THE INFLUENCE OF GEOTECHNICAL PROPERTIES ON THE PERFORMANCE OF GRAVEL WEARING COURSE MATERIALS

BY

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SUMMARY

Unpaved roads comprise about 72 percent of the national and provincial road network in South Africa and substantially more of the total southern African road network. Significant costs are incurred annually by the authorities on the maintenance of these roads and enormous sums are associated with the cost of using these roads. Specifications for the use of materials for unpaved roads in southern Africa abound, the origin of most of these, however being rather obscure. It would appear that many of them have been transferred from other countries, mostly in the northern hemisphere.

The geological materials used for construction in southern Africa, having been subjected to aeons of weathering and minimal major periods of recent glaciation, differ greatly from those in the northern hemisphere. One of the main objectives of this research was to investigate the performance of local geological materials used in wearing courses for unpaved roads and relate this to their geotechnical properties. This involved the sampling, testing and monitoring of 110 sections of unpaved road in the Transvaal and South West Africa over a period of more than three years. A large data base of gravel height, roughness, rut and corrugation measurements and the severity and extent of dustiness, stoniness, potholes, cracks, loose material, slipperiness and trafficability was compiled.

The material and performance data were then used to develop performance related specifications and prediction models for roughness progression and gravel loss with the aim of providing improved unpaved roads and consequently reducing the road user and maintenance costs, in the national interest. Some of the savings will obviously be off-set to some extent by slightly increased materials location and construction costs. These costs are, however, shown to be minimal in comparison with the potential savings to the road user, the national economy and the generally improved quality of life of many road users in southern Africa. A good knowledge of the geotechnical properties of potential wearing course materials (which are significantly dependent on the geological origin) is necessary to differentiate good gravels from poor ones.

SAMEVATTING

Ongeplaveide paaie maak ongeveer 72 persent uit van die nasionale en provinsiale padnetwerk in Suid Afrika en heelwat meer van die totale padnetwerk van suidelike Afrika. Groot kostes word jaarliks aangegaan deur die owerhede vir die instandhouding van hierdie paaie, asook enorme kosts wat in verband gebring kan word met die gebruik van hierdie paaie. Materiaalspesifikasies vir ongeplaveide paaie in suidelike Afrika is volop. Die oorsprong van die meeste hiervan is egter twyfelagtig en dit wil voorkom of dit uit ander lande, hoofsaaklik in die noordelike halfrond, afkomstig is.

Die geologiese materiale gebruik vir padkonstruksie in suidelike Afrika verskil grootliks van dié in die noordelike halfrond. Dit was onderwerp aan eeue se verwering en minimale groot-skaalse onlangse glasiale werking verskil grootliks van dié. Een van die hoof doelwitte van hierdie navorsing was om ondersoek in te stel na die werkverrigting van plaaslike geologiese materiale wat gebruik word vir die slytlaag van ongeplaveide paaie en om dit in verband te bring met hul geotegniese eienskappe. Dit het behels monsterneming en toets van materiale, asook monitering van 110 seksies ongeplaveide pad in Transvaal en Suid-Wes Afrika oor 'n tydperk van drie jaar. 'n Groot databasis bestaande uit inligting oor metings van gruishoogte, ongelykheid, spoor en sinkplaat asook inligting van graad en omvang van stof, klipperigheid, slaggate, krake, los materiaal, glibberigheid en rybaarheid is saamgestel.

Die materiaal- en werkverrigtingsdata is toe gebruik om werkverrigtingsverwante spesifikasies op te stel, asook om modelle daar te stel om verswakking in gelykheid, en gruisverlies te voorspel. Die doel is om beter ongeplaveide paaie te voorsien en gevolglik die padverbruikers- en onderhoudskoste in nasionale belang te verminder. 'n Gedeelte van die besparing sal onvermydelik deur effens duurder konstruksiekoste en moontlike langer vervoerafstande van materiaal gekanselleer word. Hiedie koste is egter minimaal in verhouding tot die potensiële besparing vir die padgebruiker, die nasionale ekonomie en die algemene verbetering in lewenskwaliteit van baie padgebruikers

in suidelike Afrika. 'n Goeie kennis van die geotegniese eienskappe van potensiële slytlaagmateriale (wat grootliks afhanklik is van geologiese oorsprong) is noodsaaklik om te differensieër tussen goeie an swak gruise.

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LIST OF ABBREVIATIONS AND SYMBOLS

The abbreviations and symbols often used in this thesis are:

Abbreviation	Term description
AASHTO	American Association of State Highway and Transport Officials
ADT	average daily traffic (total vehicles per day)
AFV	Aggregate fingers value (%)
APV	Aggregate pliers value (%)
ASTM	American Society for Testing Materials
BI	bump integrator
BS	season during which blading occurred (Visser, 1981a)
C	horizontal curvature (radius in metres)
CALC	estimated percentage of calcite (%)
CBR	California Bearing Ratio (%)
CHLOR	estimated percentage of chlorite group minerals (%)
CLASS	TPA soil classification (modified after AASHO, 1961)
COMP	in-situ relative compaction (%)
CP	cumulative precipitation since last blading (m)
Cret	Cretaceous
CRS	Crack severity
CSIT	Clegg soil impact tester
D	number of days since blading (in hundreds)
DCP	Dynamic cone penetrometer
DENS	in-situ density (kg/m³)
DR	dust ratio (P075/P425)
EMC	Equilibrium moisture content (%)
exp	exponential
F	F-value for statistical significance
FEL	estimated percentage of feldspar (%)
Fm, Form	Formation
FMC	field moisture content (%)
FME	field moisture equivalent (%)
G	vertical grade (%)
G_c	grading coefficient ((P26 - P2) x P475)/100
GI	gravel index (1 - (P2/P26))
GL	gravel thickness loss (mm)
GM	grading modulus (300 - (P425 + P2 + P075))/100
GOET	estimated percentage of goethite (%)
ILL	estimated percentage of illite-type clays (%)
Im	Thornthwaite's moisture index (Emery, 1985)
I_o	oversize index (per cent larger than 37,5 mm)
KAOL	estimated percentage of kaolin group minerals (%)
km	kilometre
km/h	kilometres per hour
kN	kilonewton
kPa	kilopascal
LAA	Los Angeles Abrasion value (%)
LABMAX	laboratory determined maximum size (mm)
LDI	Linear displacement integrator
LDI50	Roughness measurement at 50 km/h (counts/km)
LDI80	Roughness measurement at 80 km/h (counts/km)

LIST OF ABBREVIATIONS AND SYMBOLS

Abbreviation	Term description
LDQ	change in natural logarithmic value of roughness
LL	Liquid limit (%)
ln	natural logarithm (base e)
LnR	natural logarithm of rate of change of roughness
Log	logarithm (base 10)
LRA	natural logarithm of roughness after blading
LRB	natural logarithm of roughness before blading
LS	Bar linear shrinkage (%)
m	metre, minimum
MATGRP	material group (see chapter 3.2.1)
max, M	maximum
MDD	maximum dry density (kg/m^3)
MDS	Maintenance and Design System (Visser, 1981a)
MLA	predicted annual material loss (mm/yr)
MLC	estimated percentage of mixed layer clays (%)
mm	millimetre
MMP	mean monthly precipitation (m)
MM50	Roughness at 50 km/h (mm/km) (Visser, 1981a)
MM80	Roughness at 80 km/h (mm/km) (Visser, 1981a)
MODLS	modified British Standard linear shrinkage (%)
MOIST	in-situ moisture content at the time of sampling (%)
MP	average rainfall intensity during blading cycle (m/month) (Paterson, 1985)
n	number of observations in the analysis
N	Weinert's N-value (Weinert, 1980)
NAASRA	National Association of Australian State Road Authorities
NC	number of cars per day (< 3 tonnes)
NT	number of trucks per day (> 3 tonnes)
NITRR	National Institute for Transport and Road Research
NPA	Natal Provincial Administration
OCL	estimated percentage of uncommon clay minerals (%)
OMC	Optimum moisture content (%)
OTM	estimated percentage of minerals not classified individually (%)
OMCCBR	unsoaked California Bearing Ratio (%)
P075	per cent passing 0,075 mm sieve (200 mesh)
P13	per cent passing 13,2 mm sieve
P19	per cent passing 19,0 mm sieve
P2	per cent passing 2,00 mm sieve (10 mesh)
P2M	per cent finer than 0,002 mm
P26	per cent passing 26,5 mm sieve
P425	per cent passing 0,425 mm sieve (40 mesh)
P475	per cent passing 4,75 mm sieve (4 mesh)
P6	per cent passing 6,7 mm sieve
PERHEAV	percentage of heavy vehicles
PF	plastic factor (plastic limit x p75) (%^{2})
PI	plasticity index (%)
PL	plastic limit (%)
QI	Quartercar index (counts/km)

LIST OF ABBREVIATIONS AND SYMBOLS

Abbreviation	**Term description**
QI(TG_1)	roughness at time TG_1 in QI counts/km
QI(TG_2)	roughness at time TG_2 in QI counts/km
QTZ	estimated percentage of quartz (%)
r	Pearson correlation coefficient
r^2	r-squared value
R	mean roughness (mm/km) (Hodges et al, 1975)
R	annual rainfall (m) (Hodges et al, 1975)
R	radius of horizontal curvature (m) (Visser, 1981a)
RAIN	mean annual rainfall (mm)
RG_0	roughness after blading (counts/km) (Paterson, 1985)
RGM	average roughness over duration of blading cycle (QI counts/km) (Paterson, 1985)
RMSE	Root mean square error
s	second
S	season dummy variable (Visser, 1981a)
S_p	shrinkage product (BLS x P425)
SABS	South African Bureau of Standards
SAS	Statistical Analysis System
SCBR	soaked California Bearing Ratio (%)
Sev	severity
SG	Sub-group
SMEC	estimated percentage of smectite group minerals (e.g. montmorillonite) (%)
Std dev	standard deviation
SV	percentage of material passing 0,075 mm (Visser, 1981a)
SWAAR	South West Africa Department of Transport
t	t-statistic for significance
T	cumulative traffic in thousands (Hodges et al, 1975)
T1	surfacing type dummy variable (Visser, 1981a)
T2	material dummy variable (Visser, 1981a)
T7	material dummy variable
Ta	annual traffic volume in both directions in thousands of vehicles (Hodges at al, 1975)
TG_1	time elapsed since last grading (days)
TG_2	time elapsed since last grading (days)
TMH	Technical Methods for Highways
TPA	Transvaal Provincial Administration
TRH	Technical Recommendation for Highways
TRRL	Transport and Road Research Laboratory
USDA	United States Department of Agriculture
V	vertical curvature (per cent)
VERM	estimated percentage of vermiculite clays (%)
VOC	vehicle operating costs (Rand)
vpd	vehicles per day
W	road width (m)
$\bar{x}$	mean
yr	year
10% FACT	Ten percent fines aggregate crushing test (kN)
σ	standard deviation
"	inch

CHAPTER 1

INTRODUCTION

1.1 BACKGROUND

The transport infrastructure is probably the most important factor affecting the economic viability of a country, especially developing countries with strong agriculture- or mining-based economies. The road network is usually the major component of the transport infrastructure and consists of a hierarchy of structures ranging from the least important roads conveying very low traffic volumes in rural areas up to those carrying heavily loaded commercial traffic between major urban centres. Even the low volume roads in rural areas are extremely important in that they are the roads by which the farming, timber and mining industries get their products to the market.

At the bottom end of the road hierarchy are cleared earth roads and roads constructed of imported natural gravels, with single and multi-lane surfaced pavements at the top end. The ability of these roads to perform their tasks adequately under all weather conditions increases through this hierarchy, as does the cost of their construction and maintenance.

In nearly all of the developing countries and many of the developed countries a significant proportion of the road network consists of roads constructed with imported natural gravels but which are unsealed (Table 1.1). The upgrading of these roads to sealed pavements is becoming increasingly expensive and unpaved roads are of necessity going to have to be improved to carry the heavier, quicker traffic at affordable cost.

South Africa with its urban first-world economy and rural third-world economy (especially in the independent and self governing states) is in a particularly difficult situation. The often underdeveloped and generally economically strained homelands comprise many kilometres of heavily trafficked, poorly maintained unpaved roads which act as feeder routes to the urban commercial and industrial centres

TABLE 1.1: STATISTICS OF ROAD NETWORKS (1985) IN SOME COUNTRIES (IRF, 1986)

Country	Length of road network (km)	Percentage paved (%)
Great Britain	346 872	100
Germany	490 045	99
Sweden	130 691	69
Spain	318 991	56
Canada	283 796 (1984)	55 (1984)
United States	6 261 876	55
Australia	852 986	50
South Africa	183 851	28
Botswana	8 026	23
Malawi	12 192	21
Swaziland	2 723 (1982)	19 (1982)
Zimbabwe	77 927	17
Zambia	37 310	15
Kenya	64 584	12
Lesotho	4 250	12
Angola	72 300	12
South West Africa	41 701	10
Brazil	1 583 172	7

(Table 1.2). Many of these routes carry large numbers of commuter buses daily. It is therefore imperative that the routes are trafficable at all times and that user costs are minimal. Van Niekerk (pers comm, 1988) maintains that the road infrastructure is more important than the sanitation and water reticulation in a developing area, as the importance of reliable transport to the place of employment determines whether the population retains that employment and can thus afford food and accommodation in the area, i.e. if the transport is unreliable or irregular, the job may be lost.

Apart from this movement of commuters over relatively long distances (which is probably unique to South Africa), the rapid economical movement of agricultural produce to urban markets is of prime importance. Any increase in the cost of this transport by way of poorer trafficability or even the effect of rougher roads on the vehicle operating cost to the farmers will result in increased production costs and hence increased consumer prices. Poor roads may even affect the quality of, for example, farm products resulting in lower market prices or increased packaging costs.

TABLE 1.2: STATISTICS OF ROADS IN INDEPENDENT AND SELF-GOVERNING STATES OF SOUTH AFRICA (1985)

State	Length of road network (km)	Percentage paved (%)
Ciskei	2 486	20
Venda	1 036	17
KaNgwane	861	16
Gazankula (a)	972	15
Bophuthatswana	5 290	14
Lebowa (b)	5 126	11
QwaQwa	231	11
Transkei	9 482	10

a Excluding Mhala region
b Excluding Mapuleng region

Regions where tourism is important to the local economy (e.g. South West Africa (Namibia), the Eastern Transvaal and Drakensberg areas) require a high standard of unpaved roads (the traffic alone does not usually warrant upgrading to sealed roads). The quality of the roads can significantly affect the volume of traffic and influence the overall tourist potential of the region.

1.2 JUSTIFICATION OF THE PROJECT

The major costs involved in unpaved roads are the initial construction cost, the routine maintenance costs and the road user costs, their sum being the <u>total costs</u>. The accurate quantification of these costs is often difficult as they are generally not accounted for individually.

The construction costs of unpaved roads (1987) are estimated to be about R60 000/km but this obviously varies with the availability of suitable material, the quantity of earthworks required (cut, fill and drainage) and the geometric standards applied. Most unpaved roads, however, have limited earthworks. An important part of the construction of most civil engineering projects is the site investigation and material location and in general more attention should be paid to this phase of the project. Although specifications for gravel wearing course materials abound the "best available local material" is usually used. This material is usually located on past experience, with minimal expert geotechnical input, often resulting in the use of material which is not necessarily the most suitable for the prevailing climatic, traffic and maintenance conditions.

Visser (1981a and b) used a cost optimisation maintenance management programme (Maintenance and Design System (MDS)) to evaluate the maintenance strategies of the Bronkhorstspruit district in the Transvaal. The values determined by this study (Visser, 1981b) were used for an approximate analysis (Appendix 1) of the annual maintenance and road user costs for the whole of South Africa (corrected for approximately 100 % inflation between 1981 and 1987) which breaks down as follows:

Regravelling	R 97 189 714
Grader blading	R 63 360 000
Total annual maintenance costs	R160 869 714
Annual road user costs	R2 052 338 000

Significant costs are thus involved in the construction and maintenance of the road infrastructure. Even marginal improvements in material selection and maintenance management procedures should therefore result in improved roads with major savings in maintenance and road user costs. A small percentage of this saving would, however, be absorbed in the increased cost of materials location and testing and higher construction costs.

Visser and Van Niekerk (1987) note that about five per cent of the South African unpaved road network (some 6 600 km) carries more than 200 vehicles per day despite general agreement that it is economically viable to apply a bituminous surfacing to roads with a traffic volume of more than 200 to 300 vehicles per day (Richards, 1978).

The preliminary results of a transportation demand study in Lebowa indicated that some R190 million would be required to address the upgrading of those elements of the rural road network for which a viable internal rate of return could be demonstrated (Van Niekerk, 1988). On a national scale the required funding is estimated at about

R1 200 million (Van Niekerk, 1988). A study of the road network needs for the five years from 1986 to 1990 (Department of Transport, 1981) showed that about R658 million (at 1980 prices) would be needed to reconstruct and surface unpaved roads which met the current traffic warrant for upgrading. From 1990 onwards normal traffic growth would result in an increasing number of roads requiring upgrading.

The financial implications of this are enormous and it is highly unlikely that any significant proportion of the required upgrading will be possible in the foreseeable future. (In fact it is more likely that the length of road requiring upgrading will increase annually as traffic increases). Funds allocated by the World Bank (Harral and Fossberg, 1977) are biased in favour of the construction of unpaved roads. This is mainly due to the lack of maintenance of paved roads in developing areas (Harral, 1988) and a consequent sacrifice of past investments. It is therefore imperative that the heavily trafficked unpaved roads are constructed from the most suitable geological materials and can thus carry the traffic without excessive maintenance or road user costs.

1.3 OBJECTIVES OF THE PROJECT

The main objectives of the project were:

1. To investigate existing gravel roads and identify those important geotechnical and material properties affecting the performance of wearing course gravels.

2. To develop performance-related specifications for gravel wearing course materials, which can be economically applied.

3. To develop unpaved road deterioration relationships for southern African conditions for use in maintenance management systems.

1.4 STRUCTURE OF THE DISSERTATION

The historical background to research into gravel wearing course materials and the large-scale experiments carried out in the late

1960's and early 1970's in Kenya and Brazil are reviewed in Chapter 2. The limited amount of work done locally (mainly in the 1960's and early 1970's) is discussed and the requirements of ideal wearing courses are identified. A number of existing material specifications are compared and the inherent problems are identified.

The development of performance related specifications and performance prediction models required data to be collected from as many different combinations of likely material, climatic and traffic situations as possible. A statistically designed experiment resulted in the maximum benefit being obtained from the minimum number of situations. In Chapter 3 the decision process behind the experimental design is discussed and the important factors used in the experimental design of this study are described. A discussion of the work programme and testing and monitoring techniques employed, follows.

Chapter 4 includes the discussion and analysis of the performance criteria monitored. These analyses identify the geotechnical (and geological to a lesser extent) properties affecting the various performance criteria of unpaved road wearing courses. The roughness of unpaved roads is the most important factor affecting grader maintenance requirements and road user costs and is affected significantly by performance criteria such as corrugations, potholes and stoniness. The analysis of the roughness data and development of roughness prediction models is therefore dealt with separately in Chapter 5. The analysis of the gravel loss data is considered separately in Chapter 6 as this comprises an important part of the maintenance management of unpaved roads, specifically with respect to regravelling requirements and strategies.

Specifications based on the findings of Chapters 4, 5 and 6 are developed in Chapter 7 and are one of the main objectives of the study. An experiment to identify acceptability criteria for the performance of unpaved roads is described and the results are utilised in the specification development. The requirements of good specifications are identified and their relation to the proposed specifications is discussed. Specifications are developed to fulfil the requirements of different users.

In Chapter 8 the implications of the findings of the project, specifically with respect to the economic, social and political consequences are put forward. Improved construction and maintenance procedures based on the experience of this and similar projects are identified. The conclusions and recommendations are summarised in the final chapter, Chapter 9.

CHAPTER 2

EXISTING INFORMATION

2.1 INTRODUCTION

This chapter discusses the development of some of the important philosophies concerning unpaved roads since the earliest published research reports, both locally and overseas. The requirements of ideal wearing course materials are discussed and existing specifications are compared and analysed.

2.2 HISTORICAL RESEARCH

Archaeologists suggest that the the first "roads" apparently originated about 3 500 B.C. following the invention of the wheel, probably by the Sumerians in Mesopotamia (Rose, 1952). The science of road building was however inaugurated by the Romans with the construction of major roadways such as the well-known Via Appia (about 300 B.C.). With the fall of the Roman Empire (476 A.D.) the road network deteriorated to "quagmires in winter and dustbowls in summer" (Rose, 1952). The dramatic increase in travel prior to 1800 led to major advances in the science of road construction with natural materials by the pioneers of the road-building industry such as Tresaguet, Metcalf, Telford and Macadam (Rose, 1953). During the 1800s, Johnson, Gillespie and Owens (Rose, 1953) pioneered the use of sand-clay mixtures because they were cheaper, less dusty and noisy and more resilient than macadam surfaces.

The first published reference to detailed research into gravel wearing course materials (semi-gravel, top soil or sand clay) was that of Strahan (1922) who described the work which had been carried out since 1907, and in which

> "samples were secured from short stretches of existing roads whose firmness and water resisting qualities were notable by contrast with the prevailing dirt roads".

His research resulted in the specification of three classes of soil with limits as follows (Table 2.1):

TABLE 2.1: SPECIFICATIONS PROPOSED BY STRAHAN (1922)

Class	Clay (%)	Silt (%)	Total sand (%)	Sand retained on 60 mesh sieve (%)
Hard (A)	9-15	5-15	65-80	45-60
Medium (B)	15-25	10-20	60-70	30-45
Soft (C)	10-25	10-20	55-80	15-30

These specifications were developed before soil testing and classification methods had progressed very far. Clay was defined as material smaller than 0,02 mm while silt was restricted in the narrow range 0,02 to 0,07 mm and the 60 mesh sieve (apparently 0,25 mm) separated the coarse sand from the fine. Strahan (1922) noted that the coarse aggregate (i.e. greater than sand size)

> "to be valuable must be hard siliceous aggregate, preferably in sizes graded from one-fourth inch (6,7 mm) to two inch (50 mm) diameter. Rocks above three inches (75 mm) are objectionable".

The importance of the coarse sand in the general function of the soil was noted (Strahan, 1922):

- The large amount of sand, particularly the coarser grades, furnishes the hardness and resistance to wear.
- In graded mixtures, the coarse sand embedded in the fine sediments of silts and clays develops a strong mechanical bond.
- The adhesive value of the clay provides the binder which holds the sand in place during dry weather.
- In wet weather the clay softens, but the mechanical bond of the coarse sand remains to support the traffic.
- The overall density prevents the ready penetration of water.

Strahan (1929) followed up this early work with a full treatise on construction and maintenance, incorporating economic aspects. In addition he noted that surface losses (gravel loss) seldom exceeded one inch (25,4 mm) per year. The lack of knowledge on the quality of the clay ingredients with respect to adhesion and composition was a problem and investigations were recommended. Strahan (1929) noted that the work being carried out on the quality of clays by Terzaghi (1925) had opened the way to improved results with road soils.

Hogentogler et al (1929), Travers and Hicks (1933) and Hogentogler (1935) all developed the work of Strahan further, and also investigated additives such as calcium chloride, sodium chloride and even lime and cement to improve the materials. By using recently developed tests such as the Atterberg limits and hydrometer grain-size analyses, soil classification systems were developed. The particle size classifications were revised with the clay and silt fractions extended to smaller sizes. Hogentogler (1935) proposed the following limits for materials which gave good results based on observations and laboratory tests:

Passing 1" (25,4 mm) screen	100 %
3/4" (19 mm)	85-100 %
No 4 sieve (4,7 mm)	55-85 %
10 (2,00 mm)	40-65 %
40 (0,42 mm)	25-50 %
270 (0,053 mm)	10-25 %

Not more than 2/3 of the percentage passing the 40 mesh sieve should pass the 270 mesh sieve.

For the fraction passing the 40 mesh sieve the Plasticity Index should be between one and 15 and Liquid Limit less than 35.

The use of recently developed compaction techniques (Proctor, 1933) and the removal of large stones (by raking if necessary) were shown to improve the road performance (Hogentogler, 1935). Runner (1935) surveyed early United States experience and included the following specifications:

(54405)

Percentage passing 1" (25,4 mm) circular mesh screen	100
Percentage passing 1/4" (6,7 mm) sieve	35-50
Percentage wear in abrasion test - round pebbles	< 25
Percentage wear in abrasion test - broken fragments	< 15

In 1943, the Highway Research Board (HRB, 1943) recommended specifications for surface course materials based on the ASTM (ASTM, 1944) and AASHO (AASHO, 1942) specifications:

Sand clay:

Passing 1" (25,4 mm) sieve	100 %
Passing No 10 (2,0 mm) sieve	65-100 %

Grading of material passing No 10 (2,0 mm) sieve:

Passing No 10 (2,0 mm) sieve	100 %
Passing No 20 (0,84 mm) sieve	55-90 %
Passing No 40 (0,42 mm) sieve	37-70 %
Passing No 200 (0,074 mm) sieve	8-25 %

Coarse graded aggregate:

Passing 1" (25,4 mm) sieve	100 %
Passing 3/4" (19 mm) sieve	85-100 %
Passing 3/8" (10 mm) sieve	65-100 %
Passing No 4 (4,7 mm) sieve	55-85 %
Passing No 10 (2,0 mm) sieve	40-70 %
Passing No 40 (0,42 mm) sieve	25-45 %
Passing No 200 (0,074 mm) sieve	10-25 %

Note: If the mixture consists of angular particles, a higher percentage passing the No 10 sieve is desirable than if rounded particles predominate.

The plasticity index should be between 4 and 9 % and the percentage passing the No 200 sieve should not exceed two-thirds of that passing the No 40 sieve (i.e. dust ratio not more than 2/3).

Most work on wearing course gravels after this appears to revolve around the ASTM specifications devised in 1940 and published in 1944 (ASTM, 1944). Frost (1968) studied a number of specifications and concluded that classification tests were not completely satisfactory as criteria for specification purposes but should be combined with a strength criterion. The California Bearing Ratio (CBR) test was recommended. The UNESCO guide for low cost roads (UNESCO, 1971) recommended specifications published in 1952 (Road Research Laboratory, 1952) based on the ASTM specifications.

Much of the subsequent research into unpaved roads concentrated on the problem of corrugations, with numerous investigations being carried out between 1924 (Ladd, 1924) and 1980, some 28 of these being summarised by Heath et al (1980). This work is discussed fully in Chapter 4.10.

2.3 PREVIOUS STUDIES - OTHER COUNTRIES

The first of the large-scale investigations into unpaved road performance was carried out in Kenya between 1970 and 1975. This was a multidisciplinary project investigating both paved and unpaved road deterioration (Hodges et al, 1975), vehicle operating costs (Hide et al, 1975) and developing a road transport investment model (Robinson et al, 1975). The unpaved road experiment consisted of 46 sections of existing roads, each section being one kilometre long. The various combinations of two different rainfall regions and four material types (lateritic, quartzitic, volcanic and coral) together with the horizontal and vertical geometry made up the necessary experimental sections. The study resulted in a number of models relating the gravel loss, roughness, rut depth development and surface looseness to the traffic, environmental characteristics and design, maintenance and construction standards. The quality of the maintenance operation was identified as the most important factor affecting the surface condition.

In 1975 a second large experiment was initiated in Brazil along similar lines to the Kenya investigation. The "Brazil Road Costs Study" concentrated on deterioration models and road user costs for paved and unpaved roads (Visser, 1981a). The unpaved experiment involved 48 sections, each 300 m long with material type, traffic and horizontal and vertical geometry being the independent variables. Three different material types were used (lateritic, quartzitic and earth). Performance related models for the prediction of the roughness, gravel loss and rut depth were developed and incorporated into a model for the programming of maintenance and management of unpaved road networks (MDS, Visser, 1981a).

A number of other investigations on smaller scales have been primarily aimed at investigating maintenance procedures. Jones et al (1984a) described an experiment in Kenya following on from the previous one but replacing coral materials with sandstone and investigating different maintenance strategies.

Roberts (1983) conducted research into the performance of unpaved roads in Ghana, with different maintenance strategies. A simple experiment using climate, traffic and terrain as variables included 13 sections each 10 km long. Each of the sections was divided into three equal lengths and different maintenance procedures used on each.

Beaven et al (1987) carried out a number of experiments in Ethiopia using basalts as wearing course materials. There was generally poor agreement between the measured performance and the performance predicted from the models developed in Kenya (Hodges et al, 1975). It was, however noted that for more than 50 vehicles per day it was cost-effective to carry out material processing in order to remove or crush oversized material (larger than 37,5 mm).

Other experiments which have been carried out in India, the Caribbean and Bolivia (Paterson, 1985) have yielded limited data.

Paterson (1985) analysed all of the existing data and developed an "improved" set of models which form the basis of the deterioration relationships used in the Highway Design and Maintenance Model (HDM 3) for the management of low volume road networks in developing areas (World Bank, 1985).

Wearing course materials in the United States have always been closely specified by the American Association of State Highway and Transport Officials (AASHTO), American Society for Testing Materials (ASTM), the Federal Highway Administration (FHWA) and the United States Forest Service (USFS). A theoretical study of these specifications has been carried out, resulting in a relaxation of some of them being recommended (Meyer and Hudson, 1987). This relaxation, however, is generally fairly conservative allowing a maximum plasticity index for example of 15 per cent in hot dry areas (the previous maximum was 9).

An important aspect of all of the investigations which have been carried out over the last 20 years or so has been the limited input in terms of material characterisation and testing, and geological classification, with the major emphasis being on maintenance procedures, management systems and road user cost studies. Many of the experimental sections were far too long to be able to assume homogeneity of the materials.

2.4 PREVIOUS STUDIES - LOCAL

Although a number of specifications were in use in South Africa in the early 1960s, very little information as to their origin is available. Fossberg (1962) refers to the Natal Roads Department specifications which were based on the test results of a number of gravels from roads that were considered to have been constructed of "good all-the-year-round gravels".

The first serious research into the performance of gravel wearing course materials in southern Africa was initiated at the National Institute for Road and Transport Research (NITRR) in the early 1960s (Fossberg, 1963a). This resulted mainly in recommendations concerning possible future research. The importance of more care in the selection of wearing course gravels, and soil blending if necessary were emphasised (Fossberg, 1963b). Most of the work at this time, however, involved dust palliatives (especially sulphite lye) (Fossberg, 1966).

During the late 1960s, Netterberg (1969, 1978) investigated the use of calcrete in unpaved roads in South West Africa, the northern Cape and the western Transvaal. An empirical analysis of borrow-pit and in-service calcretes resulted in comprehensive specifications for calcrete wearing course gravels (Netterberg, 1978). Because of the unique properties of calcretes (e.g. self stabilization), the likelihood of these specifications being directly applicable to many other material types was doubtful.

Visser (1981b) investigated maintenance strategies for the Bronkhorstspruit district of the Transvaal and later for a portion of the Middelburg district (Visser et al, 1983). These studies were primarily aimed at comparing the results of the Maintenance and Design System (Visser, 1981a) developed in Brazil with maintenance strategies presently in use locally. Minimal attention was paid to the material characteristics and all the materials involved were necessarily classified according to the material types on which the Brazil study was based.

Theron (1983) evaluated the MDS in the Witbank district of the Transvaal and compared a number of the materials with the standard specifications of the Transvaal Provincial Administration (TPA, 1973). He concluded that many of the materials used did not comply with the specifications. Performance varied from good to bad for both the materials conforming with, and those not conforming with the specifications.

A marked deficiency of all the work carried out in South Africa has been the minimal number of different geological material types investigated.

2.5 IDEAL WEARING COURSE REQUIREMENTS

An ideal wearing course gravel should satisfy the following requirements (mostly after Netterberg, 1985 and Paige-Green and Netterberg, 1987):

- The ability to provide an acceptably smooth and safe ride without excessive maintenance.
- Adequate stability, i.e. the resistance to deformation under both

wet and dry conditions.

- The ability to shed water without excessive erosion or scouring.
- Resistance to the abrasive action of traffic.
- Freedom from excessive dust in dry weather.
- Freedom from excessive slipperiness in wet weather without excessive tyre wear.
- Low cost and ease of maintenance.

Ability to provide an acceptably smooth and safe ride

The ability of the road to provide an acceptably smooth and safe ride is one of the most important requirements and depends on a number of factors. The main factors affecting roughness and safety are discussed in more detail later in this thesis but have been identified as corrugations, stoniness, potholes and surface erosion (Paige-Green and Netterberg, 1987). All of these problems can be attributed either directly or indirectly to material properties. Stoniness should be reduced to a minimum by better material selection or construction procedures before compaction of the wearing course. The removal of oversize material also reduces the number of potholes caused by plucking of stones during grader blading. The selection of a material with a smaller erosion potential (i.e. higher cohesion) reduces erosion runnels and probably inhibits the formation of corrugations. The safety of a road appears to a large extent to depend on the directional stability of a vehicle (Paige-Green and Netterberg, 1988) which is in turn influenced by the roughness of the road.

Stability

Neither the wearing course nor the subgrade, to a lesser extent, should deform under applied loads in either wet or dry conditions. A lack of stability results in potholes, rutting and general deformation and poor wet weather trafficability. An adequate material strength (e.g. soaked CBR) and gravel thickness is necessary to ensure stability. An adequate CBR will also ensure that there is a sufficient load spreading capacity so as not to overstress weak subgrades.

Ability to shed water

Should water accumulate on the surface of the wearing course, localised areas of reduced strength due to soaking often result in the formation of potholes and depressions. Of greater concern is the formation of transverse erosion channels in certain materials due to excessive water flow rates along the cross-fall. Longitudinal erosion has less of an effect on the riding quality but is important from the material loss, maintenance and safety aspects. Adequate compaction should be achieved to improve the water shedding capability and to decrease the permeability of the wearing course as far as possible.

Resistance to abrasion by traffic

The abrasive action of traffic results in the development of ruts, the generation of loose material and an overall material loss with time. This necessitates regravelling, the most costly maintenance operation. An adequate cohesion is necessary to reduce the abrasive action of traffic. A suitably high level of compaction with water assists in the reduction of ravelling under traffic. Routine grader maintenance also helps in reducing abrasion by moving the wheel tracks and forming a thin blanket of loose material.

Freedom from excessive dust

Excessive dust is undesirable primarily from the safety aspect, but also from the effect on mechanical wear, driver discomfort and the probable effect on roadside vegetation and crops. During discussions with road users many rated the importance of dust above riding quality, especially in terms of driver and passenger discomfort.

Freedom from excessive slipperiness without excessive tyre wear

The road should not be slippery in the dry condition or become slippery when wet. This is generally overcome by the presence of a well graded fines/aggregate mix. The aggregate however, should not be of such a size that excessive tyre wear or roughness is introduced.

Low cost and ease of maintenance

Local materials are required to maintain a low construction cost. These should form a wearing course capable of being reshaped to a good riding surface during routine grader blading. Maintenance of most unpaved roads is not a problem unless excessive hard, oversize material is present. It is important to control the frequency of blading by its necessity (programmed maintenance) and not to follow a fixed period type of programme (systematic maintenance).

These requirements in relation to the serviceability, safety, vehicle operating costs and effect on the environment and agriculture have been summarised in Table 2.2. Comments on some material requirements are included.

2.6 SPECIFICATIONS

Specifications for gravel wearing course materials abound. Most countries have their own specifications, while many provincial and local authorities have their own. The origin of most of these specifications is unknown, although it is expected that some have been transferred from other regions or countries and others are based on local experience. An important aspect of all the specifications is the variation between them. Fossberg (1962, 1963a, 1963b) carried out an extensive survey of wearing course materials (Netterberg, 1985) and concluded that:

- There is no universally applicable method for the design of wearing courses for unpaved roads.
- Unlike paved roads, unpaved roads are profoundly affected by the weather.
- Specifications vary widely and the allowance made for climate, if any, also varies widely.
- Performance data and suitable test methods for wearing course materials are lacking.
- Most specifications rely heavily on indicator tests i.e. grading and Atterberg limits.
- The importance of proper compaction is stressed.

TABLE 2.2: RELATIVE IMPORTANCE OF WEARING COURSE REQUIREMENTS

Requirement	Service	Safety	Comfort	Total costs	Environment/ agriculture	Material requirements
Smooth ride:						
Roughness	C	B	B	A	B	Good grading, adequate cohesion and strength.
Stability:	A	B	B	A	B	Good grading, strength + density
Water shedding:						
Erosion	B	B	A	A	B	Adequate cohesion.
Trafficability	A	B	B	A	B	Good grading, strength + density
Resistance to abrasion:						
Loose material	C	A	B	B	C	High density and cohesion.
Gravel loss	A	C	B	A	B	High density and cohesion.
Rutting	B	B	B	B	C	High density and cohesion.
Freedom from dust:	C	A	A	C	A	Adequate plasticity index.
Not slippery:	C	A	C	C	C	Good grading, not too plastic.
Easy maintenance:	B	B	B	A	C	Little oversize.

A - very important
B - important
C - unimportant

- Plasticity should always be considered in relation to the grading.

Unfortunately, nature has not been on the side of the road engineer with the provision of wearing course materials. In arid areas a higher plasticity in the wearing course is preferred as wet conditions leading to poor trafficability are atypical and traffic is often light. Materials with low plasticity tend to ravel and corrugate. In these areas disintegration of rocks by physical weathering is typical and subordinate formation of relatively inactive clays (e.g. illite) results in low plasticity soils and gravels. The deposition of wind-blown sand is also characteristic of arid areas resulting in the addition of further non-plastic sandy material.

In wetter areas a low (but sufficient to keep the material cohesive) plasticity material is preferred, as higher plasticity material quickly becomes impassable during prolonged wet periods. The natural process of weathering, however, produces higher plasticity, clayey soils in wet areas by decomposition of the primary rock forming minerals to the higher activity clays (e.g. montmorillonite, chlorite, vermiculite).

The result of this is illustrated in Figure 2.1 where materials other than pedocretes generally plot in Zone 1. Ideal wearing course gravels would plot in the area of required materials shown as Zone 2, but, other than pedocretes, this combination is seldom found naturally. This generally results in the use of moderately cohesive materials (Zone 3) and a concomitant average performance (the point of intersection of the lines joining Zones 1 and 2 for the different climatic areas). There are of course exceptions to the rule and certain ideal materials do occur in some areas, but these have mostly been used up or are becoming depleted.

Most of the present specifications used in southern Africa and a few of the important ones used overseas and probably applied in many developing countries are summarised in Appendix 2. A guide to the behaviour of wearing course gravels compiled by the Natal Roads Department, and apparently based on the performance of in-service materials (Table 2.3) was probably used to devise some of the South

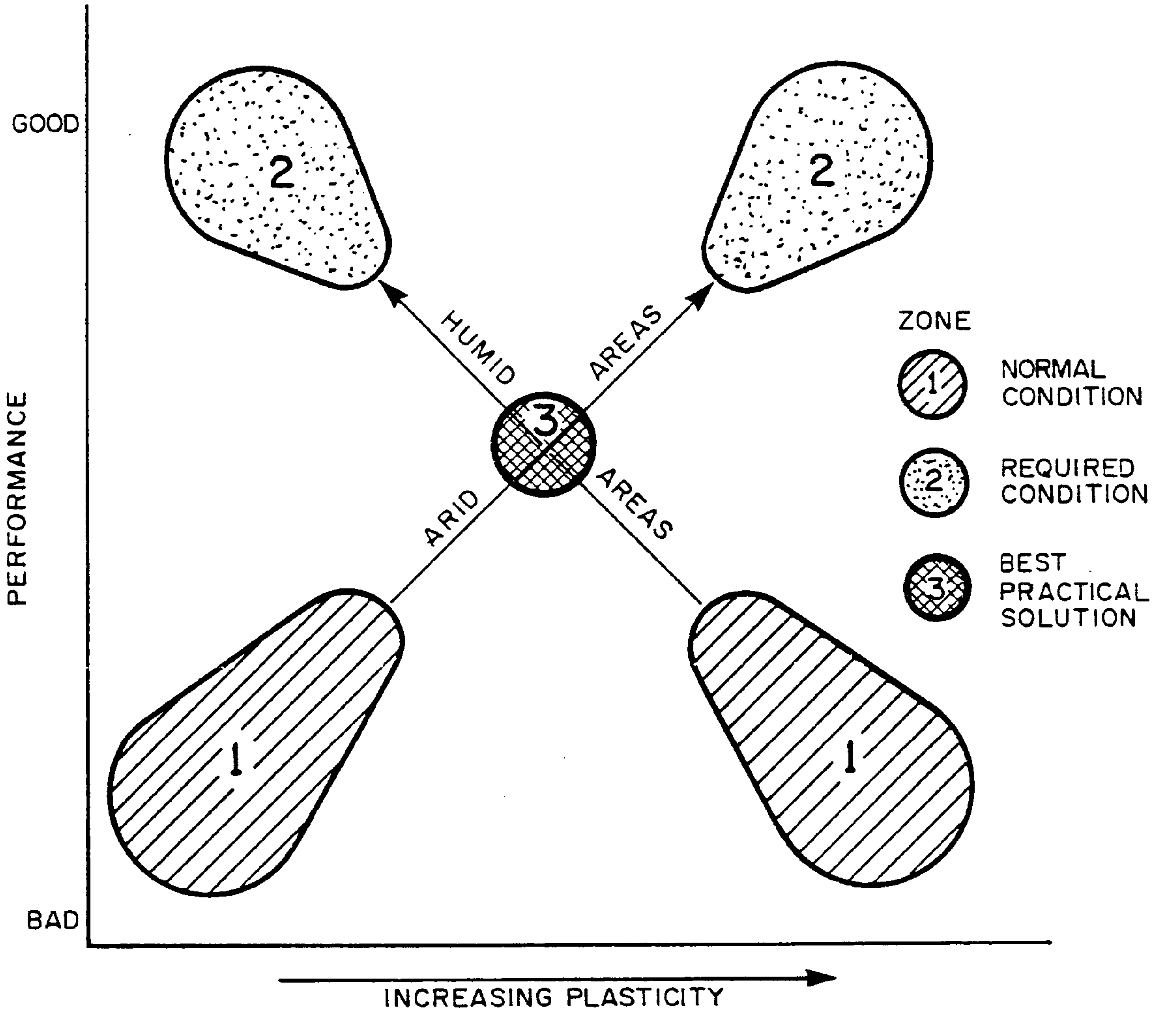

FIGURE 2.1

AN ENGINEERING DILEMMA - CLIMATE AND PERFORMANCE OF UNPAVED WEARING COURSE MATERIALS

African specifications (Fossberg, 1962).

TABLE 2.3: NATAL ROADS DEPARTMENT WEARING COURSE GUIDE (NPA, ca 1961)

Performance	Liquid limit (%)	Plasticity index (%)	Coarse and medium sand (%)	Clay (%)
Corrugates	< 20	-	> 55	-
Potholes	> 35	-	< 30	-
Dusty in dry weather	< 20	-	< 30	-
Ravels in dry weather	< 20	< 6	-	< 6
Slippery in wet weather	-	> 14	-	-
Cuts up in wet weather	-	-	< 25	> 10

The origin of most of the specifications described is not known, but there is little evidence to suggest that they have been empirically derived from performance related studies. Some of them (e.g. AASHTO, 1974) have tight grading envelopes and unrealistic plasticity criteria, and were probably derived from theoretical considerations.

It is usually not possible to carry out routine acceptance testing of materials for unpaved roads during construction due to time and cost constraints. The use of tight limits such as those specified by AASHTO (up to seven grading criteria are specified) are unlikely to be complied with for even the highest class of unpaved roads used locally. During the initial test programme carried out for material location and selection purposes, appropriate specifications should be used and a compromise between the cost of further testing or investigation and the risk of using unacceptable materials should be reached. The inherent variability of weathered geological materials precludes the use of tight specifications, and thus time-consuming tests should be limited as far as possible. The use of a greater number of quick simple tests is far more beneficial and gives a better indication of the range and potential variation of the material properties which will be used in the road.

2.7 DISCUSSION

The earliest published work on gravel wearing course materials indicated the importance of the particle size distribution. The influence on riding quality of particles larger than 75 mm was emphasised and the hardness of the particles was identified as an important property. Subsequent research into gravel wearing course materials has generally pursued the importance of grading with little emphasis on the importance of eliminating oversize material, although many specifications limit the maximum particle size.

Even though Strahan (1922) noted the importance of the various fractions of the sand component, few of the subsequent researchers have looked at this aspect separately from the overall grading. Netterberg (1978) differentiated calcrete gravel materials on the basis of grading which resulted in different material requirements depending on the fineness of the material.

Many of the older specifications rely heavily on Talbot's curves to give maximum density and hence maximum stability. These curves use the formula

$$p = 100\ (d/D)^n \tag{2.1}$$

where p is the per cent by volume of material passing sieve size d for maximum particle size D and n an exponent which is about 0,5 for maximum density (historically known as Fuller's maximum density curve). Most specifications adopt upper and lower limits of n of 0,5 and 0,3 which produces a reasonably tight envelope. Analysis of the specifications summarised in Appendix 2 indicates that for a nominal 19 mm material the coefficient for n for the 4,75 mm fraction varies from 0,12 to 0,57. The values of n for the passing 0,075 mm fractions vary from 0,19 to 0,48. A number of factors such as differential particle densities, gap-grading and the non-spherical nature of soil particles however affect the applicability of these idealised grading curves. The typical grading analysis is based on the percentage by mass, therefore different particle densities in the various fractions will result in spurious grading curves. It does, however, appear that a move away from rigid maximum density curves is occurring in the more recent specifications.

Along with the particle size distribution the plasticity index and liquid limits have been traditionally specified for wearing course gravels. Strahan (1929) noted the importance of the characteristics of the clay minerals in the behaviour of the materials. Subsequent specifications all included one or more of the plasticity index (PI), the liquid limit (LL) or linear shrinkage (LS). All of the specifications summarised in Appendix 2 contain at least the plasticity index although the limits range from slightly plastic to a PI of 30 per cent. Five of the 15 specifications allow for the effect of various climatic conditions as discussed in Section 2.6. This effect mainly takes into account the influence of the plasticity on slipperiness and stability when wet. As the climatic regimes become more arid, a higher plasticity is generally allowed. Problems may, however, be encountered in locating higher plasticity material in the arid areas. Frost (1968) investigated the use of the plasticity index and liquid limit in wearing course materials and concluded that the theoretical basis for their use was unjustified and that they should be replaced by strength tests. A major problem when comparing plasticity indices is the different sample preparation techniques and test methods used by various authorities which often results in significantly different values (Netterberg and Paige-Green, 1988a; 1988b).

Of the 15 specifications in Appendix 2, five include limits for material strength and five specify aggregate "durability" requirements. The material strength requirements are generally in terms of the soaked California Bearing Ratio (CBR) with a range of prescribed values between 15 and 60 per cent (at various compaction efforts). Frost (1968) suggested that a minimum CBR of 40 is generally associated with good roads. The CBR value is strongly dependent on the density at which it is penetrated during testing. This is often poorly defined in the specifications but the lowest value specified (15 %) is at a density of 93 per cent of modified AASHTO. The higher values are probably tested at higher relative densities. The specification of the higher CBRs (e.g. 45 at 93 per cent and 60 at 95 per cent) appear to be excessive for the type of gravel generally used in unpaved roads.

Only 14 of the sections monitored during this project had CBR strengths in excess of 45 at 93 per cent compaction. It is unlikely that many more materials would have CBR strengths in excess of 60 at the higher compaction effort. The NAASRA guide (NAASRA, 1980) specifies a minimum CBR of 60 at the expected in-situ moisture and density. The differences in sample preparation, compaction efforts, test procedures and interpretation of the results of the CBR, however, make direct comparison of results difficult (Netterberg and Paige-Green, 1988a).

With regard to aggregate strength (actually hardness and abrasion resistance) two of the specifications use the Los Angeles Abrasion value (LAA) while two are in terms of the 10 % Fines Aggregate Crushing Value (10 % FACT). The AASHTO specifications limit the LAA loss to a maximum of 50 per cent while the NPA specify a loss in the range of 30 to 60 per cent. The maximum value for the loss is specified in order to eliminate those soft materials which will degrade and form potholes, or ravel. The minimum value for the loss is, however, necessary to reduce the quantity of larger hard stones which will not abrade and result in a rough, stony surface. The other strength criterion used in the specifications is the 10 % FACT with a range of specified values between 50 and 135 kN.

Netterberg (1978) specified the hardness in terms of the Aggregate Pliers Value (APV) and Aggregate Fingers Value (AFV), two simple field tests requiring minimal apparatus.

Current specifications for unpaved wearing course materials have been fully reviewed by Netterberg and Paige-Green (1988b) and are not discussed further in this chapter. The applicability of existing specifications used in southern Africa is, however, analysed in Chapter 7.5.

The results of the experiment carried out in Kenya (Hodges et al, 1975) have been found not to be transferable to other areas (Beaven et al, 1987). A major problem with the prediction models developed in Kenya is that those for roughness and rut depth prediction were cubic equations with traffic being the only factor. These models are suitable for long term estimates of deterioration of roads with low maintenance but are unsuitable for normal maintenance management systems or short term predictions. An obvious omission in these models is a reference to the material properties.

The Brazil (Visser, 1981a) and World Bank (Paterson, 1985) models are based only on laterite and quartzitic wearing course materials. The applicability to other materials has not been studied although, on a network basis, the predicted gravel losses and roughness progressions were found to correlate fairly well with those determined in practice (Visser, 1981b).

A number of investigations into the performance of unpaved roads have been carried out since the 1960's. Most of these have, however, been restricted to tropical areas (Kenya, Brazil, Ghana, Bolivia) and their findings are thus primarily applicable to higher rainfall regions (many with no marked seasonal differences). The geological materials in these regions are generally quartzitic or lateritic, typical of the soils formed under tropical weathering conditions. Laterites are well known for their non-classical geotechnical behaviour characteristics such as self cementing properties and the difficulties with lime stabilization. Southern Africa with its relatively thin soil cover, extremes of climate with marked rainy seasons and variability of material types formed under a number of weathering regimes (mainly temperate to arid at present) thus requires material specifications adapted to the local geological and climatological conditions.

2.8 STATISTICAL METHODS

The development of the prediction models and to a lesser extent the specifications relied heavily on a statistical approach. The methods used generally followed standard techniques as described in Moroney (1951), Blommers and Lindquist (1965), Draper and Smith (1966) and the Statistical Analysis System (SAS, 1985) manuals.

Standard statistical parameters such as the mean, mode, variance, standard deviation and percentiles were liberally used in the analyses. The probability distributions of the gravel loss and roughness measurements were assumed to be normal i.e. unbounded in both directions. This is a valid assumption in nearly every case with almost symmetrical distributions around the mean.

The prediction models for roughness progression and gravel loss were derived from multiple linear regression analyses. This technique allows the investigation of any relationships between one dependent variable and two or more independent variables. The least squares method is used to estimate the "best" fit regression model, at which point the vertical differences between observed and predicted data are minimised. The value of the Pearson correlation coefficient (r) squared is an indication of the goodness of fit with a value of one indicating a perfect correlation and zero indicating no correlation. The r-squared value should be interpreted in terms of the number of data points used in the analysis. As the number of data points used in the analysis increase, a lower value of the r-squared becomes significant (GEIPOT, 1980). The root-mean-square-error (RMSE) is defined as the square root of the ratio of the error variance and the degrees of freedom (the difference between the number of data points used in the analysis and the number of terms in the model). The minimising of the root-mean-square-error in conjunction with the maximising of the r-squared and F-value was used in the selection of the best-fit model.

The analysis of variance which is the basis of the regression analyses assumes homogeneity of variances. The analysis is robust with respect to this assumption only if equal sample sizes are used. This was not the case in the analyses for roughness and logarithmic transformations were applied to force them to homogeneity for the analysis.

The F-statistic (the ratio of the explained and unexplained variances (mean squares)) indicates the statistical significance of the overall regression model. The F-statistic is compared with a theoretical tabulated vehicle (done automatically in the SAS program) to indicate the significance. Generally, if the F-value is large a highly significant regression model is indicated.

A similar analysis, the t-statistic is indicative of the significance of an individual coefficient in the presence of all other regressors (independent variables). This is also carried out automatically in the SAS program.

The 95 per cent confidence intervals were determined by assuming a normally distributed population and taking two standard deviations above and below the mean (Moroney, 1951). This is a standard statistical procedure.

2.9 CONCLUSIONS

In this chapter, the background to the following topics was given:

- past research, both overseas and local,
- requirements of ideal wearing courses,
- existing specifications,
- statistical techniques employed.

Although extensive research has been carried out on wearing course materials for unpaved roads since the early 1920's, inadequate attention has been directed towards the influence of the geotechnical properties of a material on its performance as a wearing course for an unpaved road. Most of the recent studies have concentrated specifically on the maintenance aspects of unpaved roads.

Numerous specifications (generally of obscure derivation) are currently in use in southern Africa. Their reliability and applicability in practice, however, have not been rigorously quantified and there is little evidence to indicate that any of them are performance-related.

The only performance-related studies fully documented are those by Netterberg on the use of calcretes as wearing courses in southern Africa.

CHAPTER 3

EXPERIMENTAL DESIGN, TESTING AND MONITORING

3.1 INTRODUCTION

The primary objectives of this project were to study the influence of geotechnical properties on the performance of wearing course materials, to improve existing material specifications for unpaved roads and to develop deterioration relationships for southern African materials and conditions. Fossberg (1963a), some 20 years prior to starting this project, concluded that

> "... a survey of existing gravel roads, relating performance to traffic, climate and soil conditions ..."

deserved study in South Africa. The best way of carrying out this type of project is by investigating a number of randomly selected in-service roads which cover a full range of the major factors influencing their performance. These factors are numerous but the major ones include material properties, climate, traffic volume and type, and geometrics. This chapter describes the philosophy behind the experimental design decisions and processes, the work programme followed, the testing and monitoring carried out and the methods used.

3.2 EXPERIMENTAL DESIGN OF THE STUDY

To meet the objectives of the project the fieldwork was done in accordance with a detailed experimental design. The experiment was the first of this type of investigation in southern Africa and is apparently the largest done anywhere in the world, with respect to the number of different materials.

From previous studies on unpaved roads (Fossberg, 1963a; Hodges et al, 1975; Visser, 1981a) the material properties, climate, traffic and geometrics are generally considered to be the major independent variables (factors) affecting the performance of unpaved roads. The

Brazil study was almost entirely in one climatic region (humid) with an annual rainfall of 1250 to 2000 mm (GEIPOT, 1980) and climate was not one of the factors in the experimental design. Both the Kenya and Brazil studies used factorial experimental designs successfully and no advantage would have been gained by using any other type of experimental design for this study.

A properly designed factorial experiment using typical South African material properties, climatic zones and traffic volumes as the major factors allows the results to be extrapolated over a much wider area than the original experimental area, i.e. the results of a factorial experiment in the Transvaal can be extrapolated to other parts of South Africa, Botswana, Lesotho, Swaziland, South West Africa and Zimbabwe which have similar climatic, material and traffic conditions. In fact, the results from this study which covered a total area of about 128 700 square kilometres can probably be applied to over 2 000 000 square kilometres south of the Zambezi and Kunene Rivers. For this reason and the logistics of the monitoring travelling it was decided to restrict the main project to the Transvaal. Even this resulted in a total distance of more than 250 000 kilometres being travelled during the project.

Recent work on deterioration and prediction models (Paterson, 1985) re-evaluated the data from the Brazil study for the following reasons:

- Detailed time-series observations of condition over a period of up to three years on existing sections of the road network were obtained.
- The sections were selected by factorial design and cover wide ranges of the factors.
- The measurements were systematic, frequent and reproducible.

The only drawback mentioned in the re-evaluation was the limited number of materials. The present study was designed to conform with the above reasons and, in addition, to include more levels for the material factor.

3.2.1 Materials

The use of a factorial design requires the sub-division of each factor into discrete ranges or levels. The obvious levels for the material factor would be a number of even divisions of an important property, such as plasticity index, in the range non-plastic to a plasticity of 25 or 30 percent. This is impractical as it would have required either the sampling and testing of all possible roads and then a selection of the suitable ones or else a random selection of the required number of roads. This latter procedure would in all probability result in a disproportionate representation of factor levels unless an uneconomically large number of roads was investigated.

The use of rock genesis (e.g. igneous, metamorphic, sedimentary or pedogenic) as a factor level was contemplated but the variability of the weathering products and residual material within any one group was considered to be too extreme. The classification system proposed by Weinert (1980) grouped rock types by their weathering products and geotechnical behaviour, irrespective of their genesis. Thus, for example, within the acid crystalline group, both igneous and metamorphic rocks which produce similar weathering products under similar climatic regimes, are grouped. This classification system contains nine material groups as follows:

1 Basic crystalline rocks - e.g. amphibolite, basalt, dolerite
2 Acid crystalline rocks - e.g. felsite, gneiss, granite
3 High silica rocks - e.g. chert, quartzite, hornfels
4 Arenaceous rocks - e.g. arkose, sandstone, mica schist
5 Argillaceous rocks - e.g. shale, mudstone, phyllite
6 Carbonate rocks - e.g. dolomite, limestone, marble
7 Diamictites - e.g. tillite
8 Metalliferous rocks - e.g. ironstone, magnetite, magnesite
9 Pedocretes - e.g. calcrete, ferricrete, laterite

Of these groups, diamictites and metalliferous rocks are of minimal importance in the construction of unpaved roads in the Transvaal due to their limited occurrence, and high transportation cost in the case of the metalliferous rocks. Diamictites are important construction materials in Natal, but it was considered that the material groups selected would adequately cover the range of properties exhibited by typical diamictites. The natural weathering process of carbonate rocks results in their almost total dissolution with a chert-rich residue which is used in unpaved road construction. This chert is classified as a high silica rock. Thus, six important material groups were incorporated in the analysis in the Transvaal, namely basic and acid crystalline, high silica, arenaceous and argillaceous rocks and pedocretes.

3.2.2 Climate

The choice of levels for the climatic factor was initially thought to be best based on rainfall. The Kenya study concluded, however, that for an annual rainfall of between 400 and 2 000 mm the performance of unpaved roads was independent of rainfall (Hodges et al, 1975). While the rainfall in the Brazil study was within this range it was expected that areas with an annual rainfall of less than 400 mm would be included in this study. A second climatic indicator which has been used in local paved road performance studies (Emery, 1985), Thornthwaite's moisture index (Im) (Thornthwaite, 1948), was considered. The limited range of this index in the Transvaal and the lack of well-defined, performance-related, discrete levels weighed against its use. Weinert's N-value (Weinert, 1980) was considered to be the most suitable climatic indicator for the following reasons:

- The N-value is based on annual rainfall and evaporation during the warmest month (mostly January in southern Africa) and characterises the climate into discrete geotechnical provinces. The N-value can be easily determined for any site by the following equation:

  $$N = 12\, E_j / P_a \qquad (3.1)$$

  where E_j is the computed evaporation from a shallow freewater surface during January (or the warmest month) and P_a is the total annual precipitation.
- The N-values of 2, 5 and 10 are distinct physiographical

boundaries (Weinert, 1980).

