads on which the test sites were located were gathered from the regional offices. Cost information was also collected where available. The construction details are summarised in Table 3.4.

The construction of these roads was carried out under the Department of Feeder Roads (DFR) 'Spot Improvements' programme. This DFR programme included a combination of routine and periodic maintenance activities on existing roads that were not motorable all year round. The rehabilitation works to ensure all-year passability consisted mainly of culvert construction, sectional re-gravelling and rectification of other spot defects.

As shown in Table 3.4, road works on the sites were carried out in different years. Therefore, in order to compare the costs of these works, the cost per km were normalised to the year 2004, based on reported inflation rates between the year of construction and 2004.

The maintenance activities selected for the roads under this DFR programme were at the discretion of the district engineer, after carrying out a road condition survey. Therefore, as can be seen from Table 3.4, the range of construction/rehabilitation costs is large.

As stated earlier, the figures in Table 3.4 were the costs incurred during the DFR 'Spot Improvements' programme to make existing roads motorable all year round. The costs associated with the construction of new roads using labour-based techniques were much higher than these figures as shown below:

- Cost of constructing a new 6 m wide labour-based feeder road – GHC 400 million/km
- Cost of rehabilitating a 6 m wide labour-based feeder road – GHC 300 million/km
- Cost of ditch reshaping of a labour-based feeder road – GHC 12.5 million/km
- Cost of grading a 6 m wide labour-based feeder road – GHC 3.9 million/km.

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. In most cases two samples of the gravel wearing course and one sample of the subgrade were taken from each site. 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. These plots show that the wearing course material on most of the sites was in a relatively tight grading envelope, apart from material from three of the sites being significantly finer than from the other 21 sites. However, the fine fraction of most samples was within acceptable limits and no oversized particles were noted.

The grading envelope encompassing the grading curves from all the sites is illustrated in Figure 3.2, with the tight envelope from the 21 sites illustrated separately from the wider envelope which encompasses all 24 sites. This grading envelope is typical of a country with good wearing course gravel in terms of particle size distribution. However, other properties, such as plasticity, also play a significant role in the overall performance of the wearing course.

The plasticity properties of the wearing course from each site are given in Table 3.6. Other material properties are listed in Table 3.7 and the ranges summarised in Table 3.8.

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: Grading of gravel wearing course

Site	Sample	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
AIAA	1	100	100	100	97	94	81	62	49	42	39	25	20
	2	100	100	100	96	93	79	61	48	41	36	23	18
ANAO	1	100	100	100	100	99	88	62	51	45	42	32	27
	2	100	100	100	100	98	82	47	37	32	31	23	20
AOAA	1	100	100	100	99	95	73	55	45	41	40	34	27
	2	100	100	100	100	99	84	59	49	44	42	35	29
AODU	1	100	100	98	92	82	61	47	41	37	35	27	22
	2	100	100	96	87	79	66	52	43	38	35	26	20
AOKM	1	100	100	100	98	93	74	56	49	45	44	35	24
	2	100	100	100	100	94	79	60	52	48	47	39	28
BACU	1	100	100	100	99	94	68	45	37	31	28	17	12
	2	100	100	98	–	96	73	42	35	31	29	22	18
BEPO	1	100	100	98	96	93	78	47	35	32	31	26	22
	2	100	98	98	95	92	74	34	25	23	22	18	15
BMBE	1	100	100	100	98	90	49	34	30	27	25	18	14
	2	100	100	99	94	85	50	36	31	28	26	18	15
BNBN	1	100	100	100	86	72	48	35	32	30	29	23	15
	2	100	100	100	99	92	65	46	36	29	26	14	8
BNNA	1	100	100	100	86	78	57	42	37	34	33	28	24
DABI	1	100	100	100	–	96	78	50	36	–	31	23	23
DACO	1	100	100	99	94	88	65	47	41	38	37	30	26
	2	100	100	95	89	81	57	39	33	31	30	23	19
DEWN	1	100	100	100	100	98	94	88	81	67	59	37	31
	2	100	100	100	98	92	73	58	52	43	37	23	19
DOKO	1	100	100	100	100	96	81	73	68	55	47	29	22
	2	100	100	100	96	91	72	57	48	41	36	24	18
FIDA	1	100	100	100	96	69	37	29	27	26	21	18	17
GNKI	1	100	100	100	–	98	85	58	43	–	34	26	25
LONM	1	100	100	100	100	100	97	87	74	54	46	25	16
	2	100	100	100	99	95	83	76	67	53	47	31	25
NALU	1	100	99	99	97	92	65	39	34	31	30	23	15
	2	100	100	100	97	90	62	31	24	21	20	15	10
NUSG	1	100	100	100	100	100	90	50	38	-	35	31	18
PEYI	1	100	100	100	100	99	93	43	23	21	20	20	18
SAKE	1	100	100	97	92	86	62	38	31	27	25	17	14
WAKA	1	100	99	98	96	91	68	41	33	28	25	18	14
	2	100	100	100	99	97	79	52	44	37	33	23	18
WEPA	1	100	100	100	93	85	68	52	46	44	43	39	36
	2	100	100	100	91	82	56	39	32	28	26	20	16
ZATE	1	100	100	97	96	95	82	57	45	38	35	26	21
	2	100	99	98	96	95	81	55	42	35	32	23	18