- The weathering products of the different groups used as levels for the material factor are unique for the N-value province in which the weathering occurs (Weinert, 1980).

The mean annual rainfall and Thornthwaite Index were, however, determined for each test section and included in the final analysis.

3.2.3 Traffic

The traffic volume for the various roads was available from existing Transvaal Provincial Administration records (TPA, 1982). It was initially decided to have two traffic levels, less than 150 vehicles per day (vpd) and greater than 150 vpd based on a study of the traffic counts. However, when the possible test sections were identified it was found that a split at 100 vpd was more realistic in the rural areas being investigated. During the field work, however, it became apparent that the traffic counts were not all totally accurate, and counts on or near the actual sections were carried out. These indicated that the original counts under-estimated the traffic for many of the roads.

A problem with the traffic counts is that they are generally in terms of vehicles per day with no differentiation by direction. In addition seasonal effects such as the extra traffic during the crop harvesting or holiday seasons are difficult to estimate. These problems are common to most research into roads and it is standard practice to use the traffic in both directions when analysing results (Hodges et al, 1975; Visser, 1981a). The HDM 3 model (World Bank, 1985) classifies the traffic into nine categories (not subdivided by direction), but this was considered extreme for a study of this nature. Traffic on unpaved roads is by nature very variable, very seasonal and directional and if the categories are to be used in the analyses, an estimate of the load should be used. Anything more than average daily traffic and percentage of heavy vehicles (over three tonnes) was considered an unnecessary accuracy for an inherently variable parameter.

3.2.4 Design

A factorial design similar to those used in Kenya and Brazil was used with the actual test sections selected in a stratified and clustered manner as follows:

(a) Suitable areas of each of the climatic regions were selected with the logistics of the project in mind. The primary reason for this was to reduce monitoring distances to a practical minimum.

(b) Roads conforming with the required material and traffic characteristics were identified. This was done by obtaining a list of about six roads (which were unlikely to be regravelled or upgraded in the following three years) for each of the material and traffic combinations from the Regional Engineers in each of the relevant climatic regions.

(c) The traffic counts were confirmed against the available records.

(d) Each of the roads was then visited and those roads which had suitable sections were noted. This suitability was based on the presence of a straight, flat, homogeneous section of road 300 metres long (Section A). At least 300 metres of reasonable approach was required at each end in order to reach 80 km/h during roughness monitoring. In addition a 300 metre section of road on a hill or horizontal curve was selected as close to the straight section as possible in order to investigate the influence of the road geometrics (Section B). It was not always possible to satisfy all these criteria and some roads therefore only had an A section or else had an A section with a slight gradient or curve, although this was kept to a minimum. This gradient or curvature was recorded and used in the analyses.

(e) Those sections on a convenient monitoring route were finally selected for the experiment and resulted in the experimental design matrix shown in Figure 3.1. Each of the cells of the matrix contains a road which satisfies each set of combinations. No suitable roads constructed of arenaceous materials could be found

SURFACING MATERIAL (CLIMATE)	TRAFFIC	1 ACID CRYSTALLINE	2 BASIC CRYSTALLINE	3 HIGH SILICA	4 ARENACEOUS	5 ARGILLACEOUS	6 PEDOCRETE
N < 2	< 100 vpd	576 (98/39)	796 (32/34) 1298 (25/16)	625 (57/44) 421 (89/63)		210 (92/50) 1008 (37/35)	268 (90/36)
N < 2	> 100 vpd	1560 (333/75) 219 (115/38)	383 (100/28)	P8-4 (111/14)		210 (246/34)	1110 (223/25) 1266 (108/36)
2<N<5	< 100 vpd	1439 (5/20)	1561 (61/20)	1886 (69/13)	178 (26/25)	1342 (53/15)	1717 (31/27) 942 (18/7)
2<N<5	> 100 vpd	024 (427/17) 685 (127/15)	1161 (200/10) 685 (132/14)	420 (197/20)	522 (173/14) P175-1 (148/45)	771 (236/29)	327 (395/22)
N > 5	< 100 vpd	017 (58/25) 433 (53/8)	437 (92/21) 14 (22/27)	1479 (20/26)	1141 (46/24) 508 (61/13)	508 (51/10)	167 (56/15) 1216 (11/22)
N > 5	> 100 vpd	502 (112/13)	912 (127/20)	509 (156/16)	611 (100/23)	146 (105/15)	1401 (200/50) 611 (110/30)

NOTE : THE NUMBERS IN EACH CELL ARE THE ROAD NUMBERS WITH THE NUMBER OF VEHICLES/DAY AND PERCENT HEAVY IN PARENTHESES

FIGURE 3.1

EXPERIMENTAL DESIGN MATRIX FOR SAMPLING OF UNPAVED ROADS IN THE TRANSVAAL where the N-value was less than 2. The road numbers and the traffic counts used for the design are shown for each cell. In addition 14 of the cells contain duplicate sections (replicates). Thus any differences between replicate sections (which should behave identically) can be attributed to those random factors which make up experimental error.

As there was some personal selection involved in the test site location, some of the randomness was lost. However, as no existing information was available for local conditions a great deal of time and money would have been required to obtain enough data to permit a completely randomised design.

To simplify section identification and data processing, the sections were numbered from 1 to 49 (with an A and B section for most) for monitoring and analysis purposes (Figure 3.2). During the course of the project the following sections became unsuitable and monitoring was abandoned:

Section 23 A - The section was incorporated into a bridge approach and sealed before monitoring commenced.
Section 5A and B - The road was regravelled with a different material - April, 1985
Section 22A and B - The road was reconstructed and upgraded - May, 1986
Section 34A and B - The road was upgraded and sealed - August, 1986
Section 48A and B - The road was regravelled with a different material - September, 1986

In order to extend the applicability of the results of the study as far as possible it was decided to carry out a satellite experiment in South West Africa (Namibia). The experiment was designed on the same basis as that in the Transvaal but as the N-value was about 10 for all the sections no climatic factor was used. The design matrix for this experiment is shown in Figure 3.3. The monitoring was carried out by the Department of Transport in Windhoek (SWAAR) under the supervision of the National Institute for Transport and Road Research (NITRR). All the results were sent to NITRR for analysis. The Weinert material

SURFACING MATERIAL		1	2	3	4	5	6
TRAFFIC / CLIMATE							
N < 2	< 100 vpd	26	18 29	28 22		21 19	27
	> 100 vpd	24 25	30	23		20	16 31
2 < N < 5	< 100 vpd	9	7	6	11	1	3 13
	> 100 vpd	10 14	8 15	5	12 32	2	4
N > 5	< 100 vpd	43 49	34 47	45	40 35	37	46 44
	> 100 vpd	42	41	36	39	33	48 38

NO SUITABLE SECTION FOR NUMBER 17 COULD BE LOCATED

FIGURE 3.2

IDENTIFICATION NUMBERS OF TEST SECTIONS IN THE TRANSVAAL

MATERIAL / TRAFFIC	1 ACID CRYSTALLINE	5 ARGILLACEOUS	6 PEDOCRETE	7 CALCAREOUS MIXTURE
$\leqslant$ 50 vpd	**901** * MR052 50/15 **902** DI958 10/15	**904** MR048 35/15	**905** * MR048 35/15 **906** T15/1 40/17	**907** DI535 20/15 **909** DI805 35/15
$>$ 50 vpd	**903** MR049 100/15	**908** MR057 67/9 **910** ** MR039 120/15 **913** MR091 120/15	**912** MR091 120/15 **915** MR055 90/15	**911** MR039 120/15 **914** MR056 120/15

* LATER SECTION 918 AND 919 ADDED

**RECONSTRUCTED 916 AND 917

FIGURE 3.3

DESIGN MATRIX OF SOUTH WEST AFRICAN EXPERIMENT SHOWING IDENTIFICATION NUMBERS, ROAD NUMBERS AND TRAFFIC COUNTS (vpd / % heavy) classification was used for acid crystalline and pedogenic materials, and a seventh group, calcareous mixtures was included. The behaviour of this material is probably governed by the calcareous content, although a large percentage of shale and hornfels is common. It is a commonly used material in South West Africa and was thus treated as a separate group.

A total of 91 sections in the Transvaal and 19 sections in South West Africa was investigated. A summary of the section characteristics is given in Table 3.1.

The scope and range of the parameter values of the Transvaal and SWA experimental sections is compared with those for the Kenya and Brazil studies in Table 3.2. Maps showing the locations of the Transvaal and South West African sections are provided (Figures 3.4 and 3.5 respectively).

3.3 WORK PROGRAMME

Between January, 1984 and August, 1984 each test section was visited. The 300 metre sections were measured out and carefully marked with numbered steel plates which were hung on adjacent fences at the start and end of the sections. The horizontal curvature of the curved sections was measured using a prismatic compass.

In-situ testing of the wearing course and subgrade materials was carried out at each end of the section just outside the section so as not to cause any potholes or roughness and a bulk sample (about 50 kilogrammes) was taken. At the same time a 50 metre section was laid out for gravel loss measurements (Chapter 3.4.6) and a gravel loss survey was carried out.

Monitoring of the test sections commenced during November, 1984 in the Pretoria area and in January, 1985 monitoring of all the sections on a three weekly basis was initiated. During each monitoring trip riding quality measurements were taken, rut depths and corrugations were measured and a full condition description was made. Every six months a gravel loss levelling survey was carried out on each section.

TABLE 3.1: CHARACTERISTICS OF EXPERIMENTAL SECTIONS

Variable	Transvaal				South West Africa			
	$\bar{x}$	σ	m	M	$\bar{x}$	σ	m	M
No. of sections	91			91	19			19
Grade (%)	1,5	2,4	0	8	0,5	0,6	0	2
Radius of curvature (m)	-	-	118	∞	All straight sections			
N-value	3,8	2,1	1,3	8	10	0,2	9	10
Thornthwaite Index (Im)	-6	17	-24	60	-35	0	-35	-35
Rainfall (mm/yr)	648	172	375	1100	339	25	300	375
Average daily traffic	94	87	12	427	74	45	10	150
No. of cars/day	72	76	7	354	63	38	9	128
No. of trucks/day	28	17	1	73	11	7	2	23
Road width (m)	9,1	1,9	5	14	7,5	1,6	6	10
Trafficked width (m)	6,0	1,4	3	10	-	-	-	-

$\bar{x}$ = mean

σ = standard deviation

m = minimum

M = maximum

TABLE 3.2: COMPARISON OF SECTION DATA FOR KENYA, BRAZIL, TRANSVAAL AND SOUTH WEST AFRICA STUDIES

	Kenya	Brazil	Transvaal	SWA
Total no. of sections	46	48	91	19
No. of gravel roads	37	37	91	18
No. of earth roads	9	11	0	1
Period of observation (years)	2	2,5	> 2 yrs	3 yrs
Length of sections (m)	1000	320-720	300	300
Road width (m)	-	7 to 11	5-14	6-10
Gradient (%)	0 to 5,5	0 to 8,2	0-8	0-2
Horizontal curvature (degrees/km)	0 - 200	0 to 143	0-500	0
Traffic volume (vpd)	42-403	18-608	12-388	10-150
Truck Volume (vpd)	12-136	5-477	1-113	2-23
Road roughness (QI counts/km)	52-221	20-377	12-330	18-195
Surface material	Lateritic nodular; quartzitic rounded; Volcanic angular; coral angular	Lateritic nodular; quartzitic subangular; clayey silts	Acid crystalline basic crystalline high silica arenaceous argillaceous pedocretes	quartzitic calcrete calcareous mix shale
Annual rainfall (mm/year)	400-2000	1200-2000	375-1100	300-375
Evapotranspiration balance I_m (Thornthwaite Index, 1948)	-30 to 0	35 to 100	-24 to 60	-55 to -45
Climate	semi-arid to dry sub-humid	moist sub-humid to humid	dry semi-arid to moist humid	dry arid
N-value	?	?	1,3-8	9-10

Sources: Brazil : GEIPOT (1981); Paterson (1985)
Kenya : Hodges et al, (1975)

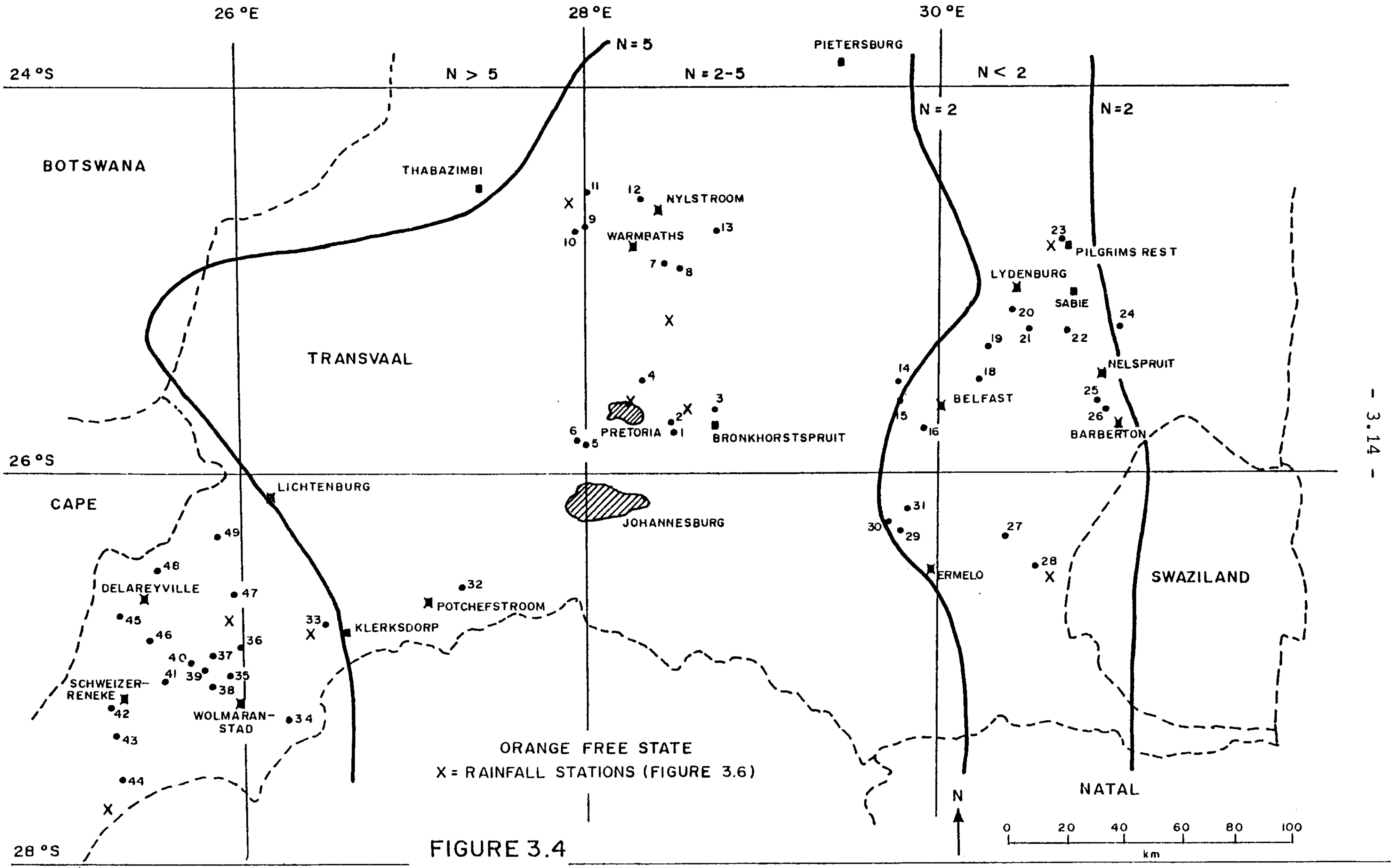

FIGURE 3.4

TRANSVAAL TEST SECTION LOCATIONS

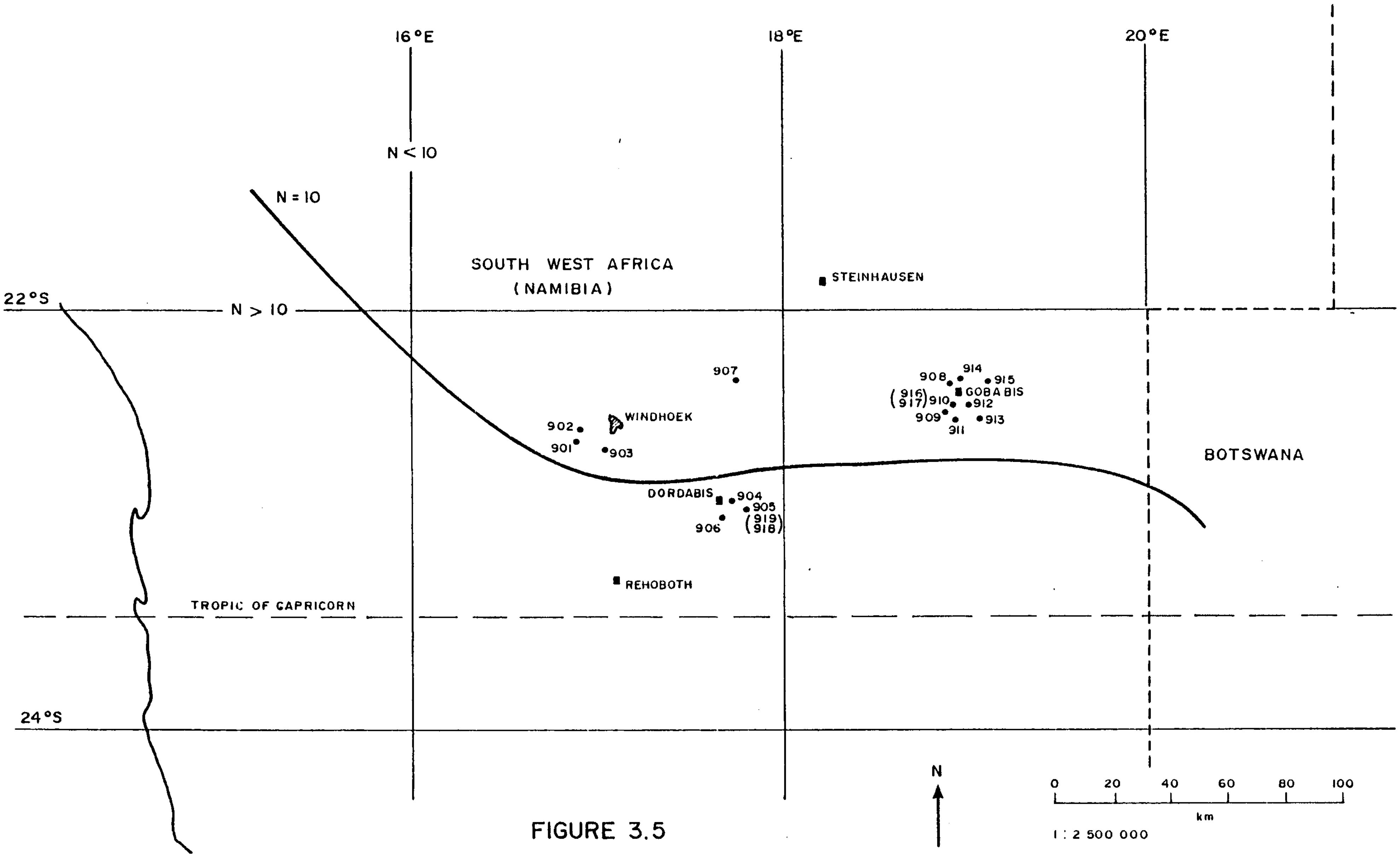

FIGURE 3.5

SOUTH WEST AFRICAN TEST SECTION LOCATIONS

During June, 1985 all of the TPA road inspectors involved in the project were requested to reduce the routine grader maintenance to not more than once every four months, in order to obtain as many measurements per grading cycle as possible. During January and February, 1987, the final monitoring trips were carried out and at the same time a quantitative estimate of the stoniness and skid resistance was obtained. Samples of the wearing course material were collected for additional testing.

When the final report stage of the project was reached (August, 1988) an extended private trip was made through the northern Cape and South West Africa to investigate unpaved roads from a tourists point of view. This trip involved in excess of 2 000 km on unpaved roads with a modern, compact, family saloon car (1985 Ford Escort) towing a four metre long low-profile caravan (the minimum ground-clearance of the combination was 125 mm). The point of view thus obtained was totally different to that obtained during monitoring of the experimental sections in a CSIR station wagon.

3.4 TESTING AND MONITORING TECHNIQUES

3.4.1 In-situ testing

In-situ testing involved mainly density, moisture and strength investigations. The normal sand replacement density test (NITRR, 1979) was considered to be a time-consuming, complicated test for what was considered would be a relatively variable parameter. Instead, a simple sand replacement test (Cernica, 1980) was chosen and duplicate tests were done on the wearing course while a single test was done on the subgrade which was generally finer grained and more homogeneous. Normal oven-dried volumetric moisture content determinations were carried out. Dynamic Cone Penetrometer (DCP) tests and Clegg Soil Impact Tests (CSIT) followed the usual procedures (Kleyn, 1975 and Clegg, 1983 respectively).

Two samples of both the wearing course and subgrade materials, one of about five kg for indicator tests and the other of about 45 kg for compaction and strength tests, were collected at the beginning and end of each section. All the material excavated from the sample holes was collected except for those stones larger than 150 mm, which were discarded. These stones were considered to be unnecessary for testing and often comprised a disproportionate quantity of the sample. This problem with the sampling and testing method as outlined in TMH 1 (NITRR, 1979) has been closely examined and a new sampling and testing procedure proposed (Paige-Green, 1988b).

The proportion of stones with an average diameter larger than 50 mm was estimated by sweeping an area of the road at each end of the sections and drawing a circle of one metre radius. The number and size of all stones larger than 50 mm in this circle was determined.

3.4.2 Laboratory testing

The laboratory testing generally followed the Standard Methods for Testing Materials (TMH 1; NITRR, 1979). The tests included Atterberg Limits, particle size distribution (sieve and hydrometer), compaction characteristics and soaked and unsoaked California Bearing Ratio (CBR). The only modifications to the test methods were for the compaction and CBR tests where the crushed oversize material (larger than 19,0 mm) was not added to the sample. The addition of this material resulted in a very coarse grading which would not compact properly. This problem, as well as the problems caused by substituting the oversized fraction with finer material is well known (Beaven et al, 1987). The laboratory strength should therefore only be taken as a guide to the actual strength. As the field and laboratory gradings, compaction and moisture content are so different the applicability of the test is questionable. In the absence of nothing better the results must be used. As the normal specified compaction for wearing course materials is about 95 % Modified AASHO density, Proctor compaction was used in the laboratory in order to save time. Other tests carried out were the aggregate pliers value (APV) and aggregate fingers value (AFV) (Netterberg, 1967), the cone liquid limit (Sampson, 1986), field moisture equivalent (AASHTO, 1982), and a modified British Standard linear shrinkage. This test was modified to be used as a possible quick, simple field indicator test and consisted of material passing the 40 mesh sieve being mixed at the field moisture equivalent and then oven dried in a British Standard shrinkage trough.

Further problems were encountered with the interpretation of the particle size analyses. The TMH 1 method (Test A1(a); NITRR, 1979) was followed where the calculation of the cumulative percentage passing is based on the total mass of the test sample. However, when oversized material is present this can lead to extreme distortion of the values determined (Paige-Green, 1988b). Depending on the proportion of material larger than 50 mm, comparison of different gradings becomes meaningless. All the results were therefore recalculated as a percentage of the material passing the 37,5 mm sieve (Standards Association of Central Africa, 1971).

A full X-ray diffraction analysis was carried out using the standard NITRR method (Paige-Green, 1978). The major primary and secondary constituents of each wearing course gravel were identified and an approximate estimate of the quantities of each was made. The problems of quantitative X-ray determination of soil clays are well known (Carroll, 1970) and are semi-quantitative at best (Karathanasis et al, 1982). This is mainly because of the variable crystallinity and chemical composition of soil clays. Direct comparison of peak heights or intensities is not possible due to the variation of the mass absorption coefficients of the clay minerals and the use of internal standards (Norrish and Taylor, 1962) requires considerable input by highly-trained personnel. No petrographic examinations were carried out because of the extreme variability of the materials and the problems of preparing thin sections.

For this work semi-quantitative estimates were obtained by comparing the diffraction traces for each section with a number of standard traces obtained from composite samples made up in the laboratory of standard minerals.

A summary of the stratigraphic units and lithology of the materials included in the experiment is provided (Appendix 3). All of these materials can be classified as highly to completely weathered rock (AEG, 1976) and many are mixtures of residual material, transported soil and/or pedogenic materials.

3.4.3 Roughness measurement

The roughness of the sections was determined using a number of vehicles fitted with Linear Displacement Integrators (LDI). The LDI was developed at NITRR during the 1970s (Van der Merwe and Grant, 1982) and is a simple, robust, response-type roughness measuring device which integrates the movement between the back axle of the vehicle and the passenger compartment above it. Each LDI is calibrated over a number of standard control sections of known roughness (Visser and Curtayne, 1982) and a calibration equation is obtained relating the LDI output to the Quartercar Index (QI). This Quartercar Index (Sayers, 1985) is determined from routine rod and level surveys of the control sections (Visser, 1982). Although the roughest calibration section has a QI of only 79 counts/km the linearity of the QI-LDI relationship has been confirmed for values up to 210 counts/km (Visser, 1983).

Before and after each monitoring trip the calibration of the LDI vehicle was tested on three of the control sections and the results checked against control charts for the mean and range of the results from five runs. Any change in the suspension, shock-absorbers or wheel balance was timeously identified and corrective action taken. Periodically, as the suspension of the vehicle aged it became necessary to recalibrate the vehicle and a new calibration equation was obtained. As the QI is a speed dependent parameter it should be determined for different speeds by calibrating the vehicle at those speeds. This is, however, impracticable each time the vehicle is calibrated and as only a few measurements were taken at 50 km/h (LDI50) it was decided to simply use a correlation between those taken at 80 km/h (LDI80) and the 50 km/h readings which were taken. The use of two correlation equations to determine the roughness in terms of QI amplifies any errors. Visser (1981a) however noted that this procedure is preferable to extrapolating a single correlation curve based on a number of relatively smooth sections to the rougher gravel sections. The readings obtained at 50 km/h (MM50) from the Mays Meters (the instruments used in Brazil which were similar to LDI's) were corrected to 80 km/h (MM80) using the following equation (Visser, 1981a):

$$MM80 = 275 + 1{,}04\ MM50 \tag{3.2}$$

The r-squared value for this equation is 0,81 with 95 per cent confidence limits of $\pm$ 2 934 mm/km. Paterson (1985) subsequently modified this equation by using a non-linear model:

$$MM80 = 2{,}22\ MM50^{0{,}831} \tag{3.3}$$

($r^2 = 0{,}75$; $n = 7654$)

The linear regression equation obtained for this work was:

$$LDI80 = 0{,}311 + 0{,}807\ LDI50 \tag{3.4}$$

($r^2 = 0{,}76$; $n = 1556$; $RMSE = 0{,}70$)

A non-linear transformation as used by Paterson (1985) resulted in the following equation:

$$LDI80 = 0{,}963\ LDI50^{0{,}934} \tag{3.5}$$

($r^2 = 0{,}83$; $n = 1555$; $logRMSE = 0{,}196$)

The logarithmic transformation results in an improved r-squared value, with an exponent almost equal to unity, indicating a small deviation from a linear relationship. Equation 3.5 is, however, better correlated than the linear model (3.4) and those of Visser (1981a) and Paterson (1985) (equations 3.2 and 3.3 respectively). Model 3.5 was used to convert all measurements at 50 km/h to values at 80 km/h where these could not be obtained by direct measurement.

It is interesting to note that the predicted roughness at 80 km/h from Model 3.2 is greater than at 50 km/h whereas the roughness at 80 km/h is always less than at 50 km/h for Model 3.4. The reason for this is not clear as Model 3.3 (the logarithmic transformation of the data used in Model 3.2) also shows the roughness at 80 km/h to be less than at 50 km/h (for values of MM50 gretaer than 100). Higher roughnesses at lower speeds are general for unpaved roads, but for smooth asphalt roads the reverse may occur (Paterson and Watanatada, 1985).

Each test section was monitored every three weeks as far as possible. Only occasional mechanical or electronic problems interfered with this routine during the project. Initially each section was tested at 80, 50 and 32 km/h with the actual result being the average of three runs in both directions. Some sections (e.g. very sharp corners) could not be tested at 80 km/h and were tested at 50 and 32 km/h only. The tests at 32 km/h were carried out in case any of the sections became so rough that testing at 80 or 50 km/h became too dangerous or was likely to damage the vehicle or LDI system. After the first couple of monitoring trips the measurements at 32 km/h were discontinued as they were time consuming and were likely to be of little use. In June, 1986 testing at 50 km/h was also omitted if the section could be tested at 80 km/h. No sections became so rough that they could not be tested at 80 km/h, even after no maintenance for up to 5 months.

3.4.4 Rut depth and corrugation measurement

During each monitoring trip the rut depth was measured. Initially this was done with a two metre straight edge and a wedge. This method was soon found to be unsuitable for gravel roads due to the stones in the wearing course which stopped the straight edge from lying flush with the surface, and the variability of the wedge readings. An AASHO type rut depth gauge was subsequently used (Visser, 1981a). Rut depths were measured in obvious wheel tracks (i.e. where no loose material occurred) every 60 m along the section. Initial analyses showed no trends and minimal rutting and as these measurements took a substantial amount of time, testing was discontinued after September, 1985 unless obvious ruts were present.

Corrugation depchs were measured with the rut depth gauge while the spacing and width were measured with a ruler or tape. An estimate of the percentage of the length of the road in each direction affected by corrugations was made.

3.4.5 Condition description

A full condition description of each section was carried out during each visit unless the section was wet (only drainage, slipperiness and trafficability were recorded) or had recently been graded. The description followed TRH 6 principles (NITRR, 1980) using a five point classification for both severity and extent (Paige-Green, 1983; 1988a). For severity a rating of one implied excellent condition while a rating of five indicated a totally unacceptable condition.

The following conditions were rated:

Dust -
Stoniness -
Potholes -
Cracks -
Loose material -
Surface drainage -
Slipperiness when dry -
Slipperiness when wet -
Wet weather trafficability -

In addition the largest stone found was measured, and the maximum depth and diameter of the pothole which had the biggest influence on the roughness of the section was recorded.

3.4.6 Gravel loss surveys

The gravel loss surveys determined the average change in height of the surface of the levelled section with respect to a set of bench marks. These bench marks were installed during the initial sampling trips and consisted of 500 mm long mild steel rods hammered into the subgrade and set in concrete about 150 mm in depth and diameter. By using shallow bench marks it could be assumed that any subgrade movement, either due to expansive clay or soil compaction would have the same effect on the bench marks as the wearing surface and would not affect the gravel loss measurements. Benchmarks out of the section used over active clays in Ethiopia (Beaven et al, 1987) were subject to significant movement, which resulted in the measurements being almost unusable. Movements of this type which affected the readings would be indicated by significant differential movement of the bench marks, none of which occurred during the project. The installation of three bench marks in each section allowed the identification of any bench mark which moved relative to the other two, whether through subgrade movement or disturbance by a grader or any other means.

The gravel loss sections were 50 metres long with two bench marks at the edge of the trafficked portion of the road at the beginning of the section and one 50 metres away, at the left hand edge of the trafficked portion of the road. During the surveys a 50 metre tape was stretched along the section and the level survey was done on a 5 metre long by 1 metre across grid.

3.4.7 Skid resistance tests

Initially estimates of the slipperiness in both the wet and dry condition were obtained during routine monitoring from the general feeling of safety at 80 km/h. Towards the end of the project it was decided to actually quantify this as far as possible. A "MOTOMETER" brake efficiency meter was obtained and locked-wheel skid tests were carried out from 50 km/h. As this device measures deceleration the actual speed does not affect the results assuming the road/tyre friction remains constant. This was confirmed by doing a few stops from different speeds. As no literature could be found for this type of test on unpaved roads a number of tests were done on various surfaces for comparative purposes. These results are discussed in Chapter 4.8.

Deceleration measurements of the test sections were done in the wheel tracks in both directions and also in areas of loose material typical of the various sections. Unless it was coincidentally raining at the time of testing it was logistically not possible to test the sections for wet weather slipperiness. However attempts were made to test as many of the sections as possible in the rain.

3.5 SUMMARY OF RESULTS

Table 3.3 contains a summary of the laboratory and in-situ test results for the Transvaal and South West African experimental sections. A full list of the test and monitoring results has been produced separately (Paige-Green, 1989).

It is evident that the site selection procedure in the Transvaal resulted in a reasonably good range of material properties being incorporated into the experiment. The plasticity index varied from non-plastic to 33 but the average of 7,5 is indicative of the bias towards low-plasticity (non-slippery) materials during material selection for unpaved roads. The site selection procedure has thus resulted in relatively few sections with very high plasticity but these materials are seldom intentionally used for unpaved roads and then only for short sections of the roads. The percent passing the 0,425 mm (40 mesh) sieve varies from 15 to 93 and includes a good range of fine to coarse materials (also shown by the grading modulus range). The range of soaked CBR (Proctor compaction) varies between zero and 99 with a mean of 26.

Comparison of the in-situ material properties of the test sections with the TPA 1973 specifications (those effective at the time most of the roads were built or regravelled) and the 1980 and 1983 specifications (there is no difference between the two) was carried out. The compliance of the results with the specifications is shown in Table 3.4.

None of the sections complied with the requirement for in place maximum size, every section having stones larger than 50 mm, with a minimum average maximum size of 132 mm and a maximum of 669 mm. Approximately half of the materials comply with the minimum GM requirement of 1,5 and the 1973 grading envelope requirements. Most of the materials not complying are too fine although only 9 per cent are too fine for the 1980/83 grading modulus (GM) relaxation of 1,2 with special approval. It should be noted that the TMH1 method of particle size analysis (as used by the TPA) results in a higher grading modulus than a value calculated on the fraction passing the 37,5 mm sieve. The presence of a few stones of about 50 mm diameter

TABLE 3.3: SUMMARY OF TEST RESULTS FOR TRANSVAAL AND SOUTH WEST AFRICA

Property		Transvaal				SWA			
	n	$\bar{x}$	σ	m	M	$\bar{x}$	σ	m	M
Liquid limit (%)	201	27	7,5	14	50	27	7,3	11	40
Plasticity Index (%)	201	7,5	5,2	0	33	9,8	4,9	1	22
Bar linear shrinkage (%)	201	3,9	2,7	0	16	5,2	2,2	1	10
Modified linear shrinkage (%)	201	3,1	2,4	0	11	3,6	1,8	0,5	7
Laboratory maximum size (mm)	201	30,4	14,4	4,75	63	-	-	-	-
Passing 26 mm sieve (%)	201	96,3	6,5	65	100	88	13	45	100
4,75 mm sieve (%)	201	77,2	16,0	30	100	61	18	28	91
0,425 mm sieve (%)	201	43,7	15,1	15	93	38	13	11	62
0,075 mm sieve (%)	201	22,4	11,8	9	76	17	7	6	38
Grading Modulus	201	1,72	0,36	0,32	2,52	1,95	0,4	1,3	2,6
AFV (%)	168	65	21	0	100	90	7	70	98
APV (%)	168	22	18	0	82	73	15	40	96
Plasticity Product	201	320	260	0	1881	380	255	36	1140
Maximum dry density (kg/m^3)	201	1962	124	1613	2309	2097	94	1936	2233
Optimum moisture content (%)	201	11,7	2,9	6,6	23,8	8,3	1,7	6	11
Soaked CBR (%)	201	25,5	19,8	0	99	23,7	11,2	6	46
CBR at OMC (%)	182	33,7	26,1	3	205	-	-	-	-
Layer thickness (cm)	201	13	4,7	3	28	12	3	7	18
In situ density (kg/m^3)	201	1924	144	1507	2195	1976	141	1685	2187
In situ moisture (%)	201	5,3	2,8	0,9	14,2	3,4	2,1	1,4	8,7

Plasticity Product = % passing 0,425 mm x PI.

TABLE 3.4: SPECIFICATION COMPLIANCE OF EXPERIMENTAL SECTIONS

	Percent of sections not complying with specifications			
Property	1973	Comments	1980/83	Comments
Lab. max. size	0		0	
In place size	100	All have large stones	100	All have large stones
% passing:				
4,75 mm	41	mostly too high	No spec.	
2,00 mm	51	mostly too high	No spec.	
0,425 mm	63	mostly too high	No spec.	
0,075 mm	21	mostly too high	No spec.	
Grading modulus (GM)	45	mostly too low	45 9	Mostly too low < 1,2
Dust ratio	14		No spec.	
Liquid limit	2	Too high	No spec.	
Plasticity index w.r.t. GM	48	42 % too low, 6 % too high	48	42 % too low, 6 % too high
Relative compaction	No spec.		62	Too low (slightly)
CBR at 93 % Mod. AASHO Comp	31	Too low	88	Too low
All	100	99 % neglecting max. size	100	100 % neglecting max. size

can thus make the difference between accepting and rejecting a material.

Most of the materials comply with the liquid limit requirements although 48 per cent of the sections have an average plasticity index with respect to the GM outside the specified limits, with 42 per cent too low and 6 per cent too high. This bias toward low plasticity materials is apparently due to a deliberate tendency to give preference to materials which it is believed will maximise skid resistance in the wet.

The compaction of the wearing course is generally slightly below specification with 62 per cent of the sections having values less than 93 per cent of modified AASHO compaction (approximately 100 per cent standard Proctor compaction). This is due to the reliance on traffic compaction with only a nominal number of passes of a grid roller during construction. Nearly all of the materials fail the 1980/83 minimum CBR requirement of 45 and 31 per cent the 1973 requirement of 15.

On an overall basis not one material satisfies either of the specifications. If the maximum size is neglected, one material satisfies the 1973 specification while none satisfies the 1980/83 specification.

3.6 CLIMATIC EFFECTS

During the course of the project South Africa was generally suffering from a major drought. For the purposes of this project it was considered unlikely to affect the results to any great extent for the following reasons:

- The performance of unpaved roads is affected more by the short term weather than the long term climate. Periodic thunderstorms and periods of intermittent drizzle are important in unpaved road performance and although the annual rainfall may be lower than normal, storms and drizzle still occur, though not necessarily to the same extent.
- An analysis of the rainfall during 1985 and 1986 for 7 stations distributed in each of the three N-value regions shows that the annual rainfall did not differ from the long-term mean to any major extent (Figure 3.6). In the wet areas ($N < 2$) the average rainfall just exceeded the long-term mean (102 and 103 %) while in the intermediate areas ($N = 2$-5) the average was just less in 1985 (96 %) and more in 1986 (102 %). In the drier areas of the western Transvaal ($N > 5$) the rainfall was only 73 per cent of the annual

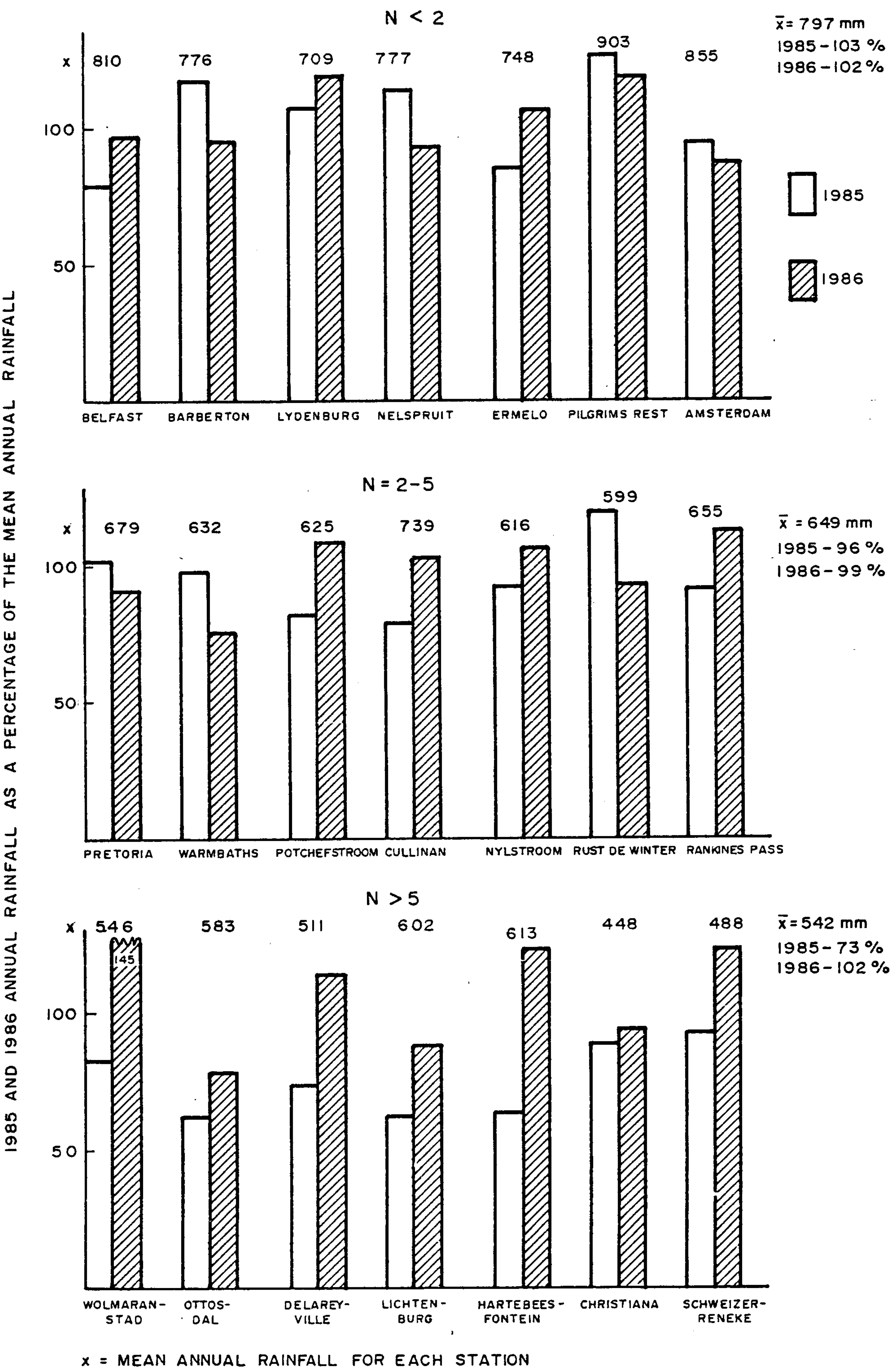

FIGURE 3.6

ANNUAL RAINFALL OF SELECTED STATIONS IN THE TRANSVAAL DURING THE MONITORING PERIOD (SEE FIGURE 3.4) mean in 1985 but increased to 102 per cent in 1986. The rainfall for all the stations over both years was 97 per cent of the long term average.

- It is therefore unlikely that the "drought" had a significant effect on the results obtained during the monitoring period.

3.7 CONCLUSIONS

As in the Kenya and Brazil studies a factorial design with material, traffic, climate and road geometrics was selected for this experiment. Although the experimental site selection was not completely random a suitable range of material properties was included in the experiment. The obvious bias towards the use of low plasticity materials in existing roads was evident in the distribution of the plasticity indices. The records from which the traffic counts were obtained were not totally accurate and the counts done during the project resulted in a change in the design matrix. However the cells of the matrix were still filled and the statistical validity of the project was not interfered with.

In all, 110 sections of road in the Transvaal and South West Africa were monitored for more than 2 years. The main consideration in the experiment was the influence of the geotechnical properties on the performance of the test sections. The effect of different maintenance strategies on the performance of the sections was not investigated to any significant extent, unlike all the previous studies which concentrated on the maintenance and not the materials.

Testing and monitoring followed well documented techniques and a carefully controlled and closely supervised, regular monitoring programme resulted in a large data base containing high quality results. An important finding of the test programme was that none of the materials in the test sections complied with the present (1980/1983) or previous TPA specifications (1973) on the basis of the specified maximum size. Even neglecting the maximum size criterion only one section complied with the early specification and none with the present one. However, many of the sections provided adequate performance, even under prolonged periods with no maintenance. It is apparent that research into material specifications and the development of performance-related specifications is thus long overdue.

The "drought" prevailing over much of southern Africa during the monitoring period was found not to have affected the study area significantly in terms of the total rainfall as a percentage of the long term average.

CHAPTER 4

GENERAL PERFORMANCE

4.1 INTRODUCTION

In order to develop performance-related material specifications it was necessary to investigate the performance of the various materials on a regular basis. During each monitoring trip a full road surface condition description was made. Each parameter shown in Figure 4.1 was classified by severity and extent where possible (Paige-Green, 1983; 1988a). In addition the longest dimension of the largest exposed stone and the depth and diameter of the most severe pothole were measured. Subsequent comparison with an equivalent rating carried out by engineers from the TPA (Paige-Green, 1987a) indicated a number of differences between the two methods. These stem mainly from the purpose of classification. For research purposes it was necessary to identify the propensity of the materials (and sections) to result in the specific condition (e.g. potholes, dust, cracks, loose material, etc.). From the TPA point of view it was necessary to identify the maintenance requirements. Thus a few potholes of severity 4 (i.e. bad) over 10 per cent of the road area would be rated as 4/5 by the TPA (i.e. the whole section needs maintenance) but as 4/1 for this project (i.e. potholes are not very prevalent). Conversely a section with no potholes would be rated 1/5 for this project (i.e. no potholes over the whole section) but 1/1 by the TPA (i.e. no potholes requiring maintenance over less than 20 per cent of the section).

Prior to analysis of the data, five data files were created to reduce computer utilisation. These were as follows:

PROPS - a summary of all the section characteristics, material properties and test results for both the Transvaal and South West Africa sections. This file contained the section number and for each section the grade (G), curvature (C), Weinert's N-value (N), Thornthwaite's moisture index (Im), the mean annual rainfall (RAIN), material type group (MATGRP), average daily traffic (ADT), per cent heavy vehicles (over 3 tonnes) (PERHEAV) and the number

NITRR CSIR
GRAVEL ROADS PROJECT

ROAD CONDITION DESCRIPTION

C		ROAD					SECTION NO		
8	1								
8	1								
8	1								
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8	1								
8	1								
8	1								
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8	1								
1	2	3	4	5	6	7	8	9	10

DUST		STONI		POTH		CRACK		LOOSE	MAT	SURF	DRAINE	SLIP DRY		SLIP WET		W.W. TRAF		MAX SIZE (mm)			POTH DEPTH (mm)			POTH DIAM (cm)			DATE					
S	E	S	E	S	E	S	E	S	E	S	E	S	E	S	E	S	E										D	D	M	M	Y	Y
11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31	32	33	34	35	36	37	38	39	40	41	42	43

FIGURE 4.1

ROAD CONDITION EVALUATION SHEET

of cars (NC) and trucks (NT). The following test results were included: in-situ density (DENS), moisture content at the time of sampling (MOIST), liquid and plastic limits (LL and PL), bar linear shrinkage (LS), modified bar linear shrinkage (MODLS), plasticity index (PI), percentage passing the 26,5 mm (P26), 19,0 mm (P19), 13,2 mm (P13), 6,7 mm (P6), 4,75 mm (P475), 2,0 mm (P2), 0,425 mm (P425), and 0,075 mm (P75) sieves, the clay fraction (percentage smaller than 0,002 mm) (P2M), the laboratory maximum size (LABMAX), the grading modulus (GM), aggregate fingers and pliers values (AFV and APV), field moisture equivalent (FME), optimum moisture content (OMC) and maximum dry density (MDD) at Proctor compaction effort), soaked and unsoaked CBR (Proctor compaction) (SCBR and OMCCBR), estimated percentage of kaolin (KAOL), smectite (SMEC), chlorite (CHLOR), vermiculite (VERM), illite (ILL) and other clays (OCL) (e.g. sepiolite, palygorskite), mixed layer clays (MLC), quartz (QTZ), feldspar (FEL), goethite (GOET), calcite (CALC) and other minerals (OTM), a quantitative estimate of the in-situ stoniness (STONES), the soil classification (a modification of the AASHO system as used by the TPA) (CLASS) and the relative compaction (COMP). This resulted in 110 observations each with 64 variables.

CONDIT - this file consisted of all the condition description data as follows: section number, the severity and extent of the dust, stoniness, potholes, cracks, loose material, surface drainage, slipperiness when dry and wet, the wet weather trafficability and the measured maximum size, pothole depth and diameter, the depth, wavelength, width and extent of corrugations and the date of description. In all, 5 515 observations each with 33 variables were included in this file.

RUTS - the average rut depth in both directions for each section and the date of measurement were contained in this file which consisted of 1 370 observations and 5 variables.

ROUGH - this file contained all the roughness measurements after correction for the instrument calibration, correction for speeds other than 80 km/h and averaging the three readings in each direction. The file contained 7 083 observations with 5 variables as follows: section number, direction, date of measurement, previous grader blading date and roughness in terms of quarter-car index.

GRLOSS - this file contained the average height of each section after each levelling survey. Altogether, 750 observations with 4 variables (section number, previous regravelling date, date surveyed and average change in height) were included.

The main objective of the analysis of the performance characteristics was to identify the important material properties which affect the behaviour of the different materials, in terms of their performance. Many factors which could not be accurately or easily quantified (such as rainfall intensity or duration, seasonal traffic during harvesting, vehicle speeds, driver characteristics, etc) affect the performance of the roads. Although prediction models were obtained for most of the variables the intention of these was to identify the important material properties and not to be able to predict the severity of any possible problems. Most of the prediction models account for about 50 percent of the total variation and have standard errors of about 0,5. The rest of the variation is apparently due to the unquantifiable factors described previously, considering that as many possible material properties as possible were quantified. The high standard error of 0,5 is probably due to the ratings being discrete integer values, with any intermediate values being of necessity rounded to the nearest integer values.

4.2 DUST

The effects and consequences of dust on unpaved roads are difficult to quantify. However the presence of dust is important for the following reasons:

- Dust affects the visibility. If the dust is too thick the possibility of accidents is increased (Sultan, 1976).
- One of the major factors leading to passenger discomfort is excessive dust. This is especially true in areas with

consistently high ambient temperatures (i.e. many parts of southern Africa) where it is uncomfortable to keep the vehicle windows continuously closed. The dust may also pose a health hazard via dust-borne diseases.

- The presence of excessive dust plays a major role in increasing the wear on moving parts of vehicles such as greased bearings and universal joints (Snyman, 1987).
- In developing areas many of the roads are unpaved. The presence of excessive dust in these areas (especially the residential areas) is both politically undesirable and socially unacceptable.
- The effect of dust on roadside vegetation and crops has yet to be quantified. Initial indications in New Zealand are that the horticultural development adjacent to unsealed roads is significantly affected by road dust (van Barneveld, 1984). It is thought possible that the revenue lost due to the decreased agricultural yield may be significant in comparison with the cost of providing a seal coat (van Barneveld, 1984). The USDA Forest Service (1973) has recorded that "for every vehicle travelling one mile (1,6 km) of unpaved roadway once a day, every day of the year, one ton of dust is deposited along a 1 000 foot (305 m) corridor centred on the road".
- Dust from unpaved roads can cause serious air pollution and pollute adjacent waterways.
- It has been shown in Kenya (Jones, 1984b) that between 25 and 33 tonnes of material per kilometre can be lost annually in the form of fine airborne material (dust) ie a layer 1,8 to 2,5 mm thick.

Despite the obvious influences of dust identified above, most of the published research experience with dust problems has been restricted to the use and efficacy of dust palliatives (Fossberg, 1966; Hoover et al, 1973; Jones, 1984b). No published work on the effect of the geological or geotechnical properties on dust emission from unpaved roads was found.

4.2.1 Method

A quantitative determination of the dust emanating from various material types was desirable. Although a number of techniques for determining dust emission are available (ASTM, 1987; van Barneveld, 1984), the following factors were considered:

- Most techniques require the collection of dust in some way. This is a very time consuming process, highly susceptible to vandalism, thus necessitating constant surveillance.
- Climatic factors, especially the rainfall and wind affect the evolution and collection of dust.
- The speeds and design of passing vehicles affect the generation of dust to a large extent.
- The elapsed time since the previous grader maintenance is important. Roads are usually at their dustiest immediately after grading.
- The dust is at its worst during the dry season, winter over most of southern Africa.

For these reasons it was thought that a regular visual classification of the dust would be more appropriate. The average dust rating over the duration of the project would be more indicative of the potential dustiness of the different materials relative to one another than a one-off numerical value. A one-off numerical value would mean very little as any variation in wind velocities and direction or the traffic type and volume would result in non-representative results (ASTM, 1987).

Taylor et al (1987) have recently developed a simple dust monitoring device (Road Dust Monitor) which is eminently suitable for the type of dust measurement required for this project. The instrument uses an infrared beam and transducer system and measures the air opacity in a duct behind a rear wheel of the testing vehicle, and should be considered for future work on unpaved roads.

For this project, however, a five-point rating system was developed by which the dust was classified by the driver of the LDI during roughness testing at 80 km/h. The severity of the dust visible in the interior rear-view mirror was rated as follows (Paige-Green, 1988a):

1 - No visible dust behind the vehicle

2 - Dust just visible through the rear window

3 - Dust easily visible, but not enough to cause driver discomfort

4 - Very dusty - major discomfort when passing approaching vehicles but not causing dangerous loss of visibility

5 - Extremely dusty - surroundings obscured to a dangerous level - totally unacceptable from driver/passenger comfort and safety aspects.

If the road surface was obviously moist a dust rating was not carried out.

4.2.2 Results

About 30 ratings were obtained for each section in the Transvaal and 50 in South West Africa. The average dust severity of all 110 sections over the monitoring period was 3,9 (σ = 0,68) with a minimum of 2,0 and a maximum of 4,97. All ratings of 1 were excluded from the analysis as the sections were assumed to be moist at the time of rating.

4.2.3 Analysis

The dust was analysed in terms of the material properties affecting its evolution from the road. The average dust severity rating over the duration of monitoring for each section was correlated with the material properties for the sections using a multiple correlation analysis (SAS, 1985). The significant variables were then analysed using a linear regression analysis (SAS, 1985) to identify the best fit equation. Although every section gave a dust rating of 5 at least once during the project (usually just after grading) the average for each section over the duration of the project (excluding the ratings when wet) is probably representative of the overall dustiness of that section.

The significant variables affecting the dustiness identified for the analysis were the material group, plastic limit (PL), fraction smaller than 6,7 (P6), 4,75 (P475) and 2,00 mm (P2), dust ratio (DR) (ratio of per cent passing 0,075 mm to per cent passing 0,425 mm sieves), feldspar content (FEL), percentage of heavy vehicles (PERHEAV), laboratory maximum size (LABMAX), soaked CBR (SCBR) and the Gravel Index (GI) (1 - ratio of the per cent passing the 2,00 mm sieve to the per cent passing the 26,5 mm sieve). Unfortunately, the 6,7 mm sieve was not used for the South West African grading analyses and was not used in the overall analysis despite it being the most highly correlated grading parameter. The material group was the most highly correlated parameter with dustiness. However, arbitrary numbers were assigned to each of the groups (e.g acid crystalline = 1; arenaceous = 4) and any correlation between these numbers and the material properties or road performance is purely coincidental as no definite relationship exists between them.

The correlation coefficient for the best-fit model below was 0,43:

$$\begin{aligned} DUST = {} & 6{,}7 - 4{,}03.GI - 0{,}16.PL - 0{,}11.FEL + 0{,}28.GI.PL \\ & + 0{,}0003.SCBR.PERHEAV - 0{,}0002.SCBR.P475 \end{aligned} \tag{4.1}$$

The percentage of feldspar in the material is a highly significant parameter but this cannot easily be quantified without specialised equipment. An analysis excluding the feldspar content but containing the percentage smaller than 6,7 mm and the aggregate fingers and pliers values together with all the other significant variables and their interactions gave an r-squared value of 0,73. However, because of the missing values for the passing 6,7 mm fraction and the AFV and APV parameters only 68 results, all from the Transvaal were used in the analysis. The standard error of this model was still high at 0,42. The model was significant at the 0,01 per cent level and was as follows:

$$\begin{aligned} DUST = {} & 12{,}57 - P6(0{,}06 - 0{,}0002.PF + 0{,}093.DR + 0{,}0017.APV \\ & + 0{,}0004.AFV - 0{,}003.LABMAX) - PL(0{,}295 + 0{,}007.APV - 0{,}003.AFV \\ & - 0{,}006.PERHEAV) - PF(0{,}017 - 0{,}0001.APV - 0{,}0001.LABMAX) \\ & + DR(11{,}45 - 0{,}121.PERHEAV) + 0{,}205.APV - 0{,}233.LABMAX \\ & - 0{,}037.PERHEAV \end{aligned} \tag{4.2}$$

where

P6 = Percentage passing the 6,7 mm sieve

PF = Plastic factor (Plastic limit x percent passing 0,075 mm)

DR = Dust ratio (ratio of percent passing 0,075 and 0,425 mm sieves)

APV = Aggregate pliers value

AFV = Aggregate fingers value

LABMAX = Maximum sieve size

PL = Plastic limit

PERHEAV = Percentage of heavy vehicles

Analyses studying the seasonal effects resulted in poor correlations as did those using the modal values for dust and various percentiles. It was considered that the modal values did not adequately differentiate the materials with respect to each other, being integer values.

4.2.4 Discussion

Model 4.2 is rather cumbersome with a relatively poor prediction capability but identifies those parameters which affect the generation of dust and should be considered in the specifications. Apart from the effect of heavy vehicles which appear to suppress dust (probably due to the multi-collinearity with rainfall) the plasticity, grading and aggregate strength are the important factors influencing the generation of dust.

During analysis of the performance of the sections and the material properties affecting the performance (see Chapter 7.5.6), it was noted that the product of the bar linear shrinkage and the percentage passing the 0,425 mm sieve was the best indicator of dustiness. This did not, however, show up in the dust analysis described here as the relationship was not linear (the best fit curve is probably parabolic) with both low and high values being dusty and intermediate values identifying those materials least prone to dustiness.

The guide to wearing course materials of the NPA (ca 1961) identifies materials with a liquid limit of less than 20 percent and a coarse and medium sand fraction of less than 30 percent as being dusty in dry weather. The results of this study indicate that the dust on 53 percent of the sections was unacceptable and on 81 percent of the sections was undesirable. However only 21 percent of the sections had liquid limits of less than 20 although 62 percent had coarse and medium sand fractions of less than 30 percent. Thirty percent of the sections rated as having unacceptable dust had a coarse fraction in excess of 30 percent. It would thus appear that if these limits are satisfactory in Natal, some other factor (e.g. rainfall or humidity) plays an important part in reducing dustiness.

It is generally postulated that dust generation is related to the silt content of the aggregate (Van Barneveld, 1984) and the threshold shear velocity of the wind generating the dust. This threshold shear velocity depends on the grain size, sorting and cohesion of the source material (Tsoar and Pye, 1987) and is of the order of 200 to 600 mm/sec. When one considers that a vehicle travelling at 100 km/h has a maximum wind shear velocity of 27 780 mm/sec and a vehicle with a ground clearance of 150 mm has a wind shear velocity gradient of 185 mm/sec/mm it is easy to see why dust is so readily generated. Once the dust has been generated it can either settle rapidly or stay in suspension in the air according to the Stokes' settling velocity relationships (Tsoar and Pye, 1987). These relationships are graphically illustrated for quartz spheres in Figure 4.2. Typical grading curves of some dust samples collected from behind a vehicle are included on the Figure.

It is clear that the dust collected behind vehicles consists almost totally of material finer than 0,075 mm with only between 15 and 25 per cent being finer than 0,002 mm i.e. the dust collected is predominantly silt. The material adhering to the back of a vehicle after travelling in very wet conditions (rain and mud) is slightly finer with about 30 per cent less than 0,002 mm in diameter. The high silt content confirms the findings of Taylor et al (1987) who suggested differences in dustiness resulting from soil properties are dependent on the relative proportion of silt-sized particles and not the clay-sized fraction. Van Barneveld (1984) noted that the silt-sized particles predominate in samples collected, irrespective of the

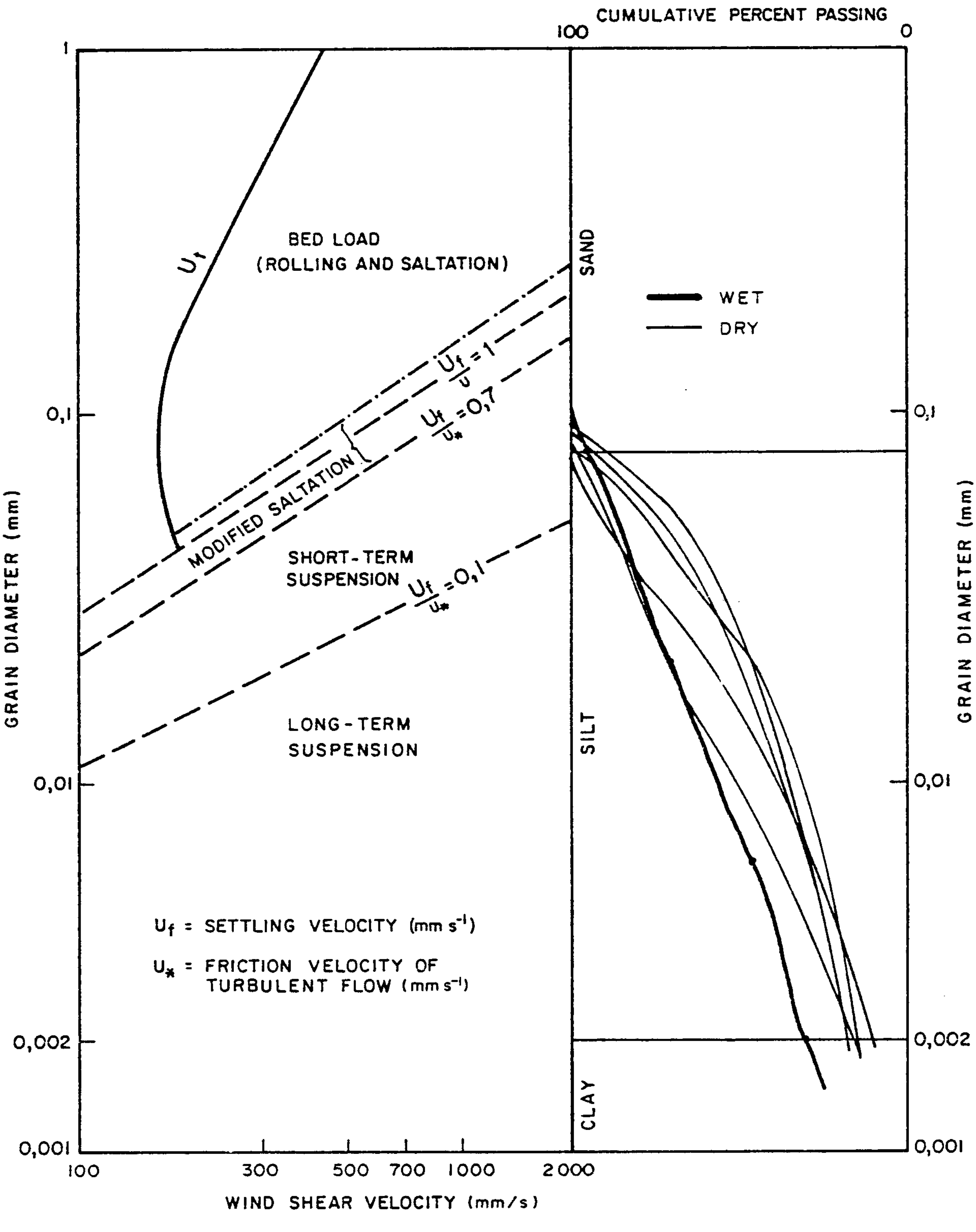


FIGURE 4.2

THEORETICAL MODES OF TRANSPORT OF QUARTZ SPHERES AT DIFFERENT WIND SHEAR VELOCITIES (MODIFIED AFTER TSOAR AND PYE, 1987) distance from the road centre line. The clay-sized fraction never exceeds 10 per cent of the sample deposited. The general paucity of clay-sized particles in samples collected may, however, be indicative of the very low settling velocity of these fine particles and general loss into the atmosphere, even under minimal wind velocities.

X-ray analyses of the dust samples collected in the dry condition showed the main components (and estimated percentages) to be quartz (70 %), feldspar (10 %), kaolin (10 - 15 %) and illite (≤ 10 %). The dust samples were a dark red-brown colour but boiling in concentrated sulphuric acid rapidly revealed white quartz particles and stained the acid green. It can thus be concluded that the quartz is coated with a thin layer of iron oxide (or hydroxide) which produces the red-brown colour.

The strong correlation between the feldspar content of the material and the dustiness is indicative of the importance of the stage of weathering in the generation of dust. The feldspar content is a good indicator of weathering as nearly all of the materials tested contain feldspar to a greater or lesser extent, which is replaced by finer, clay minerals as weathering proceeds. The determination of the feldspar content of soils and gravels with any degree of confidence requires X-ray diffraction techniques which are not readily available to road authorities. The feldspar content should therefore not be a factor in the material specifications despite its significance.

The only other minerals which had significant correlations with the dust severity rating were the smectite content (at the 0,7 per cent level) and the non-differentiated other minerals (dolomite, amphibole, pyroxene, etc) which were significant at the 0,6 per cent level.

Unlike the material group identification, the material classification in terms of the TPA modified TRB/AASHO classification system is approximately linear. As the classification changes from the A1-a to A7 groups (AASHO, 1961), the quality generally deteriorates (the materials tend to become finer and more plastic). Thus, by numbering the groups consecutively from 1 (A1-a) to 17 (A7) any correlation between the material classification and the properties or performance should be significant. No correlation between dust and the TPA classification was, however, evident.

4.3 STONINESS

The influence of stoniness on the roughness of unpaved roads was recorded as far back as 1929 (Strahan, 1929) and has been related to field performance by Paige-Green and Netterberg (1987) and road user costs by Beaven et al (1987). Most specifications provide for the exclusion of large stones and boulders but none of the sections investigated in the Transvaal or South West Africa was totally free of oversized material. Many of these sections were, however, acceptable with respect to roughness. The total exclusion of "large" stones by specification, although desirable, would therefore appear to be unnecessarily harsh. However a limit on the quantity and dimensions of large stones and boulders is definitely necessary.

The presence of large stones affects the performance of the road in a number of ways:

- The main effect is the protrusion of the stones from the road resulting in an uneven running surface and a rough road.
- An excess of large stones results in difficulty during grader blading. The grader either plucks the stones from the surface of the road causing potholes or "bounces" over the stones resulting in minimal improvement in the riding quality.
- Large stones result in poor compaction of the layer during construction as the stresses from the roller are transmitted by the stones to the subgrade without compacting the matrix between the stones. This results in a rapid loss of the uncompacted material under traffic and accelerated protrusion of the stones. In addition the uncompacted material behind the stones compacts under wheel loads and forms potholes behind the stones.
- A number of roads showed distinct corrugations due to the presence of large stones spaced at regular intervals along the road.
- The passage of wheels over the protruding stones results in a certain amount of vehicle bounce causing compaction on regaining

contact with the road and the formation of potholes behind the stones.

The importance of oversize material is clearly shown in Figure 4.3 (a and b) where the roughness measurements for sections 917 (material compacted with a grid roller i.e. normal construction practice (Plate 4.1)) and 916 (material broken down by a Rockbuster (Plate 4.2)) are plotted. The roughness of section 916 varies between about 30 and 80 counts/km with an average of about 50, while section 917 varies between 60 and 140 with an average of about 90 counts/km. Towards the end of the project it was noted that the material on section 916 was not as fine with depth, and as the surface wore down and the coarser material became exposed the roughness increased considerably.

It is interesting to note that most wearing course gravels used in the United States are crushed materials with a maximum size of 26 mm. The blading frequencies are much lower than in South Africa and yet the riding quality is generally very good.

4.3.1 Method

The severity of the stoniness was rated according to the following criteria (Paige-Green, 1983; 1988a):

1 - No stones protrude above the surrounding matrix
2 - Stones protrude above the matrix but cannot be felt or heard when traversing them in a light vehicle
3 - Protruding stones are felt and heard when driving over them
4 - Protruding stones require evasive action (avoidance and/or speed reduction) and result in a noisy, bumpy ride. A criterion used was that the driver would not subject his private vehicle to the road for an extended distance (say 5 km) at 80 km/h.
5 - The stones cause unacceptable roughness. Vehicles drive on the shoulders (i.e. not in the main wheel paths) or use alternative routes. A criterion which was used during the project was that the driver would not travel at 80 km/h in his private vehicle on that road.

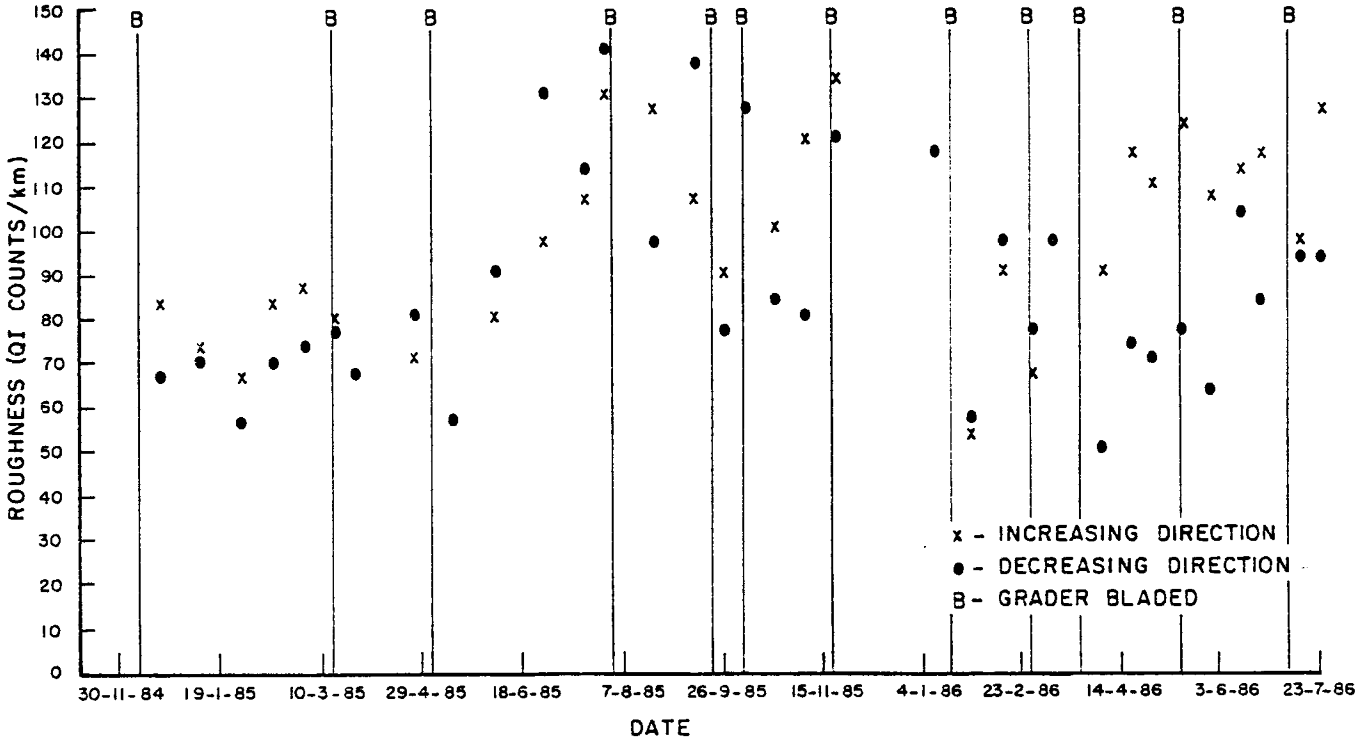


FIGURE 4.3 a

PLOT OF ROUGHNESS AGAINST TIME FOR SECTION 917 (GRID ROLLED)

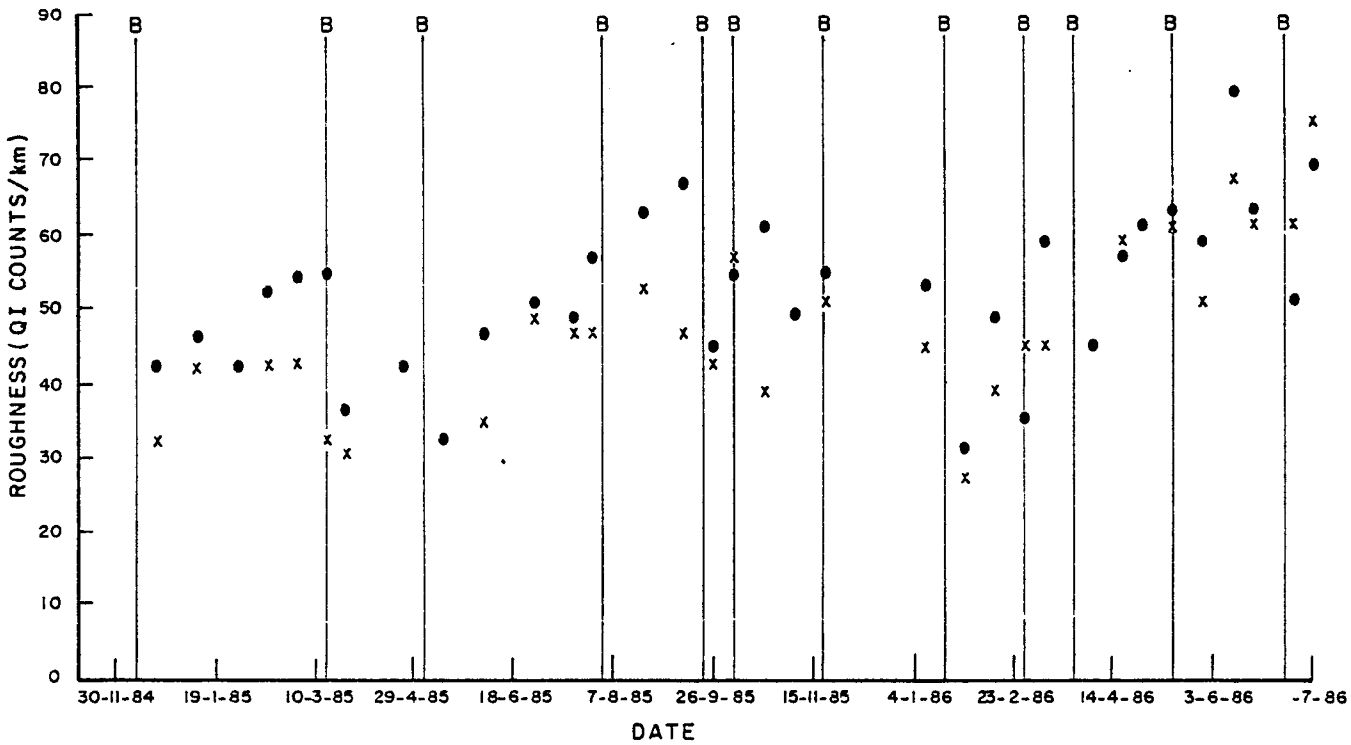


FIGURE 4.3 b

PLOT OF ROUGHNESS AGAINST TIME FOR SECTION 916 (ROCKBUSTER)

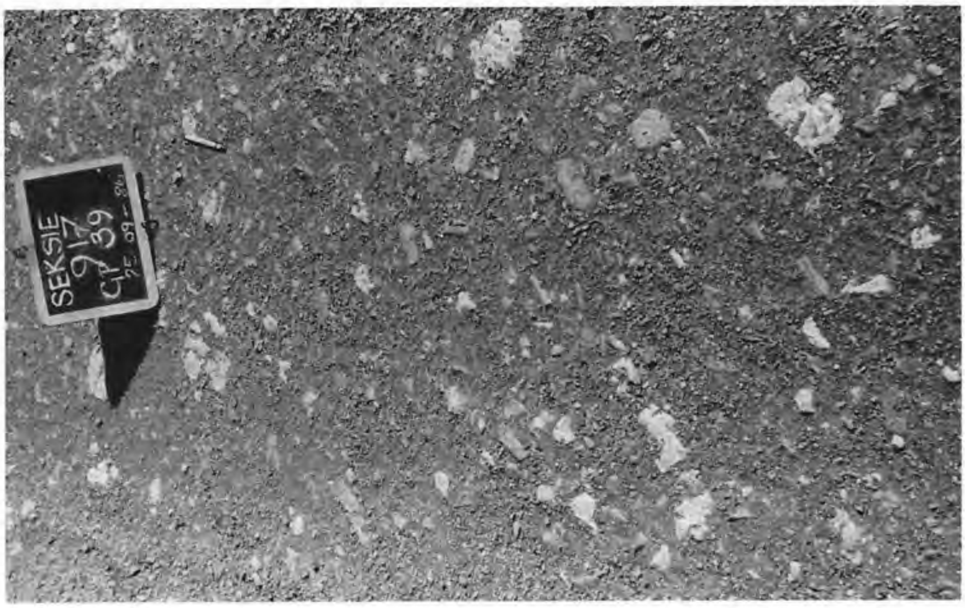


PLATE 4.1: SECTION 917 SHOWING MATERIAL AFTER COMPACTION WITH A GRID-ROLLER

PLATE 4.2: SECTION 916 SHOWING MATERIAL BROKEN DOWN BY A "ROCKBUSTER"

An attempt to correlate these ratings with the protruding heights of the stones was unsuccessful as the diameter and quantity of the stones affected the rating. It was found however that stones protruding more than about 30 mm were generally classified as a severity of 5. It was also noted that once stones up to about 200 mm in diameter protrude more than about 40 mm above the surface of the road they tend to be ripped out during routine grader blading. If the diameter of the stone is more than 200 mm the stone is usually unaffected by blading and needs to be manually removed.

The extent of the stones was estimated as the areal percentage of the road consisting of stones larger than 40 or 50 mm in exposed dimension.

4.3.2 Results

Some 5 500 results were obtained for the stoniness severity and extent and the maximum dimension of the largest stone observed in the section. These were averaged for each section. The average stoniness severity for all the sections was 3,0 while the average extent was 2,6. The average maximum stone size measured in the field for all the sections was 322,5 mm (σ = 119,6) with a minimum of 139 mm and a maximum of 674 mm.

4.3.3 Analysis

No prediction analysis for stoniness was necessary as this parameter can be easily determined from grading analyses on the material from the borrow pit. It is, however, recommended that the modified method of grading analysis proposed by Paige-Green (1988b) is utilised.

4.3.4 Discussion

The importance of removing oversize stone from the wearing course material cannot be overemphasised. This is one of the few material properties which can be "controlled" prior to use of the material. The removal of large stones can be done in a number of ways depending on the available resources:

- Screening at the borrow pit by means of a "Grizzly"
- Mechanical disintegration on the road with a mobile hammer-mill such as a "ROCKBUSTER"
- Removal from the road prior to compaction with a "Rock Rake"
- Removal from the road prior to compaction by hand-picking
- Crushing of oversize material

These techniques and their implications are discussed further in Chapter 8.3.

Although it is not imperative that every stone larger than 50 or 75 mm is removed from the wearing course material, it is strongly recommended that the percentage of oversize material is restricted to a specified value. The areal extent of the exposed oversize material in a road will depend to a large degree on:

- Size - as the size of the stones increase there is a greater chance of the stone or part of it being exposed at the surface. If the minimum dimension of the stone is greater than the wearing course thickness, compaction and shaping of the road will not be possible without breaking the stone or compacting it into the subgrade. In the latter case, the stone soon protrudes from the road and causes significant roughness, affects the safety and complicates grader blading.
- Shape - shales generally result in very platy stones while granite and dolerite stones usually tend to be more spherical. This usually results in a larger exposed surface area of shale stones in the road due to their horizontal orientation and compaction, while the more spherical stones are not exposed to the same extent.
- Quantity - the more large stones there are, the more chance there is of exposure at the surface, although a part of all stones greater than 150 mm should theoretically be exposed in the road.
- Time since construction - as the matrix surrounding the stones wears away more of the stone will be exposed, especially if the stones are rounded.

To take these aspects into account, strict specifications for each material type or stone shape would be required. However, other aspects such as the variable density of the different particle size fractions (the large stones are often more dense than the matrix particles) compound the problem and result in a simple grading analysis by mass (including the material larger than 75 mm) not being an adequate indicator.

Existing methods of grading analysis (e.g. TMH 1, NITRR, 1986) do not give an indication of the very coarse material. The method of analysis proposed by Paige-Green (1988b) gives a direct indication of the quantity of coarse material in terms of the Oversize Index (I_o).

For optimum performance the maximum particle size should be restricted to 37,5 mm. This is the size specified in the majority of specifications studied and it has been shown to be cost-effective to crush oversize material to pass 37,5 mm for roads carrying more than 50 vehicle per day (Beaven et al, 1987). A limited number of stones up to 150 mm (or 100 mm preferably), however, can be tolerated. It is important that if stones larger than 50 mm are included in the wearing course aggregate, they are not permitted to remain on the surface and in loose material after blading. Extensive damage to modern compact sedan cars and caravans (braking systems, transmissions, petrol tanks and body-work) caused by these large stones has been observed. It is thus recommended that, if a limited number of stones larger than 50 mm are permitted in the wearing course gravel, those loosened during grader maintenance should be manually removed from the road surface.

The number of oversize stones in a square metre of road 150 mm thick can be easily calculated. Table 4.1 shows the number of spherical stones (for 5 per cent oversize by mass) of 150 mm, 100 mm and 75 mm diameters theoretically in a section of road as described compacted to 2000 kg/m^3. The effect of the different stone densities used to convert the per cent by volume in the calculation to a percentage by mass (the usual practice for grading analyses) is shown.

TABLE 4.1: THEORETICAL NUMBER OF STONES IN ONE SQUARE METRE OF ROAD CONTAINING 5 PER CENT OVERSIZE BY MASS

Stone density (kg/m^3)	Number of stones of nominal diameter (mm)		
	150	100	75
3000	3	10	23
2500	3	12	27
2200	4	14	31

Except in strongly gap-graded materials, the oversize stones would, however, include various proportions of the different fractions between 37,5 mm and 75, 100 or 150 mm. Five per cent of material between 37,5 and 150 mm would therefore actually result in less than the number of stones shown.

In practice very few large stones are perfectly spherical. A simple laboratory experiment to relate the areal extent of non-spherical particles to the percentage by mass of the material using two samples was thus carried out. Artificial mixtures of various percentages of oversize material (9,5 to 13,0 mm) and matrix (passing 4,75 mm) were spread into a pan to a depth of 13,0 mm, and the areal percentage of exposed stone measured. This was considered to simulate the full-scale situation of non-spherical stones with a maximum dimension of 100 to 150 mm in a matrix passing 50 mm constructed to 150 mm thickness. Table 4.2 summarises the results of the experiment.

On average only 72 per cent of the mass of the stones is represented in the areal exposure at any one time. It is therefore theoretically probable that only 72 per cent of the number of stones indicated in Table 4.1 would be exposed at the surface at any one time.

Geologically the igneous rocks are generally harder and more likely to produce excessive oversize material. They are also prone to spheroidal weathering resulting in large boulders in a clayey matrix. Pedocretes are also prone to stoniness due to the cementitious nature of their

TABLE 4.2: AREAL EXPOSURE OF VARIOUS PROPORTIONS OF NON-SPHERICAL OVERSIZE MATERIAL

Oversize (% by mass)	Areal extent (%)			
	Quartzite		Granite	
	Mean	Std dev	Mean	Std dev
5	3,0	1,19	4,0	1,63
10	7,3	2,38	6,8	1,96
15	9,0	2,34	12,1	3,44
20	15,5	4,63	14,0	2,27

are also prone to stoniness due to the cementitious nature of their formation. Arenaceous rocks, although generally fairly soft, often have large platy particles.

The severity of the stoniness showed significant correlations (at the 0,5 % or better level) with the kaolin, illite and calcite contents (r = -0,29, 0,26 and 0,34 respectively). The correlation with the calcite content is indicative of the cementitious nature of calcite. Most of the materials with high calcite contents were calcretes which tend to be very stony because of the strong cementing action of calcite. The negative correlation with the kaolin is a good example of consequential correlation. The calcretes generally have little kaolin and a strong negative correlation exists between calcite and kaolin (r = -0,56). This multi-collinearity between the calcite and kaolin results in a significant correlation between kaolin and stoniness and emphasises the importance of analysing the cause of high correlations carefully. The correlation with the illite is attributed to illite being the prime constituent of shales which are particularly stony.

In areas with N-values greater than 5 most of the weathering of rocks is physical with disintegration predominating and only a small amount of decomposition. This results in the formation of a coarse disintegrated gravel with illite being the main clay mineral (Weinert, 1980). The stones show some correlation with the N-value (r = 0,41), confirming the predominance of disintegration in the more arid areas. This correlation is diminished somewhat by the fact that many shales and mudrocks in the wetter areas, although mostly unweathered, are soft enough to be ripped and used in unpaved roads although they contain a large proportion of oversize material.

The extent (quantity) of the stones had significant correlations with the smectite (-0,25), illite (0,47) and calcite (0,31) contents. The previous discussion on the illite and calcite contents are applicable to the quantity of stones but the absence of a significant correlation with kaolin is apparently due to the less significant correlation between calcite and stone extent. The negative correlation with smectite clays (which occur mainly in the igneous rocks) can be ascribed to the chemical weathering of these rocks. The materials with a high smectite content have generally reached an advanced stage of weathering and, whereas the partly weathered igneous rocks with minimal smectite generally have many stones, their highly weathered counterparts have very few stones larger than 50 mm.

4.4 POTHOLES

It has been shown (Paige-Green and Netterberg, 1987) that potholes play an important role in the development of roughness. Potholes can be formed in a number of ways, many of which can be prevented by improved material selection. Any depression in the road surface which affected the roughness of the road (other than corrugations) was classified as a pothole during the routine monitoring trips. Possible causes of these depressions were observed to be as follows:

- Small diameter but deep depressions result from the grading operation plucking oversize stones from the road. Sometimes these stones are dragged along the road by the grader causing elongated furrows but these are usually repaired manually while the maintenance team is on site. Stones greater than 75 mm are particularly prone to this "shoving" by the grader.
- The area immediately after (in the direction of travel) large stones (diameter more than 150 mm) is usually compacted by the traffic to form a depression with a depth approximately equal to the height the stone protrudes above the normal running surface. The depression is caused by both the bounce of the vehicles and

compaction of the material which is generally poorly compacted during construction because of the stress concentration and support of the roller by the large stones.

- With the passage of time corrugations cause vehicles to bounce and enlarge the contact area behind the corrugation into a rounded depression. Potholes formed in this way are difficult to remove during routine maintenance and remain for many months.
- The formation of depressions by any of the means described results in ponding of water during wet periods. This ponding allows the water to soak in and weaken the wearing course material which results in enlargement of the pothole by compaction and shearing under traffic. This is a self-perpetuating process (unless the depressions are timeously repaired) and can lead to large diameter, deep depressions.
- The presence of a weak subgrade can lead to the formation of large depressions during wet periods. If the wearing course becomes too thin and cannot distribute the applied loads adequately, compaction and/or shear in the subgrade occurs and a large pothole may form.
- Potholes caused by animals and insects have been noted on a number of roads. These range from small, shallow holes caused by moles and mongooses to larger depressions due to "undermining" by ants and large holes (500 mm diameter by 1 to 1,5 m deep) caused by warthog. These holes need to be carefully filled and compacted in order to eliminate traffic compaction and further development of potholes.
- The disintegration of highly cracked roads and highly weathered boulders (especially granites and sandstones) results in the formation of potholes.
- "Powder potholes" are common in the drier western parts of the region and consist of depressions filled with very powdery material, usually calcrete. No examples of these were recorded in the Transvaal test sections, but occur commonly in the northern Cape and South West Africa.

Additionally, the presence of dispersive soils (sodium rich) may lead to potholes (Visser, pers. comm) and would certainly exacerbate existing potholes in the presence of standing water.

4.4.1 Method

The potholes were rated by severity and extent on a standard 5-point scale and the depth and diameter of the pothole which appeared to affect the roughness most was measured. The depth of the potholes was generally measured in the direction of traffic flow with the rut depth gauge. If the diameter was more than the length of the gauge, a reading was taken with the gauge angled slightly or the depth under a piece of string across the hole was measured.

The severity of the potholes was rated according to the following criteria:

1 - No visible potholes
2 - Minor depressions which were visible but were not felt in the vehicle
3 - Significant potholes felt in the vehicle but not requiring evasive action
4 - Severe potholes which were obviously avoided by most vehicles
5 - Very severe potholes requiring speed reduction and careful avoidance

Both Eaton et al (1987) and Hudson et al (1987) recommend ratings for potholes based on their depth and diameter with the severity increasing as both of these increase. This results in a pothole about one meter in diameter and 50 to 100 mm deep being rated as high severity while a pothole less than 300 mm in diameter and 50 to 100 mm deep would have a low severity. The monitoring experience has, however, shown that the latter pothole would be far more severe on a passenger vehicle than the former, stressing the mechanical components to a much higher degree. For this reason none of the measured values were used in the severity rating.

An estimate of the percentage of the road over which potholes occurred was made. This was usually less than 20 per cent except for those very stony roads which had depressions caused by impact compaction behind the stones.

4.4.2 Results

The average pothole severity and extent for each section was calculated from all the results. The average severity was 2,13 (σ = 0,65) with a minimum of 1 and a maximum of 4,1 while the average extent was 1,25 (σ = 0,72; M = 3,5; m = 0). The averages of the most severe pothole depth and diameter for each section recorded during each inspection were also determined. The average pothole depth was 35,5 mm (σ = 8,3; m = 20,0; M = 59,2) and diameter was 776 mm (σ = 308; m = 291; M = 1828 mm).

4.4.3 Analysis

The analysis of the potholes attempted to identify those materials prone to the formation of potholes and those properties which were associated with potholed roads. The significant variables with respect to the severity of the potholes were found to be the grade of the section (G), Weinert N-value (N), dust ratio (DR), APV, optimum moisture content (OMC) and maximum dry density (MDD) and the percentage of kaolin in the sample. The extent of the potholes was indicated by the grade, OMC and the MDD.

As both the severity and extent of the potholes were considered important a composite pothole factor was analysed. This factor consisted of the product of the square of the severity and the extent of the potholes. The correlation for this was significantly worse than for the pothole severity by itself and the severity therefore was used for further analyses. A linear regression analysis using the significant variables and their interactions produced a maximum r-squared value of only 0,62. The APV was excluded from the model because of the missing values as was the percentage of kaolin because of the difficulty of determining it. After simplifying the model by removing the non-significant interaction and main effects the r-squared value decreased to 0,45 (RMSE = 0,49; F = 17) for the following prediction equation for pothole severity (POTSEV):

$$POTSEV = 4,2 - 0,185.G - 0,00004.N.MDD - DR(3,10 - 0,257.OMC) - 0,12.OMC \qquad (4.3)$$

All of these variables are significant at the one percent level except the dust ratio which is significant at the 3 percent level. Inclusion of the stoniness severity and extent resulted in a marginal increase in the r-squared value to 0,48. The potholes which form behind the stones are thus generally of minor consequence in relation to the overall pothole severity. The low r-squared value is caused by one of the major contributors to potholes being poor drainage, more than material properties. Ponding of water on flat sections of road with a poor crown, areas of seepage or perched water tables and at the bottom of dips usually results in pothole formation.

4.4.4 Discussion

Most potholes are formed by poor grader maintenance, inadequate drainage and the final stage of corrugation development if maintenance is delayed. The quality of the grader maintenance is of utmost importance as this dictates the shape and cross-section of the road which in turn influences the surface drainage. If water is not permitted to accumulate on the road surface potholes would be restricted in their severity.

All of the parameters included in the model reduce the pothole severity from a maximum rated value of 4,2 i.e. theoretically every unpaved road would have potholes of a severity of 4,2 if all of the factors in the model had values of zero. The implication is that potholes are an inherent part of unpaved roads. The grade of the section was the most important factor in the analysis. The formation of potholes on steeper grades is minimal as there is very little ponding of water and generally improved drainage. The importance of water is confirmed by the fact that as the area becomes more arid (N increases and rainfall decreases) the pothole severity is decreased. Contrary to expectations, an increase in the OMC and dust ratio (i.e. increased plasticity and fines) reduces the pothole severity. This is indicative of potholes being formed by ravelling of weakly cohesive material and the relatively small number of potholes formed by weak wearing courses and subgrades.

The Natal wearing course guide (NPA, ca 1961) indicates that material with a liquid limit greater than 35 percent and a coarse and medium sand content of less than 30 percent is likely to pothole. During the project, none of the sections had a pothole severity mode of 5 (i.e. unacceptable) although 11 sections had modal values of 4 (i.e. undesirable). Fifteen of the sections had liquid limits greater than 35 while 67 had a coarse sand percentage of less than 30. Of the fifteen sections with liquid limits greater than 35 percent only four exhibited undesirable potholes, while six showed potholes rated as two or less. The NPA guidelines appear therefore to be invalid for the Transvaal and South West Africa.

Netterberg (1978) noted that "wearing course materials with weak aggregate tended to break up and form powder potholes under traffic, especially when compacted dry". No powder potholes were recorded in the Transvaal where wet compaction is the norm, but many were noted in the more arid northern Cape and South West Africa where moist compaction is unusual. Some of the Transvaal sections had very weak aggregate, however, with Aggregate Pliers Values considerably lower than the 25 specification of Netterberg (1978). It would thus appear that moist compaction plays an important part in the reduction of powder pothole formation.

The successful repair of potholes in certain materials appears to be difficult. Many of the potholes in the experimental sections remained through the duration of the monitoring with little change except in their depth. It was noted that those sections constructed of chert gravels were the only ones where grader blading adequately repaired the potholes. It is, however, recommended that potholes be repaired by manually compacting good replacement gravel at about optimum moisture content. Presently pothole maintenance generally consists of loosely filling the pothole with material from the side of the road (which usually has a poor grading due to repeated grader blading and removal of fine material by wind and traffic flow and often includes grass and litter).

In summary, drainage problems are the major cause of potholes with the material type being only a minor contributory factor.

4.5 CRACKS

The presence of cracks was initially thought to have no effect on the performance of the roads and was rated as a possible simple indicator of plasticity. It was however noted that those sections with very bad cracks broke up quite badly under heavy traffic and formed potholes.

4.5.1 Method

Cracks were rated during each monitoring trip using the standard TRH 6 (NITRR, 1980) method. Although the method was devised for asphalt pavements, the definitions of the severity of distress adapted well to unpaved roads. The following definitions were used:

1 - No visible cracks
2 - Faint cracks - require close scrutiny to see them
3 - Distinct cracks - easily visible when walking at a reasonable pace
4 - Open cracks with width less than 3 mm
5 - Open cracks with width more than 3 mm

The percentage of the area of the road over which visible cracks of any severity occurred was estimated.

4.5.2 Results

About 5 500 results were used in the analysis. These results were influenced by a number of factors as follows:

1. During the wet season the cracks did not develop fully. After periods of rain the cracks were usually closed due to swell of the wearing course matrix and/or squeezing together of the upper layer of the gravel under traffic.

2. After grader blading or when a lot of loose material had been generated by traffic it was often not possible to see cracks with a severity of less than 5 or to estimate the extent.

3. On very dusty roads the cracks could not be easily discerned.

For analysis purposes the average crack severity for each section over the duration of the project was used. This value was determined separately for the wet and dry seasons with an average crack severity of 2,3 (σ = 1,09; m = 1,0; M = 5,0) and an average extent of 3,7 (σ = 1,3; m = 1,0; M = 5,0) for the dry season. The wet season results were considered to be meaningless.

4.5.3 Analysis

A number of analyses were carried out in order to identify the parameters influencing cracking and to derive prediction models. The following parameters were found to account for 70 percent of the variability:

Plastic limit (PL), liquid limit (LL), linear shrinkage (LS), Aggregate fingers value (AFV), maximum dry density (MDD), grading modulus (GM), material classification, wet or dry season and Weinert N-value (N).

Because of the inability to determine the AFV value of fine materials, this parameter was excluded from the analysis. The following model was the best obtained for the crack severity (CRS) and had an r-squared value of 0,59 and a standard error of 0,68:

$$CRS = 4,50 - 0,16.N - 1,26.GM - 0,076.PL + 0,08.LL \qquad (4.4)$$

The model is significant at the 0,01 percent level as are all the parameters except the plastic limit which is significant at the 0,5 percent level.

4.5.4 Discussion

Though the final model accounts for about 60 percent of the variability the 95 percent confidence limits of the prediction are extremely high ($\pm$ 1,36) when one considers that the predicted value should lie between 1 and 5. The variability of the ratings for each section would thus appear to be very high as was anticipated earlier.

As the N-value, grading modulus and plastic limit increase the propensity of the material to form cracks is reduced, while an increase in the liquid limit results in a significant increase in the cracking.

The importance of various other factors such as the clay mineral type and quantity, traffic, material strength, etc. were found to be insignificant or were incorporated indirectly by being closely correlated with some other property.

Other factors such as the maximum daytime temperature, rainfall intensity, subgrade plasticity and wearing course stiffness would therefore appear to affect the cracking as well as the properties identified in Model 4.4.

Apart from the indirect relationships between the plasticity and the material types, the incidence of cracks could not be related to the geological properties of the material.

4.6 LOOSE MATERIAL

Loose material is considered an important part of unpaved road performance. The generation of loose material by traffic results in a general loss of gravel, the development of ruts and influences the safety and skid resistance of the road. The presence of thick loose material affects the rolling resistance significantly, with a concomitant increase in fuel consumption. Previous studies (Hodges et al, 1975; Visser, 1981b) quantified the loose material by weighing all the loose material in a one square metre section of the road. Initially it was proposed to follow this procedure for this project, but analysis of the existing results indicated little relationship between the loose material and other properties. In addition the results which probably would have been achieved did not warrant the time which would have been expended on determining the loose material. The main reason for the poor results is probably the areal variation of the loose material over the road and the influence of grader blading.

The wheel tracks normally have the least loose gravel of the whole road while thick accumulations are common at the edge of the trafficked portion of the road ("sandwalle") and in the centre of the lane ("middelmannetjie"). It is interesting to note that all of the accidents seen on or near the experimental sections during the monitoring period were associated with total loss of control of the vehicles due to thick berms of loose material (usually more than 100 mm). During roughness testing however it was noted that the most slippery roads in the dry state were those which had a thin layer of loose, rounded gravel (usually about 5 mm in diameter) over most of the surface, such as the quartz porphyry gravels of the western Transvaal.

4.6.1 Method

A quantitative estimate of the loose material was obtained for each section during each monitoring trip. If the road had been graded within the past few days and no distinct wheel tracks were evident no rating was done. The rating was carried out with the objective of identifying those materials which were particularly prone to ravelling under traffic and would thus require frequent grader blading and regravelling.

The following severity rating system was used:

1 - No loose material at all was present
2 - Less than 10 mm thickness of loose material was present
3 - Between 10 and 20 mm thickness of loose material was present
4 - Between 20 and 40 mm thickness of loose material was present
5 - More than 40 mm thickness of loose material was present

Loose material was defined as that surface material which had no cohesion and could be freely moved by scraping with a shoe or stick. The maximum observed severity was recorded provided that this occurred over more than about five percent of the area of the section or else the most prevalent severity was recorded. A quantitative estimate of the total areal extent of the loose material (irrespective of its severity) in the section was made.

The thicknesses used for this project are significantly less than those recommended by Eaton et al, (1987), where a low severity is less than two inches (50 mm) while a high severity is more than four inches (100 mm) of loose gravel. The local monitoring experience has shown these values to be unrealistically high as thicknesses of 50 mm play an important part in influencing the control that the driver has over the vehicle.

4.6.2 Results

The average loose material for each section over the duration of the project was analysed in terms of the salient properties. The influence of time on the generation of loose material was considered important initially but observation indicated that within about three weeks of grading, the loose material reached an optimum thickness and only the extent varied after this. Redistribution of the material seemed to occur and a gradual loss by vehicle whip-off and surface erosion probably removed what was being generated.

The average loose material severity rating for all the sections was 3,0 (σ = 0,68; m = 1,43; M = 4,62) with a similar average for the extent ratings (σ = 0,67; m = 1,1; M = 4,5).

4.6.3 Analysis

The important factors affecting the formation of loose material were identified by a multiple correlation analysis. The following parameters all had Pearson Correlation Coefficients in excess of 0,35 (significant at the 0,01 percent level):

Weinert N-value (N), rainfall (RAIN), percent heavy vehicles (PERHEAV), in situ density and moisture (DENS and MOIST), liquid limit (LL), plastic limit (PL), percent smaller than 0,075 (P75) and 0,002 mm (P2M), dust ratio (DR), grading modulus (GM), optimum moisture content (OMC), maximum dry density (MDD), soaked CBR (SCBR) and material classification. A linear regression analysis of those of the variables which can be easily determined and those for which results were available for all the sections was carried out. The material classification was not used in the analysis as it is a classification variable and is not continuous.

An r-squared value of 0,59 was obtained for the best model which had a standard error of 0,44 (F = 51, significant at 0,01 percent level):

LOOSE MATERIAL = 0,68 - 1,49.DR + 0,09.N + 0,0013.MDD (4.5)

The other significant variables were all excluded during the regression analysis with a decrease in the r-squared value of only 0,02.

4.6.4 Discussion

The loose material is mainly dependent on the climatic N-value with this accounting for 37 percent of the total variability. The cohesion of the material would be expected to be an important factor influencing the loose material generation and this is taken into account in the density and dust ratio parameters where the plasticity, fine fraction grading and strength all correlate closely. It must also be noted that the N-value has an important influence on the mineralogy of the gravel (Weinert, 1980).

The 95 percent confidence limits ($\pm$ 0,88) are again somewhat larger than hoped for, indicating either that substantial variation occurred in the results or that some property which was not analysed (e.g. vehicle speed, rainfall intensity, etc.) affects the generation of loose material significantly. The rating system consisting of integer values probably also plays an important role in this high standard error.

The NPA wearing course guide indicates that material with a liquid limit less than 20 per cent, and/or a plasticity index and clay content of less than six per cent are likely to ravel in dry weather (Mitchell et al, 1979). Seven of the sections monitored had modal values for the loose material severity of 5 (i.e. unacceptable) while 32 had values of 4 (i.e. undesirable). In terms of the NPA guidelines, 23 sections had liquid limits less than 20 and 39 sections had plasticity indices less than 6. Clay fraction determinations were not done on the South West African sections but of the Transvaal sections 26 had percentages less than 6. There was, however, no correlation between the limits in the guide and the formation of loose material.

The loose material did not correlate significantly with the material groups although a poor but significant correlation ($r = 0,36$) was found with the material classification. Although the material classification is discontinuous there is a general increase in plasticity with an increase in the classification number allocated.

The use of so-called "sand-blankets" over unpaved roads provided some interesting observations. These sand blankets are theoretically a thin layer of fine material bladed over the surface of the road. It would appear that they were originally intended as a protective layer over calcrete wearing courses which had formed a crust ("blad" (Netterberg, 1967)) by avoiding breaking of the surface crust under heavy traffic and during grader maintenance. The purpose of these sand blankets seems, however, to have been obscured with time, and many grader operators now blade a loose layer of material as a standard procedure. This results in a number of problems as follows:

- Ruts are filled with permeable, loose material and form drainage channels.
- No definite crown is retained on the road, resulting in poor drainage, and ultimately a concave cross-section.
- The loose material affects the overall safety of the road significantly.
- Towing with front-wheel-drive vehicles is made extremely difficult.
- Numerous oversize stones are included in the loose material which can cause significant damage to vehicles and tyres.
- This material is particularly prone to the formation of corrugations. Corrugations removed by blading have been seen to reform within 24 hours, under less than 50 vehicle passes.
- Many sand blankets are particularly dusty.

It is recommended that the practice of blading a thick layer of loose material over unpaved roads is curtailed and sand-blankets are limited to a thin layer (not more than 15 mm) of fine material over roads with a definite crust or "blad".

4.7 DRAINAGE AND EROSION

During the monitoring a single evaluation of the surface drainage of each section was made. The drainage parameter evaluated the water shedding capability of the section and implicitly included any evidence of erosion. This resulted in one descriptor for both parameters. Although this was adequate for most of the sections, in retrospect separate evaluations would have provided a lot more useful data. Fortunately, because of the repeated visits to the sections the characteristics of each one became well known and it was possible during analysis to recall the erosion susceptibility of each section.

4.7.1 Method

The surface drainage and erosion were evaluated according to the following severity criteria:

1 - No water would accumulate or lie on the road surface.
No surface erosion was evident

2 - Shallow depressions may retain water for a limited period but most water would drain rapidly. Side drains were effective.
No surface erosion was evident

3 - Water may be retained in shallow ruts and potholes and in the side-drains.
Surface erosion was restricted to loose material.

4 - Water may be retained in depressions up to 50 mm deep. This could lead to weakening of the material by saturation and the development of potholes under trafficking.
Surface erosion affects the compacted material to a depth of less than 50 mm.

5 - Water ponds in ruts and potholes to a depth greater than 50 mm and lies in side-drains for a number of days.
Surface erosion results in channels deeper than 50 mm.

4.7.2 Results

As it was seldom raining during monitoring the surface drainage was evaluated with respect to the observed drainage characteristics e.g. cross-section, side-drains, potholes, ruts, erosion, etc. Obvious signs of poor drainage such as wheel ruts and churning were looked for. The average ratings of severity and extent for all the sections were 3,23 (σ = 0,65; m = 1,78; M = 4,62) and 3,59 (σ = 0,54; m = 2,33; M = 5,0) respectively.

4.7.3 Analysis

A correlation matrix was developed in order to identify the significant variables (at the 5 per cent level). These were the N-value (N), rainfall (RAIN), average daily traffic (ADT), plastic limit (PL), laboratory maximum size (LABMAX), per cent passing 4,75 mm sieve (P475) (the best of many significant grading parameters), grading modulus (GM), gravel index (GI), AFV, APV, OMC and relative compaction (COMP).

The r-squared value for the best model was 0,40 (significant at the 0,01 per cent level) which had a standard error of 0,55. Exclusion of the APV results reduced the r-squared value to 0,32 but included the results from all the sections.

The best model obtained for the surface drainage severity (SURDR) was the following:

$$\begin{aligned} SURDR = {} & GM(5{,}63 - 0{,}18.PL - 0{,}02.P475) + LABMAX(0{,}15 - 0{,}001.COMP) \\ & + N(0{,}06 - 0{,}002.APV) - GI(17{,}32 - 0{,}81.PL) + 0{,}04.P475 \\ & + 0{,}06.COMP + 0{,}002.ADT - 6{,}9 \end{aligned} \tag{4.6}$$

This is a fairly cumbersome model involving a number of interactions, but it identifies the grading parameters and plasticity as the main material factors.

4.7.4 Discussion

When one considers that the main factor influencing the surface drainage is the cross section and shape of the road a high correlation between the surface drainage severity and the material and environmental properties cannot be expected. The cross-section is however controllable by using well trained and experienced grader operators and not allowing the shape of the road to deteriorate to a condition which cannot be easily restored.

The rainfall intensity and duration are certainly significant parameters in the surface drainage and erosion characteristics of an unpaved road.

No significant correlation was found between the material groups and classification and the drainage and erosion problems. It was, however, generally observed that granitic materials were susceptible to erosion, but this was probably more because of their particle size distribution than the fact that they were granites.

4.8 SLIPPERINESS

The slipperiness of unpaved roads in both the wet and dry states is important from the safety aspect. This is confirmed by the average plasticity index of all the sections which is biased towards the lower end of the existing specifications. It would appear that non-slippery materials are intentionally selected.

4.8.1 Methods

During the monitoring the slipperiness rating was based on the perceived handling of the LDI vehicles at 80 km/h. It was, however, decided to obtain a more quantified value towards the end of the project and a MOTOMETER brake efficiency meter was used to measure the actual deceleration under locked-wheel skidding tests. A number of tests on various surfaces were carried out in order to check the validity of the equipment for slipperiness testing. The results of these are shown in Figure 4.4. Assuming uniform deceleration (i.e., constant tyre/road friction or skid resistance), the deceleration is independent of the speed and this was confirmed by a number of braking tests from different speeds (Figure 4.5). Safety and tyre wear considerations resulted in all the testing being done from 50 km/h.

All the A sections in the Transvaal remaining at the end of the project were tested in the dry condition and as many as possible under wet conditions. The B sections were not tested as the horizontal curvature would have made skidding dangerous and the grade would have yielded different results depending on whether testing was in the uphill or downhill direction. Testing in the wet condition relied purely on chance (whether it was raining at the time of visit to any section) and logistical considerations made testing of all the sections distant from Pretoria impossible. Testing was carried out in both directions in the wheel-paths and loose material, where it occurred, with six tests generally being done per section.

4.8.2 Results

The average deceleration of all the sections tested in the dry state (45) was 5,9 m/s^2 with a maximum of 8,0 and a minimum of 4,1 (σ = 0,57). The six sections tested wet had an average value of 5,3 m/s^2 with a maximum of 6,2 and a minimum of 4,1 m/s^2 (σ = 0,83).

4.8.3 Analysis

A correlation matrix was developed for the mean slipperiness when dry and the significant variables identified. These were the mean annual rainfall (RAIN), average daily traffic (ADT), per cent passing the 26 mm (P26) and 0,075 mm (P75) sieve, the plastic factor (PF), optimum moisture content (OMC), and dust ratio (DR). The best model obtained had an r-squared value of 0,70 with a standard error of 0,33 (Model 4.7).

$$\text{SLIPPERINESS} = 7{,}69 - \text{RAIN}(0{,}001 + 0{,}0001.\text{P75}) - 0{,}018.\text{P26} - 0{,}004.\text{PF} + 1{,}08.\text{DR} \qquad (4.7)$$

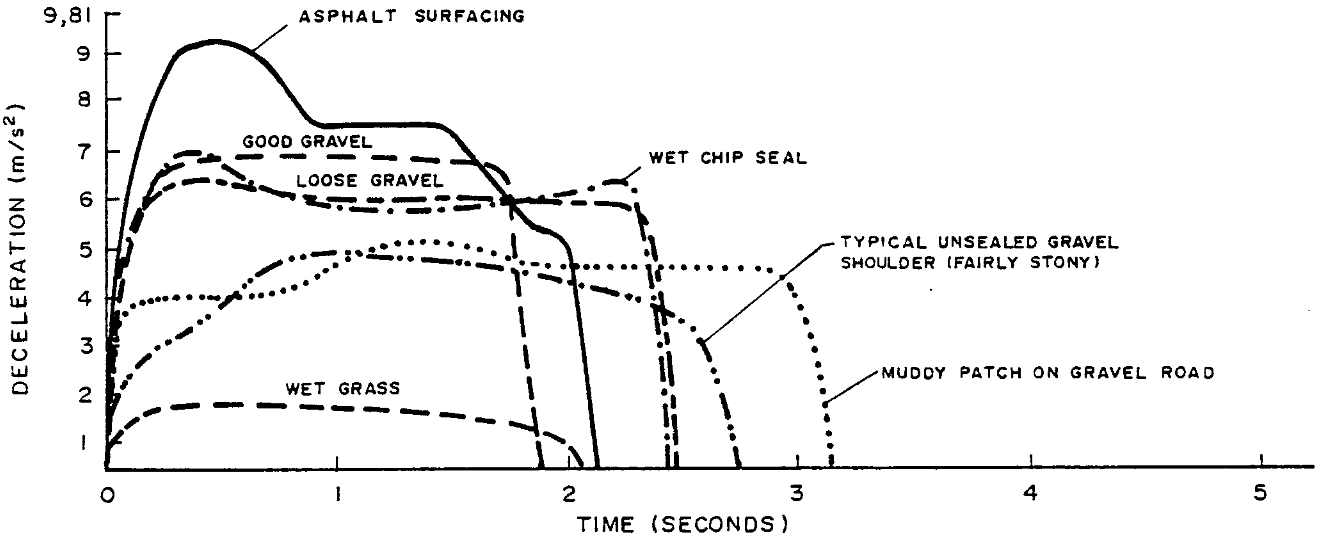


FIGURE 4.4

EFFECT OF DIFFERENT SURFACES ON DECELERATION

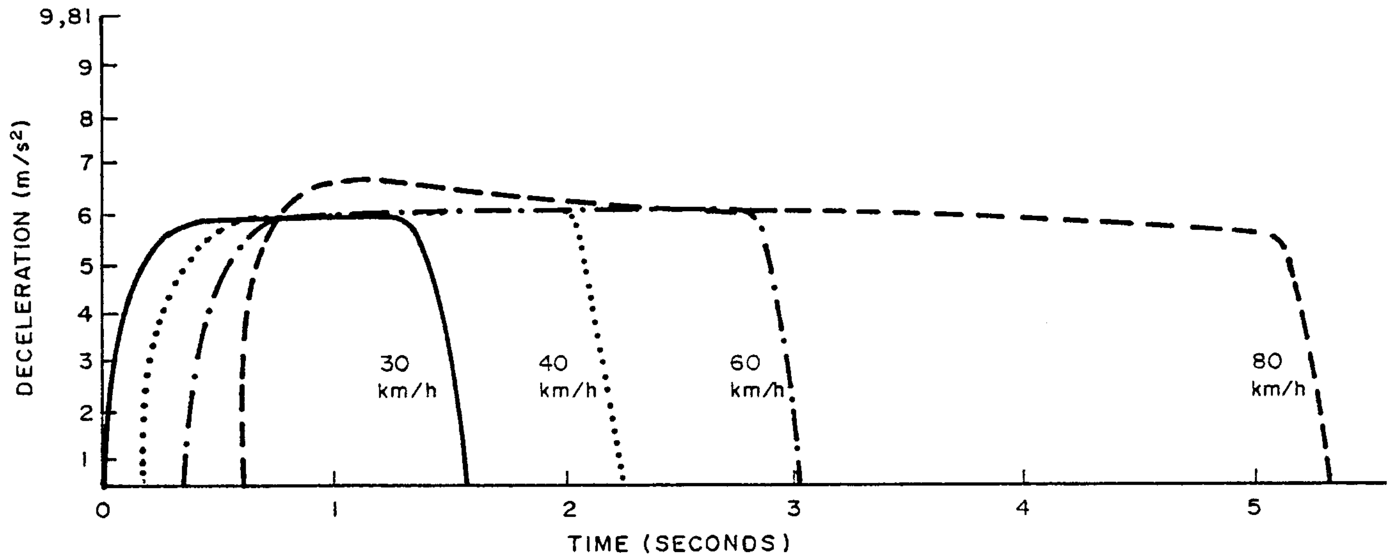


FIGURE 4.5

EFFECT OF DIFFERENT SPEEDS ON DECELERATION

The slipperiness in the wet state could not be effectively analysed due to the limited number of results. The limited results obtained indicate that the road needs to be saturated (with free water on the surface) before a significant increase in the slipperiness occurs.

4.8.4 Discussion

The slipperiness when dry is affected mainly by the plastic factor and increasing percentage passing the 26,5 mm sieve (i.e. finer material). An increase in the per cent passing 26,5 mm from 50 to 100 per cent results in a decrease of almost 1 m/s² in the deceleration. The dust ratio shows a strong correlation with the plasticity, indicative of a high cohesion increasing the skid resistance in the dry state.

SABS 1207 (SABS, 1985) prescribes a minimum deceleration for new cars of 5,8 m/s² on a dry good tarred road as a specification for braking systems. In the absence of previous work on skid resistance of unpaved roads, this value has been assumed to be indicative of a safe surface. The lowest values measured on the unpaved sections were on those constructed of igneous material (granite, basalt, dolerite, andesite and quartz porphyry) and calcretes all of which had minimum decelerations of just less than 5,0 m/s². Some of these materials (andesite, dolerite, calcrete) had extremely high quantities of coarse aggregate while others (granite, quartz porphyry) had negligible coarse aggregate. Conversely the sedimentary materials (shale, mudstone, sandstone, chert) all had minimum values in excess of 5,5 m/s². In the dry condition the roads could thus all be considered adequately skid resistant if a new car (i.e. assumed to be at its best) requires a value of 5,8 m/s².

It can be concluded from the discussion that large stones are not necessarily required for good skid resistance but an adequate cohesion (in terms of plasticity) and percentage retained on the 26,5 mm sieve should provide an acceptable skid resistance under dry conditions. The presence of loose material generally reduces the skid resistance but in a number of tests the loose material was found to accumulate in front of the skidding tyres and improve the skid resistance.