Figure 3.1
Particle size distribution of the gravel wearing course

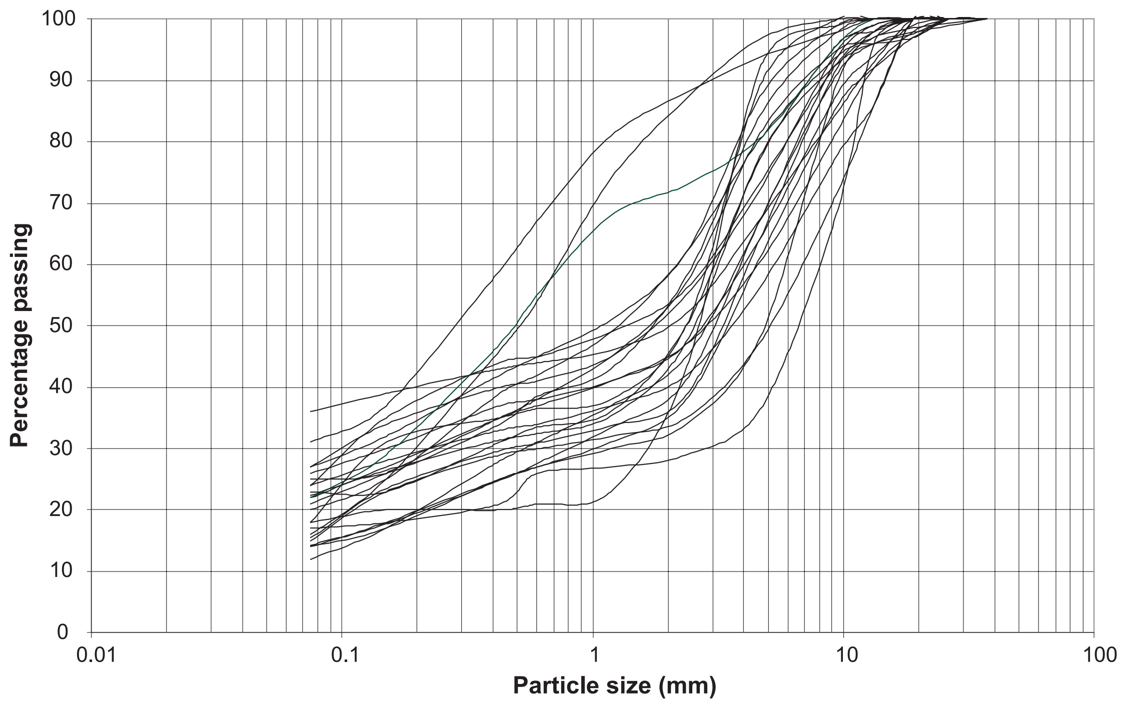


Figure 3.2
Grading envelope for the gravel wearing course

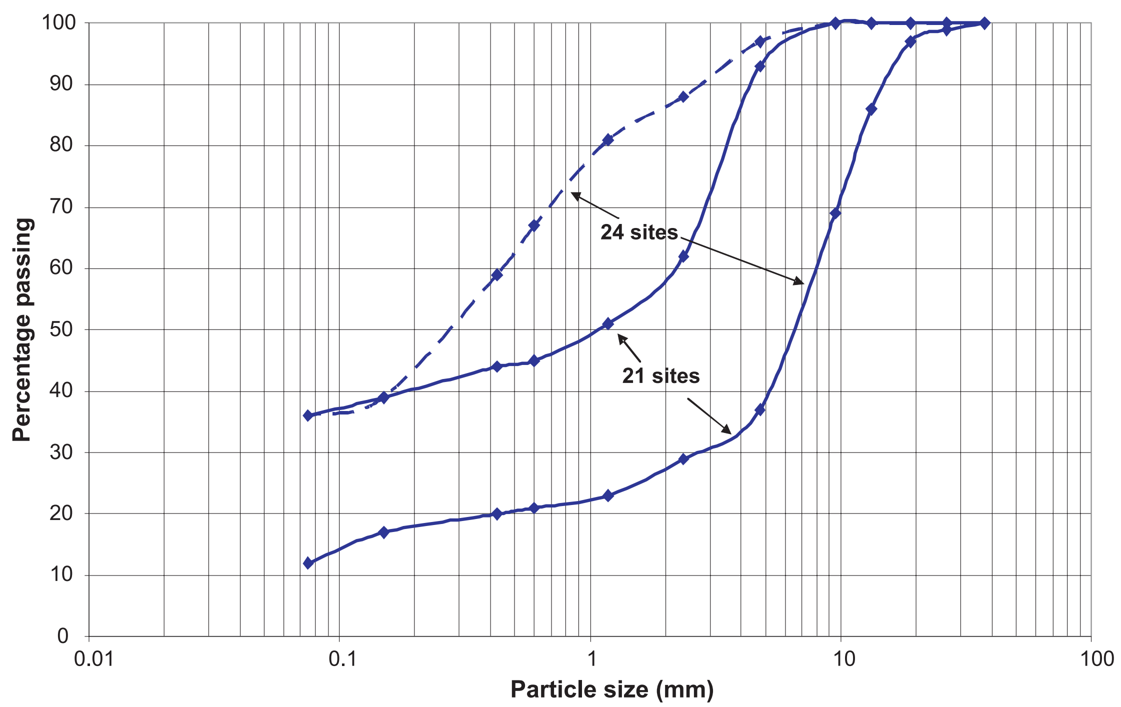


Table 3.6: Plasticity properties of the gravel wearing course

Site	Sample	Liquid Limit	Plastic Limit	Linear Shrinkage	Plasticity Index
AIAA	1	32.8	18.7	13.0	14.1
	2	32.0	20.2	5.0	11.8
ANAO	1	33.4	18.9	13.0	14.5
	2	35.2	17.8	10.0	17.4
AOAA	1	35.8	19.2	14.0	16.6
	2	47.8	25.6	15.0	22.2
AODU	1	33.0	16.1	12.0	16.9
	2	34.1	17.7	18.0	16.4
AOKM	1	30.6	16.4	13.0	14.2
	2	32.6	17.2	15.0	15.4
BACU	1	34.5	17.5	10.0	17.0
	2	37.5	21.1	14.0	16.4
BEPO	1	27.2	15.5	10.0	11.7
	2	28.3	18.1	10.0	10.2
BMBE	1	47.4	25.7	18.0	21.7
	2	46.3	25.3	15.0	21.0
BNBN	1	38.7	21.8	14.0	16.9
	2	20.9	10.2	2.0	10.7
BNNA	1	35.8	21.2	11.0	14.6
	2	37.8	20.5	15.0	17.3
DABI	1	25.0	12.4	7.0	12.6
DACO	1	47.5	26.2	17.0	21.3
	2	50.1	26.6	20.0	23.5
DEWN	1	35.0	18.0	13.0	17.0
	2	32.8	16.9	13.0	15.9
DOKO	1	25.8	15.0	5.0	10.8
	2	29.0	17.2	8.0	11.8
FIDA	1	34.5	20.5	12.0	14.0
GNKI	1	29.6	18.0	1.5	11.6
LONM	1	20.7	12.3	13.0	8.4
	2	28.2	15.3	12.0	12.9
NALU	1	29.2	13.7	14.0	15.5
	2	31.4	17.4	11.0	14.0
NUSG	1	20.6	16.3	3.0	4.3
PEYI	1	20.9	14.1	4.0	6.8
SAKE	1	22.0	15.3	3.0	6.7
WAKA	1	30.2	16.0	10.0	14.2
	2	26.7	13.7	10.0	13.0
WEPA	1	41.4	20.9	16.0	20.5
	2	39.4	20.4	16.0	19.0
ZATE	1	32.8	18.4	12.0	14.4
	2	32.2	16.0	13.0	16.2

Table 3.7: Material properties of the gravel wearing course

Site	Sample	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
AIAA	1	38	0.51	0.60	14.8	507	552	283	373
	2	39	0.50	0.62	14.0	180	426	213	363
ANAO	1	38	0.64	0.56	16.0	546	610	392	511
	2	53	0.65	0.67	16.4	310	540	348	356
AOAA	1	45	0.68	0.59	18.0	560	664	448	518
	2	41	0.69	0.57	17.2	630	932	644	742
AODU	1	53	0.63	0.65	18.6	420	593	373	353
	2	48	0.57	0.64	16.8	630	574	328	354
AOKM	1	44	0.55	0.59	19.4	572	625	341	394
	2	40	0.60	0.55	18.8	705	724	431	482
BACU	1	55	0.43	0.72	15.4	280	476	204	210
	2	58	0.62	0.70	16.8	406	477	296	379
BEPO	1	53	0.71	0.66	16.5	314	366	261	348
	2	66	0.69	0.76	14.4	223	227	157	279
BMBE	1	66	0.56	0.76	16.5	450	544	305	359
	2	64	0.58	0.74	16.6	390	546	315	380
BNBN	1	65	0.52	0.74	18.9	406	490	254	327
	2	54	0.31	0.73	14.0	52	278	86	82
BNNA	1	24	0.93	0.27	17.8	814	1080	1007	1463
	2	58	0.73	0.67	19.1	495	571	415	492
DABI	1	50	0.74	0.65	15.5	217	391	290	285
DACO	1	53	0.70	0.63	19.6	629	788	554	681
	2	61	0.63	0.71	18.3	600	705	447	505
DEWN	1	12	0.53	0.41	7.1	767	1001	526	559
	2	42	0.51	0.62	15.5	481	588	302	321
DOKO	1	27	0.47	0.53	12.7	235	506	237	331
	2	43	0.50	0.63	15.5	288	425	212	310
FIDA	1	71	0.81	0.78	14.9	252	294	238	349
GNKI	1	42	0.74	0.61	14.3	51	394	290	450
LONM	1	13	0.35	0.50	6.0	598	386	134	197
	2	24	0.53	0.51	11.3	564	606	323	383
NALU	1	61	0.52	0.72	18.0	415	459	239	211
	2	69	0.50	0.80	13.8	220	281	141	174
NUSG	1	50	0.51	0.66	17.5	105	151	77	293
PEYI	1	57	0.90	0.73	11.4	80	136	122	254
SAKE	1	62	0.56	0.74	15.5	75	168	94	214
WAKA	1	59	0.57	0.73	14.3	248	351	202	229
	2	48	0.55	0.66	15.8	330	428	233	247
WEPA	1	48	0.84	0.56	20.6	688	884	740	751
	2	61	0.62	0.73	15.9	416	493	304	327
ZATE	1	43	0.60	0.62	15.1	420	503	302	387
	2	45	0.55	0.65	14.2	421	525	287	284

Table 3.8: Range of wearing course material properties

Parameter	Measured Range
Reject Index (I_R)	0
Coarseness Index (I_c)	12 – 71
Grading Modulus (GM)	1.2 – 2.4
Grading Coefficient (GC)	6 – 21
Liquid Limit (W_L)	21 – 50
Plastic Limit (P_L)	10 – 27
Plasticity Index (I_p)	4 – 24
Linear Shrinkage (LS)	2 – 20
Shrinkage Product (SP)	51 – 767
Plasticity Product (PP)	77 – 740
Plasticity Modulus (PM)	136 – 1001
Maximum Dry Density (at 95% Mod AASHTO)	2025 - 2440

The upper limits of the grading modulus and the plasticity index were within current specifications but the lower limits were not. The current specifications and a detailed comparison between them and the observed material properties are given in Section 8.2.

The plasticity index and plasticity product of the wearing course on some sites was well below the expected lower limits. An I_p of 4 or a plasticity product of 77 is insufficient to ensure good material bonding. Lower limits for plasticity index and plasticity product equal to 11 and 300 respectively are usually necessary to ensure good performance of the wearing course.