Although only six roads were tested wet the decrease in average deceleration was generally between 4 and 40 per cent with a granite road increasing in skid resistance by 5 percent. The highest recorded decreases were for the shale roads which reduced from initial values of 6,4 to 6,7 m/s^2 to between 4,1 and 4,5 m/s^2. Free water was on the surface when the the value of 4,1 m/s^2 was measured. Again the roads with the largest proportion of oversize material (shales) were the most slippery while the granite road with no stones at all increased in skid resistance (probably due to the rapid infiltration of water and capillary cohesion of the loose material.

4.9 TRAFFICABILITY (PASSABILITY)

All weather trafficability or passability of unpaved roads is probably the main consideration for the existence of engineered unpaved roads and the necessity to import suitable gravels. Previous work carried out by Visser (1981a) indicated that a minimum soaked CBR (intermediate Proctor compaction) of 12 is necessary for passability for an ADT of 12 while a value of 18 is required for an ADT of 400. In addition the duration of the rainfall should be considered and not the intensity (Visser, 1981a). Heavy rainstorms of short duration (common in the western areas of the study region) usually result in fairly rapid drainage and drying whereas continuous light rain such as that prevalent in the eastern areas results in penetration into the wearing course and an overall weakening of the material. Visser (1981a) showed that the plasticity index and per cent finer than 0,075 mm were not adequate indicators of passability.

The trafficability is basically an indication of the load supporting characteristics of the material. The wearing course should have an adequate strength (in the weakest condition it is likely to achieve) to support the number of vehicles passing over it without excessive deformation or weakening of the trafficked surface or excessive stresses being transferred to the underlying layers which themselves may deform or shear. The weakest state is always associated with wet conditions. The wearing course deformation generally takes the form of shallow surface shearing (churning) with the formation of a progressively deepening layer of uncompacted low strength "mud". If the wearing course strength decreases to such a degree that it cannot distribute the applied stresses adequately over the subgrade deeper seated shearing occurs resulting in exposure of the subgrade and the formation of deep potholes surrounded by low strength saturated soil.

4.9.1 Methods

The monitoring of the trafficability was based on visual examination of each section during the routine visits. Any evidence of vehicles being bogged down or churning of the wearing course would indicate poor or unacceptable trafficability. No section during the course of the project showed any evidence of impassability although some of the sections showed limited areas of minor shearing and rutting which under heavier traffic may have resulted in unacceptably poor trafficability or impassability. These were generally associated with areas of inadequate drainage. The general conclusion from the observations was that the traffic was minimal during very wet periods.

The severity of the trafficability was rated according to the following criteria:

1 - No evidence of softening of the wearing course
2 - Shallow wheel tread impressions were present (< 5 mm)
3 - Marked wheel tread impressions were present (± 10 mm)
4 - Churning of the surface was evident
5 - Evidence of substantial churning and shearing of the wearing course was present

Although the passability of severity 5 sections was still acceptable the fact that all of the sections were numbered roads resulted in a conservative rating. It was considered that if the condition was extended for longer distances the effect of wheel spin and "mud-spray" would probably result in rapid deterioration of the sections. The rating was thus an indicator of the possible problems under sustained traffic in prolonged periods of wet weather.

4.9.2 Results

Very few results, none of which indicated unacceptably poor trafficability were obtained. The worst condition monitored for each section was thus used in the analysis. The average of the worst condition for all the sections was 2,6 (σ = 1,5; M = 5; m = 1). In all cases the ratings of 5 were over limited lengths of the road (generally less than 20 per cent) and were usually associated with poor drainage which resulted in ponding and churning under traffic.

4.9.3 Analysis

A multiple correlation analysis indicated that the rainfall, in situ density, liquid limit, linear shrinkage, plasticity index, optimum moisture content, maximum dry density and kaolin, smectite, illite and calcite contents were the only variables significant at the 5 per cent level. The plasticity index and calcite content have the highest positive correlation while rainfall has the highest negative correlation. Visser (1981a) showed that the soaked CBR was the best indicator of passability while the grading and plasticity factors were inadequate. Yoder (in Netterberg, 1978) indicated that the soaked CBR (Modified AASHTO compaction) above 60 was necessary for wearing courses under heavy truck traffic. The results of this study show a poor correlation between soaked CBR (Proctor compaction) (r = 0,1). This may be due to the fact that none of the sections showed any unacceptable trafficability over any significant length.

4.9.4 Discussion

Visser (1981a) provided a number of plots of various material properties against traffic with limits for passability. The maximum CBR for impassability (CIM) was defined in terms of the traffic (ADT) as

$$CIM = 8,254 + 3,745 \, Log10 \, (ADT) \qquad (4.8)$$

No definite relationships existed for material grading or plasticity parameters.

During the wide-spread flooding of the northern areas of South Africa in February, 1988 most of the unpaved roads over large areas were totally closed to traffic. Many of these roads, however, had high soaked strengths (in terms of the CBR) in terms of existing standards. Two reasons for the inadequacy of these roads are proposed.

1. Nearly all natural gravels used in unpaved roads contain clay minerals, which expand on absorption of water to a greater or lesser extent. Under flooded conditions some expansion of the clays occurs and the compaction density decreases resulting in a lowering of the strength.

2. The CBR test involves the application of a load at 1,27 mm/min which is probably adequate to allow pore-water pressure dissipation for most soils, certainly in the partially saturated state which is probably not exceeded under normal rainfall conditions. Under a moving wheel, however, the load application is much quicker, and if the material is in the fully saturated state (e.g. after prolonged total submersion during flooding) the pore water pressure will not dissipate and the material will fail under total stress conditions.

4.10 CORRUGATIONS

Corrugations typically consist of a series of ridges perpendicular to the direction of the road. Where they occur they are usually the major contributors to roughness (Paige-Green and Netterberg, 1987) and can also cause considerable damage to vehicle suspension systems (Heath and Robinson, 1980). Corrugations also lead to intermittent loss of contact between tyres and the road surface, reducing the effectiveness of steering and braking which can lead to loss of control of the vehicle. The most extreme roughness measurements recorded during the project were on badly corrugated roads. Unlike other forms of distress such as stones and potholes which can usually be avoided at reasonable speeds, corrugations often become worse as the vehicle decelerates or accelerates. Two distinct types of corrugation were recorded during the project:

a. "Loose" corrugations which consisted of parallel ridges of unconsolidated sandy material perpendicular to the direction of travel (Plate 4.3).

b. "Fixed" corrugations which consisted of hard compacted parallel ridges of material (Plate 4.4). Some of these corrugations were perpendicular to the direction of travel (usually about one metre wavelength) while others were at about forty five degrees to the direction of travel (usually 2,5 to 3,0 metre wavelength).

Corrugations were observed in a number of situations not all of which were the fault of the materials used for construction:

1. The most common cause of corrugations was apparently due to excessively sandy material with a paucity of plastic fines and minimal cohesion.

2. Very stony roads often indicated a form of corrugation probably due to the oscillation of the vehicle initiated by the stones.

3. A number of sections exhibited corrugations with a wavelength of between 2,5 and 3,0 metres and amplitudes of up to 75 mm (i.e. maximum depth 150 mm). These corrugations were of the "fixed type", at 45° to the direction of travel and were not removed by grader maintenance.

4. Corrugations generally do not occur during the rainy season.

4.10.1 Methods

The severity of corrugations was rated for all sections in the Transvaal on the following basis:

PLATE 4.3: TYPICAL "LOOSE" CORRUGATIONS WITH UNCOMPACTED CREST

PLATE 4.4: TYPICAL "FIXED" CORRUGATIONS WITH COMPACTED CREST

1 - No corrugations were visible
2 - Slight corrugations were visible but could not be felt in the monitoring vehicle at 80 km/h
3 - Significant corrugations could be felt but were not bothersome to the occupants
4 - Severe corrugations were felt and required evasive action e.g. speed reduction
5 - Very severe corrugation necessitated drastic speed reduction and affected the directional stability of the vehicle.

This severity rating was, however, not carried out for the South West Africa sections as it was only developed some time after monitoring began in South West Africa. The average amplitude, wavelength and width was measured and the areal extent of corrugations estimated for all sections in the Transvaal and South West Africa as described in Chapter 3.4.

4.10.2 Results

The average corrugation depth and spacing (wavelength) for each section over the duration of the project was determined. If no corrugations were evident a missing value was recorded and the average was determined on the measured values. Those sections where no corrugations occurred at all were given a value of zero for the analysis. The mean corrugation depth recorded for all the sections was 16,9 mm (σ = 10,5; M = 37,8; m = 0) and the mean spacing measured was 759 mm (σ = 567; m = 0; M = 2 830 mm). The average maximum depth and wavelength were 28,4 mm (σ = 18,9; M = 65,0) and 1 198 mm (σ = 912; M = 3 000 mm) respectively. It is interesting to note that the corrugation depth appears to have a maximum value, which is reached fairly quickly after maintenance, after which no further increase occurs even if maintenance is delayed.

4.10.3 Analysis

The depth of the corrugations appeared to be the factor affecting the roughness the most while the spacing was bimodal with values of about 800 to 900 mm and 2 500 to 3 000 mm. For analysis purposes the maximum measured corrugation depth and wavelength were used as these were considered better indicators of the propensity to form corrugations. The significant variables affecting the maximum depth of the corrugations (which were the same as those for the average depth) were dust ratio (DR), grade (GRADE), rainfall (RAIN), Weinert N-value (N), percentage of heavy vehicles (PERHEAV), liquid and plastic limits (LL and PL), per cent smaller than 0,075 mm (P75), optimum moisture content (OMC), maximum dry density (MDD), CBR at optimum moisture content, (OMCCBR) and quartz content.

The liquid limit and quartz content were not included in the analysis as the plastic limit correlated better than the liquid limit and the quartz content is difficult to determine. The best prediction model obtained was the following which had an r-squared of 0,57 and a standard error of 12,4:

$$\begin{aligned} CORR = {} & 93{,}2 - 37{,}7.DR - RAIN(0{,}07 + 0{,}007.GRADE + 0{,}001.OMCCBR) \\ & - 0{,}81.OMCCBR - PL(0{,}37.GRADE + 0{,}25.N) - 0{,}39.OMC.N \qquad (4.9) \end{aligned}$$

4.10.4 Discussion

The cause of corrugations has fascinated engineers for many years with theories on their origin having been published regularly since the first publication by Ladd (1924). Heath and Robinson (1980) summarised some 30 papers published between 1924 and 1978 and concluded that "corrugations are formed and perpetuated by forced oscillations at the resonant frequency of vehicles' suspension and tyre systems".

It was recorded during the project that the average wavelength of corrugations was about 800 or 900 mm on normal rural roads. The average spacing in the Kruger National Park, however, was found to be about 320 mm. The speed limit here is 40 km/h and monitoring showed that most vehicles travel at a speed of between 25 and 40 km/h. Other measurements were carried out in different areas where characteristic speeds were found with the following results:

Rietvlei Dam: Very stony road limited speeds to about 20 km/h: 200 mm

Kalahari Gemsbok Park: Speed limit 50km/h: 350 - 450 mm

Etosha National Park: Speed limit 60 km/h: 500 - 600 mm

South West Africa rural roads: Average speed ± 120km/h: 1000 - 1200 mm

Augrabies Falls: Drainage mounds restrict speeds to <20km/h: 200 mm

It would thus appear that the wavelength is proportional to the speed of the vehicles. Further evidence for this is the presence of short wavelength corrugations near areas of acceleration and deceleration (e.g. near intersections and sharp corners) on normal rural roads. Shorter wavelength corrugations tend to have smaller depths. The average depth of the corrugations (in the main study) correlates significantly (at the 0,01 per cent level) with the average wavelength ($r = 0,5$).

This would fit in with the "forced oscillation theory" (Heath et al, 1980) where both the horizontal and vertical components of the wheel movement increase with increasing vehicle speed resulting in a longer wavelength and deeper trough. The impact stresses caused by the impact of the wheel on regaining contact with the road increase as the wavelength increases (i.e. the trough of the corrugation is deepened) while the longer period with no road contact results in a more rapid spin of the wheels and greater material kick-back resulting in a higher mound of loose material. In summary it thus appears that the wavelength of corrugations is speed dependent while their depth is both speed and material dependent.

Much controversy has been generated over the movement of corrugations along the road surface (Heath and Robinson, 1980). Some workers predict movement of the corrugations in the same direction as the traffic movement (Mather, 1962; Riley and Furry, 1973) while in the field corrugations have been observed to move in the opposite direction to the traffic flow (Gresillon, 1978). Florentin and L'Heriteau (1950) suggest that corrugations move in the same direction as traffic during their formation, but in the reverse direction once they have stabilised. The proportionality of the speed and the spacing can account for both of these theories. Should the average speed of travel change for some reason (e.g. more heavy vehicles, tourist season, etc) the corrugation spacing can be expected to change, with a concomitant movement of the corrugations (forward for increased average speed; backwards for decreased average speed).

No reference was found in the literature to the formation of "fixed corrugations". Two distinct forms of these were recorded during the study and attributed to different causes.

1. Some of the sections were found to have large wavelength, fixed corrugations (between 2,5 and 3,0 metres) with troughs up to 150 mm deep which were not removed by grader blading. The corrugations were generally at an angle of about 45° to the direction of the road and the materials were generally cohesive with plasticity indices of about 10. During the blading operation it was noted that the grader rode over the corrugations and did not cut them at all. It was then found that the wavelength of the corrugations was the same as the distance between the front wheels of the grader and the grader blade which was at an angle of 45°. Normal grader maintenance will not remove these corrugations. In order to remove this type of corrugation it is recommended that the grader blade is extended out of the side of the grader and the grader drives on the uncorrugated shoulder for the first cut i.e., the grader is not affected by the corrugations.

2. Fixed corrugations with a wavelength of about 900 to 1 000 mm were common in both the Transvaal and South West Africa. These consist of well compacted wearing course material and may be formed by two processes.

   - Compaction of loose crest material - The normal corrugations formed of loose material appear in some cases to have been compacted in a moist condition under traffic. Examination of the crest material shows a fairly sandy material different to the underlying compacted wearing course material. A certain set of conditions would be necessary for this to occur as corrugations are usually flattened out by traffic in the wet season. The moisture content probably needs to be at about optimum and the percentage of heavy vehicles should be low.

- Compaction of firm trough material - Evidence was noted of the possibility that during the dry season the impact of the wheels on regaining contact with the road after passing over the crest of the corrugation causes compaction of the material in the trough. Grader maintenance spreads out the crest material but leaves unconsolidated fine material in the troughs which are not affected by the blading. This material is rapidly "whipped out" to leave deep troughs which perpetuate the process.

Fixed corrugations are not normally removed by routine grader blading. The tops of the crests are usually skimmed off and loose material is deposited in the troughs. The loose material is soon kicked back to the crests and new corrugations form. Where fixed corrugations occur heavy cutting by the grader is necessary to remove them and regular blading is required to stop them forming again. It does not help to fill them with loose material cut from the crests.

4.11 RUTS

Ruts on unpaved roads are continuous longitudinal depressions marking the wheel paths in the wearing course. Rutting is caused by the ravelling and loss of gravel from the wheel paths under traffic and/or the compaction in the wheel paths of the wearing course or subgrade under traffic loading. The rut depth has in the past been used as an indication of failure or the point of terminal use (Ahlvin and Hammitt, 1975; Paterson et al, 1975), where ruts of between 50 and 100 mm are classified as a failure condition.

The development of ruts on unpaved roads has been investigated in great detail in Kenya (Hodges et al, 1975) and in Brazil (Visser, 1981a). Both of these studies produced equations for rut depth prediction none of which were particularly good models in terms of their prediction accuracy. This was attributed to the variability in location, and therefore depth of ruts with time and traffic (Visser, 1981a).

4.11.1 Methods

The procedure followed in Brazil (Visser, 1981a) was used for this project. However after about 8 months of monitoring it was noted that the development of ruts was minimal and the measurement techniques resulted in large variations in measurements. This was particularly true for the roads with a large number of oversize stones where it was difficult to seat the straight edge and obtain consistent results. For this reason and the time involved in the collection of this somewhat variable data it was decided only to measure ruts where significant well-defined ruts occurred with the objective of identifying those materials which were particularly prone to rutting.

4.11.2 Results

The ruts were measured in both directions of the section and the average depth over the section calculated. However, as the traffic counts were for both directions the average rut depth over the whole section was used in the analysis. In all 1 370 rut depth measurements were obtained (each one the mean of 6 measurements in each wheel track of the section). The average rut depth for each section over the duration of the project was then determined. This value is biased on the high side as measurements were not carried out if no obvious ruts were observed and more measurements were thus taken when the ruts were at their deepest. The mean rut depth of all the sections was 7,8 mm with a minimum and maximum of 2,5 and 17,9 mm respectively. The values are surprisingly low considering that many of the sections were left ungraded for periods of up to 6 months or more.

4.11.3 Analysis

Attempts to predict the rate of rut development along the lines used for the Brazil study were generally unsatisfactory. The identification of those properties which led to rut development was thus carried out by a multiple correlation analysis. The significant properties (at the 1 per cent level) were the rainfall (RAlN), Weinert N-value (N), average daily traffic (ADT), percentage heavy vehicles (PERHEAV), laboratory maximum size (LABMAX), per cent passing 4,75 and

0,075 mm sieves (P475 and P75), optimum moisture content (OMC), maximum dry density (MDD) and plastic factor (PF). A multiple correlation with these parameters resulted in an r-squared value of only 0,24 for the best model (standard error = 2,79).

This low correlation is indicative of the variation in the non-material factors affecting the rut depths which were not included in the analysis (e.g. drainage, vehicle wander, trafficked road width).

4.11.4 Discussion

Eaton et al (1987) and Ahlvin and Hammitt (1975) indicate rutting of 25 mm as low severity with ruts deeper than 75 mm as high severity. The average values obtained in this study are thus all of low severity with very few measurements of more than 50 mm obtained during the study.

The factors affecting the formation of ruts are basically similar to those influencing the gravel loss. Climate and traffic have a strong influence on the rut formation while a low plasticity results in ravelling and material whip-off.

Jones (1984a) monitored the rut depths of lateritic, quartzitic, volcanic and sandstone gravels in Kenya and developed different prediction models for each material group. No laboratory test results are given in the paper, although testing was carried out. The models predict the rut depth from the traffic alone (no statistics are provided) and do not account for the material variability within the material groups (e.g. volcanic gravels in this study had PI values in the range 1,5 to 14,0). The rut depths on volcanic and quartzitic materials were predicted from the following models:

$$\text{Rut depth} = 10,11 + 0,314T + 0,0003T^2 + 0,00002T^3 \quad \text{(volcanic)} \quad (4.10)$$

$$\text{Rut depth} = 7,49 + 0,171T + 0,014T^2 - 0,00009T^3 \quad \text{(quartzitic)} \quad (4.11)$$

where T = cumulative traffic in thousands.

This model indicates that minimum rut depths of about 10 and 7,5 mm are always present and after 10 000 vehicle passes (i.e. 100 vehicles per day for over three months) the depths increase to 13 and 10 mm respectively. The curves were based on cumulative traffic counts of up to 120 000.

Under the normal grading frequencies used in southern Africa and even the reduced frequencies for this project, the rut development is often minimal. It can usually be neglected in maintenance management systems as maintenance required for roughness correction generally corrects the rutting.

4.12 CONCLUSIONS

A number of multiple correlation analyses have been carried out to identify the main factors affecting the various performance criteria of unpaved roads. Most of these models have very low r-squared values and relatively high standard errors in terms of prediction models. Results from all the standard engineering classification tests, many strength tests and a number of non-routine tests were used in the analyses and the best prediction models with this data were obtained. The fact that only about 50 per cent of the variation is accounted for by most of the models indicates that other factors not taken into account in the analyses are as important. A number of these factors have been discussed in the chapter (e.g. variation in traffic volume and speed, rainfall intensity, quality of maintenance etc.) but other unidentified factors must also be important. It is, however, unlikely that these factors can be controlled and this type of influence has to be accepted.

The interaction of different performance parameters results in problems with the interpretation of the behaviour of the materials. Potholes, rutting, trafficability and dust for example are all strongly influenced by surface drainage which in itself may be influenced by potholes and rutting.

The geotechnical properties (particle size distribution and plasticity in particular) influence the performance of the materials significantly.

The observations and measurements of corrugations support the "forced oscillation theory" as the cause of their development. New theories regarding the influence of vehicle speed on the depth, spacing and movement of corrugations have been propounded, while the differences between loose and fixed corrugations and probable causes of the latter are discussed.

As no published records of the skid resistance of unpaved roads could be found, measurements of the slipperiness of typical unpaved roads was carried out. The results indicated that at worst, dry unpaved roads are only slightly more slippery than average paved roads but are adequately safe. In the wet condition, unpaved roads become more slippery but apparently only unacceptably slippery when free water occurs on the surface.

Plasticity and grading are the main contributors to the performance of the wearing courses and the development of the specifications should concentrate on the use of these two parameters.

CHAPTER 5

ROUGHNESS PREDICTION

5.1 INTRODUCTION

The roughness of a road is the most important condition parameter affecting the vehicle operating cost of that road. The roughness is a reflection of both the surface type and properties and the maintenance standards, and has been shown to be a significant contributor to every vehicle operating cost except depreciation (Harral and Agarwal, 1975; Lu, 1985). Until the early 1970s no research had been done on roughness measurement of unpaved roads or the quantitative prediction of the roughness of unpaved roads. The main reason for this was probably the lack of suitable roughness measurement equipment.

With the development of simple, durable, response type measurement devices such as the British "Bump Integrator" (BI) and the locally constructed "Linear Displacement Integrator" (LDI) the measurement of road roughness became fairly simple and repeatable. The bump integrator is an inexpensive, simple mechanical device which is used at 32 km/h and calibrated against a beam and towed fifth wheel. The LDI on the other hand is a slightly more sophisticated electronic device which can be used at any speed (usually 80 km/h), but must be calibrated at that speed over a series of calibration sections (see Chapter 3.4.3). A comparison of the measurements of the two devices showed a good correlation but large discrepancies occurred in roads with corrugations because of the different speeds of testing (Paige-Green, 1986). The correlation between the Bump Integrator (BI) and Quartercar Index (QI) used in this study is that given by the World Bank (1985) as follows:

$$\text{BI roughness} = 630\ (\text{QI}/13)^{1,12} \qquad (5.1)$$

The first published work on roughness prediction for unpaved roads was that carried out in Kenya (Hodges et al, 1975). This project had the advantage of being in a third world country where normal maintenance is minimal. It was thus possible to include a number of sections with nil maintenance over the duration of the experiment.

This allowed some of the sections to carry in excess of 100 000 vehicles before grading although most of the sections were graded before 13 000 vehicles had passed over them. The factors which were considered to affect roughness were traffic, rainfall, gravel type and particle size distribution. The rainfall was subsequently shown to be an unimportant factor in Kenya.

Two models for the predicted roughness were obtained, one for coral gravels and another for lateritic, volcanic and quartzitic gravels as follows:

$$R = 6500 + 58.T - 1{,}02.T^2 + 0{,}017.T^3 \qquad (5.2)$$

for coral gravels ($r^2 = 0{,}61$)

$$R = 3250 + 84.T - 1{,}62.T^2 + 0{,}016.T^3 \qquad (5.3)$$

for lateritic, quartzitic and volcanic gravels ($r^2 = 0{,}94$)

where

R = mean roughness in mm/km at 30 km/h

T =cumulative traffic volume in both directions (in thousands of vehicles).

Other than the correlation coefficients (r = 0,78 and 0,97 respectively), no statistical information is provided for the regression equations. An examination of the results however indicates a strong concentration of values at traffic volumes below 13 000 vehicles and few values above 60 000 vehicles with an influential point above 100 000 vehicles for Model 5.3. This may account for the high r-squared value for Model 5.3 when the expected variability of roughness is considered. A disturbing aspect of these models for use outside Kenya is the absence of any climatic or material properties (other than material type), with the roughness being totally dependant on the traffic. Although these models may be valid in the long term for roads with very little maintenance, they are unlikely to apply to those roads with periodic maintenance where corrugations, erosion channels and stoniness have an important influence on the roughness.

The corrugations, erosion and stoniness are strongly material dependant and material properties should thus feature prominently in roughness prediction models.

It is interesting to note that only one of the sections (Hodges et al, 1975) deteriorated to a roughness level (15 000 mm/km) where vehicles created a new track next to the road. This equates with a Quartercar Index of about 220 counts/km which compares well with the local criteria for total unacceptability (QI = 200, see Chapter 7.3).

The Brazil study (Visser, 1981a) involved a programme of extensive roughness measurements which resulted in the following model for the change in the natural logarithmic value of roughness (LDQ) in terms of the Quartercar Index (QI in counts/km):

$$\begin{aligned} LDQ = D[&0{,}4314 - 0{,}1705.T2 + 0{,}001159.NC + 0{,}000895.NT - 0{,}000227.NT.G \\ &+ S(-0{,}1442 - 0{,}0198.G + 0{,}00621.SV - 0{,}0142.PI \\ &- 0{,}000617.NC)] \end{aligned} \tag{5.4}$$

where

D = number of days since last blading in hundreds

T2 = surfacing type dummy variable:

T2 = 1 if surfacing is clay

T2 = 0 if otherwise

NC = average daily car and pickup traffic in both directions

NT = average daily bus and truck traffic in both directions

G = absolute value of grade in per cent

S = season dummy variable:

S = 0 if dry season

S = 1 if wet season

SV = percentage of surfacing material passing the 0,075 mm sieve

PI = plasticity index of surfacing material (%)

This model had an r-squared value of 0,26 with a sample size of 8 276 and a standard error of 0,222. Thus, if a change in roughness of 100 QI units is predicted the actual change will be between 65 and 154 QI units with 95 percent confidence.

A second model incorporating the monthly rainfall instead of a season dummy variable (GEIPOT, 1981) was developed:

$$LDQ = D(0{,}376 - 0{,}191.T2 + 0{,}00032.NC + 0{,}001014.NT) + CP(-0{,}16 - 0{,}0354.G + 0{,}00883.SV - 0{,}0218.PI) \qquad (5.5)$$

where

CP = cumulative precipitation since the previous blading, in m

and the other variables are as defined for model (5.4).

This model has an r-squared value of 0,31, a standard error of 0,211 and used 8 276 observations. The wisdom of replacing the mean annual rainfall (variable in itself) in Model 5.4 with an even more variable parameter such as the cumulative precipitation since the previous blading (Model 5.5) in order to improve the r-squared value and standard error marginally is questionable. For prediction purposes, the latter parameter would generally be an estimate at best.

Both of these models predict the rate of change of roughness with time. This is useful for comparing various situations but cannot be used to give an indication of the absolute roughness at any time. For this purpose a datum which was defined as the roughness after blading was also predicted using the following model (Visser, 1981a):

$$LRA = (1{,}4035 - 0{,}0239.W - 0{,}0048.SV + 0{,}01694.PI + 0{,}6307.LRB + 0{,}1499.T1 + 0{,}3096.T2 + 0{,}00020.NT + 0{,}2056.BS - 0{,}01183.PI.BS) \qquad (5.6)$$

where

LRA = natural logarithm of roughness after blading

LRB = natural logarithm of roughness before blading

W = road width in metres

T1 = surfacing type dummy variable

T1 = 1 if surfacing type is quartzite

T1 = 0 if otherwise

BS = season during which blading occurred

BS = 0 if dry season

BS = 1 if wet season

and the other parameters are defined as before.

This model was based on 1 308 observations and had an r-squared value of 0,61 and standard error of 0,34. The 95 percent confidence interval for a predicted value of 100 would thus be 52 to 194.

Paterson (1985) subsequently re-worked the Brazilian data and obtained a "better" prediction model by reducing the tendency to overestimate the roughness at high levels and to underestimate at low levels of roughness. This model predicts the average roughness over the duration of a blading cycle (RGM) as follows:

$$\begin{aligned} \ln RGM = {} & 1{,}607 + 0{,}605.\ln RG_o + 0{,}174.T2 + D(0{,}0393.NC + 0{,}119.NT) \\ & + MP(0{,}370 - 0{,}069.G - 0{,}0567.PI + 0{,}00855.SV) \end{aligned} \tag{5.7}$$

where

RG_o = roughness after blading, at beginning of blading cycle, QI counts/km;

MP = average rainfall intensity during blading cycle (m/month),

and the other variables were defined previously.

This model has an r-squared value of 0,76 with a standard error of 0,239 and used 1 089 observations. An important aspect to be noted is that although the r-squared value is substantially higher than for Model 5.4 the root mean square error is a little larger indicating that the 95 percent confidence interval is somewhat larger (a predicted value of 100 counts/km would have a value of between 48 and 209 with 95 percent confidence). The number of samples in the latter analysis is also much smaller than for Model 5.4, decreasing the significance of the r-squared value (GEIPOT, 1980). The danger of using only the r-squared values for comparing different models is thus clearly confirmed.

The cubic (Hodges et al, 1975) and exponential (Visser, 1981a) models both lead to unrealistically high predictions of roughness for policies of infrequent blading unless restrained (World Bank, 1985). The model used in the Highway Design and Maintenance Manual (HDM3) (World Bank, 1985) constrains this tendency by decreasing the rate of roughness progression as roughness tends towards the maximum for a particular material. The result of this exercise was the following somewhat complex model:

$$QI(TG_2) = QIMAX_j - b[QIMAX_j - QI(TG_1)] \tag{5.8}$$

where

$QI(TG_1)$ = roughness at time TG_1, in QI

$QI(TG_2)$ = roughness at time TG_2, in QI

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

b = exp [c (TG_2 - TG_1)]; where $0 < b < 1$;

c = - 0,001 (0,461 + 0,174.ADL + 0,0114.ADH - 0,0287.ADT.MMP);

and

$QIMAX_j$ = max [279 - 421.(0,05 - MGD_j)² + 0,22.C - 9,93.RF.MMP; 150]; for section j

where

ADL = average daily light vehicle traffic (< 3 500 kg) in both directions, in vehicles/day

ADH = average daily heavy vehicle traffic (≥ 3 500 kg) in both directions, in vehicles/day

ADT = average daily vehicular traffic in both directions, in vehicles/day

MMP = mean monthly precipitation (m/month)

RF = average rise and fall of the road (m/km)

MGD_j = material gradation dust ratio defined as

$$MGD_j = \begin{cases} 1 & \text{if } P425_j = 0 \\ P075_j/P425_j & \text{if } P425_j > 0; \end{cases}$$

$P075_j$ = percent passing 0,075 mm sieve for section j

$P425_j$ = percent passing 0,425 mm sieve for section j.

This non-linear least squares regression had an r-squared value of 0,856 with a standard error of 19,8 QI units and used 1 044 observations. However, as there was no logarithmic transformation of the roughness measurements in this model, the variances are not homogeneous and the standard error should be viewed in this light. In reality, the best prediction of roughness occurs for the mean value of the independent variables used to obtain the model, becoming poorer the greater the independent variables differ from this mean (Draper and Smith, 1966). In order to constrain the prediction model at high roughness values, the maximum value of the roughness was artificially limited to a QI of 150 counts/km. This value appears to be too low for the average unpaved road in rural and developing areas in southern Africa, although it may be defined as an acceptable limit before blading is necessary.

5.2 RESULTS

During the project some 10 000 roughness measurements at various speeds were collected in the Transvaal and a further 1 500 in South West Africa. A summary of some of the statistics related to these results is shown in Table 5.1.

TABLE 5.1: STATISTICS OF ROUGHNESS MEASUREMENTS

Variable	Mean	Std. dev.	Min	Max
Number of observations*	7 005			
Roughness (QI counts/km)	77	31,2	12	329
Number of days per blading cycle	36	34,3	1	454
Number of vehicles per blading cycle	3 710	5 003	10	108 486

* - measured at 80 km/h or corrected to 80 km/h equivalent

The means and ranges of these results are comparable with those of the Kenya study, but the mean days and vehicles per blading cycle are much lower than those of the Brazil study (Visser, 1981a). The more frequent blading activity resulted in a mean roughness over the duration of the project, lower than that measured in Brazil. The Brazil study also incorporated a number of in situ or native materials (earth roads) which resulted in rougher roads.

5.3 ANALYSIS

The analysis of the data followed a similar procedure to the analysis of the Brazil data (Visser, 1981a). Each time a section was graded a new performance cycle was started. The results were analysed using the SAS programme (SAS, 1985) and used all the measurements at 80 km/h.

Where roughness could not be measured at 80 km/h, the measurements at 50 km/h were corrected to the 80 km/h equivalent using equation 3.5, and these were used.

At the outset of the analysis an Analysis of Variance of the experimental design factors (i.e. material type, climate and traffic) was carried out in order to confirm the assumptions made during the experimental design. An analysis of variance using the mean, minimum and maximum roughness measured for each section was carried out. The highest r-squared value obtained was 0,52 (significant at the 0,05 percent level) with average daily traffic and the interaction between the material group and N-value the most significant parameters. It was thus concluded that other factors such as seasonal rainfall effects, measurement variability, blading effectiveness and stoniness were probably important contributors to roughness. Some multi-collinearity between the traffic and climate was also present.

The mean rate of change of roughness for each blading and seasonal cycle was calculated from the maximum and minimum roughness in the cycle and the mean rate of roughness change for each section was determined. This value was used in a multiple correlation analysis in order to identify the significant variables affecting the roughness. These were the N-value (N), average daily traffic (ADT), grading modulus (GM), plastic factor (PF), vertical grade (G), aggregate pliers value (APV), plastic limit (PL), percentage passing the 26,5 and 0,075 mm sieves (P26 and P75 respectively) and a dummy variable for the season (S1 =1 for the dry season and 0 for the wet season). The time factor was obviously of extreme importance and was included in the analysis. A regression analysis with the natural logarithm of the mean of the roughness in both directions as the dependent variable and two- and three-way interactions of time with all the above factors (66 degrees of freedom for the model) produced an r-squared value of only 0,33 with a standard error of 0,16 for 2 687 observations. An important aspect of this is that the time factor alone accounted for 55 percent of the total r-squared value.

The best-fit model which was obtained was Model 5.9:

$$LnR = D[-6,021 + 0,00056.ADT + 2,461.GM + 0,0002.PF + P26(0,058 - 0,023.GM) + 0,0005.N.APV] \quad (5.9)$$

(r^2 = 0,25; n = 2 687; RMSE = 0,17; F-value = 125)

A second analysis using the roughness measurements in both directions was carried out and at the same time the variable APV was excluded as it could not be determined for a number of the sections. This resulted in the following model (Model 5.10), with the regression coefficients shown in Table 5.2:

$$LnR = D[-8,3 + 0,0003.PF + 0,07.S1 + 0,081.P26 + 0,0003.N.ADT + GM(3,63 - 0,035.P26)] \quad (5.10)$$

(r^2 = 0,23; n = 7 004; RMSE = 0,15; F-value = 302)

TABLE 5.2: REGRESSION COEFFICIENTS OF CHANGE OF ROUGHNESS WITH TIME (MODEL 5.10)

Parameter	Estimate	Standard deviation	t-value
D	-8,30709	0,48467	-17,14
D*PF	0,00027	0,00003	8,20
D*S1	0,06975	0,00936	7,45
D*P26	0,08118	0,00488	16,64
D*GM	3,62967	0,22463	16,16
D*N*ADT	0,00032	0,00003	12,55
D*P26*GM	-0,03507	0,00230	-15,27

If a value of 100 counts/km is predicted for the change in roughness the actual value will, with 95 percent confidence, lie between 74 and 134 counts/km.

The roughness results were re-analysed with the modified values for the particle size distribution as described by Paige-Green (1988b) in order to eliminate any variation caused by the interpretation of the grading analyses. This resulted in Model 5.11 with the t-values shown in Table 5.3.

$$\text{LnR} = \text{D}[-13{,}8 + 0{,}00022.\text{PF} + 0{,}064.\text{S1} + 0{,}137.\text{P26} + 0{,}0003.\text{N.ADT} + \text{GM}(6{,}42 - 0{,}063.\text{P26})] \quad (5.11)$$

($r^2 = 0{,}22$; n = 7 005; RMSE = 0,15; F-value = 288)

TABLE 5.3: REGRESSION COEFFICIENTS OF CHANGE OF ROUGHNESS WITH TIME (MODEL 5.11)

Parameter	Estimate	Standard deviation	t-value
D	-13,81877	0,95189	-14,52
D*PF	0,00022	0,00003	6,86
D*S1	0,06351	0,00941	6,75
D*P26	0,13705	0,00964	14,22
D*GM	6,41945	0,46791	13,72
D*N*ADT	0,00034	0,00003	13,66
D*P26*GM	-0,06340	0,00477	-13,29

If a value of 100 counts/km is predicted for the change in roughness the actual value will, with 95 percent confidence, lie between 74 and 135 counts/km. Only a marginal change in the statistics and reliability of Model 5.11 is caused by the use of the corrected grading results.

In all of these models, as in the Brazilian models, the r-squared values are poor (only between twenty and thirty percent of the variation is accounted for by the models). Low r-squared values for the prediction of pavement distress are generally reported in the literature (Middleton and Mason, 1987). The HDM3 Manual identifies high prediction errors (95 percentile confidence intervals of 20 to 40 percent) as being typical of this type of study (World Bank, 1985) and ascribed them to the large variability of material properties, drainage, surface erosion and the high roughness levels of unpaved roads. A number of reasons for the poor r-squared values in this work are proposed:

1. Although the LDI vehicles were calibrated regularly a number of factors may have affected the readings during a days monitoring. These include tyre pressure variation (due to high road surface

temperatures, low surface temperatures during rainstorms, altitude variation or high tyre temperatures caused by extremely rough roads), minor variations in testing speed (an automatic speed control was used) and the decreasing mass of fuel during a days testing. As far as possible the speed and tyre pressures (which were checked cold every morning before testing) were kept constant and testing was not carried out during rainstorms.

2. The actual wheel-paths tested may vary considerably. On recently graded sections no definite wheel-paths are present, whereas on roads which have been left ungraded for extended periods the wheel-paths become wide and considerable variation in roughness may exist within the wheel-path tested. The average reading of three test runs was used in order to take this variation into account as far as possible.

3. When a road becomes rough, or potholes or corrugations form, the traffic starts to move away from the problem areas and a decrease in the measured roughness without any maintenance having been done is recorded. This cannot be quantified or taken into account in the analyses. Many sections start with three wheel paths after grading with a fourth appearing some time later as the road becomes rougher. This results in a sudden decrease in the measured roughness in one direction.

4. At the beginning of the wet season the road roughness usually decreases especially if the traffic is light. Corrugations are flattened out by the traffic, stones may be plucked from the road or pushed deeper into the road or the wheel-path may change to avoid water-filled ruts or muddy areas. Conversely, if the traffic is heavier and the bearing capacity of the wet material is low the roughness may increase through churning and rutting.

5. The normal analysis procedure for this type of work uses the traffic in both directions, although obvious differences in the roughness in the two directions often occur. As all the environmental and material factors for any section are identical only the volume and nature of the traffic can be expected to cause

this difference. Strong evidence for this was the periodic observation on some of the sections of trucks loaded with grain or gravel. These caused rapid deterioration of the road in one direction but on returning unladen in the other direction minimal deterioration was caused.

6. The influence of temporary periods of very high traffic (e.g. construction, harvesting, detours around important routes, farmers meetings, public holidays) also causes errors during analysis. During the traffic counts specifically carried out for this project two problems of this type were recorded. A local farmers "get-together" resulted in a traffic count of 192 vpd (27 % heavy) compared to a count of 37 vpd (35 % heavy) the next day on one of the experimental sections. Similarly a train derailment near one of the sections resulted in a count of 294 vpd (21 % heavy) compared with 164 vpd (20 % heavy) over the next two days.

7. During the wet season steeply cambered corners tend to erode at right angles to the direction of travel. This is caused by the flow of water down the slope of the corner and results in a corrugation-like effect with the amplitude increasing towards the bottom of the slope. At the same time, the drainage on steeply cambered corners is often such that the water accumulates at the foot of the camber. This often results in potholes and movement of the wheel tracks towards the outside of the corner.

8. It is commonly observed that the roughness soon after grader blading is much less than before blading but within a week or so it has increased dramatically. This can usually be attributed to the loose material which was bladed over large protruding stones, potholes and corrugations being whipped off by the traffic, and the roughness rapidly returning to what it was before grading. This is especially true during the dry season where no compaction occurs. The spurious results obtained cause large residuals in the regression models leading to low r-squared values.

9. Conversely, the roughness after blading was no better or even worse than before blading in about 30 percent of the cases

recorded (Figure 5.1). This could often be attributed to loose stones being bladed from the shoulder onto the road. However, within a few days these stones were generally moved to the side of the road by traffic, with a concomitant improvement in riding quality.

10. Although the sections were chosen to be as homogenous as possible, the roughness over the 300 m length of the section often varied considerably, while the average roughness for the whole section was used in the analyses. This variation in the Kenya experiment (one kilometre long sections) must have been extremely high.

The models of Visser (1981a) and Paterson (1985) both predict the rate of change of roughness. This can be used in comparing the proclivity of various materials to result in rough roads over time and should be minimised to identify the best of a range of materials. For use in a maintenance management model such as the Maintenance and Design System (MDS) a datum from which the roughness will increase is needed to predict the road performance. The datum used by Visser (1981a) and Paterson (1985) was the roughness after blading. The prediction equation for this (Model 5.6) requires a value of the roughness before blading, which in a maintenance management system can be assigned as the maximum acceptable roughness.

A model to predict the roughness after blading was developed for use locally in the MDS. It was seldom possible to measure the roughness immediately before or after blading. Thus, in many cases the road may have been bladed some days before or after the roughness was measured. The following model for the prediction of roughness after blading was the best obtained:

$$LRA = 1{,}07 + 0{,}699.LRB + 0{,}0004.ADT - 0{,}13.DR + 0{,}0019.LABMAX \qquad (5.12)$$

where

LRA = natural logarithm of roughness after blading
LRB = natural logarithm of roughness before blading

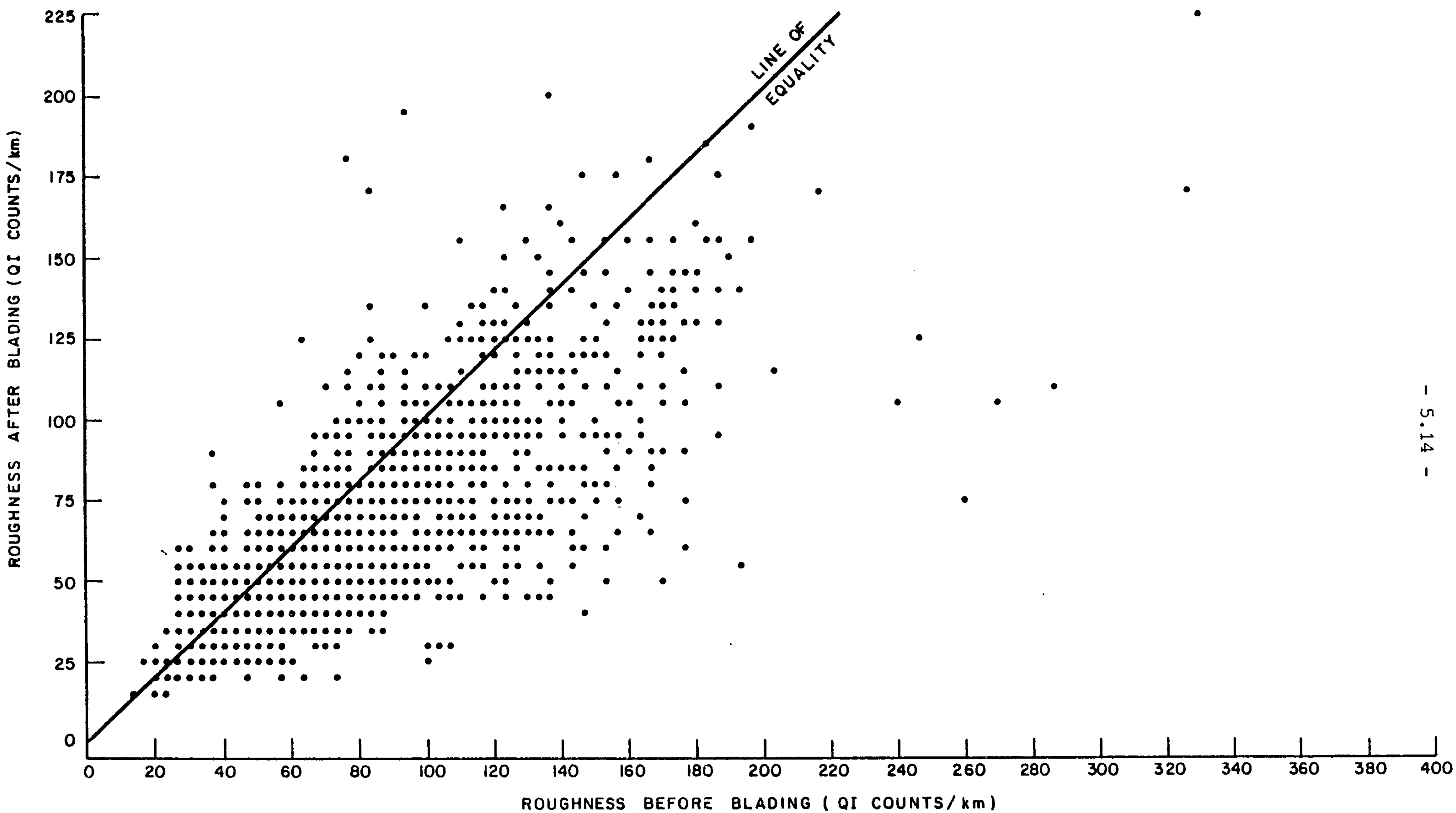


FIGURE 5.1

RELATIONSHIP BETWEEN ROUGHNESS BEFORE AND AFTER GRADER MAINTENANCE

ADT = average daily traffic

DR = dust ratio

LABMAX = laboratory determined maximum stone size (mm)

The r-squared value of this model was 0,62 with a standard error of 0,28 (i.e. if a value of 100 QI counts/km is predicted the actual value will lie between 56 and 179 counts/km. The model used 1 601 samples and is highly significant (F = 650) with all parameters significant at the 1 percent level. No change in the model occurred when the modified grading results (Paige-Green, 1988b) were used. A summary of the regression coefficients is given in Table 5.4.

TABLE 5.4: REGRESSION COEFFICIENTS OF MODEL FOR ROUGHNESS AFTER BLADING (MODEL 5.12)

Parameter	Estimate	Standard deviation	t-value
Intercept	1.07215	0,07156	14,98
LRB	0,69943	0,01521	45,99
ADT	0,00042	0,00009	4,51
DR	-0,13059	0,04884	-2,67
LABMAX	0,00187	0,00058	3,23

A marginal improvement in the r-squared value for the prediction of roughness after blading (Model 5.13) was obtained by incorporating dummy variables for the material group. Only the dummy variable (T2) for material group 2 (basic igneous rocks) was significant indicating that the basic igneous rocks tend to result in a slightly higher roughness than all the other material types. This is mainly due to the presence of large spheroidal boulders typical of weathered dolerites, andesites and basalts.

$$LRA = 0,974 + 0,704.LRB + 0,0004.ADT + 0,002.LABMAX + 0,089.T2 \quad (5.13)$$

$(r^2 = 0,62; \quad RMSE = 0,28; \quad n = 1600; \quad F = 659)$

Although the F value is slightly higher for Model 5.13 the inclusion of the material factor makes a negligible difference (one unit) to the overall prediction of the roughness after blading. Model 5.12 is therefore recommended for general use.

The roughness after blading depends primarily on the roughness before blading with LRB accounting for 98,8 percent of the total correlation. The statistics of both models are similar to those of Model 5.6, although the standard error is smaller. The main advantage of Model 5.12 over Model 5.6, however, is its simplicity with only four parameters against the ten of Model 5.6.

For optimum performance of the road, materials with a minimal rate of change of roughness and the minimum roughness which can be obtained by maintenance are the major factors affecting the choice of material. It would thus be useful to be able to predict this initial value. As most of the roads were bladed at various time frequencies and by a number of different grader operators during all seasons, it was considered that for each section the mode of the roughness after blading during the project, would be indicative of the expected roughness obtainable by routine blading for the section. Similarly, the mode of the roughness before blading was considered to be the potential maximum roughness of the section. This assumes that the sections were left until the maximum roughness for that material was reached, a valid assumption for most of the sections which were left unmaintained as long as possible.

An analysis of the roughness after blading, in terms of the mode, for each section indicated that the properties did not predict with any accuracy the potential roughness after blading, (or the potential minimum roughness which could be obtained by blading) for a particular material. The highest r-squared value obtained was 0,19 (for 110 observations). Only the particle size parameters (LABMAX, P75 and P26), plasticity index and percentage heavy vehicles showed significant correlations with the minimum roughness (significant at the 0,1 to 10 percent level). It would thus appear that the expertise of the grader operator and thus the quality of the blading operation is the most important aspect affecting the roughness after blading. A decrease in roughness after blading occurred in only 72 percent of the cases studied in South West Africa and 70 percent in the Transvaal. In the other cases the roughness was either not affected or increased (Figure 5.1).

The analysis of the maximum roughness produced similar results with a maximum r-squared value of 0,14. The only significant variables (at the 2 to 10 percent level) were the stoniness, average daily traffic and plasticity index.

Interesting correlations were obtained when the minimum roughness of each section was taken as the 5th percentile and the maximum roughness as the 95th percentile of all the results obtained for each section. This eliminated those very low and very high results which may have been spurious (e.g. after rain, incorrect tyre pressures or testing speed etc.). The significant factors influencing the minimum roughness were primarily material properties (most of the particle size and plasticity parameters, the maximum size, the stoniness and the aggregate strength tests). The maximum roughness on the other hand depended mainly on the environmental factors (i.e. traffic, rainfall, grade, curvature and compaction). The correlations were generally significant at a level of between one and ten percent but the r-squared values were still below 0,20. An inspection of the 95th percentile values indicated that the high values were obtained from those roads which were very stony and those prone to potholes and corrugations, while the low values were for the finer grained materials. The sections with the highest minimum roughness (5th percentiles) were generally those with large stones. Those sections constructed of finer grained material which corrugate and pothole and have a high 95th percentile roughness, rate significantly better in terms of the minimum roughness i.e. a substantial decrease in roughness can be obtained by blading.

5.4 DISCUSSION

The models developed for the prediction of the rate of roughness development with time and the roughness after blading are generally statistically comparable with the existing models developed elsewhere, but are easier to use and require less specialist input. No significant models for the prediction of maximum and minimum roughness could be developed, probably due to the unexplained factors which cause potholes, corrugations and erosion channels (see Chapter 4). Even including all the important material properties identified in Chapter 4 resulted in too many variables to analyse properly.

Model 5.11 indicates that the rate of roughness progression is mainly dependent on time which accounts for more than 72 percent of the explained variation. As the grading modulus, plastic factor, percent passing the 26,5 mm sieve, the Weinert N-value and daily traffic increase the rate of roughness deterioration becomes higher. The rate of roughness deterioration is greater during the dry season (S1 = 1) which is in agreement with the findings in Brazil (Visser, 1981a). This can be attributed to the compaction of stones and smoothing effect of vehicles on corrugations during the wet season, and possibly the change of wheel path to avoid water-filled potholes. It is anticipated that heavy traffic in the wet season could result in churning of weaker materials and increased roughness, but the existing situation indicates that most of the heavy traffic seems to stay off the road during periods of wet weather. The limited heavy traffic that does use the road in wet weather probably causes compaction and smoothing. In the dry season ravelling results in loose material and corrugations. The interaction between the grading modulus and fraction finer than 26,5 mm indicates that as the material becomes coarser, the influence of the grading becomes increasingly more important, and the roughness increases rapidly.

The absence of the vertical grade in the model is surprising. Although it was a significant variable, it was excluded during the modelling with no significant effect on the overall model. In Models 5.4 and 5.8, however, the grade was important as the roughness deterioration was generally found to be less on grades. Figure 5.2 shows the roughness plots of adjacent flat and inclined sections for two sections from this investigation, indicating no significant differences in roughness (i.e. the grade is of minimal importance in the overall development of roughness). One reason for this is that the stoniness, which is such an important factor in the roughness does not vary between grades and flat areas. Other factors influencing roughness such as potholes and corrugations are generally minimal on grades, but would appear to be of subordinate importance in relation

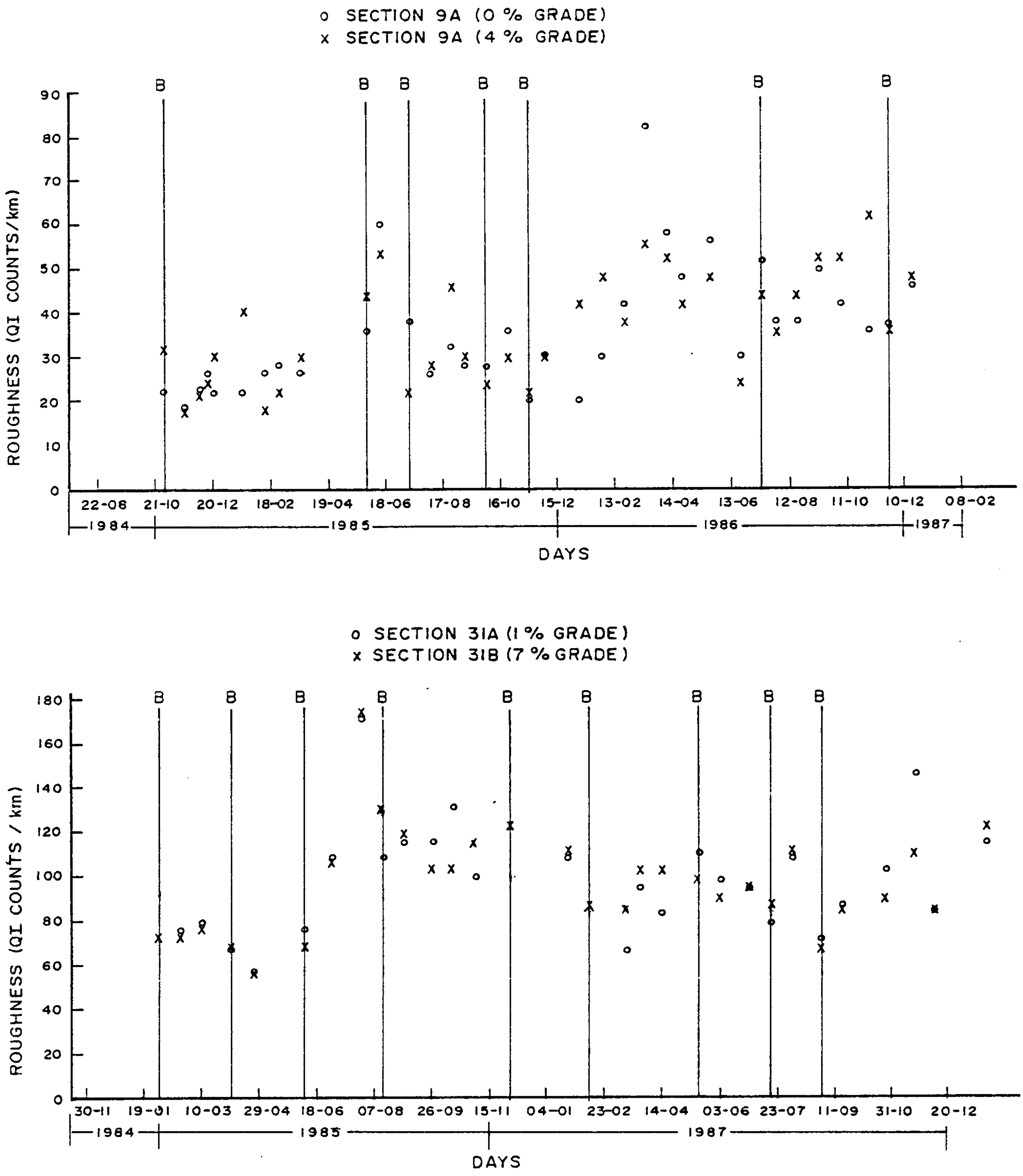


FIGURE 5.2

EFFECT OF VERTICAL GRADE ON MEASURED ROUGHNESS to the stoniness. Beaven et al (1987) noted that the gradient and curvature had little effect on the performance of the materials studied in Ethiopia.

The seasonal effect in Model 5.4 interacts with the vertical grade, material properties and traffic. This effect in Model 5.11 is independent of these parameters and the seasonal effect is a general increase in roughness in the dry season. This increase is probably mainly due to the formation of loose material and corrugations during the dry season.

An analysis incorporating dummy variables for the material groups resulted in a marginal improvement in the r-squared value (0,24) over Model 5.11 with no improvement in the standard error and a 44 percent reduction in the F-statistic (F = 170). No benefit was gained by incorporating the material type in the Model and a necessity for some specialist input would arise by its inclusion.

The incorporation of the rated severity of the stoniness for each roughness measurement decreased the standard error marginally although there was no significant change in the r-squared value. The stoniness severity is, however, not a parameter which can be determined prior to construction and varies with time. Generally, the roads with high stoniness were indicated by higher percentages retained on the 26,5 mm sieve and are thus incorporated in the model.

Table 5.5 shows a matrix of the predicted roughness for the extreme values of the variables in Model 5.11 as measured during the experiment. The values in each cell are for the predicted change in the roughness as a percentage of the initial roughness (i.e. the roughness after blading). Some of the combinations are hypothetical and unlikely to occur in nature (e.g. 70 per cent passing 26,5 mm sieve with a grading modulus of 0,32) and were not calculated. The predicted change in roughness in some cases (e.g. very light traffic with a low plastic/fine fraction) shows a small negative value and can be considered outside the inference space of the model. Similarly, the predictions for roads constructed with coarse gravels in arid areas and carrying very high traffic, indicate very high roughness values.

TABLE 5.5: PERCENTAGE CHANGE IN ROUGHNESS WITH TIME GENERATED FROM MODEL 5.11

			DAYS	10								100							
			PF	104				2100				104				2100			
			S1	0		1		0		1		0		1		0		1	
			P26	70	100	70	100	70	100	70	100	70	100	70	100	70	100	70	100
N	ADT	GM																	
10	395	2,5		24	16	24	17	29	21	30	22	728	335	782	363	1184	575	1268	619
		0,32		X	14	X	15	X	19	X	20	X	266	X	290	X	468	X	505
	10	2,5		8	2	9	2	13	6	14	7	119	15	134	23	240	79	263	91
		0,32		X	0	X	0	X	4	X	5	X	3	X	3	X	50	X	60
1,3	395	2,5		10	3	10	4	15	8	15	8	153	33	170	42	292	106	318	120
		0,32		X	1	X	2	X	6	X	6	X	12	X	19	X	73	X	85
	10	2,5		8	1	9	2	13	6	13	6	113	12	127	19	230	74	252	85
		0,32		X	-1	X	0	X	4	X	5	X	-1	X	0	X	46	X	56

The probability of this type of gravel having such high plastic factors is, however, very low. Other than these discrepancies the predicted values appear realistic.

Plots of the predicted values for some of the sections are shown in Figures 5.3 - 5.5. These plots show both the actual values recorded and the predicted values, using the roughness after blading as a starting datum. In some cases the predicted roughness has been plotted through the mean roughness of the blading cycle as well. The general tailing off of the roughness at higher values is shown by most of the sections, while the inherent variability of the roughness measurements is well exhibited especially at higher roughness levels.

From Table 5.5 and Figures 5.3 - 5.5, the importance of good grader blading is shown. The lower the roughness achieved after blading, the longer the period before blading is required again.

As the Kenya study and most of the other studies have been carried out by the TRRL, a TRRL Bump Integrator was used to measure the roughness. The measurements were taken at a speed of 32 km/h and thus the output from this instrument cannot readily be used to compare the models. Model 3.1 has, however, been used to convert the TRRL Models to QI outputs assuming that the relationships between the QI determined in Brazil and South Africa are comparable. As they are based on similar rod and level survey techniques this assumption should be valid.

The Quartercar Index was used in Brazil and the material test results are available (Visser and Queiroz, 1979) enabling the model developed locally to be compared for two sections studied in Brazil (Visser, 1981a) (Figure 5.6 and 5.7). The local model underpredicts the roughness badly on sections which were regularly maintained but overpredicts the roughness for sections with minimal maintenance. The reason for this appears to be that both of these sections are outside the inference space of the local study i.e. the traffic on section 251 was almost double the highest used to develop Model 5.11, while the time elapsed without blading for section 205 was almost 2 years. When the Brazil model (5.4) is applied to the local results a very strong tendency to overestimate the roughness for those sections with minimal

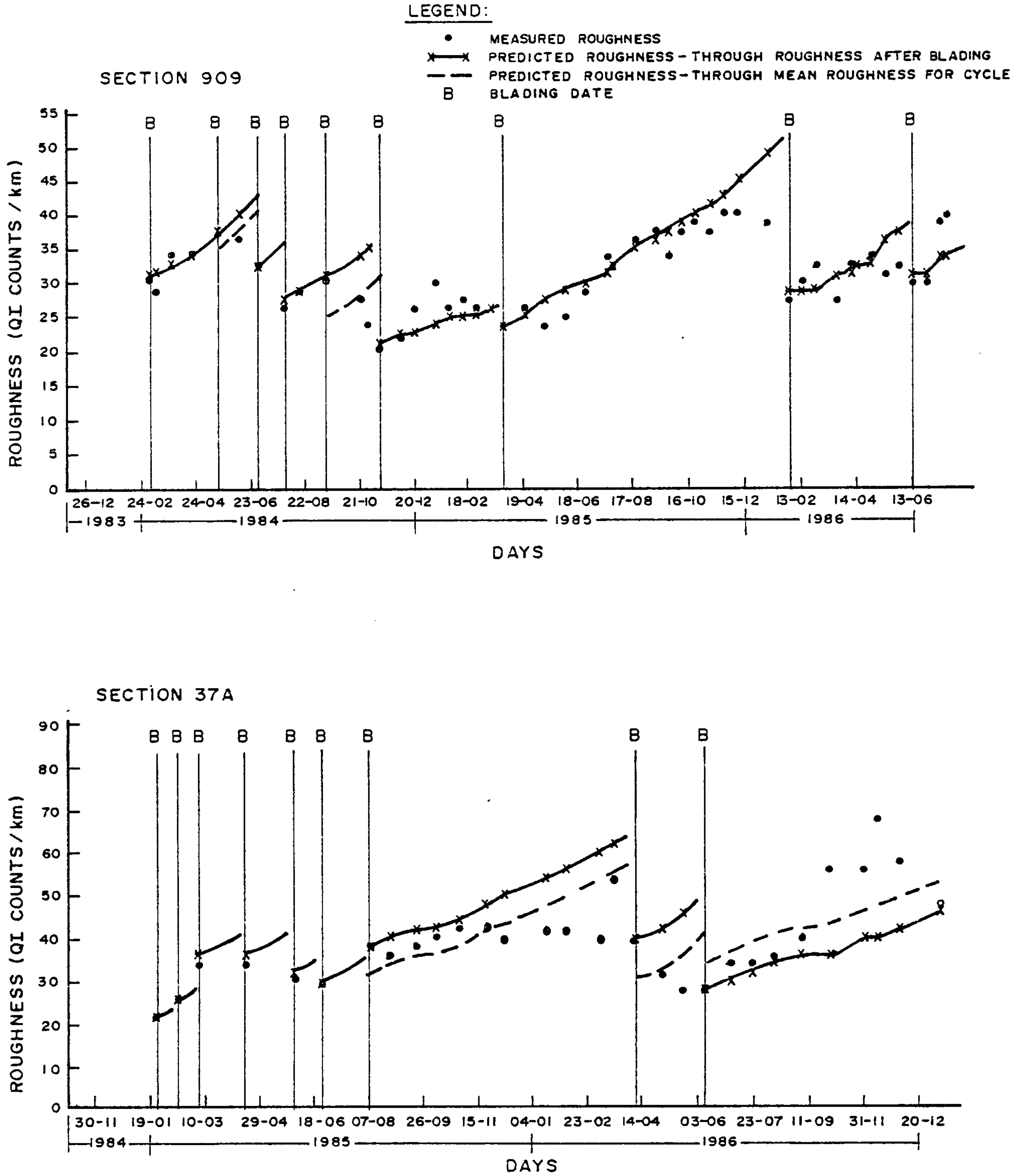


FIGURE 5.3

MEASURED AND PREDICTED ROUGHNESS FOR GOOD ROADS (QI< 80 COUNTS/km)

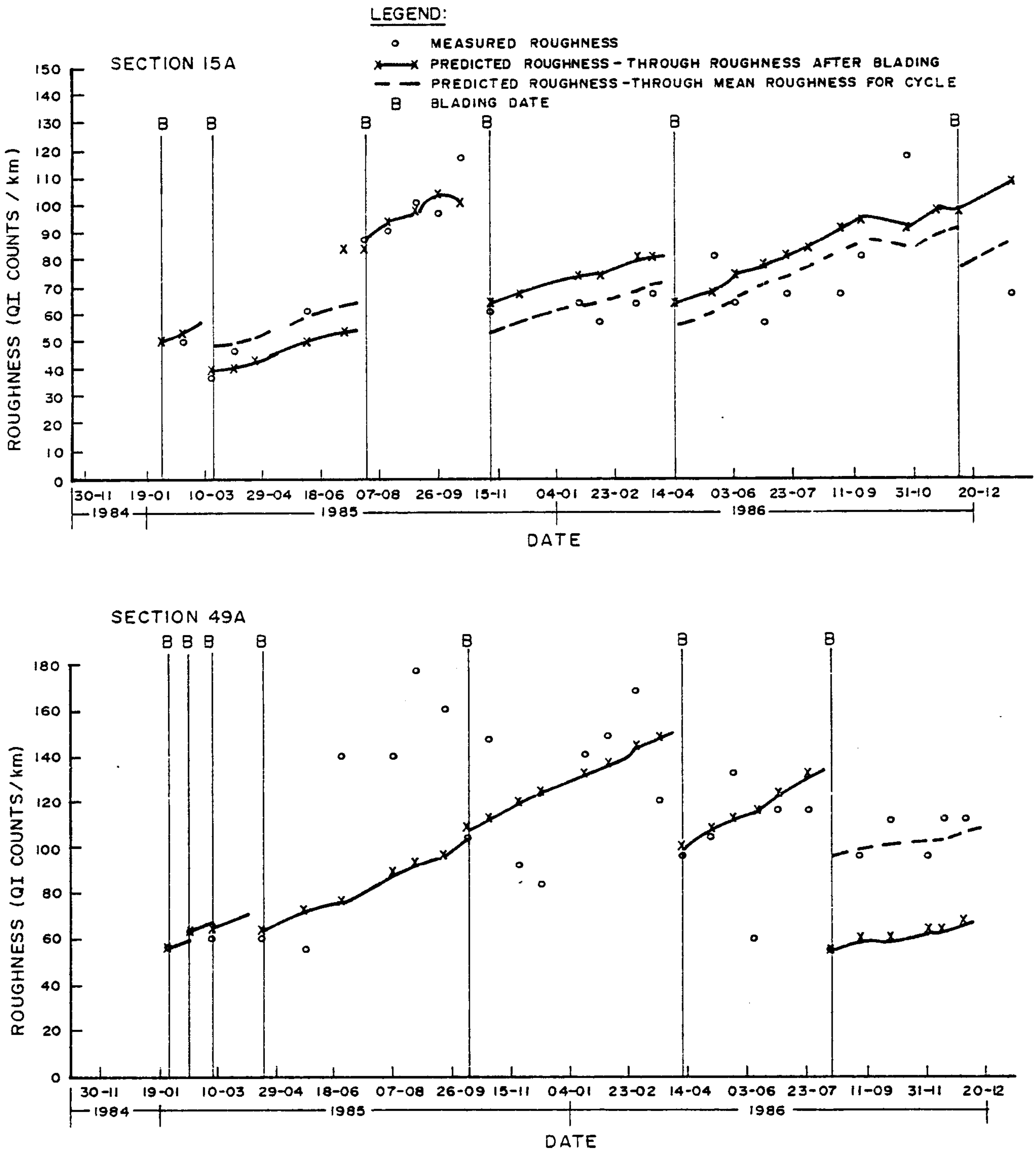
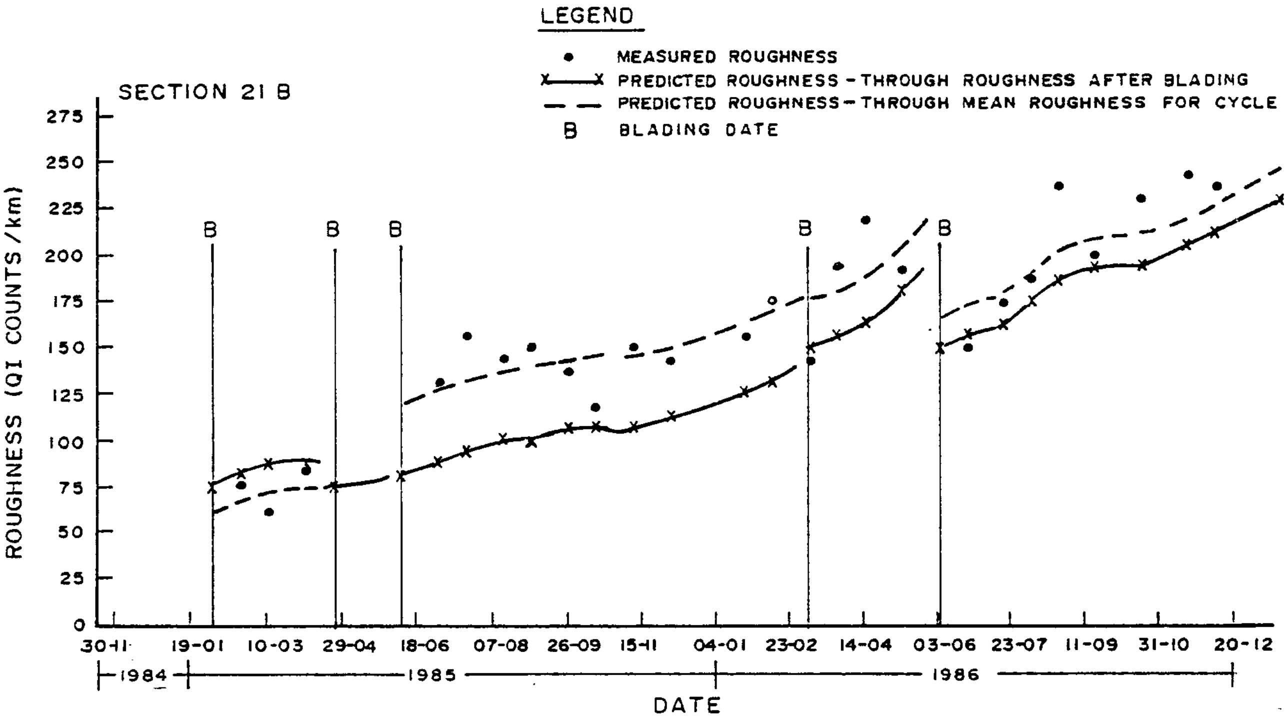


FIGURE 5.4

MEASURED AND PREDICTED ROUGHNESS FOR AVERAGE ROADS (QI ± 100-150 COUNTS/km)



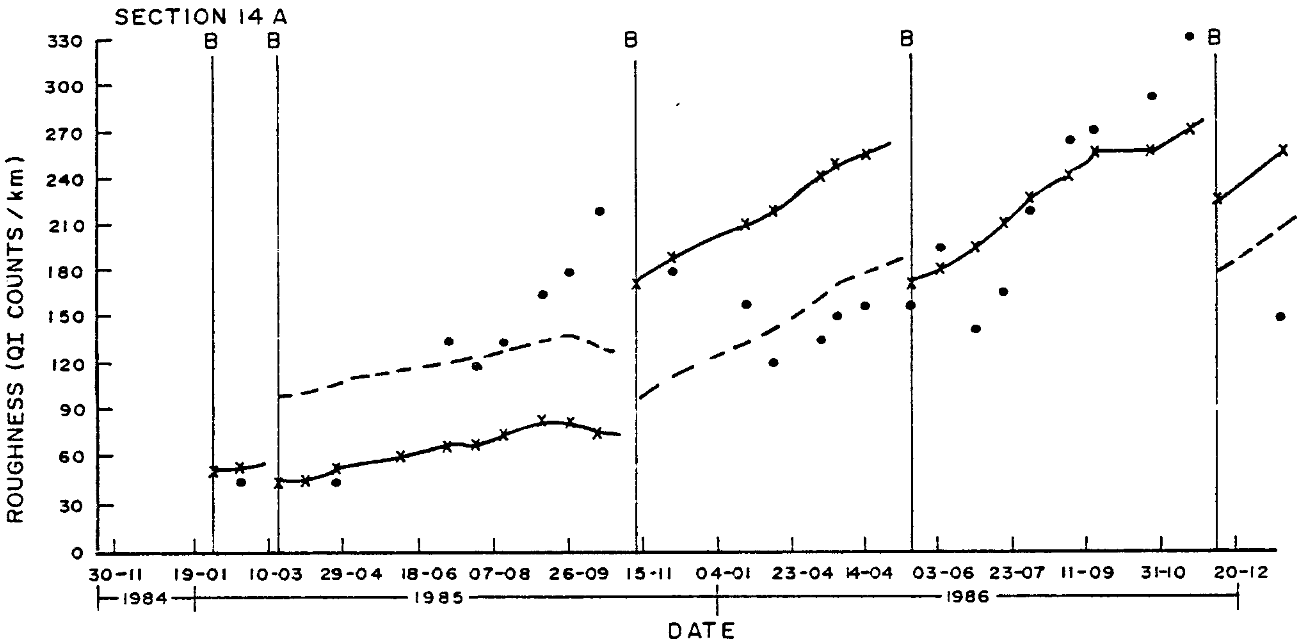


FIGURE 5.5

MEASURED AND PREDICTED ROUGHNESS FOR BAD ROADS

(QI ± 120-300 COUNTS/km)

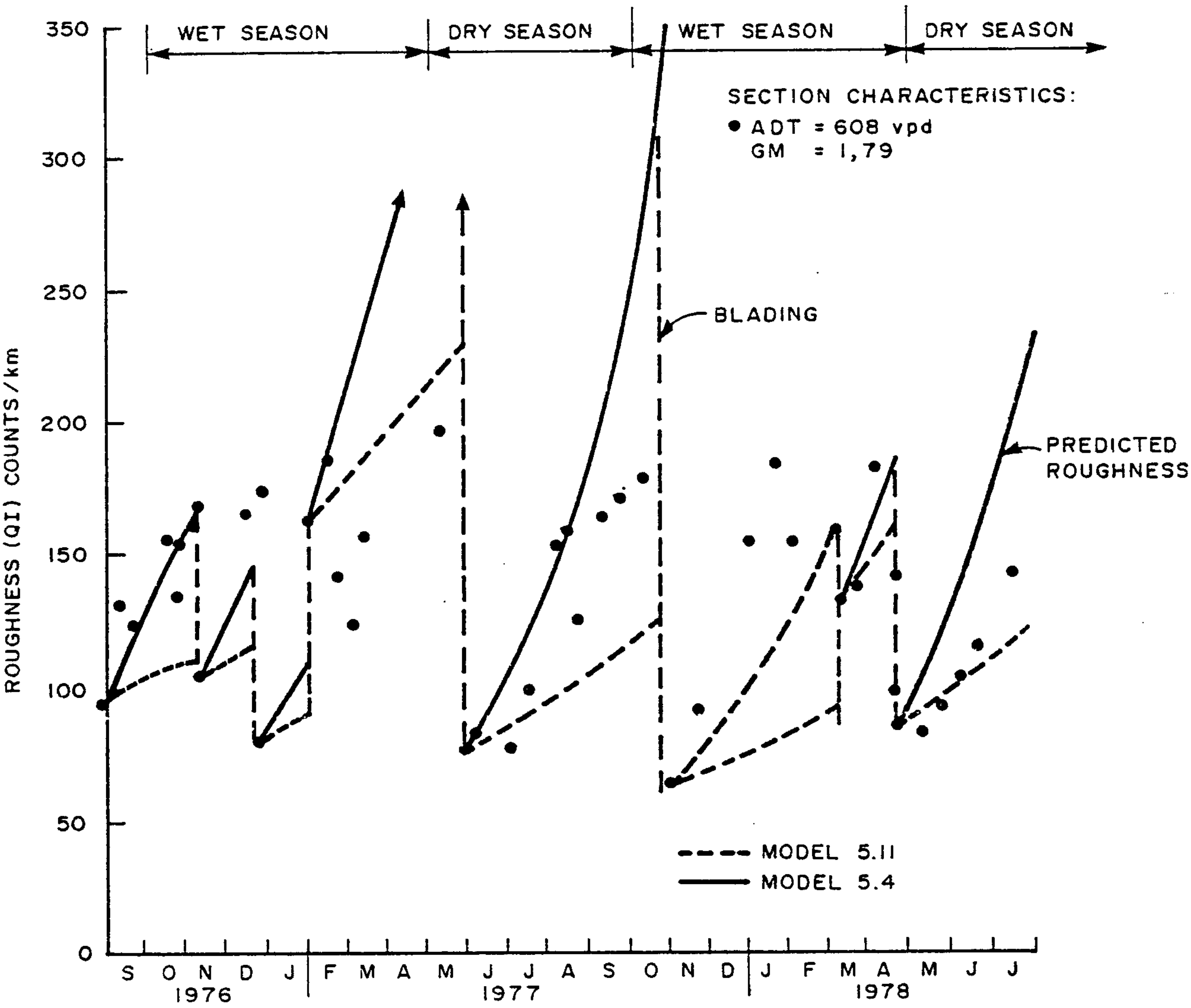


FIGURE 5.6

MEASURED AND PREDICTED ROUGHNESS ON SECTION 251 UNDER FREQUENT MAINTENANCE (After VISSER 1981a)

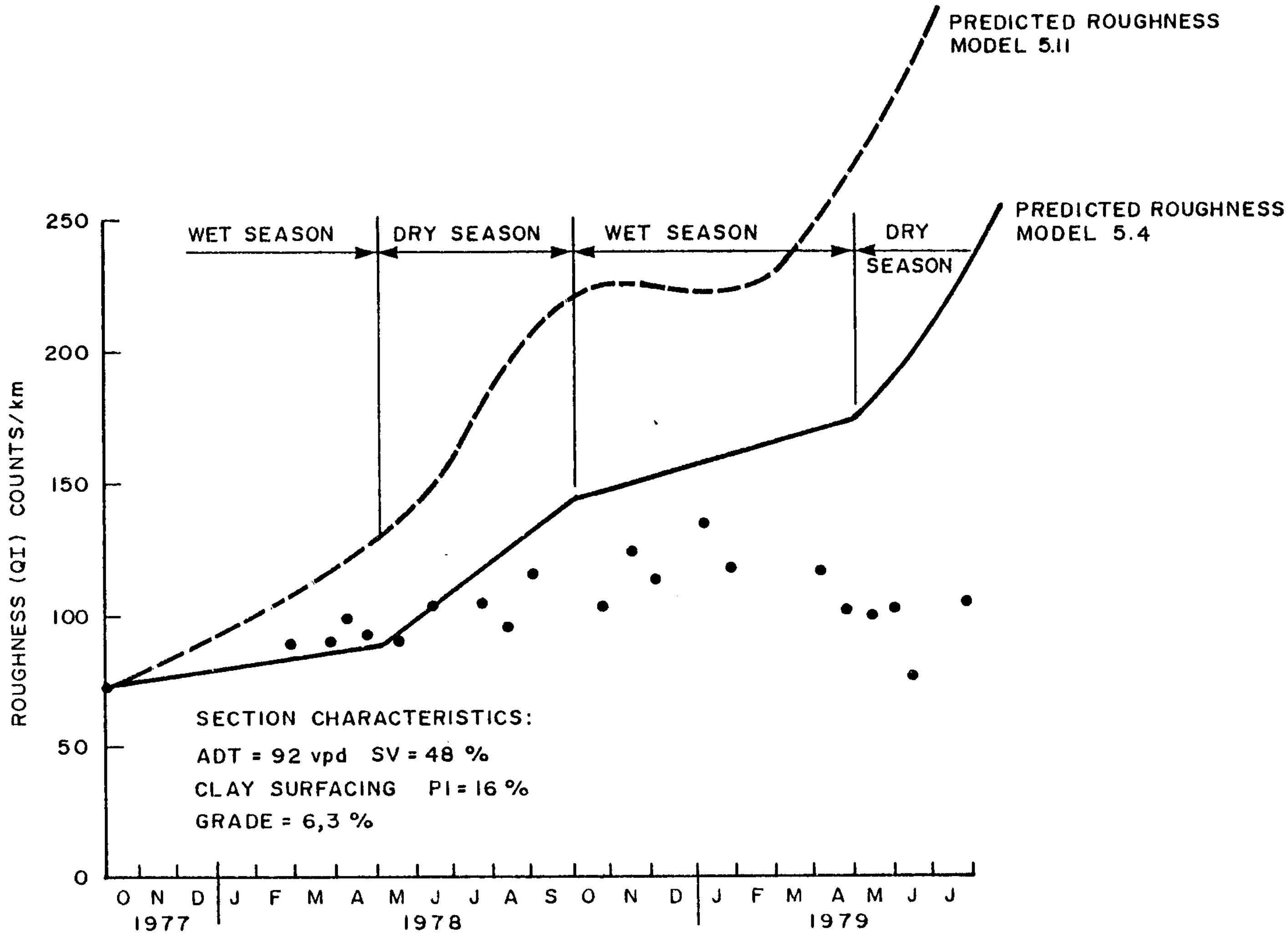


FIGURE 5.7

MEASURED AND PREDICTED ROUGHNESS ON SECTION 205 WHICH WAS NOT BLADED DURING THE OBSERVATION PERIOD (After VISSER 1981a) blading (less than twice a year) exists (Figure 5.8). The inference space of the Brazil study in this case would appear to be exceeded.

The predictive capabilities of the available models for local conditions were compared by analysing the mean roughness predicted by each of the models (Table 5.6) and the statistics of the residuals for the models (Table 5.7). Over 7 000 results were used in the analysis.

TABLE 5.6: COMPARISON OF PREDICTIVE CAPABILITIES OF VARIOUS MODELS

Model	Mean	Standard deviation	Standard error of mean	Percentage error
5.3	64,0	12,24	0,145	-20,1
5.4	96,1	87,83	1,044	20,1
5.8	66,2	30,79	0,366	-17,3
5.11	77,4	38,28	0,455	-3,3
Measured	80,0	38,49	0,457	

The Kenya and World bank models (5.3 and 5.8) underpredict the average roughness quite badly while the Brazil model (5.4) shows a significant overprediction. The local model (5.11) underpredicts by about three percent.

TABLE 5.7: STATISTICS OF RESIDUALS OF AVAILABLE PREDICTION MODELS

Model	Mean	Standard deviation	Standard error of mean
5.3	-16,0	38,21	0,454
5.4	16,0	77,30	0,912
5.8	-13,9	26,71	0,317
5.11	-2,7	26,52	0,315

The mean residuals of the Kenya, Brazil and World Bank models differ significantly from zero while that for the local model differs slightly from the optimum solution. This indicates that the models developed elsewhere are not totally adequate for southern African conditions and have been bettered by the new model. The bias towards underprediction is shown by the means (Table 5.6) and the minima and maxima (Table 5.8).

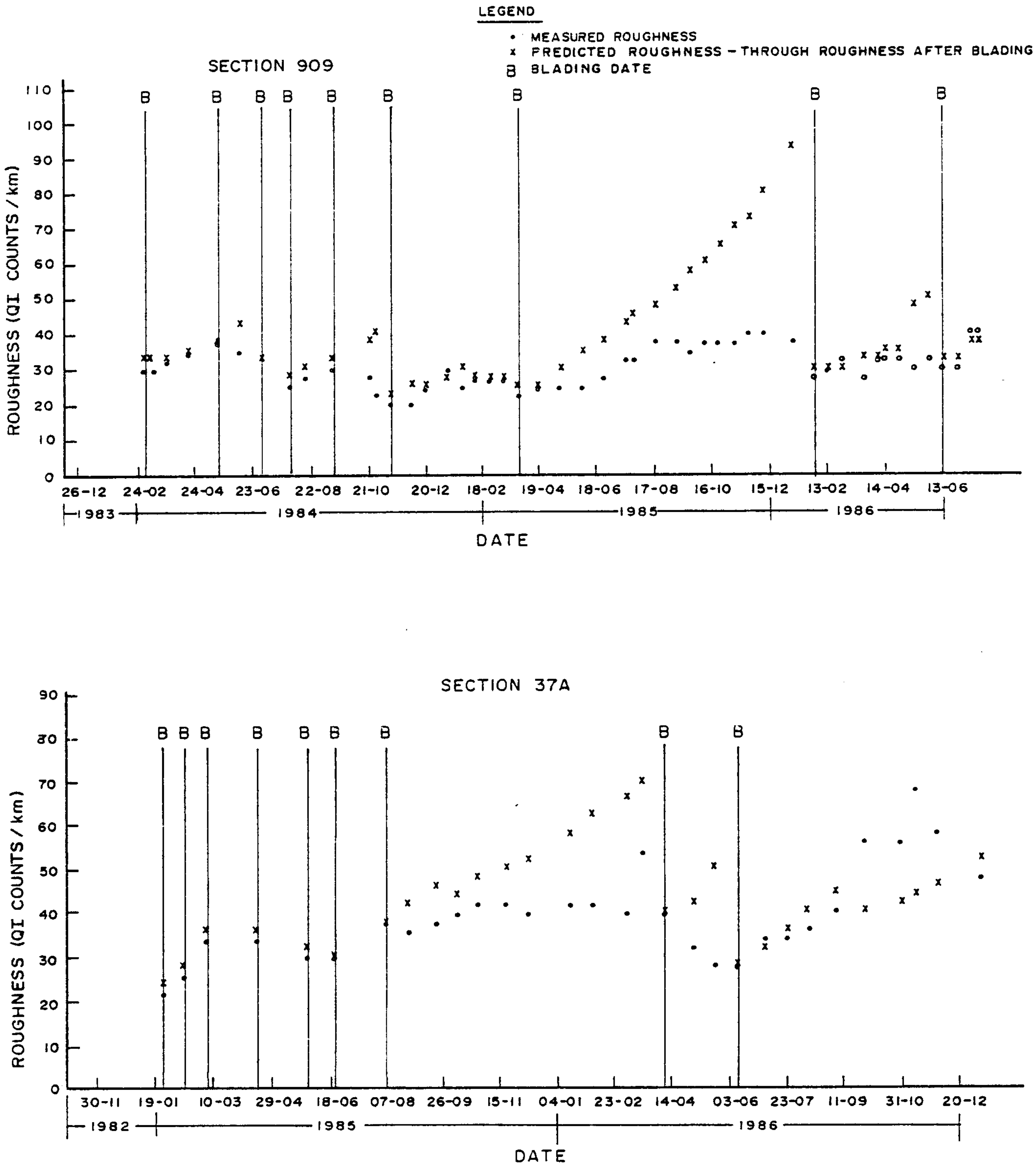


FIGURE 5.8

MEASURED AND PREDICTED ROUGHNESS (BRAZIL MODEL)

TABLE 5.8: MAXIMUM AND MINIMUM PREDICTIONS AND RESIDUALS

Model	Predicted minimum	Predicted maximum	Residual minimum	Residual maximum
5.3	56,3	294,9	-244,5	182,6
5.4	12,6	2425,9	-196,3	2294,5
5.8	12,1	229,5	-240,1	122,2
5.11	12,3	296,1	-230,1	203,4

The measured minimum and maximum roughness values were 12,0 and 329,1 respectively. All the models apart from 5.3 predict the minimum roughness very closely. The Kenya model is based on cumulative traffic and it can thus be expected that at low roughness values (i.e. shortly after grading when the cumulative traffic counts are still low) the error would be exaggerated. The extreme overprediction by the Brazil model is evident but the constraining effect of the World Bank model appears to be too limiting for local conditions. The Kenya and local models predict the maximum fairly closely, but it is recommended that a maximum of this order should not be permitted in practice. For a minimum recommended number of bladings of two per year (Visser, 1981a) the Brazil model would be relevant to southern African conditions.

Paterson (1985) showed that the roughness after blading is generally about 80 percent of the roughness before blading. The model developed in this study shows an increase in the effectiveness of blading as the roughness increases. If the roughness before blading is low (± 50 counts/km) the predicted roughness after blading shows an improvement of about 16 percent, while at high roughness values (about 200 counts/km) the predicted improvement is about 45 percent. Blading seldom returned the roughness to an approximately constant value for any section which concurs with the findings in Brazil (Visser, 1981a; Paterson, 1985).

5.5 CONCLUSIONS

A number of models for the prediction of roughness progression in unpaved roads have been developed overseas. These models are all fairly complicated, necessitating the determination of a number of geotechnical properties, an identification of the material types and the incorporation of an estimate of the average vertical grade for the total length of the link. When one considers that these models have been developed for use mainly in developing areas where computing facilities are often rudimentary and the skill levels of the roads personnel may be low, the usefulness of the models is questionable, especially in remoter areas.

The models developed in this study have eliminated the necessity of identifying the material type and of estimating the average grade for the road link. Simple indicator tests requiring minimal equipment and only basic operator training are required for the input parameters for the model. The predictive capability has, however, not been diminished through this process and, in fact, it has generally been improved for local conditions.

It can thus be concluded that the geotechnical properties, which are directly dependent on the geology in terms of the mineralogical composition, significantly affect the development of roughness of unpaved roads. Again, the particle size distribution and plasticity are the most important material properties.

CHAPTER 6

GRAVEL LOSS PREDICTION

6.1 INTRODUCTION

The prediction of the expected material loss of a gravel road is of utmost importance for both unpaved road design and maintenance planning as the gravelling and regravelling operations are the most costly construction and maintenance procedures. The loss of gravel is caused both by natural weathering (rain, wind, etc.) and by friction and whip-off from the vehicles. Timeous prediction of which roads are likely to need regravelling is important during network management and maintenance budgeting.

As far back as 1929, Strahan noted that an average gravel loss of 0,6 inches (15 mm) per year occurred. He concluded that up to 0,3 inches (7,6 mm) of this loss was due to "weather influences, grade conditions and machining", regardless of traffic density.

No further reference to gravel loss measurement is recorded until the Kenya study in 1975 (Hodges et al, 1975). The results of this study generally "did not extrapolate very satisfactorily" but the following best fit equation was given, although no statistics were included.

$$\text{Annual gravel loss} = f\left(\frac{Ta^2}{Ta^2+50}\right)(4{,}2 + 0{,}092.Ta + 3{,}5.R^2 + 1{,}88.V) \qquad (6.1)$$

where

f = 0,94 for lateritic gravel
    1,10 for quartzitic gravel
    0,70 for volcanic gravel
    1,50 for coral gravel

Ta = annual traffic volume in both directions (/ 1 000 vehicles)

R = annual rainfall (metres)

V = vertical curvature (%)

The material factor (f) depends on those properties of the material such as plasticity, particle size distribution, particle strength, clay mineralogy and probably in-situ density. The equation in this form indicates that the gravel loss of a road constructed with a volcanic gravel will be only 64 per cent of that of a similar road using a quartzitic gravel. No account is, however, taken of the variation of material properties within the two groups. This variation is probably inherent in the factors as the sections were each one kilometre long and, considering the natural variability of geological materials, it is unlikely that they were homogeneous over their entire length.

The Brazil study (Visser, 1981a) used the same basic gravel loss measurement procedure as the previous study, but reduced the number of readings per section. Instead of the measurements being taken every 2 m along the road they were taken every 5 m. A number of gravel loss prediction models were obtained the following model being the most useful:

$$GL = D(1,58 + 0,366.G + 0,083.SV - 0,210.PI + 0,0132.NC + 0,0081.NT + 420,45/R) \qquad (6.2)$$

where

- GL = gravel thickness loss in mm
- D = time period considered, in hundred days, i.e. days/100
- G = absolute value of grade in percent
- PI = plasticity index (PI)
- SV = percentage of the surfacing material passing the 0,075 mm sieve
- NC = average daily car and pick-up traffic, both directions
- NT = average daily truck traffic, both directions
- R = radius of horizontal curvature, in m

This model was based on 604 observations and had an r-squared value of 0,62 and standard error of 11,17.

Roberts (1980) conducted a series of gravel loss measurements in Ghana on a grid of 0,5 by 4,0 m. The main objective of this was to estimate the gravel loss, with a secondary objective of recording the change of shape of the road cross section. No analysis of the data is given but an average annual gravel loss of 9 mm was determined for both low and high traffic volumes. These two categories were not defined in terms of actual vehicles although high varied from 39 to 167 vehicles per day while low counts were in the range 15 to 74 vpd (Roberts, 1983). An analysis of the independent variables with respect to the performance indicated that the traffic played a major influence in the gravel loss (significant at the 1 per cent level) and the zone (climate, terrain and material type) was significant at the 5 per cent level. The vertical alignment and season showed no statistically significant relationship with the gravel loss.

Jones et al (1984a) carried out work in Kenya and compared the gravel loss on a number of roads with the loss predicted from the model developed during the earlier work in Kenya (Hodges et al, 1975). The gravel loss was generally underpredicted by about 37 per cent. The material factor in equation 6.1 was thus adjusted to account for this difference as follows (Table 6.1):

TABLE 6.1: ADJUSTED MATERIAL CONSTANTS (JONES, 1984a)

Material	Old value	New value
Lateritic gravels	0,94	1,29
Quartzitic gravels	1,10	1,51
Volcanic gravels	0,70	0,96
Coral gravels	1,50	-
Sandstone gravels	-	1,38

To investigate reasons for this difference 2 sections of road were constructed and left untrafficked and the eroding effect of rainfall was studied. The measured annual gravel loss on these roads (4,3 and 7,5 mm) was much higher than that predicted by the universal equation for rainfall-erosion losses (Agricultural Research Service, 1961) or that predicted by the original TRRL work (Jones et al, 1984). The variability of the material properties within any one material group would also influence the gravel loss. It is unlikely that discrete values for the properties in the material group would apply to a continuously variable process i.e. the plasticity and grading would probably vary considerably for different volcanic gravels depending on the conditions and stage of weathering.

Paterson (1985) used the data collected in Brazil to obtain the following gravel loss prediction equation:

$$MLA = 3,65\ (\ 3,46 + 0,246.MMP.G + KT.ADT) \qquad (6.3)$$

where

MLA = predicted annual material loss (mm/yr)
MMP = mean monthly precipitation (m)
G = grade (%)
ADT = average daily traffic in both directions
KT = max [0; (0,022 + 0,969/53700.KCRV + 0,00342.MMP.P75 - 0,0092.MMP.PI - 0,101.MMP)]
with KCRV = curvature (°/km)
and the other variables are as defined before.

More recently, Beaven et al (1987) carried out gravel loss measurements on weathered basalt roads in Ethiopia. However, the benchmarks were such that significant differential movement occurred and the results were not altogether satisfactory. The measured gravel losses were, however, considerably higher than those predicted by the model developed in Kenya (Model 6.1).

A study of the past work on gravel loss analyses indicates a number of differences between the predictions of the different models when used in areas in which they were not developed. It would thus appear to be important to develop models specifically for the materials and environmental conditions encountered in southern Africa.

6.2 RESULTS

During the project each section was surveyed 7 times at about 6 monthly intervals. As the gravel loss is a long term parameter (usually 5 years or more) the effect of seasonal loss is not important and biannual measurements were considered adequate. About 740 results were obtained altogether for the Transvaal and South West Africa sections.

Some of the sections showed small differential movement of the benchmarks during the project, and a few of the benchmarks were disturbed by the grader maintenance during the latter stages of the project. However, by using 3 benchmarks the displaced ones could be identified and one of the other two could be used as the standard in the analyses.

One problem which was encountered, especially when maintenance was delayed, was the narrowing of the trafficked part of the road. This resulted in the accumulation of loose material and even the growth of grass within the survey section. In these cases the gravel loss for the section was calculated on a reduced width i.e., without the affected readings for the full duration of the experiment.

A simple experiment in which four sections of road (two flat sections (A) and two on grades of 6 percent (B)) were levelled each day for three consecutive days was carried out in order to investigate the precision of the surveying method. The results of these surveys in terms of the mean height of the road above a standard bench mark and the standard deviation and range of the three measurements are shown in Table 6.2.

TABLE 6.2: PRECISION OF LEVELLING MEASUREMENTS (in mm)

Road no.	Mean height	Standard deviation	Range (mm)
2452 (A)	191,9	0,36	191,6 - 192,3
1342 (A)	-323,9	0,89	-322,9 - - 324,7
1342 (B)	-2710,4	3,75	-2706,2 - -2713,4
420 (B)	-1052,4	1,82	1050,0 - 1054,5

The repeatability of the survey measurements on flat sections is very good (within 1 mm) while on steep slopes and probably on steeply cambered corners, it is not as good, but still within acceptable limits. The section on road 1342 (B) was the first one laid out in the experiment and had the bench marks outside the survey section. All the other sections monitored included the bench marks in the section (e.g. 420 (B)) and a more repeatable survey was obtained as the grid was more accurately located. The standard deviation of two tangent and two curved sections surveyed in Brazil (Visser, 1981a) was 6,67 mm. Similar problems were encountered on hills in the Brazil study where a slight dislocation of the survey grid resulted in a "large" change in the average height as the average height changes but the bench marks do not. By including the bench marks in the survey grid this dislocation was minimised in the local study.

6.3 ANALYSIS

Initially the South West African data was analysed in terms of the existing prediction models developed in Kenya and Brazil and a preliminary model developed for the South West Africa data (Paige-Green, 1987b). The model was a simple best-fit linear regression using the same parameters as the Brazil model:

$$GL = D(-0{,}15.PI + 0{,}62.G + 0{,}04.ADT + 0{,}335.T7 - 0{,}87) \qquad (6.4)$$

where

ADT = Average daily traffic in both directions

T7 = 1 for calcareous mixtures and 0 for all other materials

and the other variables are defined for Model 6.2.

The predictive capability of this model for the combined Transvaal and South West Africa data was investigated and found to be inadequate. Although the mean predicted loss differed from the measured gravel loss by about 17 percent, the variance was very high (166,5) and the predicted maximum annual gravel loss was excessively high (60 mm).

The data for the Transvaal and South West Africa sections were thus combined for further analysis. The analysis was done using the Statistical Analysis System (SAS, 1985). The significant variables were initially identified using a full correlation matrix (SAS, 1985). The model was then developed using these significant variables in the

GLM (General Linear Models) programme for the maximum r-squared value and minimum Root Mean Square Error (RMSE).

The approach used for the analysis was to find the average height of the section relative to a standard benchmark each time the section was surveyed. In order to investigate the rate of change of the gravel height (i.e. the gravel loss) the intercept was removed by centralising the data for each section through the mean monitoring date (Draper and Smith, 1966). This procedure eliminates the problems described by Burger (undated) who showed that the r-squared value in a no-intercept model (i.e. zero intercept is forced) should be used with great care.

The significant variables used in the model were time (D), average daily traffic (ADT), N-value (N), the plastic factor (PF; product of plastic limit and percent passing 0,075 mm sieve), the percentage passing the 26,5 mm sieve (P26) and the vertical grade (G) in percent. All of these variables were significant at the 0,01 percent level but some multi-collinearity was obvious. Thus all of the second order interactions were analysed as well. The best fit equation was obtained from the following model:

$$GL = D\,[ADT(0{,}047 + 0{,}0027.N - 0{,}0005.P26) - 0{,}365.N - 0{,}0014.PF + 0{,}048.P26] \qquad (6.5)$$

(r^2 = 0,84; RMSE = 5,3; n = 702; F = 624)

where

GL = Gravel loss

D = Time period under consideration (days/100)

ADT = Average daily traffic

N = Weinert N-value

PF = Plastic limit x percent passing 200 mesh sieve (P75)

P26 = Percent passing the 26,5 mm sieve

The approximate 95 percent confidence intervals are gravel loss ± 10,6 mm. The t-values of the coefficients are given in Table 6.3.

TABLE 6.3: GRAVEL LOSS REGRESSION ANALYSIS (Model 6.5)

Parameter	Estimate	Standard Deviation	t-value
D*ADT	0,047	0,0091	5,13
D*N	-0,365	0,0375	-9,73
D*PF	-0,001	0,0003	-4,49
D*P26	0,048	0,0027	18,04
D*ADT*N	0,0027	0,0004	6,70
D*ADT*P26	-0,0005	0,0001	-4,98

In the eastern part of the study region the higher rainfall has resulted in the forestry industry, with a concomitant increase in traffic. This is also a more mountainous part of the country resulting in steeper grades on many of the roads. This has led to some multi-collinearity of some of the factors in the model which is unavoidable. The data was thus split at an N-value of 5 to remove some of this effect and two further models developed.

Where N < 5 (wetter eastern areas)

$$GL = D\,[ADT(0,136.N - 0,0255) - 1,7.N - 0,0073.PF + 0,077.P26 + 0,1564.P75] \qquad (6.6)$$

(r^2 = 0,88; RMSE = 5,5; n = 376; F = 440)

where the variables are defined as above.

Where N > 5 (drier western areas)

$$GL = D\,[0,222.ADT - 0,096.N + 0,031.P26 - 0,046.P75] \qquad (6.7)$$

(r^2 = 0,79; RMSE = 4,7; n = 325; F = 299)

where the variables are as defined for model (6.5).

When the influence of the oversize material on the interpretation of the grading analyses was identified towards the end of the project

(Paige-Green, 1988b) the data was re-analysed with the modified grading analyses. The following best-fit model was obtained (Model 6.8) for the full data set:

$$GL = D\,[ADT(0{,}059 + 0{,}0027.N - 0{,}0006.P26) - 0{,}367.N - 0{,}0014.PF + 0{,}0474.P26] \qquad (6.8)$$

(r^2 = 0,84; RMSE = 5,3; n = 703; F = 619)

No significant changes in the statistics of the model compared to Model 6.5 were evident although minor changes occurred in some of the coefficients. Consequently no further work was done to redevelop models to replace 6.6 and 6.7. The coefficients and the t-values for model 6.8 are shown in Table 6.4:

TABLE 6.4: GRAVEL LOSS REGRESSION ANALYSIS (Model 6.8)

Parameter	Estimate	Standard Deviation	t-value
D*ADT	0,0594	0,0159	3,72
D*N	-0,3671	0,0377	-9,72
D*PF	-0,0014	0,0003	-4,50
D*P26	0,0474	0,0027	17,84
D*ADT*N	0,0027	0,0004	6,64
D*ADT*P26	-0,0006	0,0002	-3,63

During the analyses the influence of the frequency of grader maintenance was found to be insignificant. Normal grader maintenance (graded about once a month) was carried out during the first half of the monitoring period (about 18 months) while maintenance was delayed for up to 6 months during the second half of the experiment. Comparison of the gravel losses predicted from model 6.8 with the measured gravel losses confirms this (Figure 6.1). The plot for section 30B shows excellent correlation over the duration of the project, while that for section 41A is one of the worst predictions. Section 41A however was extremely stony and the gravel loss was in fact much less than predicted due to the large exposed area of the stones. Grader maintenance resulted in the replacement of the matrix between the stones with uncompacted fine material from the shoulders

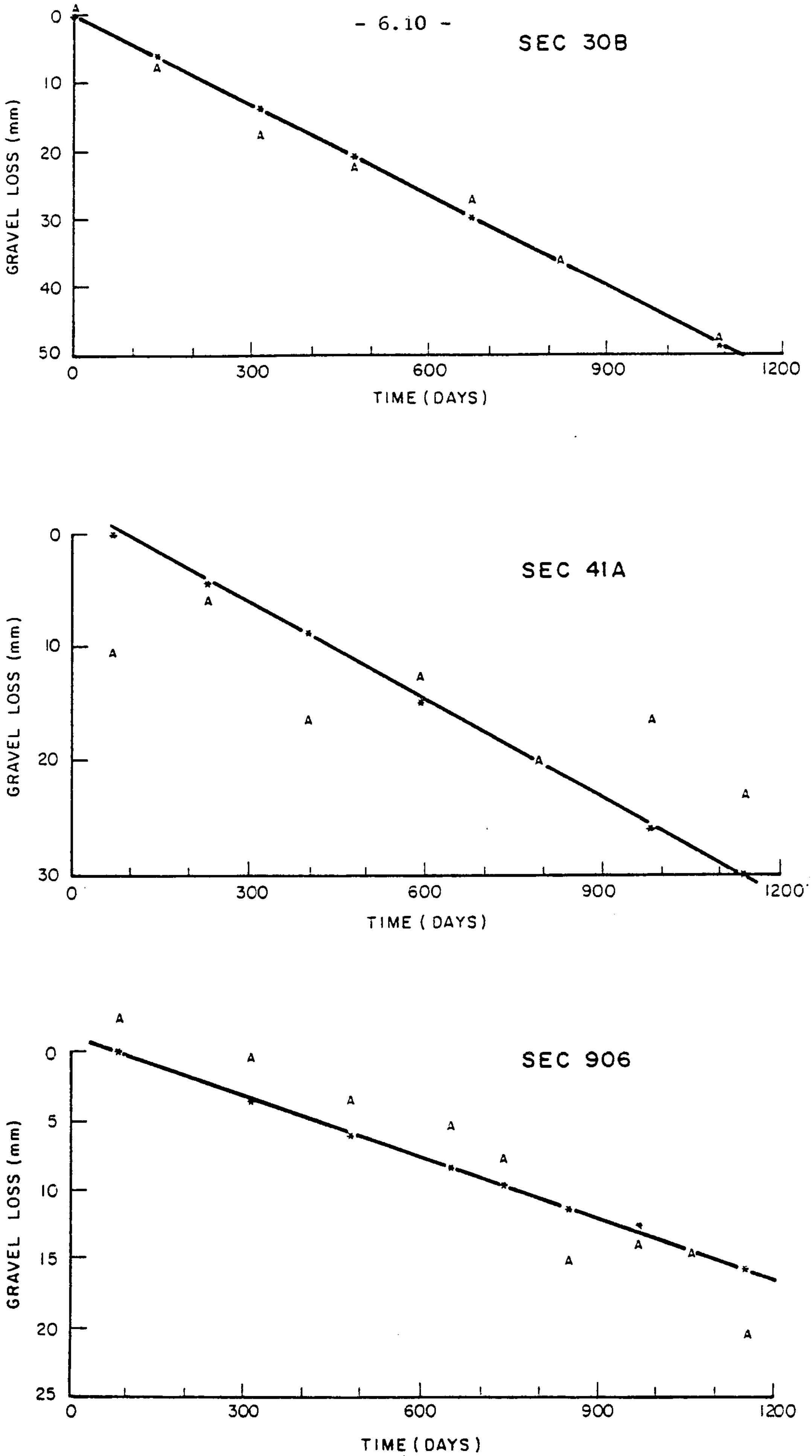


FIGURE 6.1

PLOTS OF ACTUAL (A) AND PREDICTED (*) GRAVEL LOSS and hence the variation in the measured gravel loss. The rate of gravel loss on section 906 on the other hand followed the prediction curve well for about two years before a large gravel loss occurred and then the loss returned to the predicted rate. The cause of the sudden gravel loss is not known but may be due to an unusually heavy rain storm, a period of heavy traffic or excessive cutting during grader maintenance.

A number of the level surveys taken 6 months after completion of the roughness monitoring showed very large gravel gains or gravel losses on some of the sections. These were generally those sections which became extremely rough during the latter part of the project and required heavy grading to restore the riding quality to acceptable levels.

6.4 DISCUSSION

Examination of the four models developed shows that they are generally similar. Although the grade was a significant variable (gravel loss on grades can be expected to be higher due to erosion) the multi-collinearity between this and the N-value resulted in it being excluded from the main model (6.8). The horizontal curvature was surprisingly not a significant variable in the drier western areas. It was also noted that the average daily traffic was more important than either the cars or the trucks individually. This may be due to the multi-collinearity between the percentage of heavy vehicles and the N-value (both associated with the forestry industry).

In all the models the time factor is the most important variable with the N-value also playing a major role. The gravel loss as expected is directly proportional to the time elapsed. This temporal factor is in turn clearly modified by a traffic and an environmental component. The traffic component depends on the number of vehicles per day with a higher loss as the N-value increases (less cohesion) but a smaller loss as the percent passing the 26,5 mm sieve increases (i.e. the traffic induced erosion is less as the coarse fraction decreases). The environmental component is composed of those factors affecting the erosion by wind or rain. As the N-value and the plastic factor increase the possibility of erosion is decreased while the coarser materials are less susceptible to erosion.

Similar relationships are found for the separate regions and for the different methods of carrying out the grading analyses.

It is suggested that Model 6.8 is taken for general use in South Africa as all the parameters necessary can be accurately and easily determined and the quicker grading analysis recommended by Paige-Green (1988b) can be used. All further discussion is therefore related to this model. The predicted versus actual measurements for three sections are plotted in Figure 6.1 and a matrix of the predicted gravel losses for the extreme values of ADT, N, P26 and PF used in the analysis is given (Table 6.5).

TABLE 6.5: GRAVEL LOSS PREDICTED BY MODEL 6.8 (in mm)

ADT (vpd)		10				395			
N		1,3		10		1,3		10	
P26 (%)		66	100	66	100	66	100	66	100
PF	2100	-0,1	5,1	-10,9	-5,7	32,1	8,7	54,3	30,9
	104	10,1	15,3	-0,7	4,5	42,3	18,9	64,5	41,1

The predictions of gravel loss under low traffic yield a number of negative losses (i.e. gains), especially for the high values of the plastic factor in the dry areas. It is, however, highly unlikely that the plastic factor of typical wearing course gravels would become this high in the drier areas where physical disintegration is predominant and the formation of plastic clays by chemical decomposition is uncommon. The other predicted gravel gains are less than 1 mm per year and are related primarily to the material coarseness under minimal traffic. This gravel gain is neutralised at a traffic count of 15 vehicles per day. As only one of the sections studied had a traffic count of less than 20 vehicles per day inaccuracies can be expected at these very low counts. Other than these anomalies the predicted annual gravel losses range from 4,5 mm for a lightly trafficked, fine-grained, low plasticity material in a dry area to about 65 mm for a non-cohesive, coarsely graded aggregate under high traffic in a dry area.

An interesting aspect noticed after regravelling of some of the sections was an initial rapid "gravel loss" after opening to traffic before the gravel loss became more consistent. Examination of the results from the sections which were regravelled during the project showed this to be common to them all. This phenomenon was confirmed by the Brazil data (Visser, 1981a) and was attributed to initial compaction effects.

An experiment was carried out on a newly regravelled, experimental section of road where gravel loss and density measurements were carried out at frequent intervals after construction. The road was compacted with at least three passes of a pneumatic tyred roller, an unusually high compactive effort for an unpaved road. An average relative compaction of 97 per cent Mod AASHO was obtained. The analysis of these results showed only a small initial traffic compaction followed by a consistent gravel loss of 2,5 mm per month (higher than predicted) over the next 4 months (Figure 6.2). A significant increase in the gravel loss is noted in August immediately after an unusually large number of vehicles used this road to attend an agricultural show. From the beginning of the wet season the gravel loss decreases to an amount less than the predicted loss. The increase in dry density of about 9 per cent (from the compacted density) during the initial period after construction is not reflected in a rapid initial compaction (as was the case in the roads compacted under a grid roller) although it may be incorporated in the overall gravel loss.

This experiment certainly showed the benefit of adequate compaction during construction as noted by Netterberg (1985).

The rate of gravel loss is affected mainly by the traffic, but an important overlying influence of the geotechnical properties is present. This can be seen from Table 6.5 where, for example, the annual gravel loss on a road carrying 395 vehicles per day in an area with an N-value of 1,3 will vary between 8,7 mm and 42,3 mm depending on the grading and plasticity of the material. The grading and

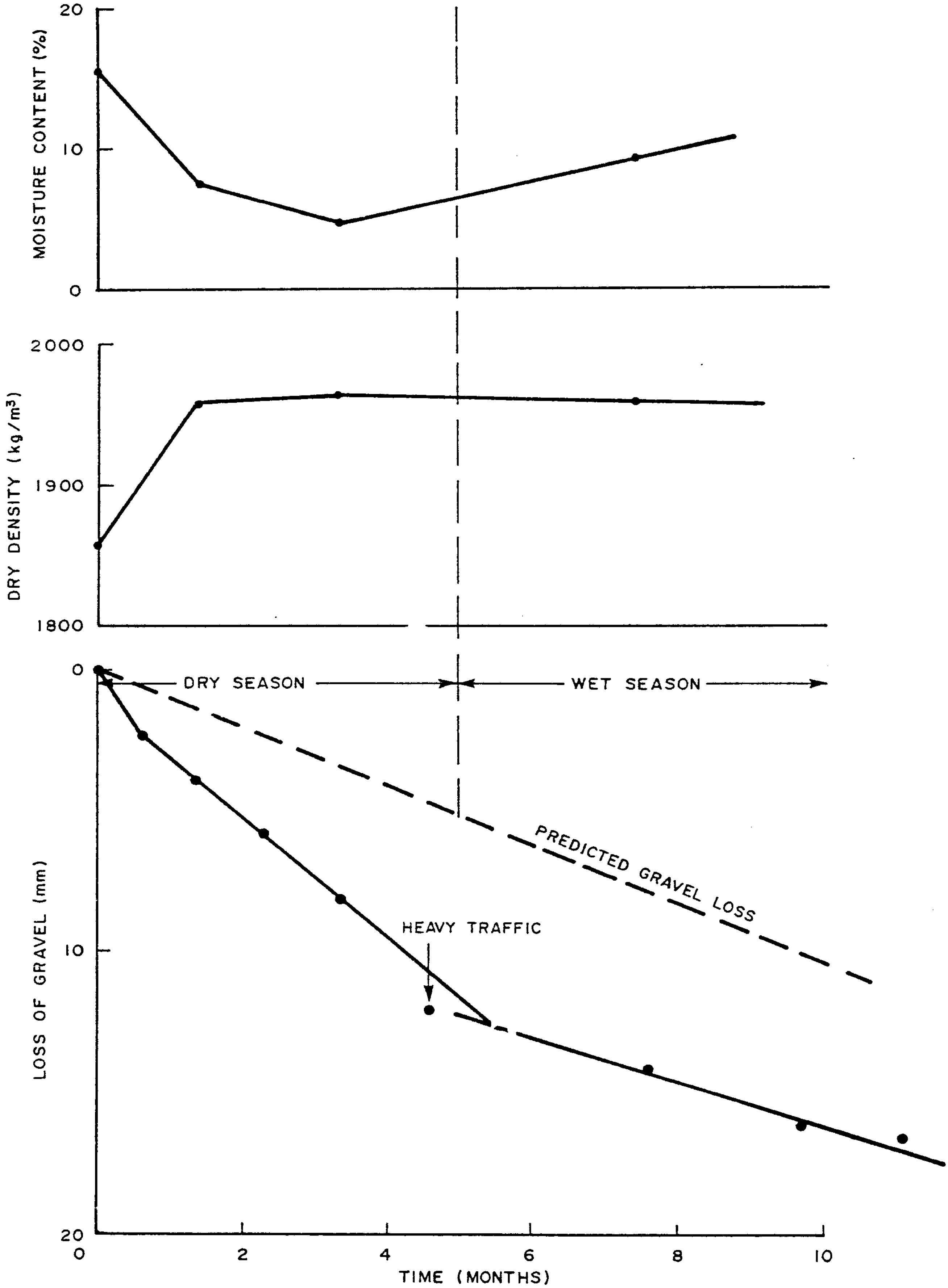


FIGURE 6.2

MOISTURE AND DENSITY CHANGES AND GRAVEL LOSS FOR ROAD 2452 plasticity are related almost entirely to the geological composition of the parent rock (or material in the case of pedocretes) and the climate under which weathering took place.

6.5 COMPARISON WITH OTHER MODELS

It is not recommended that different models are compared using statistical parameters such as the r-squared or root mean square errors, as the inference space of the models may differ considerably. A comparison of the predictive capability of the various models using the residuals was thus carried out. The residuals were calculated by subtracting the predicted annual gravel losses from the annualised differences between the first and last measured gravel heights for each section or regravelling cycle. The results of this analysis are shown in Table 6.6.

TABLE 6.6: COMPARISON OF RESIDUALS OF GRAVEL LOSS DATA USING DIFFERENT MODELS

Origin of model		Mean	Variance	Std Error of mean
Kenya	6.1	2,17	104,01	0,96
Brazil	6.2	-0,85	78,84	0,84
Kenya (Jones mod)	6.1	7,98	167,00	1,22
World Bank	6.3	5,49	55,62	0,70
South West Africa	6.4	3,12	158,98	1,19
This project	6.8	-0,87	39,97	0,59

It is noted that the closest prediction of the mean to zero of all the models is Model 6.2 derived in Brazil which underpredicts by 0,85 mm on average. Model 6.8 is marginally worse underpredicting by 0,87 mm on average. The variance of the residuals of Model 6.8 is much smaller than that of the other models indicating that the predicted values do not differ from the actual results as much and a narrower band of predicted values is obtained. An analysis of the actual measurements is summarised in Table 6.7 and shown as histograms in Figure 6.3.