3.5.2 Subgrade

Grading results were obtained for the samples of the subgrade from 22 of the 24 sites and are listed in Table 3.9. The range of the other material properties of the subgrade are summarised in Table 3.10. A plot of the grading curves from these sites is illustrated in Figure 3.3 and the grading envelope encompassing all the grading curves from the sites is illustrated in Figure 3.4.

The grading envelope for the subgrade materials encountered in the study shows a wide range. No subgrade related failures were noted during the study period. It could therefore be concluded that the subgrade had a negligible contribution towards the performance of the sites and that a wide variety of subgrades are suitable for gravel roads. A possible reason could be that these roads were low-volume with a relatively small number of heavy vehicles and therefore the contribution of pavement strength towards deterioration was relatively minor.

The wide range of properties displayed in Table 3.10 confirms the wide variety of subgrade materials in the country. The plasticity indices are within reasonable ranges as the subgrade is not subjected to traffic-induced wearing. The combination of plasticity and grading properties implies general adequacy of the in-situ CBRs from a structural point of view. The range of maximum dry densities achieved indicates that there are no inherent properties that prevent adequate densification of the material during compaction, hence the resultant good performance of the subgrade irrespective of the wide ranging material properties.

On a number of sites, the in-situ subgrade complied with the specifications associated with the wearing course material. It is recommended that under such circumstances the in-situ material should be used for wearing course and such decisions should be made at planning stage.

Table 3.9: Grading of the subgrade material

Site	Percentage Passing (mm sieve)									
	37.5	26.5	19	9.5	4.75	2.36	1.18	0.425	0.15	0.075
ANAO	100	100	100	99	83	46	33	27	19	16
AOAA	100	100	100	96	85	74	71	68	63	56
AODU	100	100	100	90	72	56	48	42	34	29
AOKM	100	100	100	99	97	94	90	87	73	57
BACU	100	100	94	89	81	72	68	57	35	23
BEPO	100	100	100	98	93	80	68	62	50	45
BMBE	100	100	100	98	91	89	87	67	53	50
BNBN	100	100	100	71	56	49	45	37	20	18
BNNA	100	100	100	93	76	68	62	51	38	36
DABI	100	100	100	98	83	50	36	33	27	22
DACO	100	100	100	99	98	97	96	93	68	60
DEWN	100	100	100	100	96	88	83	68	54	50
DOKO	100	100	100	95	80	65	56	42	26	20
FIDA	100	100	100	97	73	45	38	37	32	26
GNKI	100	100	100	100	99	92	85	81	71	50
LONM	100	100	100	100	98	93	68	33	18	14
NALU	100	100	100	99	96	91	87	64	37	17
NUSG	100	100	100	100	96	80	73	72	64	35
PEYI	100	100	100	99	96	94	92	91	74	46
WAKA	100	100	99	97	89	80	69	50	32	23
WEPA	100	100	100	92	82	72	66	59	49	43
ZATE	100	100	98	97	89	65	50	37	28	22

Table 3.10: Material properties of the subgrade

Parameter	Measured Range
Reject Index (I_R)	0
Coarseness Index (I_c)	3 – 55
Grading Modulus (GM)	0.5 – 2.1
Grading Coefficient (Gc)	2.3 – 20.4
Liquid Limit (W_L)	18 – 55
Plastic Limit (P_L)	8 – 32
Plasticity Index (I_p)	5 – 23
Linear Shrinkage (LS)	0 – 18
Shrinkage Product (SP)	0 – 1224
Plasticity Product (PP)	117 – 1136
Plasticity Modulus (PM)	218 – 1544
Maximum Dry Density (at 95% Mod AASHTO)	1900 – 2400

Figure 3.3
Particle size distribution of the subgrade

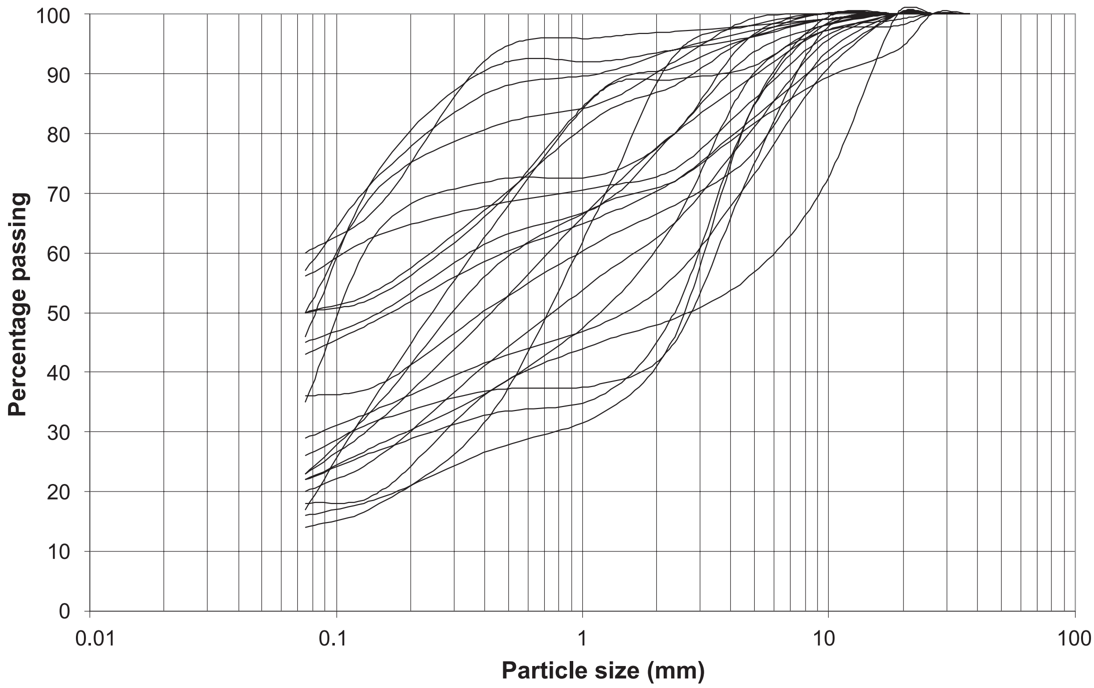
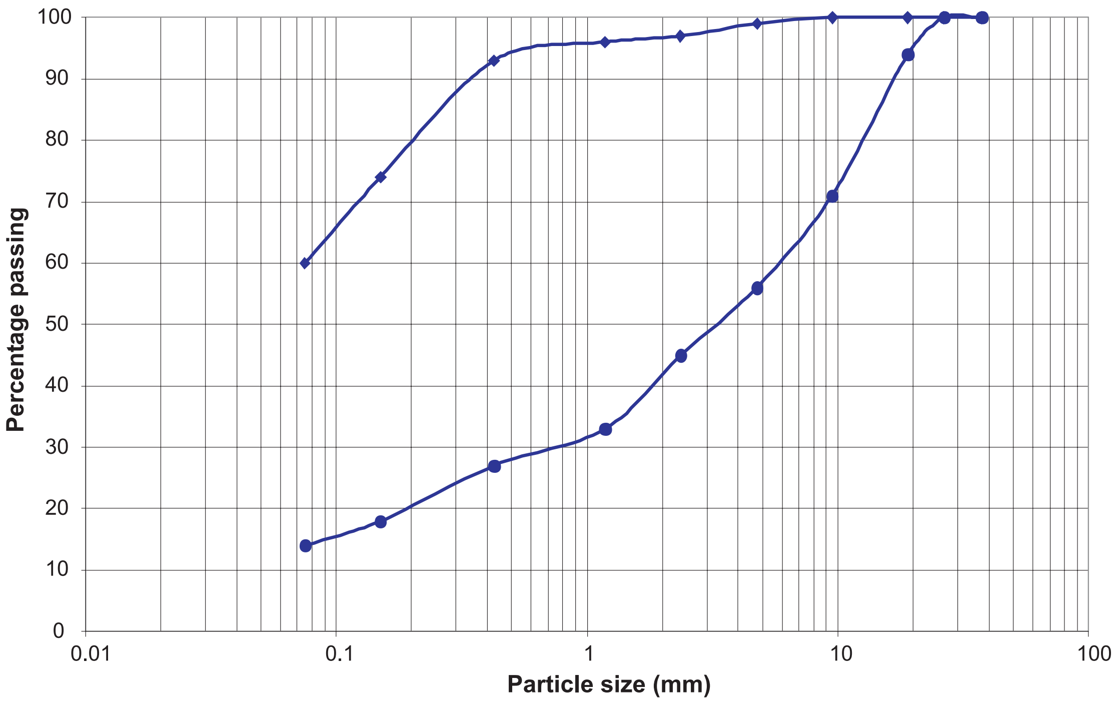


Figure 3.4
Grading envelope for the subgrade



4. Monitoring

4.1 Schedule

In order to assess the seasonal effects, the sites were monitored at the end of the dry season and the end of the wet season. The 'end of dry season' monitoring was normally carried out between February and April, while the 'end of wet season' monitoring was carried out between September and December.

The sites were monitored for a period of two years from early 2002, with each site being monitored a total of four times, as shown in Table 4.1.

Table 4.1: Monitoring dates

Site	Survey			
	First	Second	Third	Fourth
AIAA	Mar-2002	Oct-2002	Mar-2003	Nov-2003
ANAO	Aug-2002	Jan-2003	Aug-2003	Apr-2004
AOAA	Apr-2002	Nov-2002	May-2003	Apr-2004
AODU	May-2002	Dec-2002	May-2003	Apr-2004
AOKM	Mar-2002	Oct-2002	Apr-2003	Mar-2004
BACU	Jul-2002	Dec-2002	Jun-2003	Dec-2003
BEPO	Aug-2002	Jan-2003	Aug-2003	Mar-2004
BMBE	Apr-2002	Nov-2002	May-2003	Apr-2004
BNBN	Mar-2002	Oct-2002	Mar-2003	Dec-2003
BNNA	Mar-2002	Oct-2002	Mar-2003	Dec-2003
DABI	May-2002	Jan-2003	Jul-2003	Jan-2004
DACO	Apr-2002	Nov-2002	May-2003	Apr-2004
DEWN	Mar-2002	Oct-2002	Mar-2003	Nov-2003
DOKO	Mar-2002	Oct-2002	Apr-2003	Mar-2004
FIDA	Aug-2002	Feb-2003	Sep-2003	May-2004
GNKI	May-2002	Dec-2002	Jul-2003	Jan-2004
LONM	Mar-2002	Oct-2002	Apr-2003	Mar-2004
NALU	Jul-2002	Dec-2002	Jun-2003	Dec-2003
NUSG	May-2002	Dec-2002	Jul-2003	Jan-2004
PEYI	Jun-2002	Dec-2002	Jul-2003	Jan-2004
SAKE	Sep-2002	Feb-2003	Sep-2003	May-2004
WAKA	Jul-2002	Jan-2003	Jul-2003	Jan-2004
WEPA	May-2002	Nov-2002	May-2003	Apr-2004
ZATE	Jul-2002	Jan-2003	Jul-2003	Jan-2004

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

Figure 4.1
MERLIN roughness measuring device

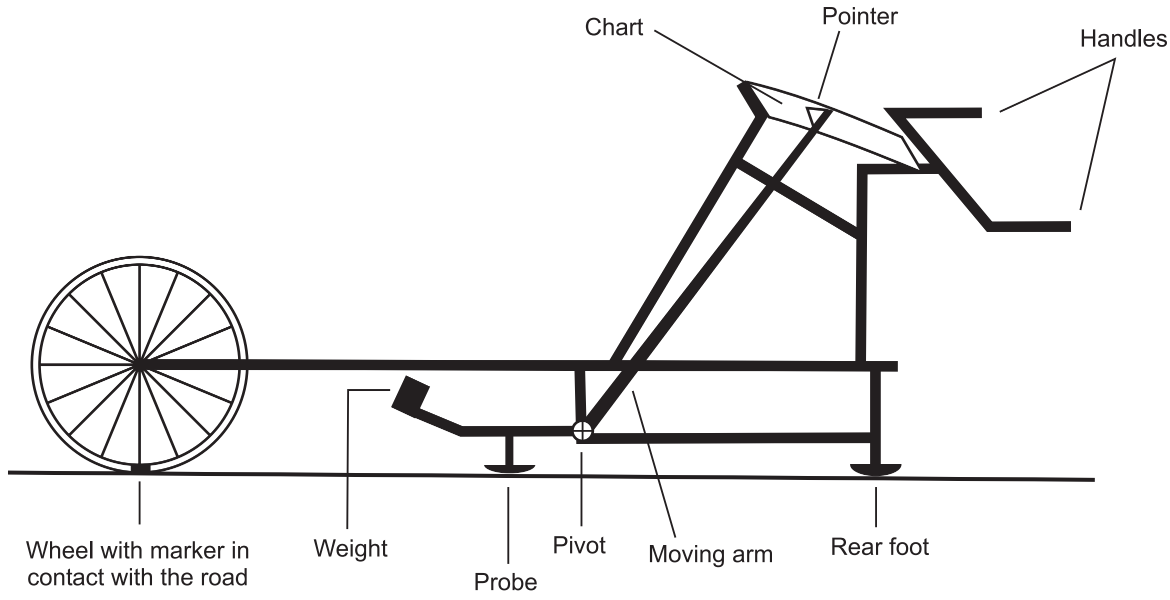
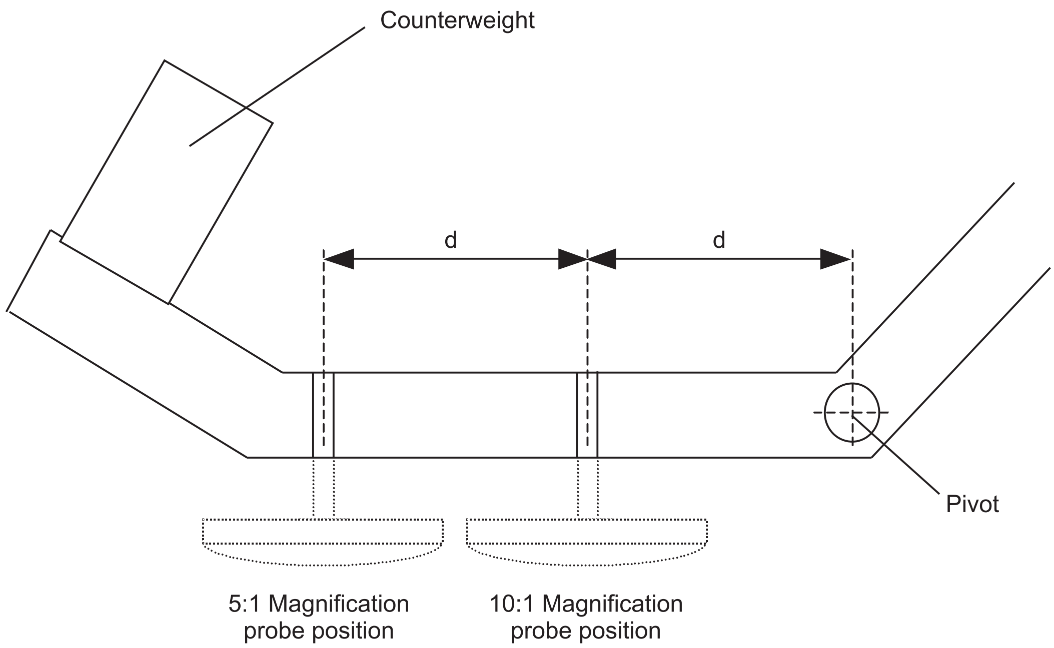


Figure 4.2
Merlin probe assembly



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 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.

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 100,000 profile heights from the 4 surveys conducted on the 24 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 F 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

Table 5.1: Gravel loss between surveys

Site	Carriageway Gravel Loss in mm			
	1st – 2nd Surveys	2nd – 3rd Surveys	3rd – 4th Surveys	Average
AIAA	33	-7	6	11
ANAO	8	-1	15	7
AOAA	3	12	9	8
AODU	21	8	9	13
AOKM	9	-10	5	1
BACU	35	-6	7	12
BEPO	29	-7	-1	7
BMBE	9	-1	1	3
BNBN	-35	63	-71	-14
BNNA	23	-13	14	8
DABI	15	-6	13	7
DACO	37	-3	–	17
DEWN	12	4	-5	4
DOKO	-3	52	17	22
FIDA	4	35	-33	2
GNKI	12	-4	–	4
LONM		-18	15	-1
NALU	27	7	6	14
NUSG	21	31	-7	15
PEYI	14	1	14	9
SAKE	8	-20	1	-3
WAKA	22	3	11	12
WEPA	15	11	-4	7
ZATE	23	-31	21	4

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 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 17 mm/year, indicating that on average the roads need regravelling every 7 to 8 years if the thickness of the gravel wearing course is 150 mm.