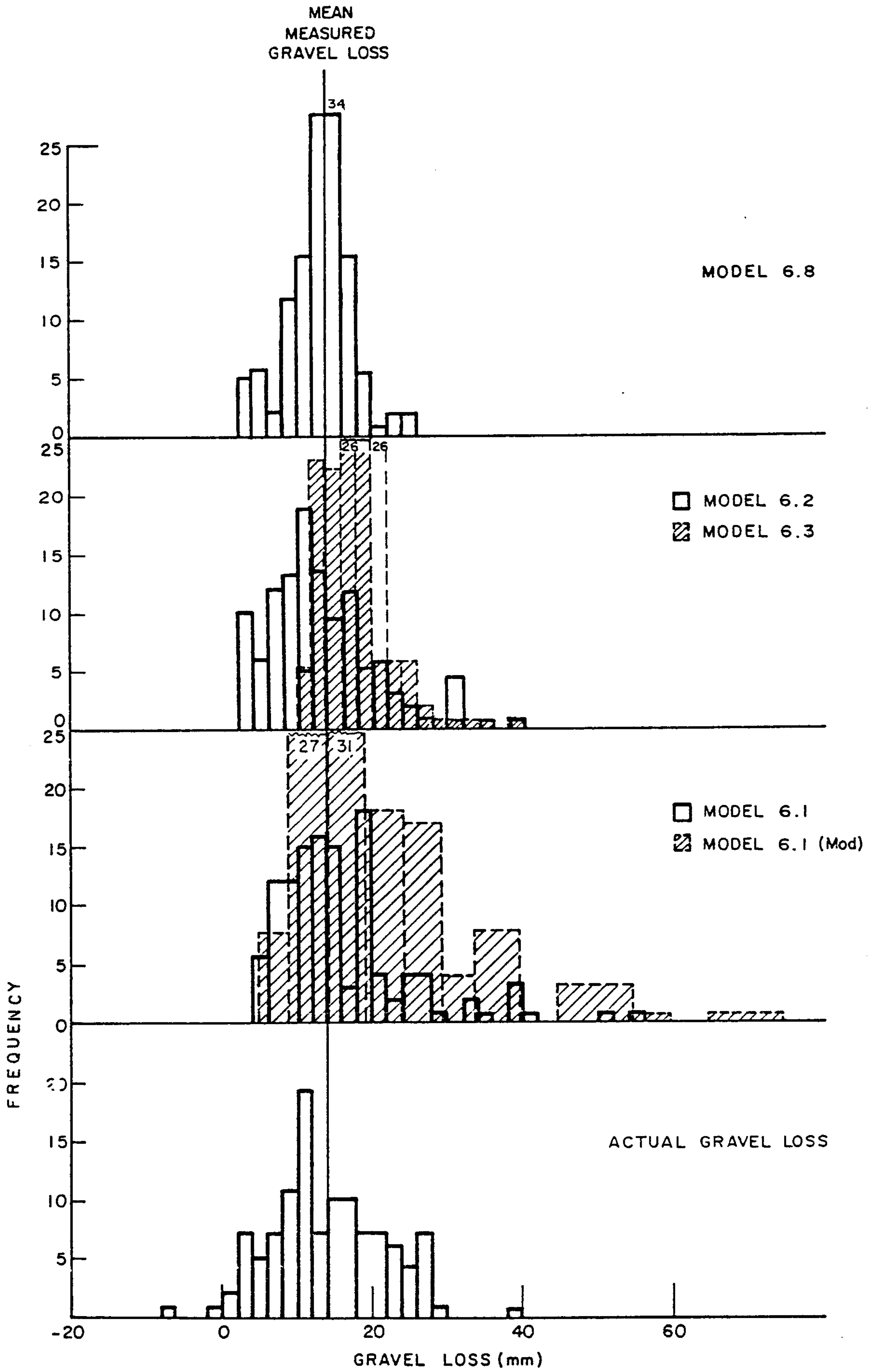


FIGURE 6.3

HISTOGRAMS OF ACTUAL AND PREDICTED GRAVEL LOSS

TABLE 6.7: SUMMARY STATISTICS OF PREDICTED AND ACTUAL GRAVEL LOSSES

MODEL	MEAN	VARIANCE	ACTUAL ERROR PREDICTED (%)
6.1	16,0	83,5	15,1
6.2	13,0	46,3	-6,5
6.1 (Jones mod)	21,8	157,5	56,8
6.3	19,4	21,7	39,6
6.4	17,0	162,3	22,3
6.8	13,1	18,7	-5,8
ACTUAL	13,9	58,8	0

An examination of the actual measured and predicted mean gravel loss and the prediction errors shows that Model 6.8 is the closest to the mean (5,8 percent under), Model 6.2 is the next best underpredicting by 6,5 percent while Model 6.1 overpredicts by 15,1 percent. The variance of Model 6.8 is much smaller than that of 6.2 indicating a much smaller range of predicted results. Inspection of the actual gravel loss measurements indicates that five of the nine results above the maximum predicted value of 25,4 mm were obtained from sections which were regravelled during the project and both of the negative values were obtained for sections which were regravelled during the project. It would thus appear that the high gravel losses are caused by excessive cutting during grader blading prior to regravelling (probably to restore the shape as the subgrade became exposed), excessive gravel loss from the subgrade material or traffic compaction of the poorly compacted wearing course after construction. The model (6.3) developed by Paterson (1985) is recommended by the World Bank in the HDM3 document for developing countries. This model overpredicts the gravel loss under South African conditions by more than 20 per cent. There is very little difference between the Brazil model and the locally developed one in terms of prediction capability but the Model 6.8 is much simpler to use and requires less input.

Application of the local model to the gravel loss measurements recorded in Kenya (Figure 6.4) and Brazil (Figure 6.5) indicates that Model 6.8 generally underpredicts the gravel loss but overpredicts for one section (G29) in the Kenya study. The predictions are more accurate for the Brazil study (Figure 6.5) conforming well with the

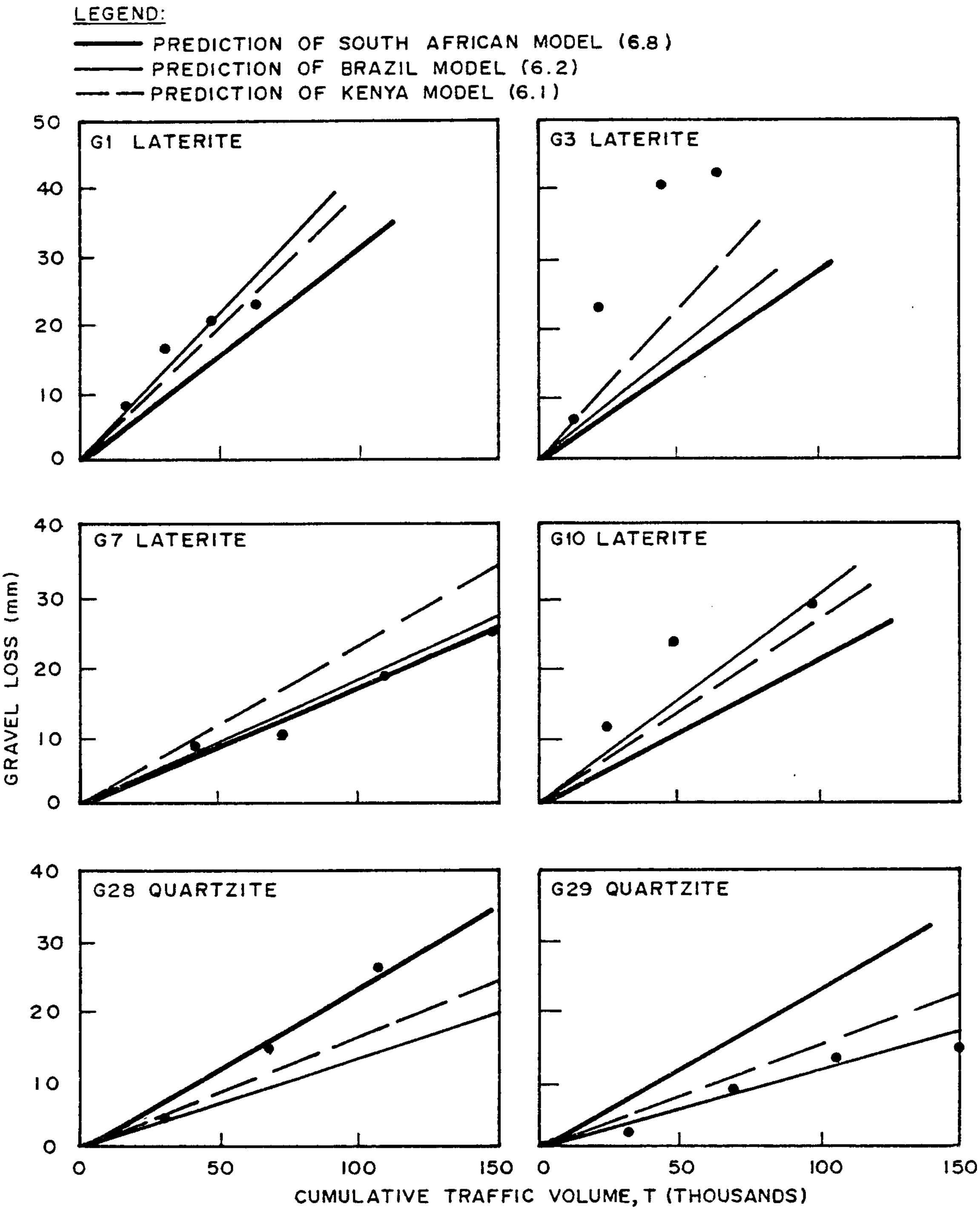


FIGURE 6.4

GRAVEL LOSS VERSUS CUMULATIVE TRAFFIC VOLUME FOR A SELECTION OF GRAVEL ROADS FROM THE KENYA STUDY (Hodges et al 1975; Visser 1981 a)

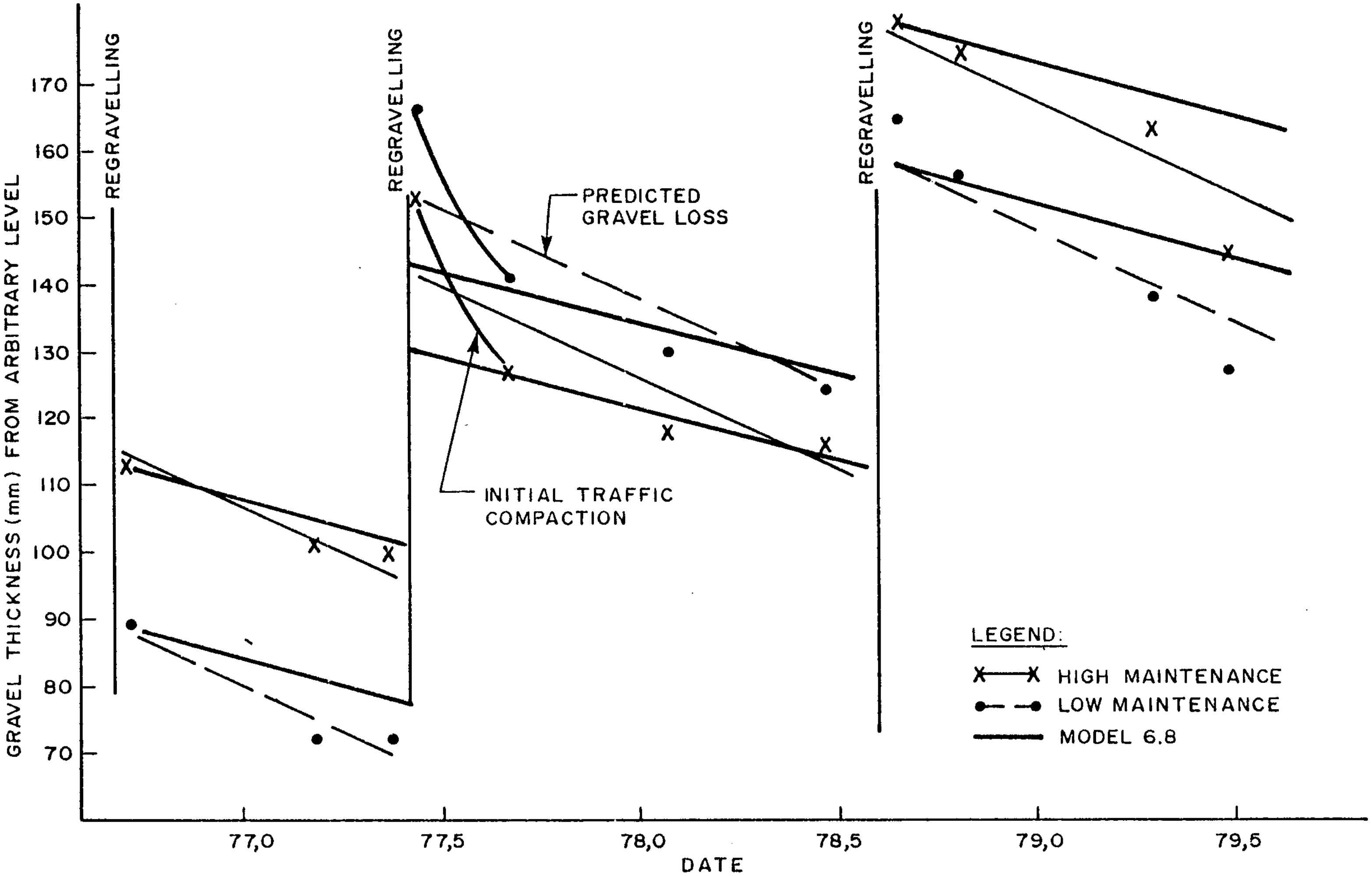


FIGURE 6.5

PREDICTED AND MEASURED GRAVEL LOSS ON SECTION 251 actual measurements for the first two cycles. The actual gravel loss in the third cycle is much higher than predicted and it would appear that either the material used had different characteristics or the traffic on the section increased dramatically. The rapid "gravel loss" between the first two measurements of the second cycle is typical of the traffic induced compaction after regravelling reported in Chapter 6.4.

6.6 CONCLUSIONS

The analysis of existing models to predict the gravel loss on unpaved roads indicated large differences between the predicted gravel losses for a number of roads in South and South West Africa and the actual losses. A model based on some 700 results from southern Africa has been developed. This model predicts the gravel loss with an accuracy of within 11 mm and should replace the model developed in Brazil for the Maintenance and Design System when used locally.

One of the main advantages of this model is its simplicity compared to the existing models. Aspects such as the vertical grade and horizontal curvature which need to be averaged for a road link are excluded from the model eliminating a major problem. No geological classification is necessary. All the parameters required can be easily obtained by relatively unskilled staff in developing areas.

As noted for the roughness prediction model, the geotechnical properties (notably the grading and plasticity again) significantly affect the gravel loss of unpaved roads.

CHAPTER 7

SPECIFICATION DEVELOPMENT

7.1 INTRODUCTION

From the discussion of the important properties affecting the road performance (Chapters 4, 5 and 6), suitable limits for the specification of the various material properties were developed. This chapter discusses the derivation of performance-related specifications for wearing course gravels based on the results of the monitoring programme and the field and laboratory testing. Although the specifications were based on a limited number of sections, they cover a wide range of material, climate and traffic regimes, and statistically are applicable over much of southern Africa.

Prior to the development of the specifications, the requirements of good specifications were identified and the applicability of the specifications in current use was investigated. An experiment to define some acceptability limits for roughness, dustiness and safety, using two rating panels was also carried out.

7.2 REQUIREMENTS OF GOOD SPECIFICATIONS

Good material specifications should fulfil a number of requirements:

- They should be simple, with as few requirements or different test methods as possible.
- The limits should be as wide as possible and not restricted to a narrow range of a significant property (e.g. a tight grading envelope), but adequately comprehensive to accept suitable materials and reject unsuitable materials. Experience has shown that strict specifications result in their being totally ignored, or in excessively high costs to locate and transport suitable material. The trend in recently specified limits is toward simplification (e.g. the most recent TPA (1983) and DOT (1985) specifications have all but eliminated grading requirements).
- The properties specified should require inexpensive, quick, easily

done tests which are repeatable and reproducible and necessitate minimal sophisticated equipment and a relatively low level of operator training.

- Specifications should be practical to implement and apply to the total area for which they are intended.
- Specifications should adequately define important properties (indirectly if necessary) such as cohesion and strength and eliminate obvious problems such as oversize material.
- Specifications should preferably be in terms of existing test methods or combinations of results from existing methods, although scope exists for the development of simple new methods.
- Specifications should be based on performance related studies.
- Although specifications should be rigidly adhered to, the facility should exist for the user to appreciate the consequences of the use of materials outside the specified limits e.g. the implications of increased construction, maintenance and road user costs, increased dust, poor safety standards.

The specifications were developed bearing all of these factors carefully in mind.

7.3 EXPERIMENT TO DEFINE ACCEPTABILITY CRITERIA

Acceptability criteria for paved roads are well defined and have been summarised by Visser (1981a). Acceptability criteria for unpaved roads, however, have generally consisted of various criteria such as those affecting the profitability of haulage routes in the forestry industries of the USA and Canada (Paterson et al, 1975; Visser, 1981a). These criteria are suitable for this type of service but are unsuitable even for developing countries where the all weather trafficability of important routes is necessary. In the southern African situation where together with the all weather trafficability, the road roughness, passenger comfort and safety are considered equally important, acceptability criteria for these parameters needed to be defined.

Visser (1981a), instead of considering a set of subjectively determined standards proposed a set of limiting conditions, mainly related to maintenance philosophies. For this study, it was necessary to quantify acceptability criteria for road roughness, passenger comfort and safety in order to relate the material properties to these parameters.

In order to quantify these parameters two rating panels were used. The first one consisted of 8 engineers, road inspectors and materials technicians from the TPA and one from South West Africa who carried out a full condition description as used in the routine monitoring of the gravel road experimental sections. In addition a rating of the roughness, dustiness and safety was obtained. This was followed up four days later by a panel made up of a cross section of the travelling public. This 14 member panel consisted of black and white males and white females and included motoring and other journalists, a farmer, a rallyist and an engineer and geologist. In this case only the ratings for roughness, dustiness and safety were requested. The results of this experiment are fully described by Paige-Green and Netterberg (1988) and are summarised below.

For both panels 5 of the experimental sections were selected with measured roughness values in the range QI = 30 (very good) to QI = 184 (thought to be totally unacceptable). The dust was generally in the range average to unacceptable as rated during the project. The engineers' panel was asked to drive through the sections at 80 kph and rate the three parameters on a 5 point scale as follows:

1 - excellent
2 - good
3 - average
4 - bad
5 - unacceptable

This rating was done in both directions although the roughness differed in the two directions. They were then asked to carry out a full condition description. The same 5 point scale was used for severity and the areal extent was rated as follows:

1 - 0 - 20 % (of area of travelled surface)
2 - 20 - 40 %
3 - 40 - 60 %
4 - 60 - 80 %
5 - 80 - 100 %

The results of this rating varied considerably from those which were based on the standard method used during the project. After some discussion it became apparent that the TPA staff were rating the sections in terms of required maintenance. This differed from the NITRR approach which rated the sections in terms of the propensity to form potholes, loose material etc.

In addition the TPA ratings generally overestimated the extent, usually classifying the extent as 100 % of the section. Should any distress occur the whole section would need maintenance and not just a few potholes patched or the excess loose material removed in places. For research purposes it was considered more important to classify the worst severity in the section (provided it was significant in the section i.e. occurred over say at least 5 % of the area or affected the riding quality or safety). It was also important to estimate the area affected by that condition as opposed to the TPA rating of the whole section, e.g. if there were only a few potholes, mainly with a severity of 3 but one or two with a severity of 5 the rating would be - severity 5, extent 1.

The public rating panel visited the same sections four days later and rated the roughness, dustiness and safety. A practice section was rated first so that any problems could be clarified and the participants knew what was required. Each section was only rated in one direction and the actual roughness in that direction was measured.

An analysis of variance of the roughness rating matrix (Paige-Green, 1987a) indicated that the variation between the raters and sections was significant at the 0,01 level. This agrees with the ratings carried out during the AASHO Road Test (Hutchinson, 1963). The overall reliability (a measure of the repeatability) of the raters and ratings was 0,76 and 0,987. These figures compare well with the values obtained by Curtayne et al (1972) of 0,661 and 0,978 for a panel rating paved roads in South Africa (a value of 1 indicates perfect repeatability and 0, no repeatability).

7.3.1 Roughness rating

The roughness ratings were generally fairly consistent with the range of ratings within a panel only differing by more than 2 units on three of the sections, one being the practice panel for the public. It must be noted that some of the sections had fairly wide wheel paths and the ratings may not apply to exactly the same paths each time. This is taken into account as far as possible by using the average of three results for the measured roughness.

In order to get an acceptance criterion the average of each panel and section was determined. These were regressed against the measured roughness (in QI counts/km) of the sections. A good correlation was obtained for the following equation:

$$\text{Mean rating} = 0{,}018 \text{ ROUGHNESS} + 1{,}26 \qquad (7.1)$$

($r^2 = 0{,}93$; RMSE = 0,25; 95 % confidence limits $\pm$ 0,5)

The regression equations for the TPA and public when compared are as follows:

$$\text{TPA Rating} = 0{,}018 \text{ ROUGHNESS} + 1{,}28 \qquad (7.2)$$

($r^2 = 0{,}91$; RMSE = 0,35)

$$\text{Public rating} = 0{,}017 \text{ ROUGHNESS} + 1{,}24 \qquad (7.3)$$

($r^2 = 0{,}97$; RMSE = 0,17)

Analysis of these relationships (Figure 7.1) yield the following criteria for measured roughness (QI rounded to the nearest 10) (Table 7.1):

TABLE 7.1: CRITERIA FOR MEASURED ROUGHNESS

Severity		Proposed	Engineers	Public
Excellent	(1)	0 - 40	0 - 40	0 - 40
Good	(2)	40 - 100	40 - 90	40 - 100
Average	(3)	100 - 150	90 - 150	100 - 160
Bad	(4)	150 - 200	150 - 200	160 - 220
Unacceptable	(5)	> 200	> 200	> 220

The proposed values (Paige-Green and Netterberg, 1988) indicate that the roughness of gravel roads should be maintained at values less than 150 QI and preferably less than 100 counts/km (i.e. good). During the routine monitoring of the experimental sections many sections were rated by the different LDI drivers and the following tentative limits were proposed:

		QI
Severity	1	0 - 45
	2	45 - 90
	3	90 - 135
	4	135 - 180
	5	> 180

From Figure 7.1 it is observed that the TPA rating was apparently more variable than that of the public. This is probably due to the TPA ratings being in both directions while the higher measured roughness of the two directions was used in the analysis. An interesting aspect was that the public considered a slightly higher value (QI of 162 against 146) as the lower limit of a bad section. Both of these limits are higher than that proposed tentatively during the monitoring.

7.3.2. Dustiness

As no quantitative value for the dustiness was available the ratings of the panels were compared with the standard method used during the routine monitoring of the project. The rating panels used subjective values on the 5 point scale while the standard used more objective ratings (e.g. Severity 4 - the silhouettes of cars are visible in the

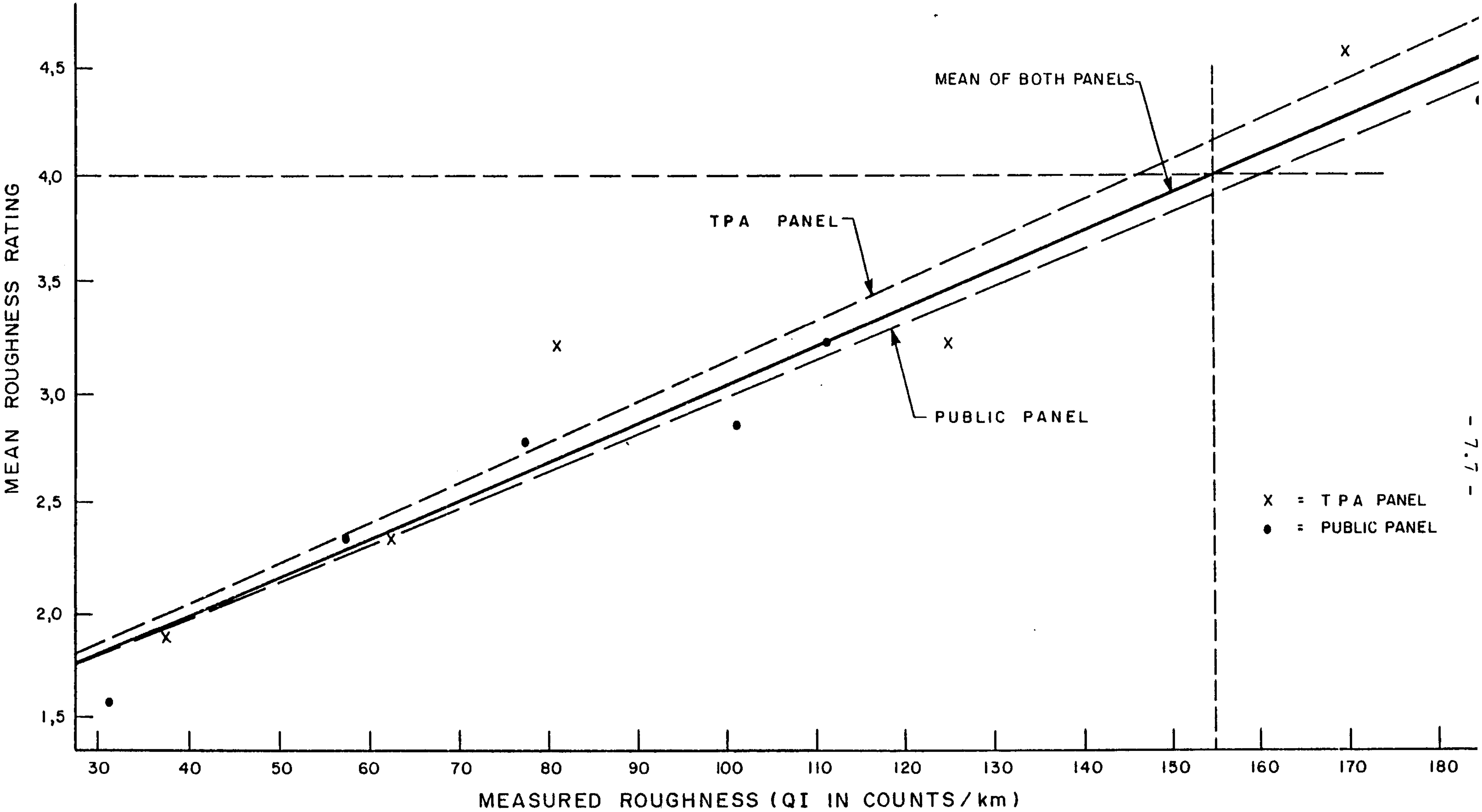


FIGURE 7.1

REGRESSION LINE OF MEAN RATED ROUGHNESS AGAINST MEASURED ROUGHNESS (QI COUNTS/km) dust while for a rating of 5 a car is totally obscured by the dust at 80 km/h).

The average dust for all the sections was less than 4 (i.e., no section was regarded as bad or unacceptable) although two sections (Sections 2 and 4) were rated 4 by the standard method. Although it was indicated by a number of the panel members that dust is an important aspect it would appear that provided visibility is not totally obscured the dust is tolerable. A comparison of the panel ratings and project ratings is given in Table 7.2.

TABLE 7.2: COMPARISON OF PANEL AND PROJECT DUSTINESS RATINGS

Section	Project rating for eng panel	Engineers rating	Project rating for public panel	Public rating
8	4	2,3	3	2,4
7	3	2,7	3	3,0
4	4	3,0	4	3,6
1	4	2,7	3	2,9
2	5	3,1	4	3,6

The project ratings differed for the two panels as the dustiness was affected by rain between the two panel ratings. The public panel rated the dust marginally higher than the engineers on every section even though the dust was rated lower on 4 of the sections by the project standard. The public therefore requires a stricter dust standard (i.e. less dust) than the engineers.

From this rating it can be assumed that those roads which consistently gave ratings of 5 during the project monitoring (when not affected by rain) are the only ones unacceptably dusty. The mean for the public panel was consistently slightly higher (i.e. worse) than that for the engineers.

7.3.3 Safety

The analysis of the safety aspect was difficult to relate to any one characteristic of the road. All the sections had an average safety rating of less than 3 (i.e. considered to be of better than average safety), except Section 4. This had an average rating of over 4 from both the TPA panel and the public panel. The main reasons given for the poor safety were the loose material ("sandwalle"), the roughness and the unpredictability when driving on it.

Skid resistance tests on this section using a MOTOMETER brake efficiency meter indicated that the skid resistance was adequate when compared to most other sections and substantially better than Sections 7 and 8. It appears therefore that the public opinion of safety is directly related to the directional handling characteristics of the vehicle under normal conditions and not under emergency braking situations.

In summary it can be concluded that:

1. The acceptable limit for road roughness was found to be a Quartercar Index (QI) of 200 and the desirable limit 150. This compares closely with the value of 180 and 135 estimated during the monitoring programme.

2. The roughness ratings of the TPA and Public panels were very close to each other with the average for the Public panel being surprisingly higher (rougher) than that of the TPA.

3. The rating of dustiness showed that the dust is generally acceptable provided vehicles are not totally obscured by the dust.

4. Safety criteria seem to be based entirely on the vehicle directional handling under normal conditions with little regard to emergency braking situations.

7.4 ACCEPTABILITY LIMITS FOR SPECIFICATION DEVELOPMENT

The acceptability limits discussed in this section are defined as the critical levels beyond which the roads fail to provide the service expected from them (i.e. become unacceptable) and are applicable only to performance criteria. This acceptance should not to be confused with the acceptance testing of materials for specification compliance.

Acceptability criteria for the roughness, dustiness and safety under local conditions have been derived for unpaved roads (Paige-Green and Netterberg, 1988). These criteria are, however, defined for the worst conditions which would be accepted. For the purpose of specification development it was necessary to classify the performance of the various properties monitored in terms of their acceptability over a period of time. The results of the condition evaluation (Paige-Green, 1988a) and performance monitoring could be analysed in terms of a number of statistical parameters (i.e. mean, maximum, mode, minimum, various percentiles, etc). The mode (the most frequently occurring value) was considered to be the most useful of these parameters for those performance criteria with discrete values (e.g. dust, potholes) for the purpose of specification development. Use of the mode would exclude such spurious measurements as those affected by rain, recent grader blading, or poor subjective assessments. It was noted during the monitoring that an equilibrium condition was reached for most of the performance criteria fairly soon after maintenance and the condition seldom deviated from this equilibrium, until affected by blading or rainfall.

During analysis the mode for each parameter was determined but was found to be unsuitable for those properties with continuously variable values (i.e. roughness, rut depth, corrugation depth, even though for example corrugations would form and deepen to an equilibrium depth, after which they seldom varied by more than about 5 mm until the section was bladed or rain fell) as these results were often multi-modal and the first observation which had more than one value was taken as the mode by the analysis programme (SAS, 1985). The 90th percentile was therefore taken for these properties as this was considered to exclude spurious results and be indicative of the condition that would not be exceeded 90 per cent of the time. The performance of each section was then classified for each property as desirable, undesirable or unacceptable according to Table 7.3.

The criteria for selecting the values in Table 7.3 were based mainly on the acceptability criteria experiment (Paige-Green and Netterberg, 1988), the rating criteria (Paige-Green, 1988a), existing information and the experience gained from having travelled some 90 000 km on unpaved roads during the project. As it is usual (and recommended) that most unpaved roads are bladed periodically during the year all the results for roughness recorded longer than 4 months after blading were excluded from the analyses so as not to bias the results excessively. Very few roads should, in practice, be bladed less than three times per year, but for the purpose of this project, blading was limited as far as possible to less than once every four months. This allowed the performance in the longer term to be investigated.

The performance acceptability criteria (Table 7.3) of the dust, cracks, loose material, slipperiness when wet and passability were based solely on the most frequently occurring ratings (modes) for the severity as rated during the monitoring (Paige-Green, 1988a). The areal extent of the stoniness, potholes and surface drainage affected the performance of the road significantly, but to a lesser degree than the severity. The product of the square of the severity rating and the extent rating was thus considered to be representative of the influence of the parameter on the performance. All of these criteria are unitless values based purely on the ratings of severity and extent obtained during monitoring.

As no data existed on the skid resistance of unpaved roads, no basis for the decision on acceptability limits for the deceleration was available. The recommended minimum deceleration for new cars on good surfaces is 5,8 m/s^2 (SABS, 1985) and a value just less than this was considered to be the lower limit of desirability for dry unpaved roads. None of the sections had values less than 5,1 m/s^2.

It was considered that a maximum of 20 mm for the 90th percentile values of the rut and corrugation depths was desirable. Although a

TABLE 7.3: PERFORMANCE CRITERIA USED IN SPECIFICATION DEVELOPMENT

Property	Criterion	PERFORMANCE		
		Desirable	Undesirable	Unacceptable
Dust	Sev	≤ 3,5	3,5 - 4,5	≥ 4,5
Stoniness	Sev²x ext	< 25	25 - 32	> 32
Potholes	Sev²x ext	< 25	25 - 32	> 32
Cracks	Sev	≤ 3,5	3,5 - 4,5	≥ 4,5
Loose mater.	Sev	≤ 3,5	3,5 - 4,5	≥ 4,5
Surf. drain.	Sev²x ext	< 25	25 - 32	> 32
Slip (dry)	Deceler.	> 5,5	4,5 - 5,5	< 4,5
Slip (wet)	Sev	≤ 3,5	3,5 - 4,5	≥ 4,5
Passability	Sev	≤ 3,5	3,5 - 4,5	≥ 4,5
Ruts	Depth (mm)	< 20	20 - 30	> 30
Corrugation	Depth (mm)	< 20	20 - 30	> 30
Roughness	QI	< 100	100 - 150	> 150

Sev = severity; ext = extent; Deceler = deceleration (m/s^2); QI = Quartercar index (counts/km).

roughness value of 150 counts/km was found to be the upper limit for desirability from the acceptability experiment (Paige-Green and Netterberg, 1988), the limit for desirability in terms of the 90th percentile could not be set at this value. A maximum value of 100 counts/km was considered desirable, while a value of more than 150 counts/km was taken as unacceptable.

7.5 APPLICABILITY OF EXISTING SPECIFICATIONS

Prior to developing new specifications, it was considered prudent to analyse the performance of the sections studied in terms of their overall acceptability with respect to their compliance with the existing specifications applicable to southern Africa. The TRRL (1981) and UNESCO (Odier et al, 1967) limits used by some commonwealth countries in southern Africa are generally the basis of the Botswana (Min of Works and Comm, 1982) specifications and were not considered separately. The main objective of this analysis was to investigate which specifications (if any) were adequate, and what the main deficiencies of the inadequate specifications were so as to eliminate these problems in future specifications. Table 7.4 shows the total percentage of sections complying with all the material limits (except maximum size where specified which eliminated all of the sections studied). This is sub-divided by the observed performance. The percentage of sections not complying with the specifications but providing at least a desirable performance is also shown for each specification.

Most of the materials rejected by the TRH 14 (NITRR, 1985a) specification had either plasticity indices which were too low, or the percentages passing the 0,425 mm and 4,75 mm sieves were too large. The Natal Roads Department (NPA, ca 1985) and Swaziland (Shepherd, 1986) requirements include the TRH 14 grading limits but specify a narrower range for the plasticity index. The lower plasticity index requirement (6 per cent) of Natal, results in more of the lower plasticity materials being accepted than the TRH 14 and Swaziland requirements. The Department of Transport (1985) and Transvaal Roads Department (1983) use the same specifications with the preferable minimum limit for the soaked CBR of 45 eliminating most of the materials. Those materials with a CBR of 45 or more generally have plasticity indices less than 6.

The Orange Free State (Van der Walt, 1973) specification requires a minimum PI of 10 and CBR of 25 which excludes many of the experimental sections. The percentage passing the 2,0 mm sieve is specified by the CPA (Provincial Administration of the Cape of Good Hope, 1983) and this together with the minimum plasticity of 8 eliminates nearly all of the sections. The Botswana (Ministry of Works and Communic, 1982) specification is adapted to the prevailing aridity and restricts the PI to between 15 and 30. This excludes many of the materials in this study, but the difference in climate should be considered. The Botswana specification makes allowance for the use of the Netterberg (1978) specification for calcretes which improve the situation significantly as discussed below. South West Africa (Von Solms, 1987)

TABLE 7.4: COMPLIANCE OF STUDY SECTIONS WITH EXISTING SPECIFICATIONS

Specification	Total	Sections complying (%) By performance Desirable	Undesirable	Unacceptable	Sections not complying but giving desirable performance (%)
TRH14	12,2	4,7	5,6	1,9	32,6
DOT	2,7	0,9	0,9	0,9	36,4
TPA	2,7	0,9	0,9	0,9	36,4
NPA	22,5	7,5	10,3	4,7	29,8
OFS	4,5	1,8	1,8	0,9	35,5
CPA	14,0	7,5	5,6	0,9	29,8
Botswana	3,8	1,9	1,9	0	33,6
Lesotho	0	0	0	0	37,3
Swaziland	15,0	5,6	4,7	4,7	31,7
SWA	3,6	0,9	1,8	0,9	36,4
Netterberg	29,0	14,1	10,2	4,7	23,2

has much the same climatic conditions as Botswana but allows for much lower plasticity materials. Most of the materials are however rejected by the restriction on the grading modulus with more than half of the acceptable materials having GM values in excess of 1,7 while more than 40 per cent of the unacceptable sections have materials with grading moduli within the limits. The Lesotho (Tsekoa, pers comm, 1986) requirement of a minimum CBR of 60 eliminates all but five of the sections, these five, however, defaulting on the plasticity index (generally less than 6) or the percentage passing the 200 mesh sieve.

The specification developed by Netterberg (1978) specifically for calcretes in dry areas accepts the highest percentage of the desirable materials. A number of the acceptable materials are only just rejected by virtue of marginally low AFV or APV values. The relative success of these specifications for the range of different materials in this study is indicative that the performance of wearing course gravels is not significantly dependent on the rock type. The specification excludes a number of materials which perform acceptably but with slightly low plasticity indices and liquid limits. Some of the finer grained materials with more than 75 per cent passing the 40 mesh sieve performed adequately but cannot be tested against the specifications as no limits for these fine materials are specified by Netterberg. In addition, values for the aggregate fingers value and aggregate pliers value cannot be determined when the material is fine and the 9,5 to 13,2 mm fraction is very small. Of the five unacceptable materials which would be accepted by the Netterberg specification, three perform unacceptably by virtue of the percentage of oversize material while one section had bad grader-induced fixed corrugations.

In terms of the 18 calcrete sections of road included in the study, of which eight were classified as giving a desirable performance during the project, two as undesirable and seven as unacceptable, the Netterberg criteria identify the following:

- Three of the eight desirable materials comply with the limits, three have marginally low APV values, one a low liquid limit and the other sample was too fine grained (87 per cent passing the 40 mesh sieve) for application of the specified limits. Using the finest specified limit all the requirements were satisfied except the plasticity index which was too low (6 instead of 8).
- One of the undesirable sections was accepted and the other rejected because of a low liquid limit. The material accepted was rated undesirable because of its stoniness and would probably provide an acceptable roughness without the excessive oversize material.
- Three of the unacceptable materials complied with the limits, two having excessive oversize material and the other, having almost worn through the wearing course, was prone to the formation of potholes. The four materials rightfully rejected had one or more of a low liquid limit, a high linear shrinkage, an excessive LS x percent passing the 40 mesh (0,425 mm) sieve and a low AFV.

The performance guide for wearing course materials used in Rhodesia (now Zimbabwe) (Mitchell et al, 1975) utilised the plasticity product (PI x per cent < 0,075 mm) and the coarseness index (mass of material passing 37,5 mm but retained on 2,36 mm as a percentage of material passing 37,5 mm). The performance of the sections monitored during this study are plotted on the chart of Mitchell et al (1975) (Figure 7.2). It must be noted that the test results have been corrected for the different test methods (i.e. PI increased by 3,5 to account for the BS 1377 method used in Rhodesia compared to TMH 1 (NITRR, 1986; Sampson and Netterberg, 1984) and particle size analysis re-calculated on a maximum size of 37,5 mm) (Standards Association of Central Africa, 1971). The material in two of the sections is classified as being too fine but both perform well whilst the material used for 20 of the sections is classified as too coarse. There was, however, an almost even division between the performance of the sections in terms of their acceptability (7 desirable, 7 undesirable and 6 unacceptable). About 50 per cent of the sections in zones C (performs well under wet and dry conditions) and D (performs reasonably well during wet conditions but corrugates in dry weather) perform desirably. In zone E (poor all round performance) 30 per cent of the sections perform well. The bias towards the use of coarser, lower plasticity materials locally is clearly evident in Figure 7.2.

A later specification (Rhodesia Ministry of Works, 1979) recommended in Rhodesia characterises the material in terms of the plasticity index and grading modulus (Figure 7.3). The upper limit of the grading modulus excludes one unacceptable material and could perhaps be reduced (say to 2,3) which would exclude two further unacceptable materials. The lower specified grading modulus excludes 20 materials, nine of which perform desirably. Only 12 materials comply with the specified limits, six of which perform desirably. Fifty two of the materials are excluded, even by the extended limits which make allowance for climate. Of these, 18 sections provided a desirable performance.

As a whole none of the specifications differentiate between the desirable, undesirable and unacceptable materials adequately although the Netterberg (1978) specification shows the most potential to do this. All of them, except that of Netterberg (1978) reject thirty per cent or more of the acceptable materials, whilst there is only a slightly better chance of the accepted materials performing desirably than performing unacceptably.

A summary of results of some of the standard tests used in most of the specifications for the three categories of performance is given in Table 7.5.

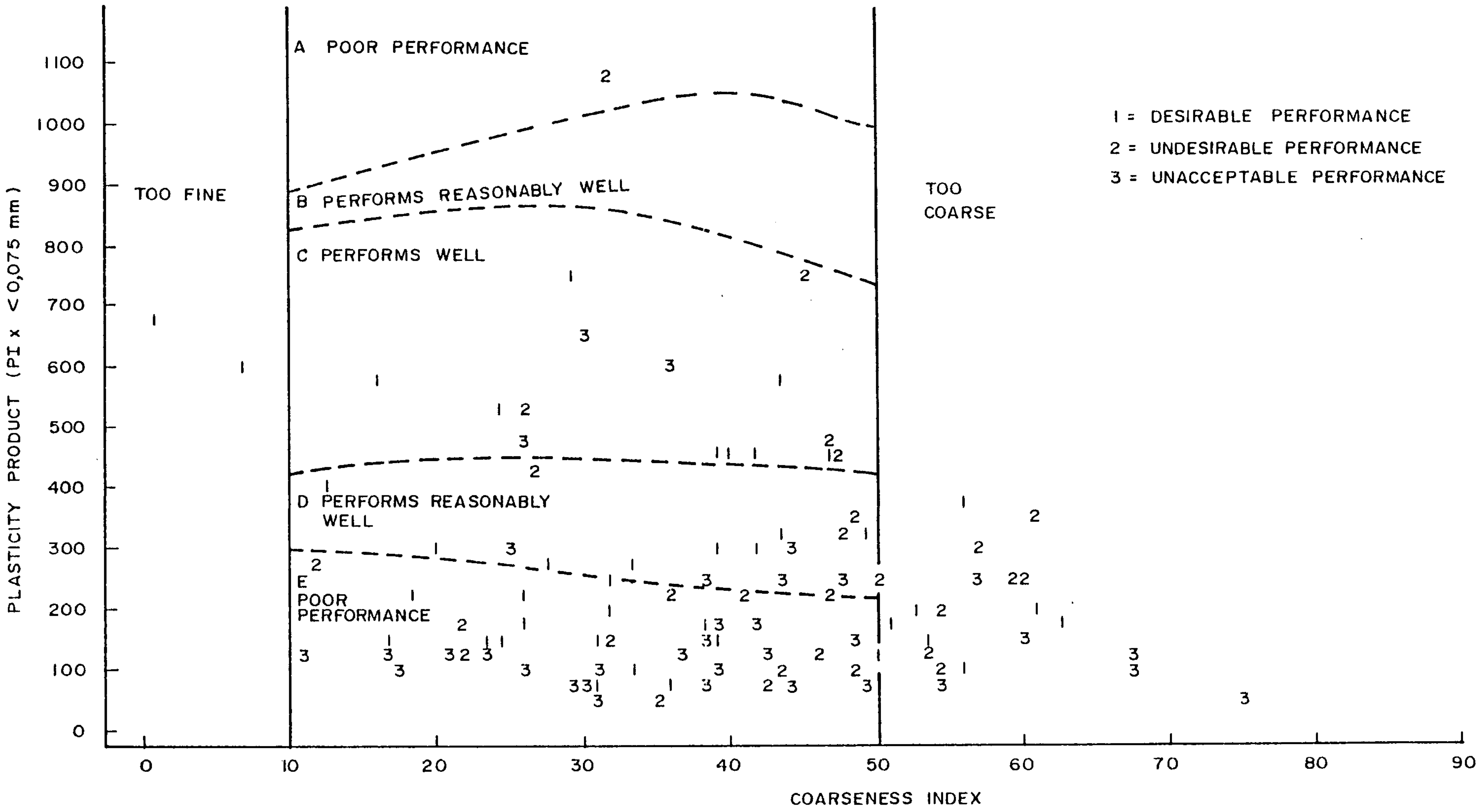


FIGURE 7.2 *PERFORMANCE OF SECTIONS IN RELATION TO 1975 RHODESIAN SPECIFICATION*

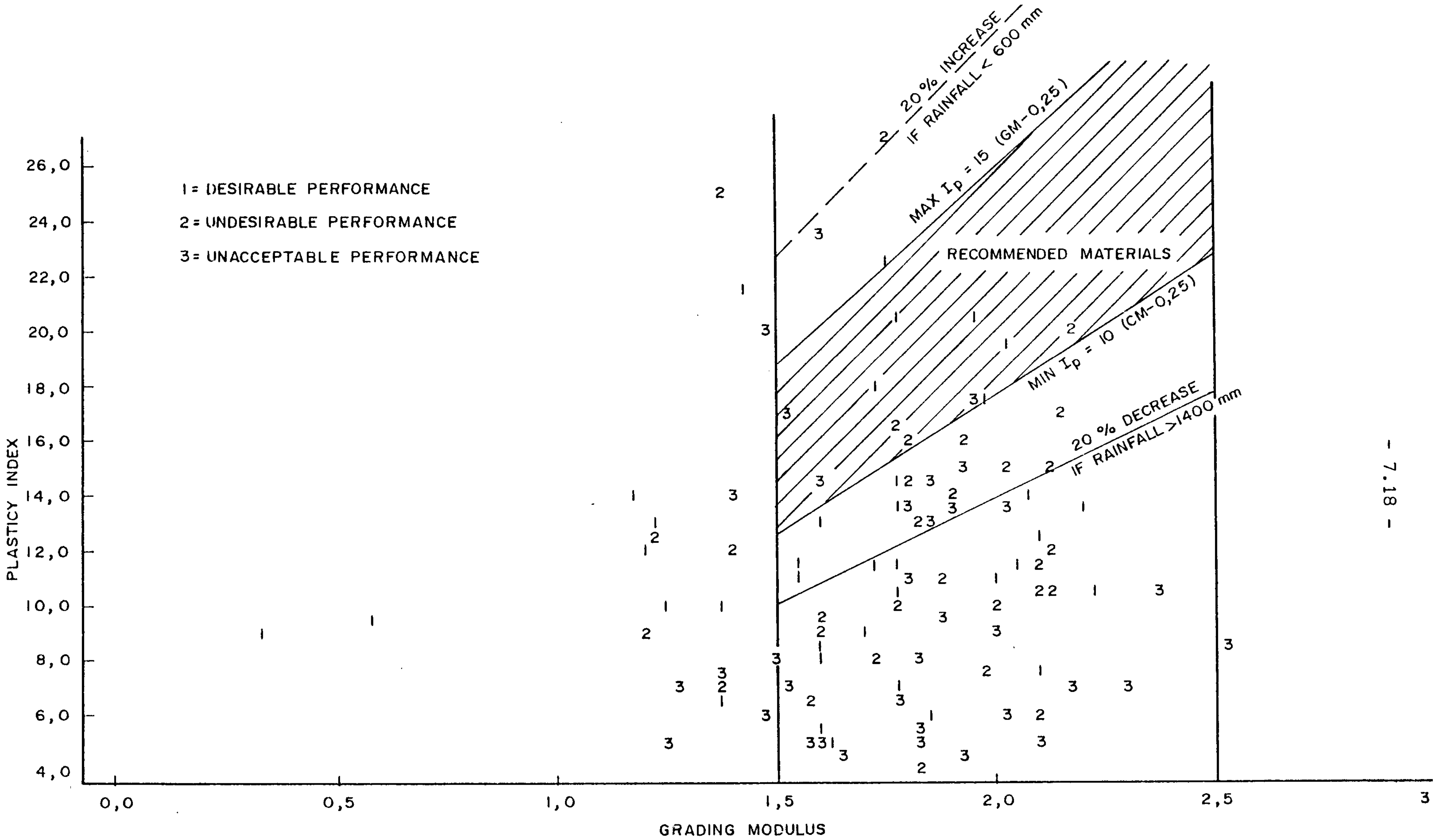


FIGURE 7.3 *PERFORMANCE OF SECTIONS IN RELATION TO 1979 RHODESIAN SPECIFICATIONS*

TABLE 7.5: SUMMARY OF TEST RESULTS FOR DIFFERENT PERFORMANCES

Performance	PI	LL	%<0,425	%<0,075	GM
Desirable	1,5-19,0	16 - 40	23 - 92	12 - 75	0,32-2,28
Undesirable	0,5-24,0	17 - 42	22 - 61	10 - 46	1,23-2,30
Unacceptable	NP-20,0	NP - 46	16 - 71	6 - 32	1,26-2,50

The properties on which most of the specifications investigated rely so heavily do not differentiate between the performance of the sections adequately, with a substantial overlap between properties for the different performances. The Rhodesian and Netterberg limits are the best predictors at present, both using the plasticity and grading in conjunction with each other, as recommended by Fossberg (Netterberg and Paige-Green, 1988b).

One of the features which does come out of this discussion is the influence of oversize material. In a direct comparison of the performance of the sections studied, many of the sections contain excessive oversize stones which result in an unacceptable performance. Had these stones not been in the sections, the materials may have performed adequately. In the development of the specifications this has necessarily had to be taken into account and for some of the stony sections it has had to be assumed that had the stones not been present the sections would have performed better. This assumption was, however, based on a good personal knowledge of the materials studied and the performance of both the test sections and adjacent sections of road. No other valid means of taking this problem into account could be found.

The presence of excessive oversize material renders the interpretation of the grading analyses difficult, and comparison of results almost meaningless. Many of the sections contain boulders substantially larger than the 63 or 75 mm sieve, normally the largest sieves used (NITRR, 1979 and 1986). The underestimation of the percentage passing the smaller sieve sizes (and the consequent overestimation of the grading modulus) caused by these large stones (noted more than 60 years ago (Strahan, 1922)) results in a number of potentially acceptable materials being rejected by many of the specifications. This is illustrated in the case of one kilogramme of test sample (NITRR, 1979 and 1986) which contained a stone of about 60 mm diameter in a finer matrix with all the material passing the 26,5 mm sieve. The percentage passing the 0,425 mm sieve was determined as 27 while as a percentage of the material finer than 37,5 mm it was 41. This makes a significant difference to the specification compliance.

Bearing in mind that the finer fraction (passing the 2,00 mm sieve) is the major contributor to the strength and cohesion of the material (Strahan, 1922), this should be accurately quantified. It is therefore suggested (Paige-Green, 1988b) that all grading analyses for unpaved roads are standardised with the percentage retained on the 37,5 mm sieve being calculated as a percentage of the dried, bulk field sample, and reported as the Oversize index (I_o) while the particle size distribution of the material is calculated on the weight of sample passing the 37,5 mm sieve (Standards Association of Central Africa, 1971). This allows a direct comparison between materials with significantly different coarse fractions without affecting the interpretation of results on finer materials (maximum size less than 37,5 mm).

The standard maintenance procedures resulted in some sections having potholes or corrugations throughout the monitoring period. Had the potholes been filled by hand with suitable material and compacted with added water, many of them would have undoubtedly been repaired. The standard procedure of grader blading does not repair potholes adequately. Some of the corrugations were of such a wave-length that they were not removed by grader blading and were probably made worse as the grader bounced over them during blading.

For the specification development the particle size analyses were standardised as percentages passing the 37,5 mm sieve in order to make the finer fractions directly comparable. This ensures that, provided a consistent test technique is used in future (Paige-Green, 1988b), the specifications are transferable for different materials .

7.6 ANALYSIS OF RESULTS

In order to develop performance-related specifications it was necessary to classify the overall performance of each of the sections. The classification of the individual performance criteria (Paige-Green, 1988a) according to Table 7.3 allowed a value to be put to each rating (desirable = 0; undesirable = 1; unacceptable = 2). The overall quality of all the roads studied was then rated by the following equation:

QUALITY = 2.Dust + Stones + Potholes + 2.Loose material + Surface drainage + Ruts + Corrugations + 4.Roughness

The roughness, being the most important criterion (the major contributor to vehicle operating costs) was weighted by a factor of 4 and the dustiness and loose material which are major criteria affecting the safety were weighted by a factor of 2. The other parameters were not weighted, but the cracking (which plays a minimal role in the performance), and slipperiness and passability which were not fully quantified were excluded from the equation. The stones, potholes, drainage, ruts and corrugations are all included to a lesser or greater extent in the roughness and were thus not weighted further. This resulted in a ranking of the sections in terms of their overall performance and classification into desirable, undesirable or unacceptable performance (quality less than 10, 10 to 14 and greater than 14 respectively). These values were subjectively determined by consensus of those involved in the routine monitoring and the findings of the acceptability experiment (Paige-Green and Netterberg, 1988).

The performance of all the sections was related to the material properties to develop specifications. The analysis involved identifying those material properties and limits which would provide the ideal wearing course according to the criteria identified in Chapter 2.5.

7.6.1 Smooth ride

The smooth ride is the most important aspect of an unpaved road as the roughness is the major contributor to the vehicle operating cost. A low roughness is obtained by minimising oversize material and eliminating corrugations, erosion channels and potholes. The presence of oversize material is one of the few material properties which can be controlled and those sections which were excessively rough because of large stones were accounted for in the analyses. The overall roughness (including the effects of corrugations, potholes and erosion channels) was analysed in terms of a number of the material properties with the shrinkage product (linear shrinkage x per cent passing 0,425 mm sieve (Netterberg, 1969)) being the best indicator. A minimum value of 100 for the shrinkage product rejects 4 desirable, 18 undesirable and 9 unacceptable sections on the basis of roughness. Most of the sections with shrinkage products less than 100 were particularly prone to the formation of corrugations and even those which were excessively rough due to oversize stones would probably remain rough if the oversize material was removed, due to corrugations. Thirty three desirable sections, 33 undesirable sections and 5 unacceptable sections have shrinkage products in excess of 100. However, many of the undesirable sections would probably perform desirably with proper maintenance. The 5 unacceptable sections with a shrinkage product larger than 100 all contained excessive oversize material or had maintenance-induced corrugations.

The oversize material should therefore be eliminated in order to reduce the roughness to acceptable levels. This is of particular importance in urban areas where the number of potential projectiles used in vandalism, especially during periods of urban unrest, should be reduced. An Oversize Index of up to 5 per cent can be allowed as discussed in Chapter 4.3.4.

7.6.2 **Safe ride**

The perceived safety of unpaved roads is apparently mainly concerned with the directional stability of the vehicle (Paige-Green and Netterberg, 1988), the most important property affecting the vehicle handling on a dry road being the presence and quantity of loose material. Under wet conditions the slipperiness and drainage is the most important aspect and is discussed under the relevant section.

The generation of loose material depends both on the material properties and the compaction during construction. Adequate wet compaction reduces ravelling significantly (S F Poolman, pers comm). Materials with a shrinkage product of less than 100 have a 60 per cent chance of forming undesirable or unacceptable loose material, whilst those with a value in excess of 100 have only a 26 per cent chance. When the material becomes well sorted (i.e. poorly graded (Rosenak, 1963)) or becomes excessively coarse the materials are prone to ravel, as the interlocking capability of the particles diminishes. These two aspects are taken into account by the grading coefficient (difference between the per cent passing the 26,5 mm and 2,0 mm sieves x per cent passing 4,75 mm sieve/100). This function not only defines the quantity of gravel in the sample, but relates it to the sorting of the material. A value in excess of 34 results in ravelling even when the shrinkage product is more than 100.

7.6.3 Deformation resistance

The resistance to deformation results from the material having an adequately high stability (or strength) at the in-place density and moisture to avoid shearing, and having an adequate thickness and strength to spread the applied loads sufficiently to avoid over-stressing the subgrade.

The latter requirement is seldom a problem in southern Africa as most of the subgrades are sandy materials, and the water-table is generally deep. The subgrades on the test sections generally had similar laboratory soaked CBR values (Proctor compaction) (mean = 22; σ = 17) to the imported wearing course materials (mean = 25; σ = 17), but lacked the cohesion to form a hard wearing course in the unconfined condition. This type of sandy material is usually quick draining and therefore not subject to high pore-water pressures under loading. Very few examples of subgrade failure were recorded during the project, these generally being associated with deep potholes which were not repaired and thus retained water for prolonged periods, or thin wearing courses.

The strength of the material is generally defined in terms of the California Bearing Ratio (CBR) in most specifications. This test is, however, subject to a number of problems, especially when carried out on materials with a significant coarse fraction (in excess of 19,0 mm). The coarse fraction is either broken down and added to the finer material (NITRR, 1979 and 1986), replaced by finer material (Beaven et al, 1987) or left out all together (RRL, 1952; Country Roads Board, 1982; GEIPOT, 1980). The material strength obtained by any of these methods is not a true reflection of the actual strength. It is recommended that for unpaved road testing the test is only carried out on the material passing the 19 mm sieve, which will probably result in a slightly lower (but repeatable) value, as the coarser material in the road is likely to increase the stability and strength (Beaven et al, 1987).

In most cases where N is less than 5 the "worst" moisture condition is probably less than the condition at which the soaked CBR is carried out, apart from areas adjacent to potholes which retain water. Where N is greater than 5 the "worst" condition is unlikely to exceed the optimum moisture content except perhaps in the upper 25 mm of the road. However, certain areas which obtain continuous rain for prolonged periods may have soaked materials.