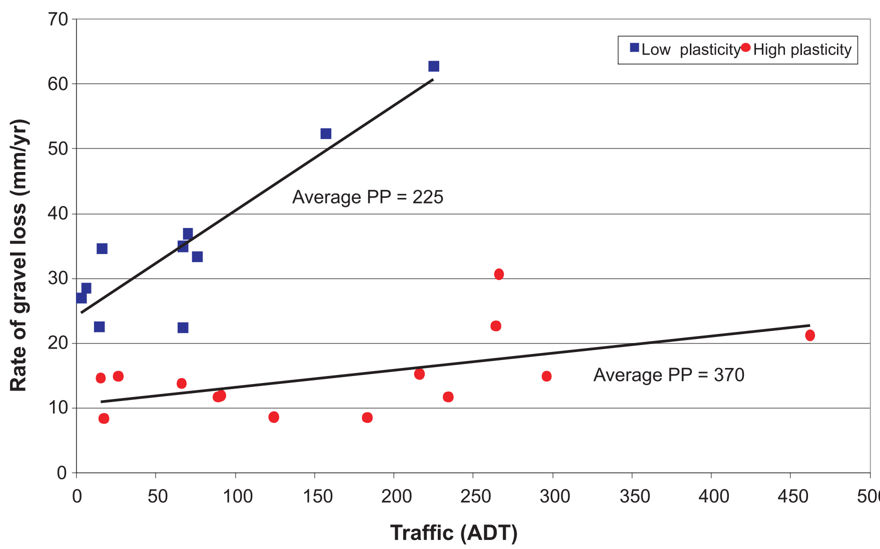
A more detailed examination of the gravel loss rates was conducted to determine the influence of variables such as traffic and material properties. The typical rates of gravel loss have been plotted against ADT as illustrated in Figure 5.1.

Two distinct groups were evident from this plot. The average plasticity of one group of sites was low, average plasticity product (PP) of 225, with the average PP of the other group being 370. The rate of gravel loss from the 'low plasticity' sites was significantly higher than the rate from the 'high plasticity' sites as shown in Table 5.2.

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

ADT	Gravel Loss (mm/year)	
	Low Plasticity	High Plasticity
20	27	11
50	32	12
100	40	13
250	64	17

Figure 5.1
Rates of gravel loss vs traffic and plasticity



The values in Table 5.2 indicate that the rate of gravel loss on the low plasticity sites was approximately a factor of three times higher than the rate for the high plasticity sites. These rates of gravel loss indicate that roads constructed with material that has high plasticity will need regravelling after approximately 12 years for traffic levels up to 100 vpd, whereas roads constructed with low plasticity material will need to be regravelled after approximately 5 years for low trafficked roads (ADT < 20) and after 3 years for higher trafficked roads (ADT = 100).

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, 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 5 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.

Based on these performance criteria 12 sites were classified as good, 7 sites were classified as moderate and 5 sites as poor. The details are given in Table 5.4.

Table 5.3: Performance criteria

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

The next stage in the analysis was to determine causes or reasons for differences in performance. Each site has been examined individually. Based on the results of the analysis, the current specifications may be reviewed and if necessary revised.

The assessment of the performance of the various test sites is dependent on the type and material properties of the wearing course. Unfortunately the type of wearing course was not described sufficiently accurately as all gravel was referred to as laterite (see Table 2.5). This made the assessment of performance more difficult because the type of wearing course is an important factor. Other factors that influence performance such as level of compaction at the construction stage are not covered in this assessment because this information was not available to the project team.

AIAA was a site with a fine wearing course on a moderate slope and in a dry zone. The material was fine with maximum nominal particle size of 9.5 mm and moderate coarseness (I_c = 38). Plasticity was moderate with I_p = 14 and PP = 283. The performance of the wearing course was moderate and the reason for this could be the combination of moderate plasticity and moderate coarseness. It is likely that that the material could have performed better if the plasticity had been higher or the coarseness had been lower.

ANAO was a site in a wet climate (1415 mm/year) with a fine wearing course with a coarseness index, I_c, of 38. The I_p of 14.5 is moderate and a PP of 610 is high. The wearing course on this site performed poorly but there are no obvious reasons for the poor performance.

AOAA, AODU, and AOKM were sites situated in wet areas and consisted of wearing course that had moderate to high coarseness (I_c = 45, 53 & 44 respectively). The I_p was moderate to high (17, 17 & 14 respectively) and the PP was high (448, 373, & 341 respectively). The wearing course on these sites performed very well. This

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	If	I_c	GM	PP
AIAA	M	42	76	539	8.9	14	20	38	1.79	283
ANAO	P	70	15	1415	16.2	15	27	38	1.69	392
AOAA	G	8	234	1357	23.7	17	27	45	1.78	448
AODU	G	9	462	1106	21.8	17	22	53	1.96	373
AOKM	G	7	183	1485	21.2	14	24	44	1.76	341
BACU	M	58	67	1094	19.9	17	12	55	2.15	204
BEPO	P	398	6	1415	9.0	12	22	53	1.99	261
BMBE	G	8	124	1138	11.9	22	14	66	2.27	305
BNBN	M	31	225	1485	9.7	17	15	65	2.21	254
BNNA	G	12	264	1485	11.4	15	24	58	2.01	350
DABI	M	45	26	1115	5.7	13	23	50	1.96	290
DACO	M	53	70	1138	18.1	21	26	53	1.90	554
DEWN	G	13	91	539	15.6	17	31	12	1.22	526
DOKO	M	36	157	1485	1.0	11	22	27	1.58	237
FIDA	P	217	16	1119	9.3	14	17	71	2.33	238
GNKI	G	17	89	1115	2.7	12	25	42	1.83	290
LONM	G	10	216	1285	2.3	8	22	13	1.51	134
NALU	P	737	3	1094	21.2	15	16	61	2.16	239
NUSG	G	16	266	1115	4.2	4	18	50	1.97	77
PEYI	G	23	66	1115	1.2	7	18	57	2.19	122
SAKE	M	27	17	556	18.1	7	14	62	2.23	94
WAKA	M	36	67	974	17.2	14	14	59	2.20	202
WEPA	G	8	296	1106	5.2	21	36	48	1.69	740
ZATE	P	140	14	973	3.5	14	21	43	1.87	302

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

performance can be attributed to the high plasticity which effectively enhances bonding within the gravel matrix.

BACU was located in a wet climate and had a wearing course with high coarseness (I_c = 55, If = 12). The I_p was high (17) and the PP was low (204). The performance of the wearing course on this site was moderate, most likely because of the combination of low PP and high coarseness. This indicates that the plasticity of the material is better defined using PP rather than I_p because it is the quantity of the clay that has more influence than the mineralogy of the clay itself.

BEPO performed poorly. The I_p was relatively low (12), the PP was moderate (261) and the coarseness was high (I_c = 53). This indicates that a combination of high coarseness and low plasticity is likely to have resulted in the poor performance of the wearing course.

BMBE showed good performance. The wearing course had a high coarseness (I_c = 66) and high plasticity (I_p = 22 and PP =

305). The good performance can be attributed to the high plasticity despite the fact that coarseness was high.

BNBN performed moderately. The coarseness was high (I_c = 65) and the I_p was high (17), but the PP was moderate (254). The high coarseness is likely to have contributed to the moderate performance of the wearing course.

BNNA was in a wet area. The wearing course had a high coarseness (I_c = 58), a moderate I_p (15) and a high PP (350). High plasticity, in particular the high PP, contributed to the good performance in spite of the high coarseness.

DABI and DACO performed moderately. The wearing courses had moderate and high I_p (13 & 21 respectively) and moderate and high PP (290 & 556 respectively). The coarseness for both sites was high (I_c = 50 & 53 respectively). The high coarseness is likely to have compromised the performance of the wearing course on these sites, although the high plasticity on DACO should have resulted in a better performance on this site.

DEWN was a site in a relatively dry area. The plasticity of the wearing course was high (I_p = 17, PP = 526) and the coarseness was low (I_c = 12). This combination of high plasticity and low I_c contributed to the good performance of the wearing course.

DOKO performed moderately. The wearing course had a low coarseness (I_c = 27). The I_p was moderate (11) and the PP was low (237). The low PP is likely to have contributed to the moderate performance due to diminished bonding.

FIDA was in a wet region and performed poorly. The I_p was moderate (14) while the PP was low (239). The coarseness was high (I_c = 71). The combination of high coarseness and low plasticity contributed to the poor performance of the wearing course.

GNKI and LONM exhibited good performances. The plasticity was moderate and low (I_p = 12 & 8, PP = 290 & 134 respectively). The coarseness was moderate and low (I_c = 42 & 13 respectively). The good performance of GNKI is attributed to a combination of moderate plasticity and moderate coarseness, while that of LONM is due to the low coarseness despite the low plasticity.

NALU performed poorly. The I_p was moderate (16) and the PP was low (239). The coarseness was high (I_c = 61). The poor performance is a result of a combination of low plasticity and high coarseness.

NUSG and PEYI performed well. The wearing courses exhibited low plasticity (I_p = 4 & 7, PP = 77 & 122 respectively). The coarseness was high (I_c = 50 & 57). Based on the performance of other similar sites, the performance of these two sites should have been poor. However, there is another influencing property and that is the maximum nominal particle size. The maximum nominal particle size for both sites was 4.75 mm with 40% and 50% respectively retained on the 2.36 sieve. This implies that all of the wearing course gravel was less than 5 mm in size, while 40 – 50% was less than 2.36 mm. This indicates that the material is clayey-silty-gravelly sand. The good performance is likely to have been a result of its fine gradation.

SAKE performed moderately but was very close to a good performance with 27mm/year gravel loss for an ADT of 100. The plasticity was low (I_p =7, PP = 94) and the coarseness was high (62). Again, the performance of this site would be expected to be poor. There are no obvious reasons why the wearing course performed reasonably well, other than possibly the material type, the description of which is not available. One notable aspect is that the site was situated in a dry climate.

WAKA performed moderately. The I_p was moderate (14), the PP was low (202) and the coarseness was high (I_c = 59). The moderate I_p may have prevented the wearing course from performing poorly.

WEPA had a wearing course that had high I_p (21), very high PP (740) and moderate I_c (48). The high plasticity must have contributed to the good performance of this site.

Table 5.5: Observed roughness during each survey

Site	Observed IRI (mm/km)				
	1st Survey	2nd Survey	3rd Survey	4th Survey	Average
AIAA	8.7	5.3	5.9	4.3	6.1
ANAO	4.6	4.1	3.9	4.2	4.2
AOAA	5.4	4.6	5.6	7.1	5.7
AODU	5.0	6.0	4.4	7.8	5.8
AOKM	7.2	7.1	4.6	8.3	6.8
BACU	5.0	4.6	3.9	5.3	4.7
BEPO	5.3	4.8	5.3	6.6	5.5
BMBE	4.6	4.6	4.6	4.1	4.5
BNBN	7.8	12.6	12.2	7.3	10.0
BNNA	7.5	7.9	6.6	7.8	7.4
DABI	4.5	4.3	3.5	4.2	4.1
DACO	8.1	8.2	7.4	5.8	7.4
DEWN	7.3	8.2	8.5	8.4	8.1
DOKO	8.1	8.4	5.8	10.1	8.1
FIDA	3.2	3.5	2.7	7.4	4.2
GNKI	4.7	4.2	3.5	4.2	4.1
LONM	5.3	5.2	7.7	19.0	9.3
NALU	8.2	6.3	6.0	5.8	6.6
NUSG	3.5	4.6	3.3	6.0	4.4
PEYI	3.9	3.1	3.1	4.8	3.7
SAKE	3.6	3.6	4.6	5.4	4.3
WAKA	6.2	4.4	4.9	7.7	5.8
WEPA	5.8	5.9	6.3	12.1	7.5
ZATE	3.6	5.6	3.7	4.1	4.3

ZATE performed poorly. The plasticity was moderate (I_p = 14, PP = 302), as was the coarseness (I_c = 43). The wearing course, however, was not well graded and this may be one of the contributing factors to the poor performance, but a moderate performance would be expected from a wearing course with these properties.

5.2 Roughness

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

The average roughness of all the sites over the monitoring period was evaluated as 5.9 IRI, which indicates that the labour-based gravel roads were in a relatively good condition. As expected, the higher trafficked roads had higher roughness values than the lower trafficked roads as shown in Table 5.6.

Table 5.6: Roughness vs ADT

No. of Sites	ADT	IRI
7	< 50	4.7
7	50 – 100	5.7
3	100 – 200	6.5
7	> 200	7.2

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.

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

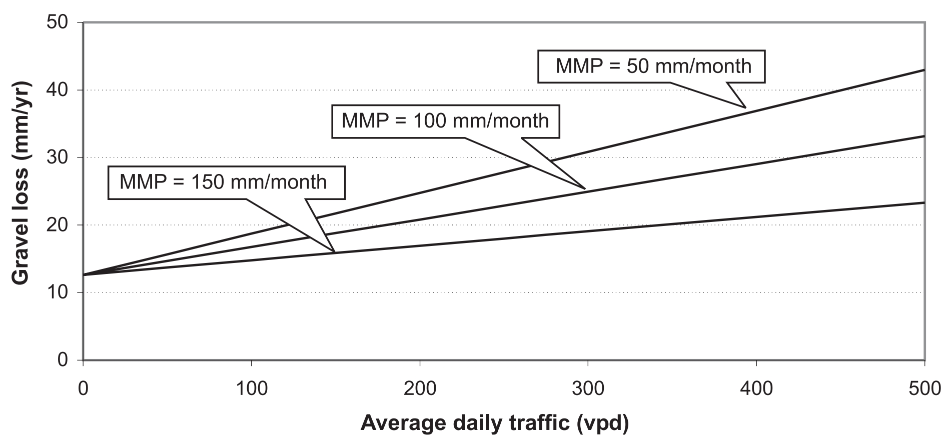
where

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

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}
AIAA	33.4	17.2	1.9
ANAO	14.7	14.5	1.1
AOAA	11.8	16.22	0.8
AODU	21.3	23.5	0.9
AOKM	8.6	17.5	0.5
BACU	35.0	15.2	2.3
BEPO	28.6	13.7	2.1
BMBE	8.7	13.6	0.6
BNBN	62.8	13.7	4.7
BNNA	22.7	15.4	1.4
DABI	14.9	14.0	1.1
DACO	36.9	14.3	2.6
DEWN	12.0	18.1	0.7
DOKO	52.4	15.7	3.3
FIDA	34.6	13.8	2.6
GNKI	11.8	16.3	0.7
LONM	15.3	19.4	0.8
NALU	27.0	14.4	1.8
NUSG	30.7	27.3	1.1
PEYI	13.8	15.8	0.9
SAKE	8.5	14.4	0.6
WAKA	22.5	15.9	1.4
WEPA	15.0	17.8	0.8
ZATE	22.6	13.3	1.7
Average	**23.6**	**16.3**	**1.5**

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.

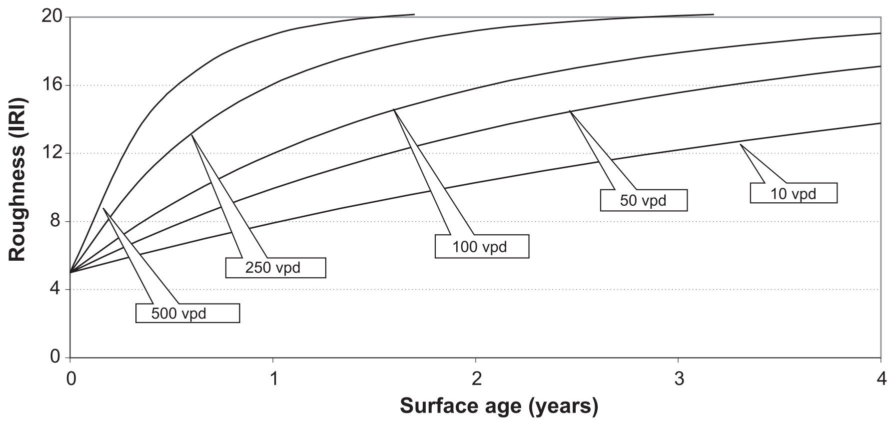
The results in Table 6.1 show that the HDM-4 predicted rates of gravel loss were higher than those observed on 10 of the sites and lower than observed on 14 of the sites. The average value of the gravel loss calibration factor K_{gl} was 1.5, which indicates that, on average, the amount of gravel lost on these labour-based roads was 50% 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 2.4 and the average value for the high plasticity sites was 0.85, indicating that the observed rates of gravel loss in this study were more sensitive to PP than indicated by the HDM-4 model. These findings are summarised in Table 6.2.