It was thought that the moisture contents in the wearing courses of the test sections at the time of sampling (field moisture content) (FMC) would vary considerably, unlike that of surfaced roads where an equilibrium is reached within about 2 years of surfacing (Emery, 1985). Where $N \leq 5$, the average field moisture content (FMC) to OMC ratio was 0,49 ($\sigma = 0{,}20$) and where $N > 5$ the average value was 0,39 ($\sigma = 0{,}22$). It must be noted that the sections with $N < 5$ were generally sampled towards the end of the wet season while the roads in the drier areas were sampled early in the dry season. However, roads in both areas were sampled in both sunny and rainy conditions. The maximum recorded FMC:OMC ratio was 102 percent with only three results over 100 percent. Emery (1985) found that the mean equilibrium moisture content (EMC) (the point at which the FMC equilibrates in paved roads) to OMC ratio varied between 0,71 and 1,05 ($\sigma = 0{,}29$ to 0,45) for subgrades, 0,7 to 0,83 ($\sigma = 0{,}26$ to 0,28) for subbases and

0,53 to 0,63 (σ = 0,16 to 0,24) for bases, under bitumen surfaced roads (average coefficient of variation = 0,38, 0,35 and 0,33 respectively). Where $N \leq 5$ the coefficient of variation for unpaved wearing courses is 0,41 and where $N > 5$ the value is 0,56. Significantly more variation occurs in the wearing courses of unpaved roads (as found by Visser, 1976). A more intensive investigation into the moisture changes in unpaved roads associated with rainfall and seasons is recommended.

It can, however, be stated with 95 percent confidence that the ratio of the field moisture content to the optimum moisture content will be less than 0,89 where $N \leq 5$ and less than 0,83 where $N > 5$. The difference in the CBR between a soaked test and one done at OMC or 90 per cent of the optimum moisture content is usually substantial, especially in those natural gravels containing an appreciable amount of clay, which results in particularly moisture sensitive materials. Some increases in the CBR between the soaked determination and the values at OMC and 90 percent of OMC (Proctor compaction effort) are shown in Table 7.6.

TABLE 7.6: MOISTURE SENSITIVITY OF SOME SAMPLES

Sample	Description	Soaked CBR	CBR at OMC	CBR at 90% OMC
10534	Weathered shale	6,9	16,5	30,0
10588	Fine ferricrete	1,4	5,3	15,0
10610	Coarse ferricrete	36,0	45,7	43,5
10677	Chert gravel	7,9	12,3	42,0
10654	Red powdery shale	12,0	16,0	46,5
10700	Decomposed granite	8,6	11,5	18,0
10985	Gravelly quartz porphyry	82,9	94,8	180,0

As none of the sections studied in the project (including heavily trafficked timber haul roads in wet areas) became impassable during the project, specification limits with respect to impassability cannot be derived solely from the results of this project. However, the fact that some of the sections had soaked CBR values (at Proctor compaction) as low as zero, confirms the conclusion that the specification of soaked CBR values is unnecessarily harsh for many conditions, and it is unlikely that even during periods of sustained rainfall wearing courses become fully saturated.

Measurement of the moisture contents on a number of compacted calcrete gravels (OMC = 7,2 to 12,7) after the usual 4 days soaking (NITRR, 1986) showed that the changes in moisture content during soaking depend strongly on the degree of compaction achieved. At Mod AASHO compaction the average change in moisture content was 1,85 percentage points (σ = 0,65) whilst at standard Proctor effort the average change was 4,2 percentage points (σ = 1,7). This is of course purely an indication of the extra moisture required to fill the air voids as all the samples were fully saturated after 4 days soaking. The importance of good compaction to preserve the stability of the wearing course, both by increasing the stability and decreasing the permeability and porosity, is thus confirmed.

The evidence indicates that in practice, except for the upper few centimetres, it is unlikely that a well-compacted unpaved road will become soaked even under severe rainfall conditions.

Although the CBR at 90 percent of OMC would be adequate 95 percent of the time under most conditions prevailing in southern Africa, the CBR at OMC should be used as the design criterion. This reduces both the time for testing and the number of moulds which need to be compacted as each mould compacted during the density determination can be penetrated and a moisture sensitivity versus strength curve can be obtained quickly and at minimum cost. The necessity for higher compaction can be judged from the moisture/strength relationships.

Visser (1981a) presented a plot of the soaked CBR (Standard Proctor compaction effort) against the traffic, and all materials with a CBR greater than 15 at 100 vehicles per day and greater than 18 for 500 vpd remained passable during the wet season. Specifications currently in use limit the soaked CBR from values as low as 15 at 93 per cent Mod AASHO density to not less than 60 at a 95 to 98 per cent BS compaction (heavy). This is an extremely wide range which could have serious cost implications during material selection.

Because none of the sections studied became impassable during the monitoring period, recourse to other available work in conjunction with the findings of this project was necessary in order to define limits for stability. Ahlvin and Hammitt (1975) have provided a nomograph for the relation between load, tyre pressure, CBR and repetition of load for unsurfaced soils in which a maximum rut depth of 50 to 75 mm can be expected (under nil-maintenance conditions). A second nomograph in which minimal distortion of the road (with no maintenance) is allowed uses the tyre pressure, stress repetitions and surface CBR strength. The CBR used in these nomographs is not clearly identified but is probably at Mod AASHO compaction.

Taking typical South African tyre pressures of 620 kPa (Van Vuuren, 1973) and 800 kPa (for modern radial ply truck tyres (A Thomas, pers comm)) these models (Figure 7.4 and 7.5) predict the following number of coverages possible for heavy vehicles (40 kN wheel load, normally the worst condition) and different material strengths (Table 7.7):

TABLE 7.7: PREDICTED NUMBER OF VEHICLE PASSES (40KN WHEEL LOADS) FOR DIFFERENT MATERIAL STRENGTHS AND TYRE PRESSURES (AHLVIN AND HAMMITT, 1975)

Material CBR	Figure 7.4		Figure 7.5	
	620 kPa	800 kPa	620 kPa	800 kPa
5	50	10	< 20	< 20
10	3 000	950	< 20	< 20
15	6 500	4 000	< 20	< 20
25	>10 000	>10 000	< 20	< 20
40	>10 000	>10 000	40	20
60	>10 000	>10 000	480	260
100	>10 000	>10 000	>10 000	6 000

It can be seen that provided some deterioration of the riding surface is allowed and maintenance is carried out, a material with a CBR of 5 in the worst condition prevailing is inadequate for the number of heavy vehicles typically using local roads during wet weather (80 TO 160 40 kN wheel loads (plotted on Figure 7.4), equivalent to the passage of some 2000 light vehicles) while a minimum CBR of 10 is adequate. If, however, no deformation is allowed and no maintenance is carried out, a CBR of about 50 at the worst condition prevailing would

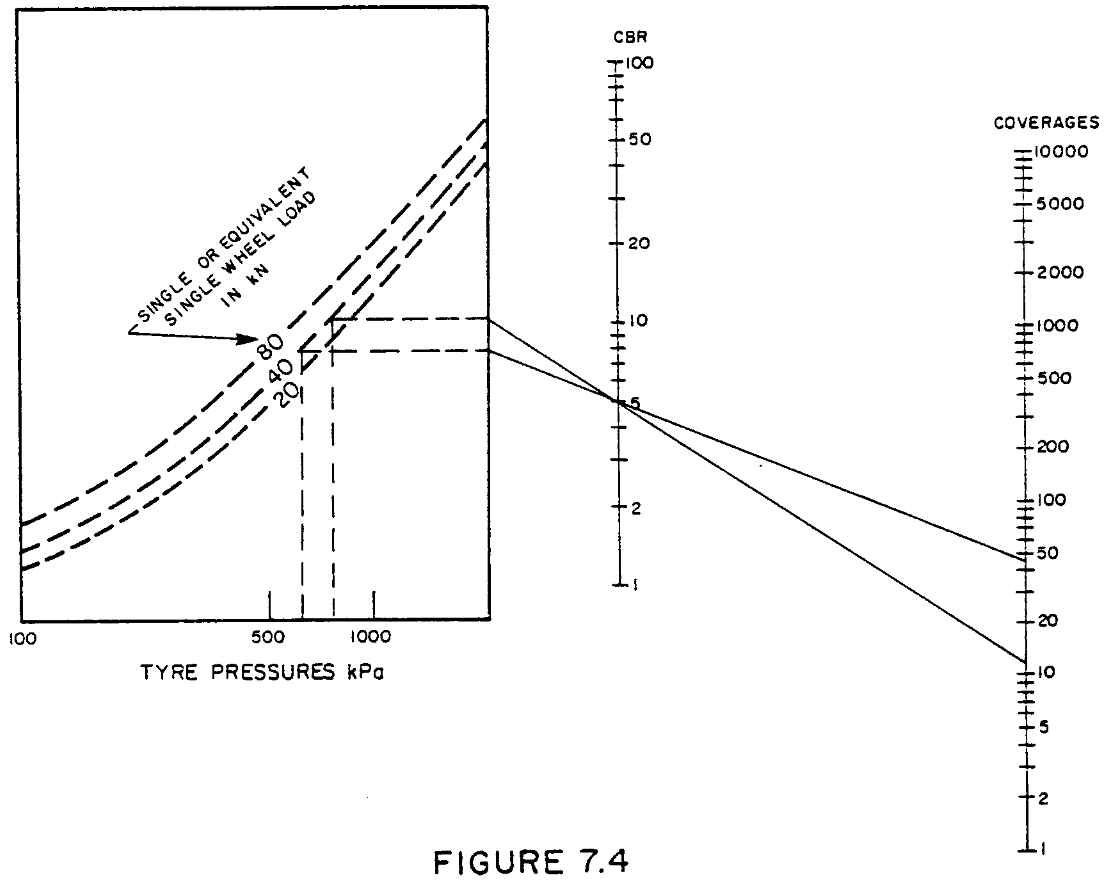


FIGURE 7.4

RELATION BETWEEN LOAD, REPETITIONS, TYRE PRESSURE AND CBR FOR UNSURFACED SOILS.

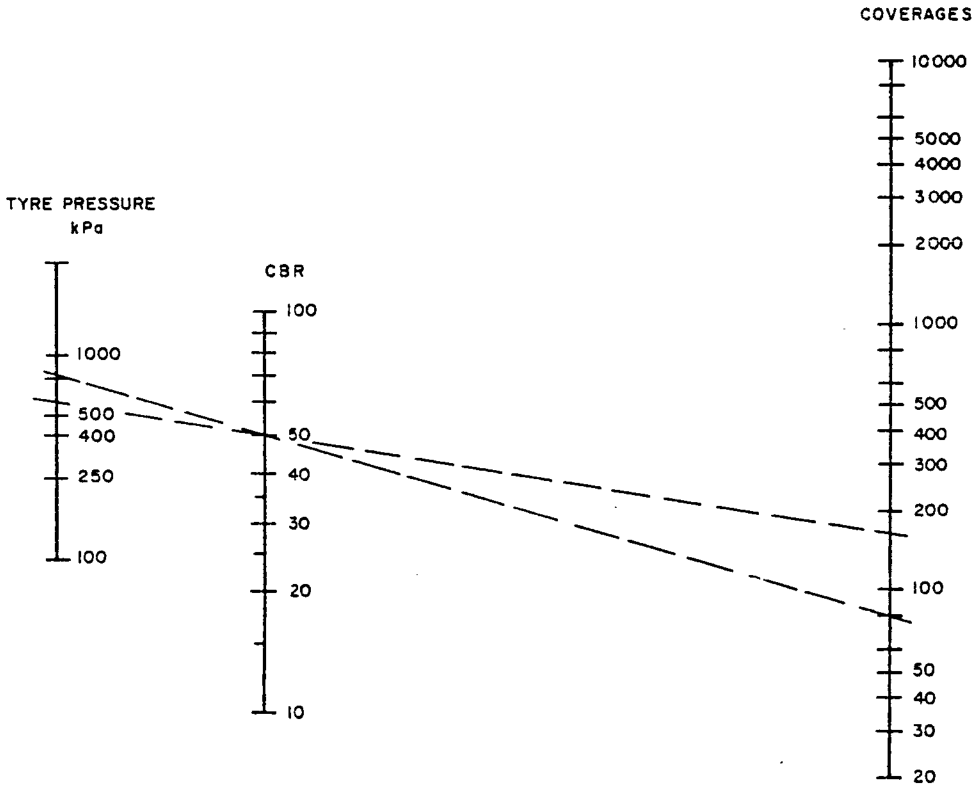


FIGURE 7.5

SURFACE STRENGTH REQUIREMENTS FOR AGGREGATE-SURFACED ROADS be required for between 80 and 170 40 kN wheel coverages (plotted on Figure 7.5). The actual CBR necessary, allowing some deformation before routine maintenance, is probably somewhere between these two figures and it is proposed that a value of 15 at 95 per cent Mod AASHO compaction and optimum moisture content is adequate for most roads.

It must be noted that this recommendation is based on the findings of Ahlvin and Hammitt (1975), Visser (1981) and the local monitoring programme. Many of the roads with soaked 95 per cent Mod AASHO CBR values of much less than 15 proved successful in practice and the recommendation of 15 probably carries a significant factor of safety for many climatic and traffic conditions.

7.6.4 Water shedding ability

The water shedding ability is extremely important as it affects the material strength and slipperiness in wet conditions. It is mainly a function of the shape of the road (a maintenance problem) and the compaction density achieved. Water should not be allowed to accumulate on the surface of the road or be retained long enough to soak in to the layer. High quality maintenance grading is necessary to ensure an adequate cross-fall, and eliminate ruts, potholes and other forms of ponding.

The wearing course material should be erosion resistant, especially on vertical grades. Run-off down poorly shaped grades often results in the water accumulating in the centre of the road and causing extensive scouring. On slopes steeper than about 4 per cent, any extra cost incurred in reducing erosion will soon be recovered by the necessity for less frequent regravelling. The crossfall should be maintained at about 4 per cent to allow adequate run-off but not at velocities which will cause erosion. Excessive super-elevations on corners are often the cause of erosion across the road (i.e. down the camber). They should be limited to a maximum of about 8 per cent.

Adequate compaction results in a tight, dense layer which retards the penetration of water and resists erosion significantly better than loosely compacted materials. Materials with a grading coefficient of more than 34 will generally not compact to a dense layer as a definite gap-grading occurs in the gravel component. Compaction should, however, always be in excess of 95 per cent Mod AASHO, and moist compaction under suitable rollers is essential. The present practice of a limited number of passes with a grid roller followed by traffic compaction is unacceptable for optimum performance, and further research into the optimum compaction procedures and standards is necessary.

Materials with a grading coefficient of less than 16 (these are generally fine-grained, poorly graded soils) are particularly susceptible to erosion and should be avoided in wet areas.

7.6.5 Abrasion resistance

The abrasion resistance of the wearing course is mainly influenced by the cohesion and particle interlock of the material. The use of materials which will resist abrasion results in a reduced necessity for regravelling, the major maintenance costs associated with unpaved roads. All those materials with a shrinkage product of less than 100 tend to ravel under traffic and should be avoided. The influence of the strength of the aggregate particles on the performance has previously been shown to be important (Netterberg, 1978). However, the present study has indicated no relationship between the aggregate strength and the abrasion loss on the road. This confirms the findings of Von Solms (1987).

7.6.6 Dustiness

Dust from unpaved roads should be reduced as far as possible. Dust is a major contributor to poor safety (in terms of visibility), vehicle occupant discomfort, the wear of vehicle components (such as universal joints) and has an unquantified but significant effect on roadside vegetation and agriculture (Van Barneveld, 1984). During monitoring of the sections, every section had dustiness ratings of 5 (unacceptable) at some stage, usually shortly after grader blading. The dust rating for all the sections generally showed a strong mode, this being the characteristic dust rating for the section.

Attempts to eliminate the dust through the selection of relatively dust-free materials should be made but it is often not possible to locate suitable materials within reasonable haul distances. Very few materials are dust-free and little correlation could be found between dustiness and material properties. Materials with a shrinkage product of more than 240 have only a 30 per cent chance of having a dustiness of 4 (undesirable (Paige-Green, 1988a)) or less. If the shrinkage product is less than 100 this probability increases to 40 per cent and with a value of between 100 and 240 the probability is about 60 per cent.

It is often difficult to find adequately dust free materials and recourse should be made to some of the available dust palliatives (e.g. ligno-sulphonates, magnesium chloride, etc). Although it is difficult to analyse the effectiveness of dust palliatives in terms of the cost/benefit ratio, the social consequences of dusty roads, for example in townships are extremely important (i.e. the benefits of dust reduction or elimination are very high) and should be taken into account. Further research into the different dust palliatives and their economics is, however, long overdue.

7.6.7 Slipperiness

The slipperiness of a dry unpaved road is affected by the presence of fine rounded gravel, which acts as "ball bearings" between the vehicle and the road. None of the roads studied, however, was slippery under normal driving conditions during the project. The slipperiness in wet weather, on the other hand, is affected mainly by the clay and fine material content of the wearing course, and the water shedding ability of the road. The influence of the aggregate larger than 37,5 mm has been shown to be of relatively minor importance in terms of slipperiness.

Those materials with a shrinkage product of more than 370 tend to become slippery when wet irrespective of their grading coefficients. A maximum particle size of 37,5 mm is adequate to provide stability in the upper layers of the wearing course and prevent the shearing and churning of the top layer under the tractive forces of vehicle wheels.

Aggregate of this size contributes to the skid resistance, without influencing the roughness of the road or causing excessive tyre wear.

7.6.8 Cost considerations

The materials should be locally derived with minimal haul distances, but the cost of providing a material with an adequate performance should be balanced against the life-cycle cost of the road (including road user costs). In most cases the cost of hauling better materials further distances, or blending or processing poor local materials is rapidly offset by the reduced blading maintenance and road user costs (Szkutnik, 1983) and a longer period before regravelling is required.

The exclusion of large stones simplifies the blading maintenance considerably. The frequency of blading is reduced, the pothole-filling and oversize removal is minimal and wear on the grader-blades is significantly reduced.

7.6.9 Suited to prevailing conditions

The choice of materials (and design of the road) should be suited to the local environmental and traffic conditions. The specifications are developed in such a way that materials outside the recommended limits may be used under certain conditions e.g. erodible or slippery materials may be suitable in arid areas (with adequate warnings for road users) or material likely to ravel or corrugate may often be acceptable for very lightly trafficked roads.

7.7 DEVELOPMENT OF SPECIFICATIONS

As well as fulfilling all the requirements of ideal wearing course materials the specifications were developed to comply with as many of the requirements of good specifications as possible (see Sec 7.2).

It was considered that any new specifications should be use-specific and should allow different limits for the different classes of service provided. The following classes of unpaved roads were identified:

- Rural roads
- Urban roads
- Haul roads

7.7.1 Rural roads

Unpaved rural road links are are mainly farm to market or intervillage roads, generally measured in tens of kilometres and should as far as possible be smooth, safe, comfortable all weather roads which cause minimal disturbance to the environment (both in terms of the dust emission and scarring of the countryside from old borrow pits) while requiring minimal maintenance. This is obviously a rather tall order but certain of the sections studied during the project certainly fulfilled all of these requirements.

The following specifications for materials for rural roads are proposed (Table 7.8):

TABLE 7.8: RECOMMENDED MATERIAL SPECIFICATIONS FOR UNPAVED RURAL ROADS

Maximum size:	37,5 mm
Oversize index (I_o):	≤ 5 per cent
Shrinkage product (S_p)[a]:	100 - 365 (max. of 240 preferable)
Grading coefficient (G_c)[b]:	16 - 34
CBR:	≥ 15 at ≥ 95 per cent Mod AASHO compaction and OMC[c]

a S_p = Linear shrinkage x per cent passing 0,425 mm sieve
b G_c = (Per cent passing 26,5 mm - per cent passing 2,0 mm) x per cent passing 4,75 mm)/100
c tested immediately after compaction

The specifications for shrinkage product and grading coefficient are shown schematically in Figure 7.6 together with the actual data points on which they are based. The CBR requirements are not shown as they were derived from a different source and are in effect a second

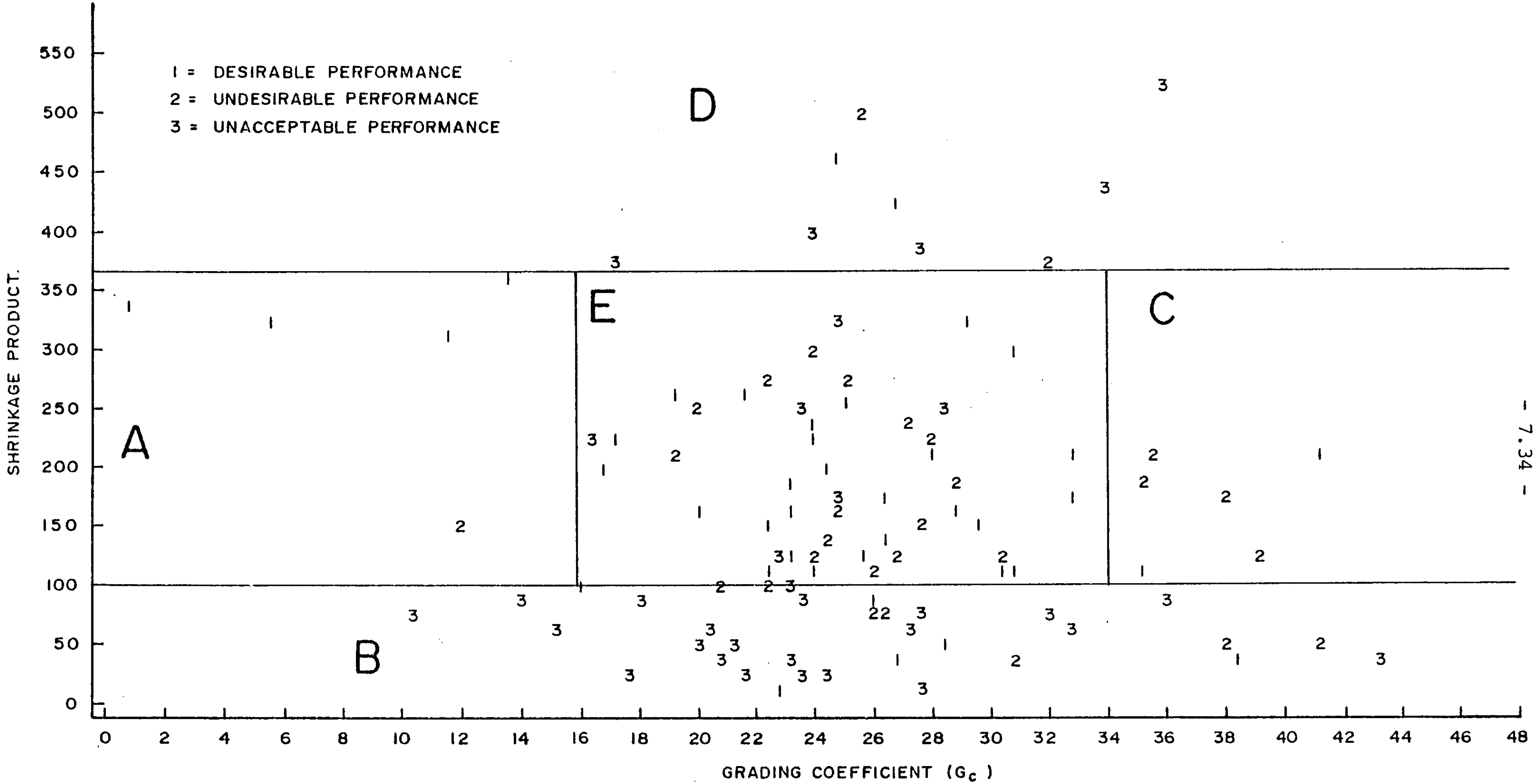


FIGURE 7.6 *RELATIONSHIP BETWEEN SHRINKAGE PRODUCT, GRADING COEFFICIENT AND PERFORMANCE*

performance criterion (passability). The following conclusions can be drawn about each zone as defined in the figure:

A - Materials in this area generally perform satisfactorily but are finely graded and particularly prone to erosion: they should be avoided if possible, especially on steep grades and sections with steep cross-falls and super-elevations. Eighty per cent (four) of the sections in this zone were classified as providing a desirable performance (but required constant labour intensive maintenance over short lengths and had high gravel losses) and 20 per cent (one) section as undesirable.

B - These materials generally lack cohesion and are highly susceptible to the formation of loose material and corrugations. Sixty three per cent (22) of the sections in this zone were classified as unacceptable while 17 per cent (6) of the sections which were desirable fell into this zone. All of these materials did, however, form corrugations and loose material, but the roads were wide enough to accommodate lateral movement of the traffic or were maintained prior to the full development of the corrugations. Twenty per cent (7) of the sections performed undesirably, mainly because of corrugations. These sections were generally on corners or near intersections where the traffic was moving at a slower speed resulting in less well-developed corrugations.

C - These materials are generally comprised of fine, gap-graded gravels lacking adequate cohesion, which results in ravelling and the production of loose material. Two of the sections were desirable, four undesirable, but none was unacceptable.

D - Materials with a shrinkage product in excess of 365 tend to be slippery when wet. Five of the sections in this zone performed unacceptably (due to stoniness in four cases and potholes formed by excessive cracking in the other) whilst two performed undesirably and two desirably.

E - Materials in this zone perform well in general provided the oversize material is restricted to the recommended limits. Six of the eight samples performing unacceptably contained excessive oversize material while the other two had grader induced corrugations (3 m wavelength) which remained throughout the duration of the project. Sixteen of the sections (32 per cent) performed undesirably, most of them being dusty or containing excessive oversize material, but less than those classified as unacceptable. Fifty two per cent of the sections in this zone performed desirably.

The above specifications accept a number of materials with unacceptable dust but 13 roads which were not classified as unacceptable in terms of their overall performance would be eliminated by lowering the shrinkage product to 240. This was considered unnecessarily harsh for rural roads. Attempts should be made, however, to locate materials with a shrinkage product of less than 240 as far as possible.

For very lightly trafficked rural roads (less than 50 vpd) the requirements can be relaxed and a higher level of dust and roughness can be tolerated. On economic grounds it would be hard to justify the maintenance of roughness at levels less than 100 QI counts/km on roads carrying less than 50 vehicles per day. Social, political or other reasons (e.g. tourism) may of course dictate otherwise. The maximum acceptable roughness for the lightly trafficked roads can be taken as the limit of acceptability determined in the acceptability criteria experiment (i.e. 200 QI counts/km (Paige-Green and Netterberg, 1988)). None of the sections had 90 percentile values for the roughness in excess of 200 counts/km, implying that all the roads were acceptable for light traffic 90 per cent of the time, even under conditions of reduced grader maintenance (only three times per annum in most cases). An analysis in terms of road user costs would, however, be necessary for each road in order to delineate the exact limit of traffic for which <u>almost</u> any material could be used. In terms of the dustiness, for a road carrying less than 50 vpd (i.e. less than 5 vehicles per daylight hour on average, with minimal vehicle interaction) the discomfort and safety aspects associated with dust, although undesirable, are generally tolerable for even the dustiest roads, and a limit for the shrinkage product of 365 is allowable. Materials for these roads should, however, be selected to comply as closely as possible with the specifications, especially the maximum size which is the only property which can be relatively easily controlled.

Figure 7.7 shows plots of the performance of the sections by climate and traffic and Figure 7.8 shows the performance of the sections by material group. Little correlation between the performance and material groups is evident. The plots can, however, be used to obtain site-specific information depending on the prevailing traffic and climate and the proposed material group to be used.

7.7.2 Urban roads

Unpaved urban roads generally consist of a network of short road links and need primarily to provide a dust-free surface which will not become excessively loose during dry weather or muddy during wet weather and which requires minimal maintenance. An important positive factor in these roads is that very few heavy vehicles use them whilst vehicle travel speeds are generally low, and should be encouraged to remain low, as pedestrians and children often use these roads as paths and playgrounds. Their main use is as residential access roads within townships in developing areas and not as arterial or major collector streets which carry buses and delivery vehicles (these roads should be constructed to normal paved road standards (NITRR, 1985b; Department of Development Aid, 1987). It is recommended that the access roads be restricted to normal vehicles by using cul-de-sac type roads (or less desirably loop roads) serving a limited number of residences, with no through traffic.

The driver and passenger comfort consideration is therefore of relatively minor importance and the main objectives are to eliminate as far as possible the dustiness, drainage and erosion problems and muddiness when wet at minimum cost. The formation of loose material should be reduced as far as possible.

The following specifications are proposed for urban roads (Table 7.9):

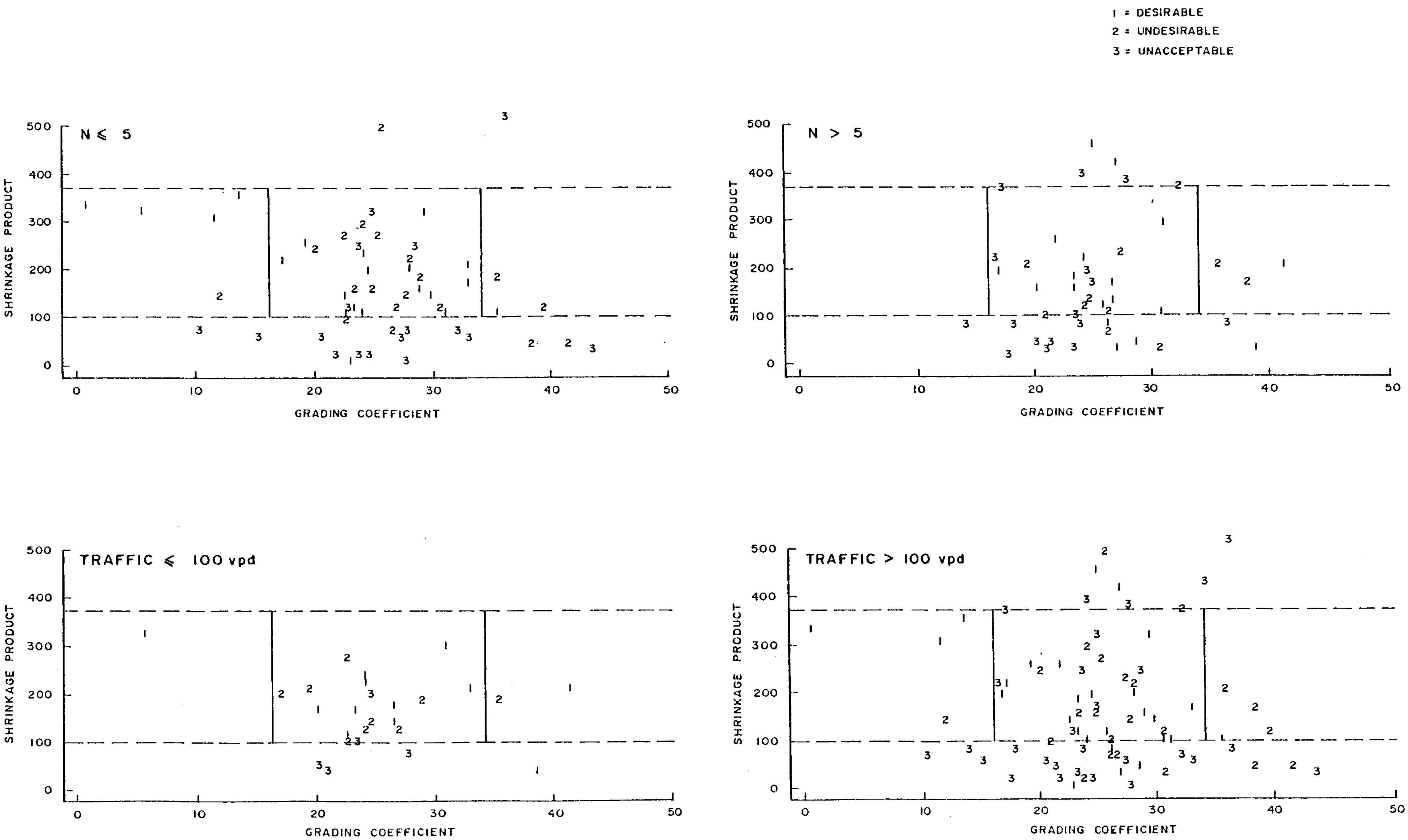


FIGURE 7.7 *PLOTS OF PERFORMANCE BY TRAFFIC AND CLIMATE*

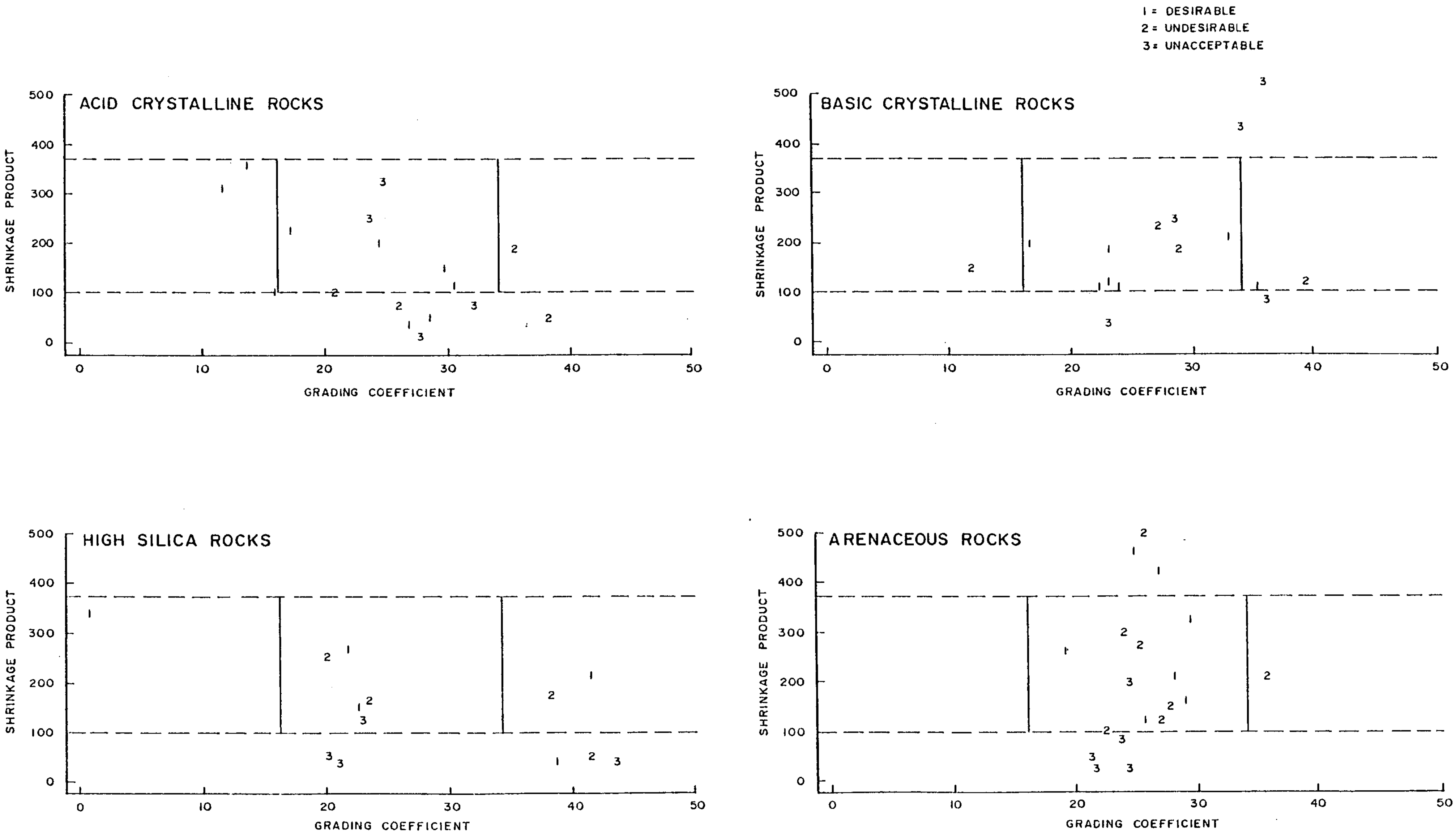
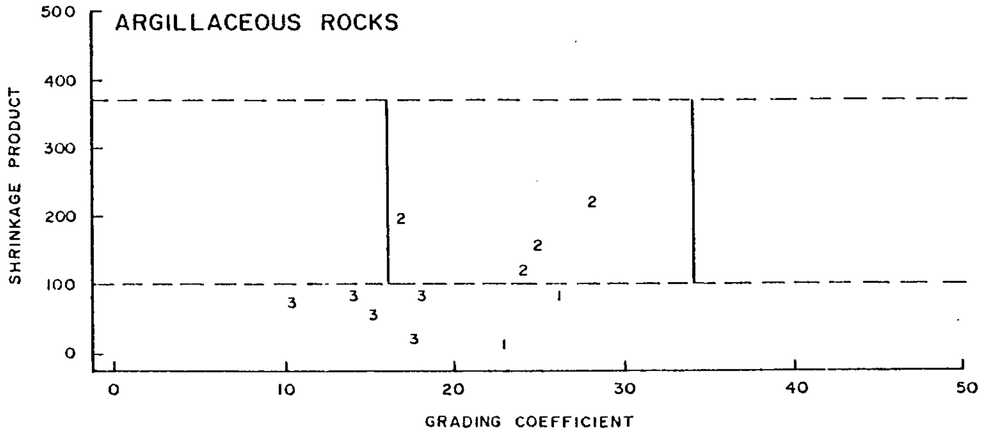
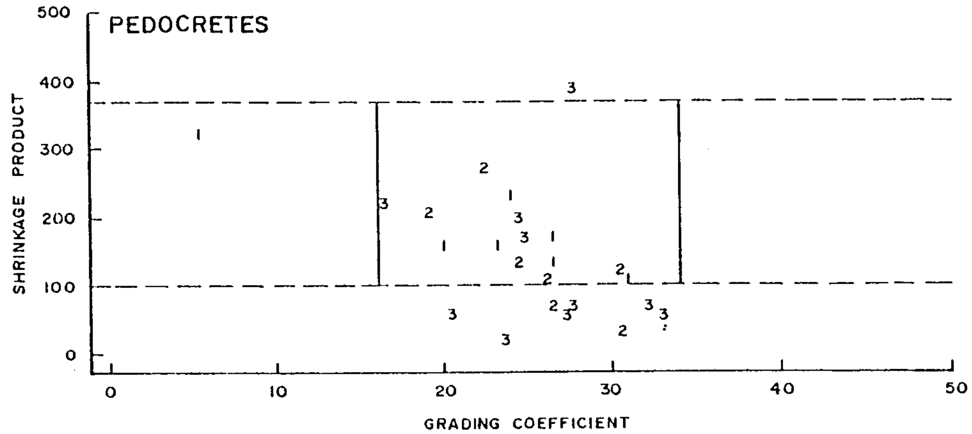
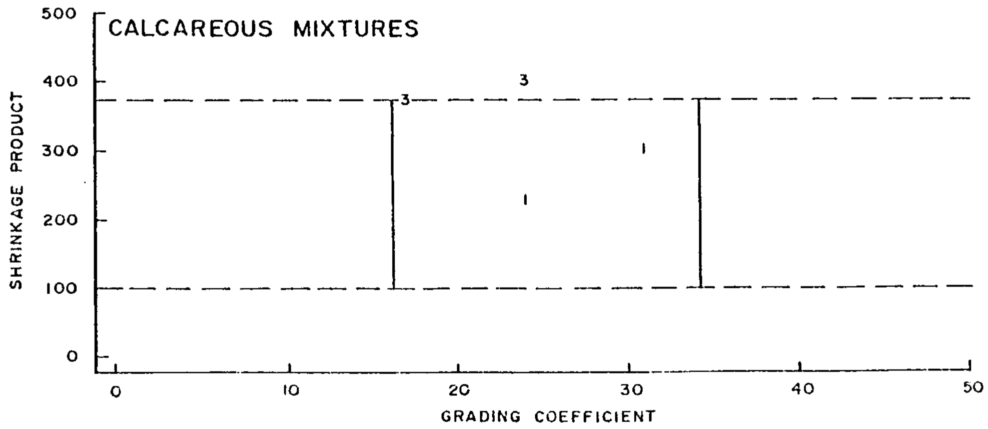


FIGURE 7.8 *OVERALL PERFORMANCE OF TEST SECTIONS BY MATERIAL GROUP*







1 = DESIRABLE
2 = UNDESIRABLE
3 = UNACCEPTABLE

FIGURES 7.8 (CONT) *OVERALL PERFORMANCE OF TEST SECTIONS BY MATERIAL GROUP*

TABLE 7.9: RECOMMENDED MATERIAL SPECIFICATIONS FOR UNPAVED ROADS IN URBAN AREAS

Maximum size:	37,5 mm
Oversize index (I_o)	0
Shrinkage product (S_p):	100 - 240
Grading coefficient (G_c):	16 - 34
CBR: ≥ 15 at ≥ 95 per cent Mod AASHO compaction and OMC	

In comparison with the limits for rural roads it can be seen that the limits for the oversize index have been reduced to eliminate stones whilst the shrinkage product has been reduced to a maximum of 240 to reduce the dust as far as practically possible.

7.7.3 Haul roads

Unpaved haul roads are generally restricted to fairly limited lengths in forests, quarries and mines, but are required to be always passable for high numbers of very heavy vehicles at minimum total cost (i.e. sum of construction, maintenance and user costs) and not excessively slippery when wet. The materials used for the construction of haul roads should therefore have an adequate strength under the "worst" moisture conditions likely to occur in the area and be able to support the high wheel loads and tractive forces common to the vehicles using these roads. Obviously, an element of risk needs to be considered in this decision, in order to balance the "worst" moisture condition with the cost of obtaining better materials.

Slipperiness in the wet is an important factor on mine and forestry haul roads where significant vertical grades are common, mainly from the service viewpoint of hauling up steep slopes, but also from the safety aspect of vehicles descending steep slopes. Slippery roads which provide minimal traction for vehicles result in churning and loosening of the surface with the production of increasing thicknesses of weak material which leads to poor trafficability.

Dust should be eliminated as far as possible for safety and vehicle component wear reasons.

The material selected for an unpaved haul road should therefore have the following properties (Table 7.10):

TABLE 7.10: RECOMMENDED MATERIAL SPECIFICATIONS FOR UNPAVED HAUL ROADS

Maximum size:	75 - 100 mm
Oversize index (I_o):	≤ 10 per cent
Shrinkage product (S_p):	100 - 365 (max preferably < 240)
Grading coefficient (G_c):	16 - 34
CBR:	≥ 15 at ≥ 95 per cent Mod AASHO compaction and 4 days soaking

An increase in the maximum size and Oversize index (within limits) has been proposed as haul-vehicles generally have large tyres (often with lower tyre pressures) and travel at slower speeds than on rural and urban roads. Road user costs will therefore not be as sensitive to the stoniness. A soaked CBR is recommended in order to allow for the greater traction forces exerted by the heavily-loaded vehicles usually associated with haul roads. Vertical grades on haul roads are usually steeper than those found on rural and urban roads.

7.8 DISCUSSION

The simplicity and brevity of the new specifications (Tables 7.8, 7.9 and 7.10) is significant when compared with existing material requirements for unpaved roads. Only a simple bar linear shrinkage test, a compaction and CBR test and a grading analysis through the 37,5 mm, 26,5 mm, 2,0 mm and 0,425 mm sieves are necessary for acceptance testing. When using only these sieves it is often not necessary to wash the materials unless they are very clayey: brushing of the fractions retained on each sieve with a hard bristle brush is adequate.

One problem with nearly all material specifications is that the specifications apply to the material after compaction (NITRR, 1985a). Wearing course materials are probably more likely to degrade during construction than say base or subbase materials, due to their weaker nature. If the shrinkage product is borderline the particles retained on the sieves with openings greater than 0,425 mm should be examined and an estimate of the potential increase in the per cent passing the 0,425 mm sieve (and hence shrinkage product) made before the material is accepted. The use of a durability mill test (Sampson and Netterberg, 1988) may be a useful indicator of potential degradation during construction and deserves investigation.

Climate and traffic are conspicuous by their absence in these specifications. The experiment was designed to investigate the influence of these factors on the performance of unpaved roads and although they appear in the prediction models for gravel loss and roughness progression no improvement in the specifications could be achieved by incorporating them.

A number of the existing specifications incorporate the aggregate strength. The main reason for this is to limit the coarse aggregate which will break down under traffic and often lead to powder potholes (Netterberg, 1978). Analysis of the influence of aggregate strength showed no correlation with performance, with both hard and soft aggregates (in terms of the AFV and APV) giving good and bad performance.

It is interesting to note that the limits for the shrinkage product are very similar to those of Netterberg (1978) which were determined from the performance of a number of calcretes in the western Transvaal and South West Africa.

Many of the recent specifications (NITRR, 1985a; DOT, 1985; TPA, 1983) only specify limits for the grading, plasticity and strength. The problems caused by oversize material in the existing method for particle size analysis described before may have a significant influence on the grading results and affect the acceptance or rejection of materials.

The importance of good construction and maintenance procedures cannot be overemphasised. The moist compaction, to densities in excess of 95 per cent Mod AASHO, alone results in significant improvements in the performance of unpaved roads. It is important to provide an adequate crown and a definite crossfall during construction and to maintain it. The crossfall should be about 4 per cent which allows water to drain rapidly, thereby reducing the possibility of the material becoming saturated and shearing under traffic, but not enough to cause erosion. Adequate side-drains should be provided to remove the water from the vicinity of the roadway.

The occurrence of periodic flooding in South Africa should not be underestimated. During the extensive flooding at the end of 1987, numerous unpaved roads were closed to traffic for extended periods. Discussions with some of the Authorities involved indicated that the problems were not impassability but extensive erosion and wash-aways. If the drainage is inadequate even the best materials will not perform satisfactorily. Those roads constructed of materials with marginal CBR strengths at OMC should be noted and during periods of extensive flooding, the traffic should be limited as far as possible or the roads closed.

Compliance with requirements of a good wearing course material.

If the specifications and construction requirements are followed the road should fulfil all the requirements of an ideal wearing course as specified previously (See Chap 2.5).

Compliance with requirements of good specifications.

The new specifications which have been developed comply with all the requirements for good specifications defined earlier (Sec 7.2) except for perhaps not accepting all suitable materials. Although all the test results available were used in the analysis, no single test or combination of different tests predicts the performance of all materials with 100 per cent confidence. The specifications as defined do, however, accept most of the materials which will perform well and reject all of the materials which perform unacceptably. Those existing roads which perform undesirably but are apparently constructed of materials complying with the specifications (except for the oversize criterion) can generally be improved by better compaction, removing the oversize material or programmed maintenance.

Reliability

The specifications as proposed reject all unacceptable materials (after accounting for the stoniness and maintenance problems) and 13 (12 per cent) of the materials rated as acceptable. Although these materials were rated as having an acceptable weighted overall performance over the duration of the project, they should not normally be used without examining the consequences of their use, as they were prone to corrugations, ravelling, erosion and slipperiness which required labour intensive maintenance or was temporarily undesirable from the safety viewpoint. Sixteen materials (15 per cent) which were rated as undesirable would be accepted by the specifications. Removal of the oversize material, routine maintenance (the sections were not maintained for 4 months or longer) and acceptance of a higher dust level would result in their being rated as acceptable. Twelve (11 per cent) of the sections rated undesirable are rejected by the specifications. Seven of these are susceptible to erosion, slipperiness or ravelling while 5 formed corrugations, but were graded regularly or occurred on wide roads which allowed lateral movement of vehicles.

The specifications therefore reject all materials susceptible to corrugations, erosion, ravelling and slipperiness and accept all materials which will provide a good riding quality and minimal loose material. A number of dusty materials are, however, accepted by the specifications. If the shrinkage product is limited to between 100 and 240, 60 per cent of the sections will have a dust rating of 4 (undesirable but acceptable) or less, but if the limit is extended to 370, 50 per cent of the sections have unacceptable dust.

In general, the proposed specifications are apparently a marked improvement on the specifications currently used locally (Table 7.11).

TABLE 7.11: COMPLIANCE OF STUDY SECTIONS WITH EXISTING AND NEW SPECIFICATIONS

Specific-ation	Sections complying (%)				Sections not complying but giving desirable performance (%)
	Total	By performance			
		Desir-able	Undesir-able	Unaccept-able	
TRH14	12,2	4,7	5,6	1,9	32,6
DOT	2,7	0,9	0,9	0,9	36,4
TPA	2,7	0,9	0,9	0,9	36,4
NPA	22,5	7,5	10,3	4,7	29,8
OFS	4,5	1,8	1,8	0,9	35,5
CPA	14,0	7,5	5,6	0,9	29,8
Botswana	3,8	1,9	1,9	0	33,6
Lesotho	0	0	0	0	37,3
Swaziland	15,0	5,6	4,7	4,7	31,7
SWA	3,6	0,9	1,8	0,9	36,4
Netterberg	29,0	14,1	10,2	4,7	23,2
New specs	47,0	28,0	15,0	4,0	12,0

7.9 CONCLUSIONS

Numerous specifications for unpaved roads are currently in use in southern Africa, none of which are successful for all geological materials. Performance-related specifications have thus been derived for the identification of suitable gravel wearing course gravels.

The specifications are simple, requiring minimal testing (a quick grading analysis, a linear shrinkage test and a CBR) and should be valid for various types of road carrying any traffic in any climatic area (scientifically valid for practically all of southern Africa). The importance of removing the oversize material, limiting the shrinkage product to the specified values and adequately compacting the wearing course has been clearly identified.

In order to ensure all weather trafficability, an adequate material strength is required. As none of the sections on which the specifications were developed became impassable during the project, tentative limits based on theoretical considerations and empirical studies elsewhere have been proposed.

CHAPTER 8

PRACTICAL IMPLICATIONS OF FINDINGS

8.1 INTRODUCTION

The practical implications of the findings of this project are extremely significant, affecting all the costs associated with unpaved road networks (some increasing but most of the significant ones decreasing). Socio-economic and socio-political implications are equally important. Application of these specifications during the materials location and construction of unpaved roads in southern Africa will result in both improved riding quality and performance and reduced maintenance costs. This will result in substantial maintenance savings and a better satisfied travelling public. In addition significant savings on road user costs due to smoother roads will accrue in the national interest.

8.2 MATERIAL LOCATION AND SELECTION

Optimum use of the proposed specifications which have been developed will obviously necessitate increased attention to materials location and testing prior to the construction or regravelling of unpaved roads. Presently many of the materials used for unpaved roads are obtained from old borrow pits, most of which provided adequate materials in the past. However, traffic has increased, both in quantity and mass, vehicles developed typically for local conditions tend to have harder suspensions than the older imported vehicles and axle weights on many of these unpaved roads are in excess of the legal limits. Many of the older borrow pits have been extended beyond their original specified limits and to much greater depths resulting in a deterioration of the material as a gravel wearing course. Many of the good wearing course gravels have been depleted or are rapidly running out.

In order to optimise the maintenance and road user costs it is imperative that the best material is utilised and this requires adequate prospecting by experienced staff, confirmation of the material properties by adequate laboratory testing, careful delineation of the areal limits and depth of proposed borrow pits, and adequate supervision during excavation.

The implications of this extra work are increased costs. An estimate of the increase in costs is difficult as existing costs are difficult to obtain and the cost will depend on the state of existing borrow pits, the length of the road to be built or regravelled and the experience of the prospector. Many materials are located by field technicians of the Provincial Roads Departments who probably spend about a week in the field for a job. The prospecting is generally based on experience (often extensive, but more recently becoming less as older staff retire) with very little recourse to modern techniques.

The use of an experienced engineering geologist will result in the use of modern techniques (air photo interpretation or other remote sensing techniques where they already exist) with a sound scientific basis. It is unlikely that the increase in cost will exceed about R5 000 or R10 000 for a typical 30 or 40 km rural road (i.e. R250 to R333 extra per kilometre).

Methods for the location and identification of potential wearing course material borrow pits need to be researched further. A survey of existing borrow pits from which good wearing course materials have been obtained should be carried out. As the geological material is not an adequate indicator of potential suitability of the material, aspects such as the land form (NITRR, 1978) should be investigated. A great deal of useful information is probably available from existing data bases (NITRR, 1978).

In many cases the material will not comply exactly with the proposed specifications, but it would certainly be very close.

Presently, minimal testing is carried out on proposed borrow pits. Normally five samples (one from each corner and one from the centre) are tested in the laboratory. With the newly proposed specifications, substantially more testing can be carried out for the same price as only values for the bar linear shrinkage, a dry grading (percentage passing the 37,5, 26,5, 4,75, 2 and 0,425 mm sieves) and a CBR at 95 per cent Mod AASHO compaction and optimum moisture content are required. It is recommended that CBR penetrations are carried out on all the density moulds to get an idea of the moisture/density/strength sensitivity analysis. If no laboratory is available DCP testing of proof rolled sections may be considered. Testing will be significantly quicker because of the reduced preparation for grading and shrinkage and as the CBR moulds do not have to soak for four days.

8.3 MATERIAL PREPARATION

Once the best available material is located the borrow pit should be opened and adequate material for the proposed job excavated and stockpiled. This material is unlikely to meet each of the requirements specified in Chapter 7.6 and will therefore require some additional work in the borrow pit as follows:

a) If the shrinkage product of the material is too high, the material should be blended with a lower plasticity material to dilute the shrinkage. The fines content (passing 0,425 mm sieve) can also be adjusted by blending to correct the shrinkage product.

b) If the shrinkage product is too low, plastic or other finer material should be added to correct it.

c) If the grading coefficient is incorrect blending of a suitable gravel is necessary to correct it. It should be noted that the grading coefficient is a product of the percentage gravel and the percentage passing the 4,75 mm sieve. Blending should take into account the cause of the material being outside specification e.g. the gravel content is high but it is too fine.

d) The gravel may contain excessive oversize material. This should be removed in the borrow pit. Probably the most cost effective way is by labour intensive hand-picking. If a large proportion of oversize is present and a large quantity of material is to be processed it may be economic to crush the material instead of wasting most of it. Other means of overcoming problems with oversize material are by screening (Grizzly), rock rakes (Gallup, et al, 1974) or "ROCK-BUSTER" type hammer mills (Gallup et al, 1974; Fisher, 1979). Economics dictate that excessive oversize material should never be hauled to the road unless it is to be "Rockbusted". Even this is preferably done in the borrow pit. Many instances were observed during the project where large boulders were hauled to the road with the (false) expectation that they could be broken under a grid roller. This usually resulted in the boulders being embedded into the subgrade during compaction and protruding through the wearing course soon after trafficking of the road.

A general economic analysis of the increased costs of material processing is of no benefit as each case will depend on unique factors e.g. proximity and availability of equipment, quantity of material to be processed, percentage of oversize material, haul distance of material for blending, etc). The use of manual labour for hand-picking of oversize material in the borrow-pit is, however, unlikely to increase the cost of the material significantly.

8.4 PAVEMENT DESIGN

Numerous design procedures for unpaved roads have been published (Hammitt, 1969; Ahlvin and Hammitt, 1975; Greenstein et al, 1981; Alkire, 1987; Hudson et al, 1987; Mathur et al, 1988). None of these design procedures is applicable to local conditions as they mostly take the failure criterion as a rut depth of 75 mm, this being considered excessive for local urban and rural roads.

A design procedure for unpaved roads has been proposed by Paige-Green (1988c) and will not be discussed fully here. The proposed procedure optimises the material thickness to avoid subgrade overstressing, to provide the required design life and to account for the loss of thickness due to traffic compaction. No major financial gains or losses result from the design, but the new design method together with the ability to estimate the life of the wearing course, assists greatly in the future programming of maintenance and rehabilitation of the road network. For the first time in South Africa, a scientific basis for the design of unpaved roads is available. Previously, nearly all unpaved roads were built with a nominal thickness (usually 150 mm) of gravel. The design can now take the effects of material quality, traffic, climate, construction procedures and subgrade strength into account to provide a road with a known life before rehabilitation is necessary.

8.5 CONSTRUCTION CONSIDERATIONS

The use of the proposed specifications will probably result in a minimal increase in the construction cost. Other recommendations of this thesis will, however, result in significant increases in the construction cost compared with the present situation but will result in substantial savings in maintenance and road user costs.

The only increase in construction cost arising from the new specifications will probably be the result of increased haulage costs. Where the nearest available "suitable" material is used at present, the proposed specification may result in greater haulage distances as the borrow pits may be further from the section being constructed.

The present cost of gravel (assuming a nominal haulage of two or three kilometres) is about R6/m³ (about R6 300 per kilometre for a standard 7 m wide 150 mm wearing course). An extra haul cost of 40 c/m³ km would increase this cost to about R10 000/km if the haul distance was taken as a conservative 10 km (i.e. an increased cost of some R4 000/km).

The importance of moist compaction has been clearly shown by Poolman (1988). Adjacent sections of "identical" material were constructed in the Namib desert. Three sections were moist compacted and two were dry compacted (the standard practice over much of the arid part of southern Africa). The performance of the sections as monitored is shown in Figure 8.1 (after Poolman, 1988).