Table 6.2: Calibration factors for gravel loss

Sites	K_{gl}
Low plasticity	2.4
High plasticity	0.85
All	1.5

Figure 6.2
Roughness progressions on unsealed roads with no maintenance



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

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

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

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

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 < 3500 kg) in both directions, in vpd

ADH = average daily heavy traffic (GVW ≥ 3500 kg) 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 average value of K_c

Table 6.3: HDM-4 roughness calibration factors

Site	Construction Year	ADT	Observed Roughness IRI	Calibration Factor K_c
AIAA	2001	76	6.1	0.45
ANAO	1998	15	4.2	0.4
AOAA	2001	234	5.7	0.2
AODU	1996	462	5.8	0.05
AOKM	1990	183	6.8	0.15
BACU	2002	67	4.7	0.6
BEPO	2002	6	5.5	1.9
BMBE	2001	124	4.5	1.0
BNBN	1990	225	10.0	0.3
BNNA	1988	264	7.4	0.1
DABI	2002	26	4.1	0.75
DACO	2001	70	7.4	0.8
DEWN	2001	91	8.1	0.7
DOKO	1987	157	8.1	0.25
FIDA	2000	16	4.2	0.5
GNKI	2000	89	4.1	0.25
LONM	1994	216	9.3	0.2
NALU	2002	3	6.6	2.5
NUSG	2002	266	4.4	0.25
PEYI	1998	66	3.7	0.2
SAKE	2001	17	4.3	1.0
WAKA	2002	67	5.8	0.9
WEPA	2000	296	7.5	0.2
ZATE	2002	14	4.3	1.2
Average				**0.6**

for all the sites was 0.6, which indicates that the rates of roughness progression observed on the sites were, on average, lower than those predicted by HDM-4.

A further examination of the calibration factors revealed that as the traffic levels increased, the values of the calibration factors generally decreased as illustrated in Table 6.4. This indicates that the effect of increased traffic levels in HDM-4 is much higher than observed on the sites in Ghana, despite the fact that the roughness on the higher trafficked sites was generally higher (see Table 5.6).

Table 6.4: Roughness calibration factors by traffic volumes

No. of Sites	ADT	K_c
7	< 50	1.2
7	50 – 100	0.6
3	100 – 200	0.5
7	> 200	0.2

7. Life-Cycle Cost Methodology

As mentioned in Section 1.4, this study in Ghana 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 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

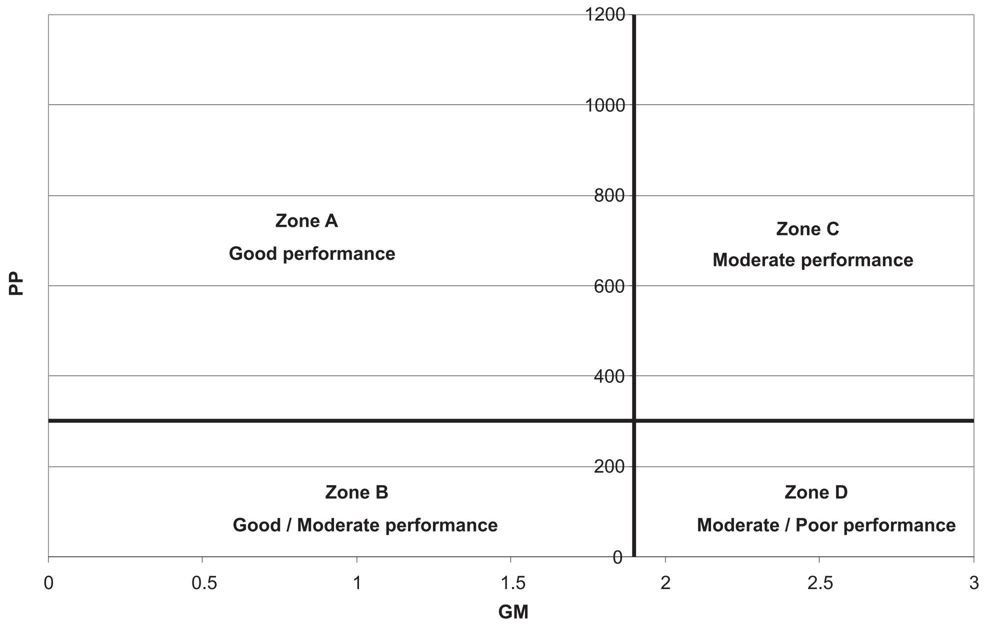


Figure 7.2
Regravelling frequency

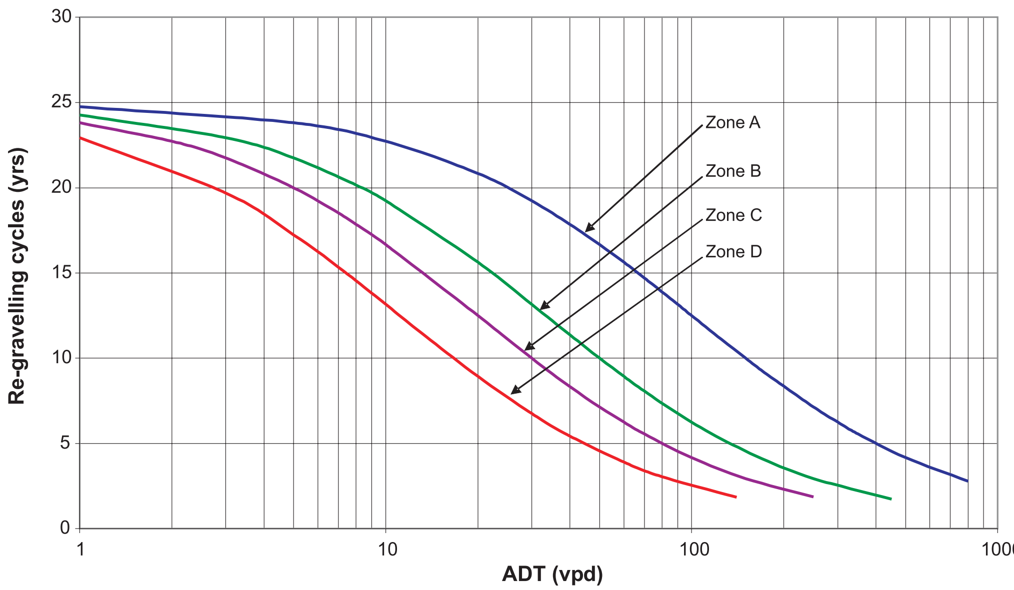
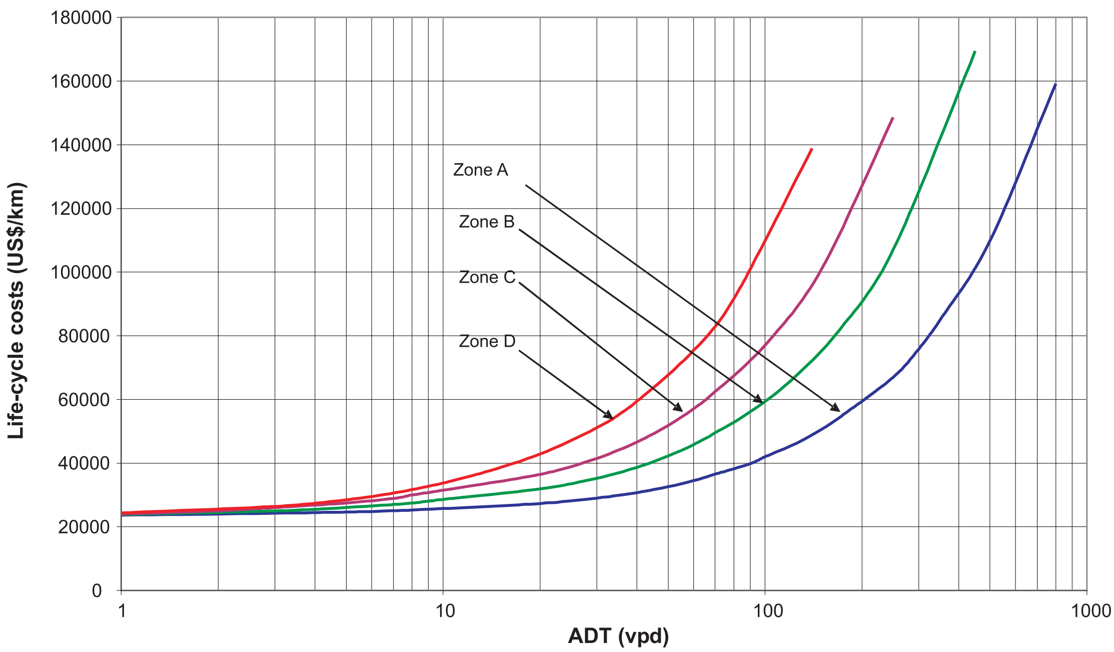


Figure 7.3
Example of life-cycle costs



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 a low plasticity wearing course, the rate of gravel loss was high, ranging from 25 mm/year for low trafficked roads (ADT < 20) to 40 mm/year for higher trafficked roads (ADT = 100). For sites with a 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 courses with low plasticity are less durable and therefore less economical than wearing courses with relatively high plasticity.
iii) The performance in terms of gravel loss of the high plasticity and low plasticity wearing course tend to converge with reduction in ADT. This indicates that for very low volume roads, materials of low plasticity may be used without increasing the maintenance costs significantly. 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 wearing courses 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 country's minimum soaked CBR required for wearing course material should be adhered to. In Ghana this is 20% m.d.d. at 95% Mod AASHTO. 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 50% higher than the HDM-4 predicted rates, giving an average value of 1.5 for the gravel loss calibration factor K_{gl}. However, the average value for K_{gl} for the low plasticity sites was evaluated as 2.4 and the average value for the high plasticity sites was 0.85. This indicates that on wearing courses with low plasticity, the rates of gravel loss are approximately 2½ times higher than those predicted by HDM-4, whereas for high plasticity wearing courses the rates are similar to 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 0.6 for the roughness calibration factor K_C. The values of K_C decreased as traffic levels increased, indicating that the effect of increasing traffic in HDM-4 is much higher than observed in Ghana.

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 (by weight)	
	Class 1	Class 2
37.5	100	100
20	95 – 100	85 – 100
10	65 – 100	55 – 100
5	45 – 85	35 – 92
2	30 – 68	23 – 77
0.425	18 – 44	14 – 50
0.075	12 – 32	10 – 40

From the values in Table 8.1, the limits of the GM can be evaluated as follows:

Class 1: $1.56 \le GM \le 2.40$

Class 2: $1.33 \le GM \le 2.53$

The specifications for the plasticity of the wearing course in terms of I_p are as follows:

Wet Zones: $6 \le I_p \le 15$

Dry Zones: $8 \le I_p \le 20$

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.

Table 8.2: Grading envelopes of monitored sites

Sieve sizes	Percentage Passing (by weight)	
	Good and Moderate Performance	Good Performance
37.5	100	100
26.5	99 – 100	100
19	97 – 100	97 – 100
13.2	86 – 100	86 – 100
9.5	72 – 100	78 – 100
4.75	48 – 97	49 – 97
2.36	34 – 88	34 – 88
1.18	23 – 81	23 – 81
0.6	21 – 67	21 – 67
0.425	20 – 59	20 – 59
0.15	17 – 39	17 – 39
0.075	12 – 36	14 – 36

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

There are substantial overlaps amongst the different grading envelopes. Based on the adequate performance of the monitored sites, it appears that there is scope for widening the grading specifications. In particular, the analysis indicated that finer materials performed better than coarser materials when other properties, such as plasticity, were similar. It is therefore proposed that the grading envelope is relaxed to accommodate finer materials. The recommended grading envelope is given in Table 8.3 and plotted in Figure 8.2.

Figure 8.1
Comparison of grading envelopes

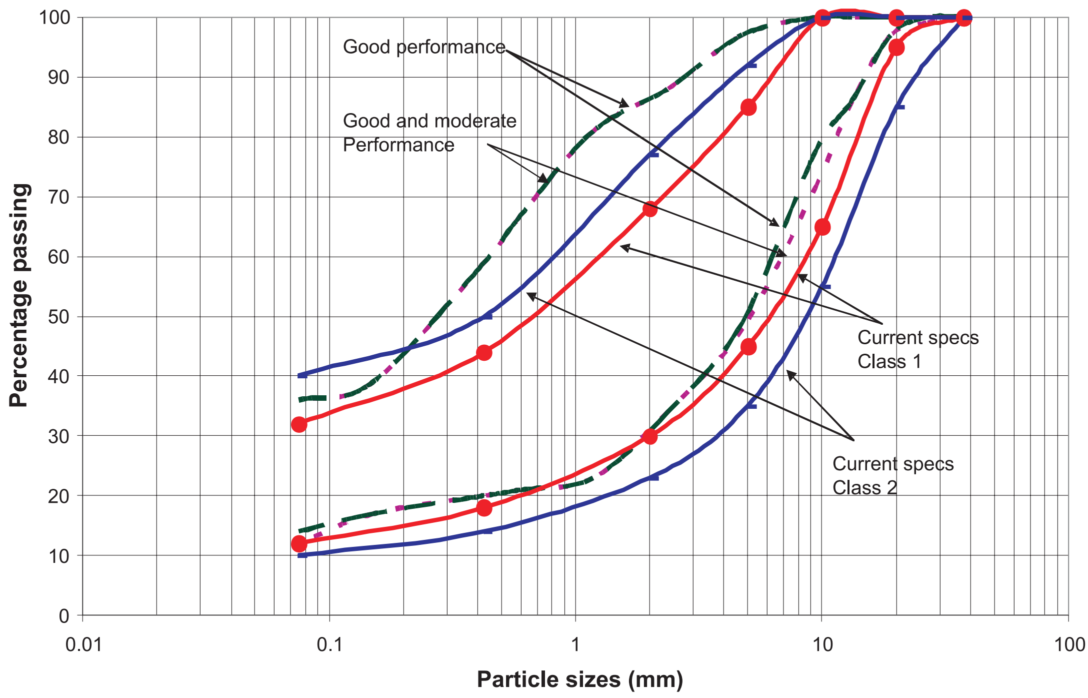


Figure 8.2
Recommended new grading envelope

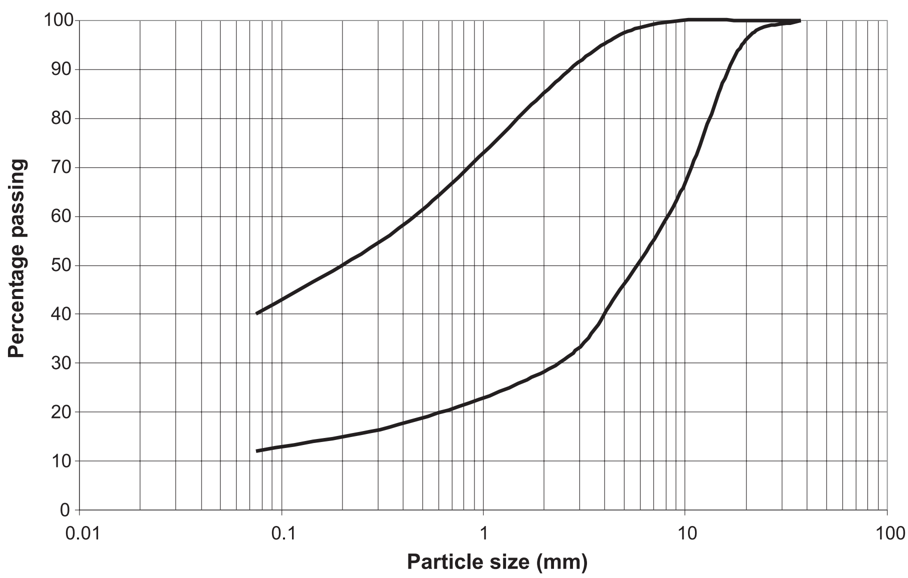


Table 8.3: Recommended new grading for gravel wearing courses

Sieve Size (mm)	Percentage Passing (by weight)
37.5	100
20	95 – 100
10	65 – 100
5	45 – 97
2	30 – 88
0.425	18 – 59
0.075	12 – 40

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

$$1.13 \leq GM \leq 2.40$$

The analysis showed that it is possible to use wearing course that has an I_p as low as 4 and 6 on condition that coarseness is low ($I_c \leq 30$) or the maximum nominal size of the gravel particles is ≤ 5 mm. Material with high I_p can also be used provided the PP is less than 800. Wearing courses with higher I_p or PP can be used in dry climates (< 700 mm/year).

The recommended new specifications for the plasticity of gravel wearing courses 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.

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 (Uganda 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.

Table 8.4: Recommended specifications for plasticity of gravel wearing courses

Parameter	Acceptable	Conditionally Acceptable	Reject
I_p	10 – 20	5 – 9[1] 20 – 24[2]	< 5 > 24
PP	300 – 800	80 – 300[1] 800 – 1000[2]	< 80 >1000

Notes: [1] - $I_c \leq 30$ *or maximum nominal size of the gravel particles* ≤ 5 *mm*
[2] - *Rainfall* < 700 *mm/year*