It is clear that after four months of traffic (about 35 vpd) the wet compacted sections were in a very good condition while the dry compacted sections were extremely rough and should have been bladed

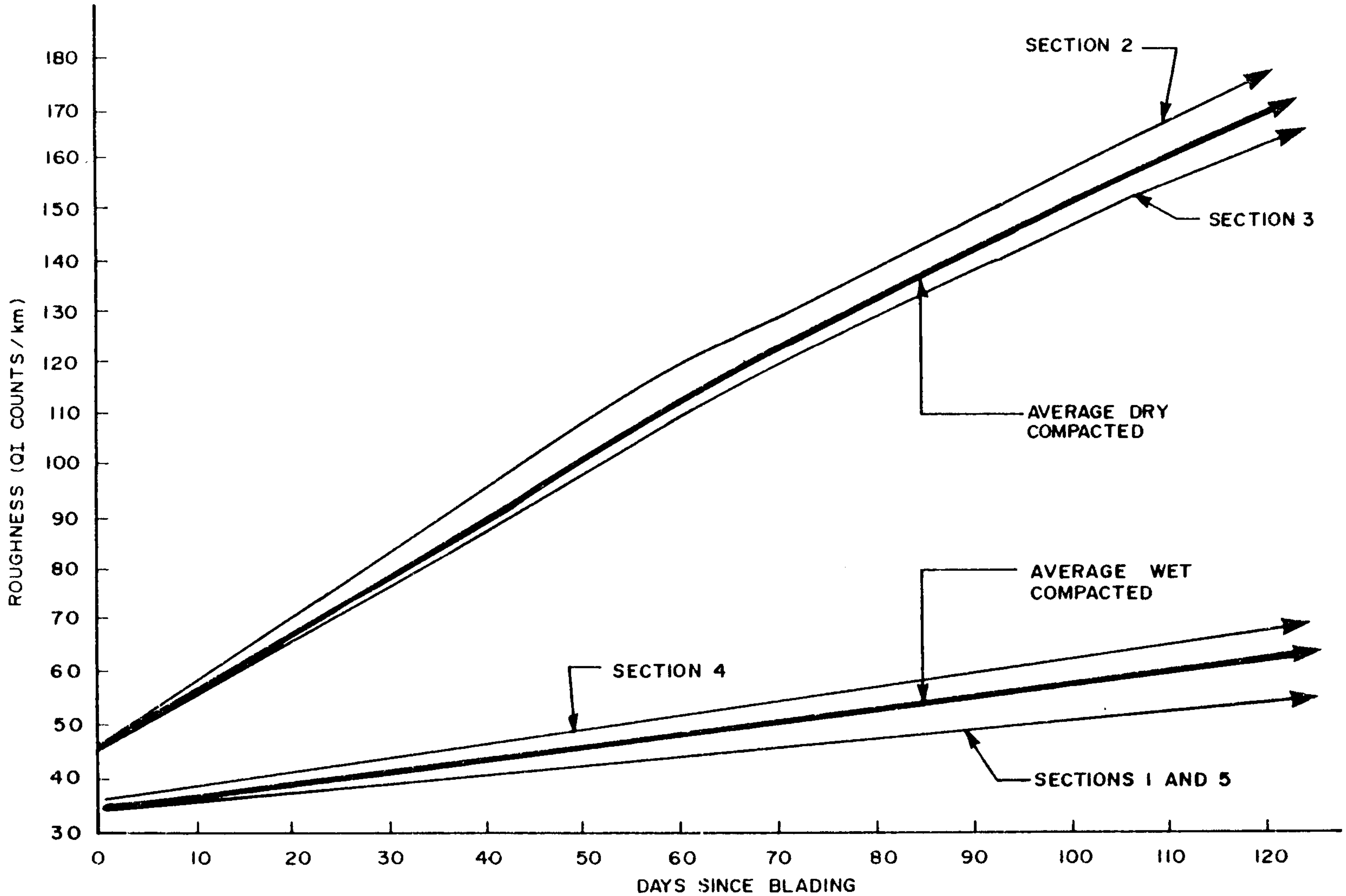


FIGURE 8.1 *EFFECT OF MOIST COMPACTION ON ROUGHNESS (AFTER POOLMAN, 1988)*

two or three times to keep the roughness below about 100 QI counts/km. The wet compacted sections were also less dusty than the dry compacted sections and drained better during thunderstorms (Poolman, 1988). The difference between the discounted total present worth of the two sections (assuming the same maintenance was applied to both sections and a discount rate of 7 per cent) was R15 830/km (Poolman, 1988).

Compaction to a high density is also important. This was clearly shown by the experiment discussed in Chapter 6.4, where a pneumatic tyred roller produced a compacted density of 97 percent of Mod AASHTO with only three passes and produced a much better road than the grid rolled equivalent.

8.6 MAINTENANCE

The use of the new specifications will result in a number of maintenance advantages. The materials specified will not require as much routine maintenance as many of the materials presently in use as problems with corrugations, stony roads, ravelling (loose material) and bad erosion will be significantly reduced.

Present grader maintenance costs are about R400 per day with an average about 40 blade kilometres per day (i.e. about 8 road kilometres, depending on the width of the road). If, by using the proposed specifications, the blading frequency can be reduced by 50 per cent, the savings to the authority concerned on a 40 km road presently being graded once a month would be R2 000 per annum. This is, however, unlikely to be a direct saving as the machine would be used instead on roads with heavier traffic which need more frequent blading (i.e. once every two or three weeks) to maintain an acceptable riding quality. With the anticipated increase in the population and traffic in the future, the number of roads carrying heavier traffic are going to increase (see Chapter 1.2) and a better road will result in substantially reduced road user costs (Chapter 8.7).

The use of maintenance programming instead of the systematic maintenance presently carried out over much of southern Africa results in an overall improvement of the riding quality of the road network with significantly reduced road user costs. At present the MDS (Visser, 1981a) is applied locally but this uses prediction models developed in Brazil. Although the results of the MDS are consistent with local observations, its accuracy and scope require improvement (Visser and Curtayne, 1987). It is anticipated that the application of the models developed in this thesis to the MDS will go a long way towards increasing the validity of the MDS under local material, traffic and climatic conditions.

Maintenance programming results in an improved road network by identifying those roads which deteriorate rapidly and indicating that they should be graded more frequently than those roads which deteriorate at a slower rate. From model 5.12 it can be seen that the roughness after blading depends primarily on the roughness before blading and thus by avoiding excessive deterioration of the road the general standard of the road is improved.

It is of course equally important that the maintenance team is experienced and that the standard of maintenance is of a high quality. Maintenance practices and procedures are fully described by Ferry (1986).

8.7 ROAD USER COSTS

Unpaved roads constructed with the specified materials will provide a substantially better service to the road user. Removal of oversize stones and the reduction (elimination in most cases) of corrugations will improve the riding quality with concomitant savings in road user costs. The road user costs for three typical roads, two with excessive oversize material (sections 34A and 919) and one susceptible to the formation of bad corrugations (section 26A) are compared below with the expected costs of the same roads constructed with material complying with the proposed specifications. Vehicle operating costs at the typical roughnesses measured on roads infrequently maintained (bladed every three months with average QI of 160 counts/km on stony sections and 130 on corrugated sections), frequently maintained (bladed every month with average QI of about 100 counts/km on stony roads and 85 on corrugated roads) and constructed with material complying with the proposed specifications (average QI about 50 counts/km) are compared in Table 8.1. In order to eliminate the influence of road geometry on the road user costs, it is assumed that the roads are 10 km long and straight and flat. Horizontal curvature and positive road gradient both increase the vehicle operating costs. The road user costs are calculated using the modified HDM 3 model (du Plessis and Rust, 1988).

TABLE 8.1: EFFECT OF ROUGHNESS ON VEHICLE OPERATING COSTS

Section No	QI Counts/km	Total vehicle operating cost R/yr
34A	160	345 305
	100	247 723
	50	187 390
26A	130	470 866
	85	362 393
	50	290 974
919	160	122 454
	100	87 682
	50	67 139

Section 34A - 73 cars and 19 heavy vehicles per day
Section 26A - 74 cars and 48 heavy vehicles per day
Section 919 - 30 cars and 5 heavy vehicles per day

The implications of improving the quality of unpaved roads on the vehicle operating costs for a short link of a road are significant. An improvement of the riding quality of section 26A from 160 to 100 counts/km results in an annual saving in vehicle operating costs of R108 473 while an improvement to 50 counts/km would save R179 892 per year. This monetary saving would be accompanied by an increase in the average speed of the heavy vehicles of about 30 per cent. Most of the heavy vehicles on this road haul timber from nearby forests to the local saw-mills and although the authority who constructs and maintains this road would have higher initial costs (less than R20 000 assuming one borrow-pit along the section) and the unchanged or slightly reduced road maintenance costs, the savings to the timber industry (and the country as a whole due to improved productivity) would be significant. When viewed on a district or regional basis the figures are astounding.

It is estimated that there are in excess of 200 000 km of unpaved roads in southern Africa (provincial, homeland, recreation, forestry and mine haul, urban, etc.). Assuming that these roads carry on average 50 vehicles per day with 20 per cent heavy (a conservative estimate as many buses and agricultural vehicles use these roads) and the present average roughness is 100 QI counts/km (also very conservative as homeland, parks and haul roads are generally much rougher) an improvement in the average roughness to a QI value of 60 counts/km would result in an annual decrease in the vehicle operating costs from R2 672 974 000 to R2 142 279 900 (i.e.. an annual saving in vehicle operating costs of R530 694 100).

These analyses and discussions are all related to road roughness values of 160 QI counts/km or less. The relationship between road roughness (QI) and vehicle operating costs (VOC) is exponential and takes the form

$$VOC = EXP\ (a + b.QI) \qquad (8.1)$$

where a and b are constant values related to the vehicle characteristics and road geometry (du Plessis and Rust, 1988). Some of the values of the road roughness measured during the project were consistently higher than 160 counts/km with a maximum measured value of 320 counts/km. The predicted vehicle operating cost (per 1000 km) at a roughness of this magnitude (from the appropriate values of a and b in Model 9.1) would be R1747,25 for cars (R452,64 at QI =50) and R5909,99 for trucks (R962,99 at QI = 50). These figures are, however, outside the inference space of the original data from which Model 8.1 was derived (max QI = 200) and should be viewed with caution. The vehicle operating cost of buses is much higher than the values used for medium trucks (5,4 tonnes) in the analyses carried out with costs of R1323,44, R1614,70 and R2122,20 at QI values of 50, 100 and 150 counts/km.

The estimates of traffic volume, the ratio of light to heavy vehicles and the average road roughness used in this discussion are generally conservative and the figures derived are at best optimistic. The average road roughness in many of the homeland areas is closer to 200

QI counts/km (maintenance is minimal) and the traffic often consists mainly of buses. The actual total vehicle operating costs for the southern African region can thus be expected to be substantially higher than those derived in this study.

Forder (1987) discussed the experience of a single bus company with respect to tyre problems. In northern Natal many of the roads became so rough and eroded (with exposed stones) after cyclone Demoina that most tyres were scrapped prematurely for his company. The estimated costs of replacement tyres in 1987 was almost 1,7 million Rand. As many of the roads were in remote areas tyre problems resulted in major delays to the bus service, passenger inconvenience and generally an unacceptable erosion of the quality of service.

8.8 NATIONAL ECONOMY

The influence of the findings of this project on the national economy relate mainly to the foreign exchange benefits. Fuel, lubricants, tyre and parts consumption are directly related to road roughness (Hide, 1975; Du Plessis et al, 1988) and these are significant in terms of foreign exchange.

Fuel

Actual quantities of fuel imported and consumed in southern Africa are not available (for strategic reasons) but it would appear that a significant portion of the South African foreign exchange account is directed towards fuel purchases.

Although the percentage contribution of fuel to the total vehicle operating cost decreases as the road roughness increases, the fuel cost generally rises (Figure 8.2). The marginal increase in fuel cost with increasing roughness appears minimal but in terms of the total consumption is significant.

If the average roughness of the network can be reduced from the estimated present value of more than 100 QI counts/km to a value of around 60 counts/km a saving of some R11,5 million (Appendix 4) in

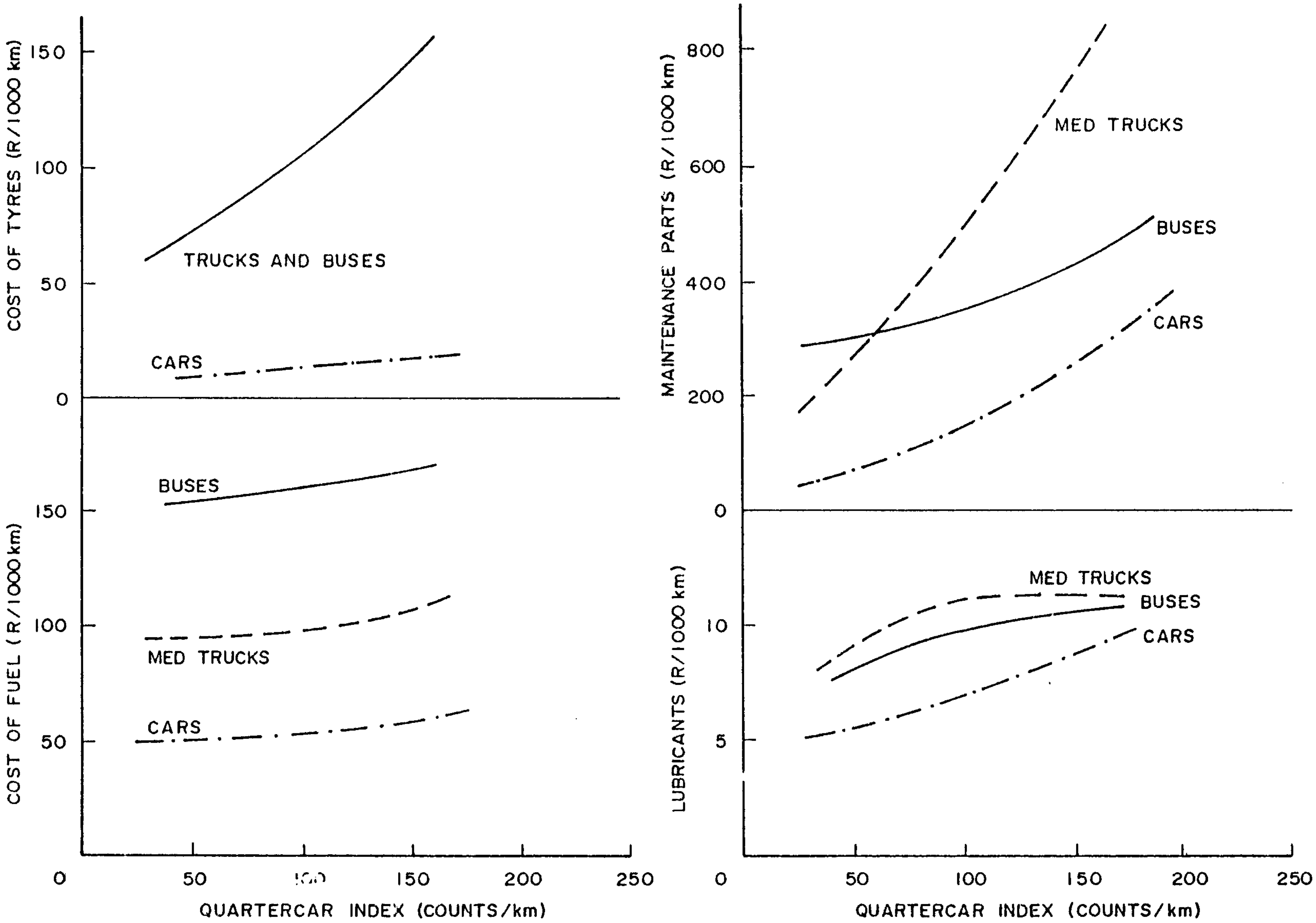


FIGURE 8.2 *EFFECT OF ROUGHNESS ON COSTS OF TYRES, FUEL, LUBRICANTS AND MAINTENANCE PARTS FOR CARS, MEDIUM TRUCKS AND BUSES*

fuel could be expected (assuming an economic cost of fuel of R0,60 per litre (Du Plessis and Rust, 1988) a large percentage of which would be in foreign exchange. Present world oil prices and rand/dollar exchange rates indicate that imported oil costs about 30 c/litre, the remaining cost being the refining, and transport costs i.e. probably half of the economic cost is foreign exchange. Assuming half of South Africa's oil needs are imported (the other half being produced by SASOL) annual savings in foreign exchange of R2 875 000 could be expected.

In practice, better roads generally result in higher speeds and concomitant increased fuel consumption. Unless law enforcement was increased much of the savings would be consumed by the higher petrol consumption at the increased speeds.

Lubricants

A saving in foreign exchange on reduced use of lubricants would be achieved by reducing the average road roughness (Figure 8.2). By improving the average road roughness from 100 to 60 QI counts/km an annual saving of R4 635 000 can be made (Appendix 4). At least 25 per cent of this would be foreign exchange savings (i.e. R1 160 000).

Tyres

Much of the rubber used in the manufacture of tyres is imported and significant savings (especially in bus and truck tyres) can be achieved by reducing the roughness of unpaved roads. Figure 8.2 shows that a decrease in road roughness results in a significant saving in tyre costs. An improved unpaved road may also result in significant savings from reduced punctures.

A decrease in the average road roughness from 100 to 60 QI counts/km would result in an annual saving in tyre costs of R15 250 000 (Appendix 4). It is estimated that at least twenty per cent of this cost would be in foreign exchange on the imported rubber (i.e. R3 000 000).

The increase in passenger delays and disruption to bus services on rougher roads caused by a larger number of breakdowns and punctures (Forder, 1987) has a significant effect on the national economy in terms of productivity and unscheduled trips to effect repairs to the buses and maintain the service.

Maintenance parts

It is conservatively estimated that about thirty per cent of the important maintenance parts are imported. The maintenance parts are the major contributor to the vehicle operating costs (together with the associated labour) for trucks and buses and the second major contributor (after depreciation) for cars. As the road roughness increases the cost of maintenance parts increases significantly (Figure 8.2) and major savings in imports can thus be obtained by decreasing the road roughness.

A general decrease in the average unpaved road roughness from 100 to 60 QI counts/km would result in an annual saving of R289 500 000 (Appendix 4) of which at least 30 per cent (i.e. almost R87 million) would involve foreign exchange.

General

The main contribution to the national economy of improving the unpaved road network would be a significant saving in foreign exchange. A total estimated annual saving in foreign exchange of at least R96 500 000 can be expected if the average road roughness is improved from 100 to 60 QI counts/km. In practice many of the roads in developing and rural areas have roughness levels far higher than 100 counts/km and many of these roads carry a significantly higher traffic and larger proportion of buses and heavy vehicles than has been assumed for the calculations on which the analyses were based. No geometric effects (curvature or vertical grades) were included in the analyses thereby biassing the results on the low side. The final figure is therefore a conservative estimate.

With the possibility of increased sanctions against South Africa and a possible worsening of the Rand/dollar exchange rate the importance of reducing the imported fuel and rubber requirements becomes increasingly significant.

Other aspects which affect the national interest are the effects of accidents. It is difficult to quantify the cost of accidents on unpaved roads, but smoother, less slippery and less dusty roads should theoretically result in a decrease in the accident rate. The higher speeds associated with a better unpaved road network may, however, increase the number and severity of accidents.

8.9 SHOULDERS FOR SEALED ROADS

Many thousands of kilometres of rural road in southern Africa have unsealed shoulders. Some of these roads carry up to 15 000 equivalent vehicle units per day (1 truck = 3 cars) and in wet weather extensive damage is done to the shoulders by heavily loaded vehicles stopping on the shoulders.

Existing specifications for unpaved shoulders for sealed roads are generally the same as the existing specifications for wearing course gravels for unpaved roads (TPA, 1983; Dept of Transport, 1985). The new specifications for wearing course gravels proposed in this work (preferably those for urban areas (Section 7.7.2) to reduce the dust nuisance on major routes) should therefore replace the existing shoulder material specifications.

It is considered that this will result in an improvement in existing shoulders and reduced grader and drain maintenance requirements.

8.10 OTHER ROADS

The roads studied during this project were all provincial numbered roads in the Transvaal and South West Africa. However, thousands of kilometres of unpaved roads exist in urban areas, public and private forests and recreation areas, and national, municipal and privately owned parks and nature reserves. In addition, thousands of kilometres of unpaved strategic and tactical military roads and landing strips are constructed and maintained in southern Africa.

The expected performance of these roads varies considerably. Some of the roads are required to be passable all the time and thus require routine maintenance while others are seldom used and then only by off-road or four-wheel-drive vehicles. In many cases, however, the economic viability of a nature reserve, forest or recreation area depends on a reliable all-weather road system. Many of the smaller enterprises cannot afford to own and maintain road graders or alternative maintenance equipment and it is thus important to construct the road (probably by contract) of the highest quality materials in order to reduce the maintenance to an absolute minimum.

The use of a suitable material specification from the three derived in this study will result in both improved service and reduced maintenance. In many of these cases the extra expenditure required to locate the materials and construct the road to a standard appropriate to the traffic and needs of the road will soon be recovered by savings in maintenance and vehicle operating costs.

8.11 OTHER IMPLICATIONS

Resources of good quality gravels are rapidly being depleted. Many of the existing specifications for unpaved road materials are so tight that materials potentially suitable for low-volume paved roads are accepted for wearing courses. These materials should be conserved for future use in paved roads. The proposed specifications will allow the use of many materials not suitable for paved roads and thus allow conservation of better gravels for future paved road use.

In many townships in developing areas, erosion of the unpaved roads results in blocking of drains and culverts. This necessitates excessive maintenance which is seldom available and results in unsanitary conditions and breeding areas for flies and mosquitoes. The proposed specifications will eliminate those materials particularly prone to erosion or provide advance warning of possible erosion problems if only erodible material is available. Contingency plans for drainage maintenance can therefore be built into the road design.

Socio-economically, an improved transportation system will result in an increased quality of life in urban and rural developing areas. Better roads will lead to a more efficient transportation infrastructure with less break-downs, lower transportation costs (less maintenance, fuel and spares required) and quicker travel in greater comfort.

Socio-politically, an improved quality of life in the developing areas with fewer muddy and slippery roads, less dust, improved public health conditions and a visibly better road network is tremendously important to the country as a whole.

8.12 CONCLUSIONS

The findings of the project and the new specifications which have been proposed have significant implications for the roads authorities, the road users and the national economy.

In order to implement the proposed specifications increased attention will need to be directed towards the material location, testing and processing. Increased haul distances for wearing course materials may be necessary resulting in increased construction costs. It is estimated that the increased costs involved with implementing the specifications are unlikely to exceed R6 000 per kilometre at the time of construction. This increased cost would need to be borne by the roads authorities.

Some of the increased construction cost may be recouped by the road authorities by reduced maintenance costs but it is anticipated that the programming of maintenance will make optimum use of the available plant to reduce road user costs and thus will not result in any direct savings to the roads authorities.

The major benefit of implementing the proposed specifications would be significant savings in road user costs with annual savings well in excess of R530 million ultimately being possible. Of this total, almost R100 million would be in terms of foreign exchange savings.

The proposed specifications are equally applicable to shoulder materials for paved roads and significant savings are possible in terms of the maintenance of these shoulders.

A substantial improvement of the quality of life in developing urban and rural areas can be expected through implementation of these specifications.

CHAPTER 9

SUMMARY OF CONCLUSIONS AND RECOMMENDATIONS

9.1 SUMMARY OF CONCLUSIONS

- Although extensive research has been carried out on wearing course materials for unpaved roads since the early 1920's, minimal attention has been directed towards the influence of geotechnical properties on the performance of unpaved roads. Recent studies have concentrated specifically on the maintenance aspects.

- Numerous specifications (generally of obscure derivation) are currently in use in southern Africa. Their reliability and applicability in practice, however, have not been rigorously quantified and there is little evidence to indicate that any of them are performance-related.

- Of the published work, only the work by Netterberg, specifically on the use of calcretes as wearing courses in southern Africa, is performance-related.

- As in the Kenya and Brazil studies a factorial design with material, traffic, climate and road geometrics was selected for this experiment. Although the experimental site selection was not completely random a suitable range of material properties was included in the experiment.

- In all, 110 sections of road in the Transvaal and South West Africa were regularly monitored for more than 2 years.

- The main consideration in the experiment was the influence of the geotechnical properties on the performance of the test sections. The effect of different maintenance strategies on the performance of the sections was not investigated, unlike all the previous studies which concentrated on the maintenance and not the materials.

- Testing and monitoring followed well documented techniques and a carefully controlled and closely supervised, regular monitoring programme resulted in a large data base containing high quality results.

- None of the materials in the test sections complied with the TPA specifications on the basis of the specified maximum size. Even neglecting the maximum size criterion only one section complied with the early specification and none with the present one. However, many of the sections provided a satisfactory performance, even under prolonged periods with no maintenance.

- The "drought" prevailing over much of southern Africa during the monitoring period was found not to have affected the study area significantly in terms of the total rainfall as a percentage of the long term average.

- Multiple correlation analyses were carried out to identify the main geotechnical properties and environmental factors affecting the various performance criteria of unpaved roads.

- Most of the models developed to predict the rated performance of the roads have unacceptably low R-squared values and relatively high standard errors in terms of prediction models. The fact that only about 50 percent of the variation is accounted for by most of the models indicates that other factors not taken into account in the analyses are as important (e.g. variation in traffic volume and speed, rainfall intensity, quality of maintenance etc.).

- The interaction of different performance parameters results in problems with the interpretation of the behaviour of the materials. Potholes, rutting, trafficability and dust, for example, are all strongly influenced by surface drainage which in itself may be influenced by potholes and rutting.

- The geotechnical properties (particle size distribution and plasticity in particular) influence the performance of the materials significantly.

- The observations and measurements of corrugations have resulted in a confirmation of the "forced oscillation theory" as the cause of their development. Vehicle speed has been shown to be the major factor determining the depth, spacing and movement of corrugations. The presence of both loose and fixed corrugations is discussed and traffic compaction is identified as the probable cause of the latter.

- Measurements of the slipperiness of typical unpaved roads were carried out. The results indicated that at worst, dry unpaved roads are only slightly more slippery than average paved roads but are adequately safe. In the wet condition, unpaved roads become more slippery but apparently only unacceptably slippery when free water occurs at the surface.

- The plasticity and grading characteristics of materials are the main contributors to their performance as wearing courses gravels.

- A number of models for the prediction of roughness progression in unpaved roads have been developed overseas. These models are all fairly complicated, necessitating the determination of a number of geotechnical properties, an identification of the material types and the incorporation of an estimate of the average vertical grade for the total length of the link.

- The new models developed in this study have eliminated the necessity in existing models for identifying the material type and of estimating the average grade for the road link. Simple indicator tests requiring minimal equipment and only basic operator training are required for the input parameters for the model. The predictive capability has, however, not been diminished through this process and, in fact, it has generally been improved for local conditions.

- The geological origin of the wearing course materials do not directly affect the roughness of unpaved roads. The geotechnical properties, which are directly dependent on the geology in terms of the mineralogical composition, on the other hand significantly affect the roughness development.

- The analysis of existing models to predict the gravel loss on unpaved roads indicated large differences between the predicted gravel losses for a number of roads in South and South West Africa and the actual losses.

- A new model for the prediction of gravel loss has been developed. This model predicts the gravel loss with an accuracy of within 11 mm over the design life of a typical unpaved road (wearing course about 150 mm thick) and should replace the model developed in Brazil for the Maintenance and Design System when used locally.

- Like the model for roughness progression, this model is simple compared to the existing models. Aspects such as the vertical grade and horizontal curvature which need to be averaged for a road link are excluded from the model eliminating a major problem. No geological classification is necessary. All the parameters required can be easily obtained by relatively unskilled staff in developing areas.

- As noted for the roughness prediction model, the geotechnical properties (notably the grading and plasticity again) significantly affect the gravel loss.

- Although numerous specifications for unpaved roads are currently in use in southern Africa, none of them are successful for all potential wearing course materials.

- Performance-related specifications have been derived for the identification of suitable gravel wearing course gravels.

- The specifications are simple, requiring minimal testing (a quick grading analysis, a linear shrinkage test and a CBR) and should be applicable to various types of road carrying any traffic in any climatic area in South Africa (scientifically valid for practically all of southern Africa).

- The importance of removing the oversize material, limiting the shrinkage product and grading coefficient to the specified values and adequately compacting the wearing course has been clearly identified.

- In order to ensure all weather trafficability, an adequate material strength is required. As none of the sections on which the specifications were developed became impassable during the project, tentative limits based on theoretical considerations and empirical studies elsewhere have been proposed.

- The use of these specifications together with good construction and maintenance procedures should result in a general improvement of unpaved roads country-wide as existing roads are gradually regravelled according to these specifications.

- Together with the routine regravelling of roads and non-routine regravelling of roads requiring excessive maintenance, a maintenance management system should be implemented to optimise the grader maintenance in terms of maintenance and road user costs.

- Although previous work in Kenya and Brazil has shown that a classification of the material type is necessary to predict wearing course gravel performance, this study has shown that for South African conditions the material properties in terms of plasticity and particle size distribution are the only valid means of predicting the performance.

- The performance of natural wearing course gravels has been shown to be independent of the geological origin and geotechnical classification of the gravel, and is entirely dependent on the material properties (mainly plasticity and particle size distribution).

- Existing classification systems, although based on the correct properties appear to have excessively wide ranges for classification based material selection processes.

- The findings of the project and the new specifications which have been developed have significant implications for the road authorities, the road users and the national economy.

- In order to implement the proposed specifications increased attention will need to be directed towards the material location, testing and processing. Increased haul distances for wearing course materials may be necessary resulting in increased construction costs. It is estimated that the increased costs involved with implementing the specifications will not exceed R10 000 per kilometre. This increased cost will need to be borne by the road authorities.

- Some of the increased construction cost may be recouped by the road authorities by reduced maintenance costs but it is anticipated that the programming of maintenance will make optimum use of the available plant to reduce road user costs and thus may not result in any direct savings to the roads authorities.

- Significant savings in road user costs are envisaged with annual savings in excess of R530 million ultimately being possible. Of this total, almost R100 million would be in terms of foreign exchange savings.

- The proposed specifications are equally applicable to shoulder materials for paved roads and significant savings are possible in terms of the maintenance of these shoulders.

- A substantial improvement of the quality of life in developing urban and rural areas can be expected through implementation of these specifications.

9.2 RECOMMENDATIONS

- It is recommended that as existing roads are routinely regravelled and those roads which perform unsatisfactorily and require excessive maintenance are regravelled, the proposed specifications are used. At the same time a data base of the material, traffic and climatic characteristics is built up and a maintenance management system is put into practice.

- Further work on cost-effective dust palliatives, wet-weather trafficability, the effect of increased compaction, maximum allowable subgrade strains in unpaved roads and improved geotechnical classification systems is recommended.

- An investigation into the optimum methods for the location of suitable materials for unpaved roads should be carried out.

- Maintenance should concentrate on the retention of the crown of the road and the crossfall, more than on the reduction of roughness. Monitoring has indicated that the roughness is only decreased by blading about 70 per cent of the time. Restoration of the shape on the other hand results in improved drainage and less rutting with a concomitant improvement in riding quality in the longer term. Grader blading should be carried out in a programmed manner, and the roads should not be allowed to deteriorate too far or else routine blading will not correct the deficiencies.

- Improved training for grader operators is strongly recommended. This should include such aspects as the importance of the pavement cross-section and good drainage and the effect of loose material on the road.

- Further research on the use of sand blankets is necessary. Many grader operators spread a thick layer of material from the side of the road over the road. This results in unsafe conditions and increases the vehicle operating costs and should be discouraged.

- A number of the sections monitored were excessively wide (up to 14 m with 10 m trafficked widths). This allows considerable lateral movement of the traffic but also results in a substantial extra maintenance cost, requiring one or two extra grader passes. Road widths should thus not be increased above the safe minimum.

9.3 SUMMARY OF FINDINGS

The specifications which were developed and the prediction models for roughness progression, roughness after blading and gravel loss are summarised below. The definitions of the symbols are given in the front of this thesis.

Recommended material specifications for unpaved rural roads

Maximum size:	37,5 mm
Oversize index (I_o):	≤ 5 per cent
Shrinkage product (S_p):	100 - 365 (max. of 240 preferable)
Grading coefficient (G_c):	16 - 34
CBR:	≥ 15 at ≥ 95 per cent Mod AASHO compaction and OMC

Recommended material specifications for unpaved roads in urban areas

Maximum size:	37,5 mm
Oversize index (I_o)	0
Shrinkage product (S_p):	100 - 240
Grading coefficient (G_c):	16 - 34
CBR:	≥ 15 at ≥ 95 per cent Mod AASHO compaction and OMC

Recommended material specifications for unpaved haul roads

Maximum size:	75 - 100 mm
Oversize index (I_o):	≤ 10 per cent
Shrinkage product (S_p):	100 - 365 (max preferably < 240)
Grading coefficient (G_c):	16 - 34
CBR:	≥ 15 at ≥ 95 per cent Mod AASHO compaction and 4 days soaking

Tests according to TMH 1 (NITRR, 1979) except grading according to Paige-Green (1988b) and plus 19 mm material discarded in CBR test.

The model to predict the change in roughness (CR) of an unpaved road after any selected number of days (D = No of days/100) is as follows:

$$CR = \exp\{D[-8{,}3 + 0{,}0003.PF + 0{,}07.S1 + 0{,}081.P26 + 0{,}0003.N.ADT + GM(3{,}63 - 0{,}035.P26)]\}$$

The following model is recommended for the prediction of the roughness after grader blading (RA):

$$RA = \exp(1{,}07 + 0{,}699.LRB + 0{,}0004.ADT - 0{,}13.DR + 0{,}0019.LABMAX)$$

The recommended model for the prediction of gravel loss from an unpaved road is as follows:

$$GL = D\ [ADT(0{,}059 + 0{,}0027.N - 0{,}0006.P26) - 0{,}367.N - 0{,}0014.PF + 0{,}0474.P26]$$

CHAPTER 10

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APPENDIX 1: MAINTENANCE AND ROAD USER COST ANALYSIS (1987 Rand)

Regravelling - each road once every 7 years on average
(132 000 km / 7) x (R5 154/km) = R 97 189 714

Grader blading - each road once a month on average
(132 000 km x 12) x (R40/km) = R63 360 0006

TOTAL = R160 867 714

The calculated cost of maintaining the 29 745 km of unpaved roads in the Transvaal (21% of the South African total) would therefore be:

21 % of R160 867 714 = R33 782 219/yr

The 1986/87 budget for the maintenance of unpaved roads in the Transvaal was R28 994 500. Therefore, (assuming the maintenance programmes of the various provinces are similar) the estimate of R160 867 714 for the provincial unpaved road network in South Africa would appear to be reasonably accurate.

Road user costs are difficult to quantify because of the effect of the variable traffic volumes on different routes. An estimate can, however, be made as follows:

Provincial unpaved roads	-	132 000
Average daily traffic	-	75 vpd; 12 % heavy
Average road roughness	-	QI = 70 counts/km
Vehicle operating costs	-	Cars: R491,20 per 1000 vehicle km
(du Plessis & Rust, 1988)		Trucks: R1130,90 per 1000 veh km

Estimated annual vehicle operating cost = R2 052 338 000

If the roads are bladed 25 % less than the optimum frequency then the road user costs are increased by about 33,3 per cent (Visser, 1981a): i.e. R2 735 766 554/yr.

If the roads are bladed 25 % more than the optimum frequency then the road user costs are decreased by about 33,3 per cent: i.e. R1 366 857 108/yr.

APPENDIX 2: EXISTING SPECIFICATIONS

A summary of existing specifications is included in this appendix. These are most of the specifications presently in use in southern Africa and some of the more important ones which are probably used in other developing countries.

Some rounding-off and metrication has been done in order to make the various specifications more easily comparable. Most of the specifications are complete other than neglecting qualitative statements such as "hard, durable, angular gravels should be used".

Both the 1973 and 1983 specifications for the Transvaal are included as most of the roads selected for the investigation were either constructed or regravelled prior to 1983.

SPECIFICATIONS SUMMARISED

AUTHORITY (Reference): TRH 14 (NITRR, 1985)

Maximum size: 50 mm but preferably 37,5 mm.

Sieve Size (mm)	% Passing			
	Nominal Size (mm)			
	37,5	26,5	19,0	13,2
37,5	100			
26,5	85-100	100		
19,0	70-100	80-100	100	
13,2	60-85	60-85	75-100	100
9,5				
4,75	40-60	45-65	50-75	60-100
2,00	25-45	30-50	35-55	45-70
0,425	15-40	15-40	18-45	25-50
0,075	7-30	7-30	7-30	7-30
0,053				
Grading modulus				
Dust ratio				

Limits of consistency	Climate		
	N < 5	5 < N < 10	N > 10
Liquid limit Plasticity index Linear shrinkage	8 - 14	14 - 20	14 - 20+

Aggregate strength and durability :

Material strength:
CBR (%)
CBR swell (%)

Relative compaction: ≥ 93 percent Mod AASHTO density.

Comments: For calcretes use specification of Netterberg (1978).

<u>AUTHORITY</u> (Reference): Department of Transport (1958)

Maximum size:

Sieve Size (mm)	% Passing			
	Nominal Size (mm)			
	Gravel	Semi-sand clay	Sand-clay	
37,5	100			
26,5	75-100			
19,0	70-90			
13,2	60-80			
9,5				
4,75	40-60			
2,00	25-50	50-80	80-100	
0,425	10-35	20-55	32-70	
0,075				
0,053	4-15	8-25	12-30	
Grading modulus Dust ratio				

Limits of consistency	Material type*		
	1	2	3
Liquid limit (%)	≤ 35	≤ 35	≤ 35
Plasticity index (%)	6 - 15	6 - 12	6 - 12
Linear shrinkage (%)	≤ 7	≤ 4,5	≤ 4,5

Aggregate strength and durability : ≤ 25 % sodium sulphate loss after 5 cycles

Material strength:
CBR (%)
CBR swell (%)

Relative compaction:

Comments: * Material type 1 = gravel
2 = semi-sand clay
3 = sand-clay

AUTHORITY (Reference): Department of Transport (DOT, 1983)

Maximum size: 40 mm.

Sieve Size (mm)	**% Passing**			
	Nominal Size (mm)			
37,5 26,5 19,0 13,2 9,5 4,75 2,00 0,425 0,075 0,053				
Grading modulus Dust ratio	≥ 1,5			

Limits of consistency	**Climate**		
Liquid limit (%) Plasticity index (%) Linear shrinkage (%)	6-10+3(GM)		

Aggregate strength and durability :

Material strength:
CBR (%) ≥ 45 at 95 percent Mod AASHO density
CBR swell (%)

Relative compaction: ≥ 93 per cent Mod. AASHO density.

Comments: The GM may be reduced to 1,2 with the permission of the Engineer.

AUTHORITY (Reference): Transvaal (TPA, 1973)

Maximum size: 50 mm.

Sieve Size (mm)	% Passing Nominal Size (mm)			
37,5				
26,5				
19,0				
13,2				
9,5				
4,75	35-85			
2,00	28-70			
0,425	16-45			
0,075	10-30			
0,053				
Grading modulus	≥ 1,5			
Dust ratio	≤ 0,67			

Limits of consistency	Climate		
Liquid limit (%)	≤ 40		
Plasticity index (%)	6 - 10+3(GM)		
Linear shrinkage (%)			

Aggregate strength and durability :

Material strength:
CBR (%) ≥ 15 at 93 % Mod AASHO density.
CBR swell (%)

Relative compaction: ≥ 93 % Mod AASHO density.

Comments: Grading Modulus may be reduced to 1,2 with engineers approval.

AUTHORITY (Reference): Transvaal Roads Dept. (TPA, 1983)

Maximum size: 40 mm.

Sieve Size (mm)	% Passing			
	Nominal Size (mm)			
37,5 26,5 19,0 13,2 9,5 4,75 2,00 0,425 0,075 0,053	100			
Grading modulus Dust ratio	≥ 1,5			

Limits of consistency	Climate		
Liquid limit (%) Plasticity index (%) Linear shrinkage (%)	6 - 10+3(GM)		

Aggregate strength
and durability :

Material strength:
CBR (%) ≥ 45 at 93 % Mod AASHO density.
CBR swell (%)

Relative compaction: ≥ 93 % Mod AASHO density

Comments: Grading Modulus may be reduced to 1,2 with engineers approval.

AUTHORITY (Reference): Natal Roads Department (NPA, ca 1985)

Maximum size: 100 mm.

Sieve Size (mm)	% Passing			
	Nominal Size (mm)			
37,5 26,5 19,0 13,2 9,5 4,75 2,00 0,425 0,075 0,053	THE SAME AS TRH 14			
Grading modulus Dust ratio				

Limits of consistency	Climate		
Liquid limit Plasticity index Linear shrinkage	20 - 35 6 - 15		

Aggregate strength and durability : Los Angeles Abrasion 30 - 60 %.

Material strength:
CBR (%)
CBR swell (%)

Relative compaction: $\geq$ 93 per cent Mod AASHO density.

Comments:

AUTHORITY (Reference): Orange Free State (Van der Walt, 1973)

Maximum size:

Sieve Size (mm)	% Passing			
	Nominal Size (mm)			
37,5	100			
26,5	90-100			
19,0	75-100			
13,2	60-95			
9,5				
4,75	35-85			
2,00	25-70			
0,425	20-45			
0,075	15-30			
0,053				
Grading modulus	≥ 1,5			
Dust ratio				

Limits of consistency	Climate*		
	1	2	3
Liquid limit (%) Plasticity index (%) Linear shrinkage (%)	14 - 18	12 - 16	10 - 14

Aggregate strength and durability : 10 % FACT ≥ 135 kN (≥ 90 kN in drier areas)

Material strength:

CBR (%) ≥ 25 % at 95 % relative compaction.
CBR swell (%) ≤ 1 %

Relative compaction:

Comments: * The province is divided into 3 regions depending on climate. PI may be as high as 20 for calcretes and 21 in the south west of the area.

AUTHORITY (Reference): Cape Province (Provincial Administration of the Cape of Good Hope, 1983)

Maximum size: 50 mm but preferably 37,5 mm.

Sieve Size (mm)	% Passing			
	Nominal Size (mm)			
	37,5	26,5	19,0	13,2
37,5				
26,5				
19,0				
13,2				
9,5				
4,75				
2,00	26-45	30-50	35-55	45-70
0,425				
0,075	7-30	7-30	7-30	7-30
0,053				
Grading modulus				
Dust ratio				

Limits of consistency	Climate		
	dry	wet	
Liquid limit (%)			
Plasticity index (%)	8 - 20+	8 - 20	
Linear shrinkage (%)			

Aggregate strength and durability :

Material strength:
CBR (%)
CBR swell (%)

Relative compaction: ≥ 95 % of Mod. AASHTO density.

Comments: In wet areas a lower PI is desirable. For pedogenic materials the upper limit of PI may be exceeded.

AUTHORITY (Reference): Fossberg (1963)

Maximum size:

Sieve Size (mm)	**% Passing**			
	Nominal Size (mm)			
	Lower	Upper		
37,5				
26,5	100			
19,0	70-100	100		
13,2				
9,5	50-80	80-100		
4,75	35-65	65-85		
2,00	25-50	50-70		
0,425	13-30	30-50		
0,075	5-15	15-30		
0,053				
Grading modulus				
Dust ratio				

Limits of consistency	**Climate**		
	Lower	Upper	
Liquid limit (%)			
Plasticity index (%)	6 - 8	8 - 20	
Linear shrinkage (%)			

Aggregate strength and durability :

Material strength:
CBR (%)
CBR swell (%)

Relative compaction:

Comments:

<u>AUTHORITY</u> (Reference): Netterberg (1978).

Netterberg devised these specifications from in-service materials and departs from the traditional method of specifying grading envelopes and plasticity reqirements. The specification of various properties depends on the fraction passing 0,425 mm as follows.

Sieve Size (mm)	% Passing 0,425 mm			
	20-40	41-50	51-60	61-75
Maximum size (mm)	53	53	53	38
Liquid limit (%)	30-65	22-48	22-40	18-36
Plasticity index (%)	9-22	7-23	5-13	8-13
Bar linear shrink (%)	2,0-9,5	2,7-9,0	2,0-5,5	2,0-5,0
Linear shrinkage x % < 0,425 mm	70-340	130-395	100-320	130-330
Min AFV (%)	65	60	35	40
Min APV (%)	20	20	15	-

Comments: Linear shrinkages between 2,0 and 2,7 and values of the linear shrinkage x % passing 0,425 mm between 70 and 100 may cause looseness and dust. A minimum APV of 14 is permissible if the AFV > 75.

AUTHORITY (Reference):South West Africa (L von Solms, 1987)

Maximum size: 37 mm if N < 10 ; 25 mm if N >10

Sieve Size (mm)	% Passing			
	Nominal Size (mm)			
37,5 26,5 19,0 13,2 9,5 4,75 2,00 0,425 0,075 0,053				
Grading modulus Dust ratio	1,1-1,9			

Limits of consistency	Climate		
	N < 10	N > 10	Kalahari
Liquid limit (%) Plasticity index (%) Linear shrinkage (%)	6 - 15	4 - 18	4 - 20

Aggregate strength and durability :

Material strength:
CBR (%) ≥ 25 but preferably ≥ 45 if N < 10.
CBR swell (%)

Relative compaction: ≥ 95 per cent Mod AASHO.

Comments: It is important to get the right combination of Grading Modulus and PI. For roads with less than 75 vpd the grading modulus may go up to 1,9, a per cent passing 0,075 mm of 15-40 and fineness index of 50-400 is preferred. If N < 10 the minimum grading modulus should be 1,5 with a minimum of 20 per cent passing 0,075 mm and a minimum fineness index of 150.

AUTHORITY (Reference): Botswana (Ministry of Works and Comm., 1982)

Maximum size: 20 mm but up to 20 % may be between 20 and 38 mm.

Sieve Size (mm)	% Passing			
	Nominal Size (mm)			
	19,0	13,2	4,75	
37,5				
26,5				
19,0	100			
13,2	90-100	100		
9,5				
4,75	60-85	75-100	100	
2,00	45-70	50-75	80-100	
0,425	25-40	25-45	20-45	
0,075	10-25	10-25	10-25	
0,053				
Grading modulus (min)	1,6	1,55	1,3	
Dust ratio				

Limits of consistency	Climate		
	All areas		
Liquid limit	≤ 55		
Plasticity index	15 - 30		
Linear shrinkage	8 - 15		

Aggregate strength and durability : 10 % FACT ≥ 50 kN or modified AIV ≥ 40%.

Material strength:
CBR (%)
CBR swell (%)

Relative compaction: ≥ 95 per cent Mod. AASHO density.

Comments: Not less than 10 per cent should be retained on successive sieves, except the largest pair. For calcretes the specification of Netterberg (1978) should be used.

AUTHORITY (Reference): Lesotho (D.T. Tsekoa, pers. comm., 1986)

Maximum size: 37,5 mm.

Sieve Size (mm)	**% Passing**			
	Nominal Size (mm)			
	Recomm.			
37,5	100			
26,5				
19,0	80-100			
13,2				
9,5	55-80			
4,75				
2,00	30-50			
0,425				
0,075	5-15			
0,053				
Grading modulus	1,5-2,8			
Dust ratio				

Limits of consistency	**Climate**		
	All areas		
Liquid limit	≥ 30		
Plasticity index	6 - 15		
Linear shrinkage			

Aggregate strength and durability :

Material strength:
CBR (%) minimum of 60 at in-place density
CBR swell (%)

Relative compaction: 95 % to 98 % Mod AASHO.

Comments: The grading may be outside the envelope as long as it is uniform.

AUTHORITY (Reference): Swaziland (J. Shepherd, pers. comm. 1986)

Maximum size: 37,5 mm but up to 50 mm allowable.

Sieve Size (mm)	**% Passing**			
	Nominal Size (mm)			
37,5 26,5 19,0 13,2 9,5 4,75 2,00 0,425 0,075 0,053		SAME AS TRH 14		
Grading modulus Dust ratio				

Limits of consistency	**Climate**		
Liquid limit (%) Plasticity index (%) Linear shrinkage (%)	8 - 20		

Aggregate strength and durability :

Material strength:
CBR (%)
CBR swell (%)

Relative compaction: ≥ 93 per cent Mod. AASHO density.

Comments: For PI in the range 8 - 10 % the percentage passing 0,075 mm must be greater than 20. If the PI is between 8 and 10 % or 14 and 20 % the TRH 14 grading envelopes must be adhered to.

AUTHORITY (Reference): Rhodesia (1979)

Maximum size:

Sieve Size (mm)	% Passing			
	Nominal Size (mm)			
37,5 26,5 19,0 13,2 9,5 4,75 2,00 0,425 0,075 0,053				
Grading modulus Dust ratio	1,5-2,5			

Limits of consistency	Climate		
Liquid limit (%) Plasticity index (%) Linear shrinkage (%)	< 70 10(GM-0,25)- 15(GM-0,"%)		

Aggregate strength and durability :

Material strength:
CBR (%)
CBR swell (%)

Relative compaction:

Comments:

AUTHORITY (Reference): Malawi (Ministry of Works, 1978)

Maximum size: 20 mm.

Sieve Size (mm)	**% Passing**			
	Nominal Size (mm)			
37,5				
26,5				
19,0	100			
13,2				
9,5	80-100			
4,75	60-85			
2,36	45-70			
0,300	20-40			
0,075	10-25			
0,053				
Grading modulus				
Dust ratio				

Limits of consistency	**Climate**		
Liquid limit (%)	≤ 40		
Plasticity index (%)	≥ 8		
Linear shrinkage (%)			

Aggregate strength and durability :

Material strength:
CBR (%)
CBR swell (%)

Relative compaction:

Comments: The fineness index (PI x Passing 0,075 mm) shall not exceed 600.

AUTHORITY (Reference): TRRL (TRRL, 1981)

Maximum size:

Sieve Size (mm)	% Passing			
	Nominal Size (mm)			
	19,0	9,5	4,75	
37,5				
26,5				
19,0	100			
13,2				
9,5	80-100	100		
4,75	60-85	80-100	100	
2,00	45-70	50-80	80-100	
0,425	25-45	25-45	30-60	
0,075	10-25	10-25	10-25	
0,053				
Grading modulus				
Dust ratio				

Limits of consistency	Climate *		
	1	2	3
Liquid limit (%)	≤ 35	≤ 45	≤ 55
Plasticity index (%)	4 - 9	6 - 20	15 - 30
Linear shrinkage (%)	2 - 5	3 - 10	8 - 15

Aggregate strength and durability :

Material strength:
CBR (%)
CBR swell (%)

Relative compaction:

Comments: > 10 % retained between each pair of sieves except the last pair. Nominal size 9,5 and 4,75 may have up to 35 per cent > 38 mm provided the passing 4,75 is within the limits.

* Zone 1 - Moist tropical to wet tropical - PI and LL may be higher for concretionary materials.
Zone 2 - Seasonally wet tropical
Zone 3 - Arid and semi-arid

AUTHORITY (Reference): AASHTO (AASHTO, 1974)

Maximum size:

Sieve Size (mm)	% Passing			
	Nominal Size (mm)			
37,5				
26,5	100			
19,0				
13,2				
9,5	50-85	60-100		
4,75	35-65	50-85	55-100	70-100
2,00	25-50	40-70	40-100	55-100
0,425	15-30	25-45	20-50	30-70
0,075	8-15	8-20	8-20	8-25
0,053				
Grading modulus				
Dust ratio	≤ 2/3	≤ 2/3	≤ 2/3	≤ 2/3

Limits of consistency	Climate		
Liquid limit (%)	≤ 35		
Plasticity index (%)	4 - 9		
Linear shrinkage (%)			

Aggregate strength
and durability : Los Angeles Abrasion value not exceeding 50.

Material strength:
CBR (%)
CBR swell (%)

Relative compaction:

Comments:

AUTHORITY (Reference): UNESCO (UNESCO, 1971).

Maximum size: 19 mm.

Sieve Size (mm)	**% Passing**			
	Nominal Size (mm)			
	19	9,5	4,75	
37,5				
26,5				
19,0	100			
13,2				
9,5	80-100	100		
4,75	60-85	80-100	100	
2,00	45-70	50-80	80-100	
0,425	25-45	25-45	30-60	
0,075	10-25	10-25	10-25	
0,053				
Grading modulus				
Dust ratio				

Limits of consistency	**Climate**		
	1	2	3
Liquid limit (%)	≤ 35	≤ 40	≤ 55
Plasticity index (%)	4 - 9	6 - 15	15 - 30
Linear shrinkage (%)	2 - 4	3 - 7	7 - 14

Aggregate strength and durability :

Material strength:
CBR (%)
CBR swell (%)

Relative compaction:

Comments: Not less than 10 % should be retained between each pair of sieves except the largest pair. Up to 35 % of stones larger than 38mm may occur in the 9,5 and 4,75 mm nominal size material provided that the material passing 4,75 mm is within the limits. The climatic zones are:

Zone 1 - moist temperate and wet tropical
Zone 2 - seasonal wet tropical
Zone 3 - arid

AUTHORITY (Reference): NAASRA (1980)

Maximum size:

Sieve Size (mm)	% Passing			
	Nominal Size (mm)			
37,5 26,5 19,0 13,2 9,5 4,75 2,00 0,425 0,075 0,053	DEPENDS ON MAXIMUM SIZE			
Grading modulus Dust ratio				

Limits of consistency	Annual rainfall (mm)		
	< 400	> 400	
Liquid limit (%) Plasticity index (%) Linear shrinkage (%)	≤ 35 4 - 15 ≤ 6	≤ 35 4 - 9 ≤ 3	

Aggregate strength and durability :

Material strength:
CBR (%) ≥ 60 at expected in-situ moisture and density
CBR swell (%)

Relative compaction:

Comments: Maximum dry compressive strength at least 2,8 MPa.

APPENDIX 3: SUMMARY OF STRATIGRAPHIC UNITS AND LITHOLOGY

Section	Stratigraphic unit	Lithology
1	Silverton Shale Formation	Shale
2	Silverton Shale Formation	Shale
3	Post Cretaceous on Dwyka Form	Ferricrete
4	Post Cretaceous on Ecca Form	Ferricrete
5	Malmani Subgroup	Chert
6	Malmani Subgroup	Chert
7	Letaba Formation	Amygdaloidal basalt
8	Letaba Formation	Amygdaloidal basalt
9	Lebowa Granite Suite	Granite
10	Lebowa Granite Suite	Granite
11	Alma Graywacke Formation	Sandstone
12	Alma Graywacke Formation	Sandstone
13	Post Cretaceous on Irrigasie Form	Calcrete
14	Rustenburg Layered Suite	Ferrodiorite
15	Rustenburg Layered Suite	Norite/anorthosite
16	Post Cretaceous on Vryheid Form	Ferricrete
18	Vaalian Intrusives	Diabase
19	Silverton Shale Formation	Shale
20	Silverton Shale Formation	Shale
21	Undifferentiated Pretoria Group	Shale/mudrock
22	Malmani Subgroup	Chert
23	Malmani Subgroup	Chert
24	Nelspruit Granite	Granite
25	Kaap Valley Granite	Hornblende granite
26	Kaap Valley Granite	Hornblende granite
27	Post Cretaceous on Vryheid Form	Ferricrete
28	Vryheid Formation	Gritstone
29	Karoo Sequence Intrusives	Dolerite
30	Karoo Sequence Intrusives	Dolerite
31	Post Cretaceous on Vryheid Form	Ferricrete
32	Undifferentiated Pretoria Group	Sandstone/shale
33	Hospital Hill Subgroup	Shale/sandstone
34	Karoo Sequence Intrusives	Dolerite
35	Undifferentiated Witwatersrand SG	Sandstone/shale
36	Makwassie Formation	Quartz porphyry
37	Recent lake deposit	Mudstone
38	Post Cretaceous on Vryheid Form	Ferricrete
39	Vryheid Formation	Sandstone
40	Vryheid Formation	Sandstone/calcrete
41	Allanridge Formation	Andesite
42	Undifferentiated Swazian Intrusiv	Granite/gneiss
43	Undifferentiated Swazian Intrusiv	Granite/gneiss
44	Post Cretaceous on Allanridge Fm	Calcrete
45	Kareefontein Formation	Quartz porphyry
46	Post Cretaceous on Allanridge Fm	Calcrete
47	Allanridge Formation	Andesite
48	Post Cretaceous on Allanridge Fm	Calcrete
49	Undifferentiated Swazian Intrusiv	Granite/gneiss

APPENDIX 3 (cont): SUMMARY OF STRATIGRAPHIC UNITS AND LITHOLOGY

Section	Stratigraphic unit	Lithology
901	Kuiseb Formation	Mica schist
902	Kuiseb Formation	Mica schist
903	Kuiseb Formation	Mica schist/quartzite
904	Mulden Group	Phyllite/hornfels
905	Post Cret. on Mulden Group	Calcrete
906	Post Cret. on Mulden Group	Calcrete
907	Post Cret. on Damara Sequence	Calcrete
908	Kamtsas Formation	Conglomerate
909	Post Cret. on Kamtsas Form	Calcrete/hornfels
910	Kuibis Subgroup	Shale/hornfels
911	Kuibis Subgroup	Hornfels/calcrete
912	Post Cret. on Kuibis Subgroup	Calcrete
913	Kuibis Subgroup	Shale/hornfels
914	Post Cret. on Kamtsas Form	Hornfels/calcrete
915	Post Cret. on Kamtsas Form	Calcrete
916	Kuibis Subgroup	Shale/hornfels
917	Kuibis Subgroup	Shale/hornfels
918	Post Cret. on Mulden Group	Calcrete
919	Post Cret. on Mulden Group	Calcrete

APPENDIX 4: INFLUENCE OF ROUGHNESS ON FOREIGN EXCHANGE

In order to analyse the influence of the roughness of unpaved roads on foreign exchange transactions in South Africa a number of assumptions and estimations were necessary:

- The total unpaved road network in southern Africa was estimated at 200 000 km (provincial, national states, parks and industry).
- Each road was assumed to carry 50 vehicles per day of which 10 were classed as medium trucks (i.e. Tare weight 5 400 kg).
- The average road roughness at present was 100 QI counts/km and use of the proposed specifications would result in an average roughness of not more than 60 counts/km.

The average annual distance travelled would therefore be:

Cars - 200 000 x 40 x 365 = 2 920 000 000
Trucks - 200 000 x 10 x 365 = 730 000 000

The road user costs (per 1000 vehicle km) at a roughness of 100 and 60 counts/km were (after du Plessis et al, 1987):

	QI = 100 cnts/km	QI = 60 cnts/km
Cars -	R 570,10	R 471,92
Trucks -	1381,70	1046,95

Fuel

For cars, at QI = 100 fuel makes up 9,2 per cent of the total operating cost and at QI = 60, 10,8 per cent. For trucks these figures are 9,1 and 7,0 respectively.

Therefore, cost of fuel:

	QI = 100	QI = 60
Cars	R 153 151 664	R 148 824 691
Trucks	76 656 716	69 548 889
Total	229 808 380	218 373 580

Saving = R11 434 800

Lubricants

For cars, at QI = 60 and 100 lubricants makes up 1,2 per cent of the total operating cost. For trucks these figures are 0,9 and 0,8 respectively.

Therefore, cost of lubricants:

	QI = 100	QI = 60
Cars	R 19 972 800	R 16 527 200
Trucks	8 066 500	6 876 600
Total	28 039 300	23 403 800

Saving = R4 635 500

Tyres

For cars, at QI = 100 tyres make up 2,2 per cent of the total operating cost and at QI = 60, 2,1 per cent. For trucks these figures are 7,6 and 7,5 respectively.

Therefore, cost of tyres:

	QI = 100	QI = 60
Cars	R 36 623 224	R 28 938 134
Trucks	75 648 075	58 084 786
Total	102 271 299	87 022 920

Saving = R15 248 379

Maintenance parts

For cars, at QI = 100 maintenance parts make up 24,1 per cent of the total operating cost and at QI = 60, 16,9 per cent. For trucks these figures are 36,7 and 29,7 respectively.

Therefore, cost of maintenance parts:

	QI = 100	QI = 60
Cars	R 401 190 772	R 254 931 184
Trucks	370 171 019	226 989 230
Total	771 362 019	481 920 414

Saving = R289 441 605