


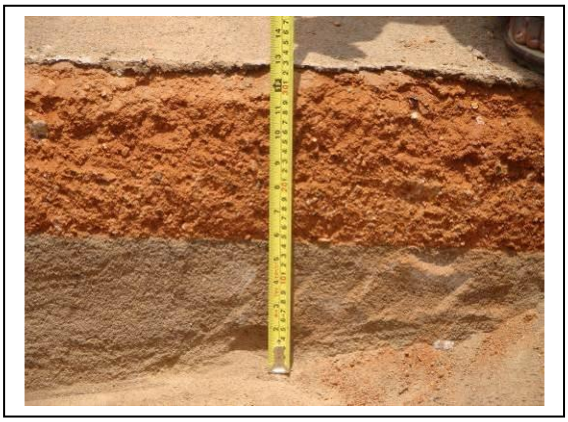

Back Analysis of Previous Constructed Low Volume Rural Roads in Mozambique

AFCAP/MOZ/001/G

Final Report

CPR1612

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June 2013

This project was funded by the Africa Community Access Programme (AFCAP) which promotes safe and sustainable access to markets, healthcare, education, employment and social and political networks for rural communities in Africa.

Launched in June 2008 and managed by Crown Agents, the five year-long, UK government (DFID) funded project, supports research and knowledge sharing between participating countries to enhance the uptake of low cost, proven solutions for rural access that maximise the use of local resources.

The programme is currently active in Ethiopia, Kenya, Ghana, Malawi, Mozambique, Tanzania, Zambia, South Africa, Democratic Republic of Congo and South Sudan and is developing relationships with a number of other countries and regional organisations across Africa.

This material has been funded by UKaid from the Department for International Development, however the views expressed do not necessarily reflect the department's or the managing agent's official policies.

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Preface

This is the Final Report for the Back Analysis Project. The report provides information on the activities carried out and the data that was collected during the execution of the project.

The report gives a brief on the preliminary activities and the reconnaissance surveys carried out during the early stages of the project. More details of these early stages are given in the Inception Report.

The report also gives a detailed account of the field surveys particularly the data that was collected from the measurements carried out in the field and the materials tests results.

Finally, the report provides results of the analysis and the recommendations and conclusions from the results of the analysis

Acknowledgments

TRL acknowledges the valued contribution and collaboration from ANE, particularly the General Director of ANE, the Directorate of Maintenance (DIMAN) and the Provincial Delegations of ANE, particularly the Delegates, the technical departments and the provincial materials laboratories and the Engineering Laboratory of Mozambique (LEM). TRL also acknowledges the contributions made by the Provincial Consultants, the Road Fund in Mozambique, Crown Agents, DFID (through UKAid) and SIDA (Swedish International Development Administration)

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Executive Summary

This report presents the findings of the Back Analysis Project and recommendations that were derived from the observations and results of the analysis of the information and data that were obtained.

In 2008, ANE initiated the Rural Road Investment Programme (RRIP) supported technically by the Africa Community Access Programme (AFCAP). The programme spanned over three Phases, 1, 2 and 3, the latter of which started in mid-2011. The main focus of the programme was to design and construct low volume roads using locally available materials and techniques that are not compliant with the current standards and specifications, with a view to providing additional research data to feed into the current development of specifications, work norms and guidelines for provision of low volume sealed roads (LVSRs) in Mozambique.

During the monitoring and evaluation of the performance of the pilot projects, it emerged that there was insufficient data to carry out a retrospective evaluation of the previously constructed roads. Hence in mid-2012, the Back Analysis Project was initiated to bridge the data gaps within the RRIP/AFCAP Phases 1, 2, and 3. These phases, including the Back Analysis Project are all complementary and essential components of the ongoing programme.

The Back Analysis Project has sought to evaluate the performance of low volume roads constructed 10 years ago and earlier. The criteria that were used for the selection of the study sections included the road classification (N1, R400, Unclassified), traffic levels, age of road, construction type, current pavement condition and the local knowledge and expertise of the ANE staff. On that basis, 21 sections on 8 roads in six provinces were deemed representative test sections and selected for the study in consultation with ANE.

The standards that were used for the design and construction were reviewed as part of the project. The condition of the roads has been assessed using visual and structural surveys. Samples of the constituent road layers were also taken from site and tested in the laboratories in Mozambique and the UK. The visual surveys, field investigations and laboratory test results were then aggregated and analysed to evaluate the performance of the test sections.

1. Road condition surveys (cross sections, cracking, potholes, patching, drainage, rutting and roughness).
2. PRIMA deflection surveys – similar in principle to the FWD and used to obtain information on deflection and elastic moduli of pavement at the centre of load.
3. DCP Survey- used for the rapid measurement of the in-situ strength of unbound pavement layers using the DN 800 Method.
4. Test pits - to obtain full information about the properties of each pavement layer at the test site.
5. Laboratory testing of surfacing, roadbase, subbase and subgrade materials obtained from the test pits.
6. Laboratory testing of control samples which included borrow pit materials for cement content tests and fresh bitumen from Mozambique and UK for control parameters and bitumen ageing simulation.

Lab testing of bases, subbases and subgrade

The tests carried out on samples of pavement materials obtained during excavation of test pits included grading, plasticity, in-situ moisture, maximum dry density and optimum moisture contents, soaked CBR and for some samples CBR at optimum moisture content (OMC). Tests were also carried out on stabilised materials including cement content tests and bitumen content for the emulsion treated bases (ETB).

Test carried out on bitumen recovery on surfacing samples include penetration tests, Brookfield viscosity, kinematic viscosity, ductility, field ionisation mass spectrometry (FIMS). Tests carried out on aggregate obtained from the surfacing samples include grading and petrography.

Control samples of fresh bitumen 80/100 and 70/100 were obtained from Mozambique and UK respectively in order to compare the test results of the recovered bitumen with the fresh bitumen and to carry out laboratory simulation of long term in-service ageing.

The tests that were carried out included rolling thin film oven test (RTFOT) to simulate in-service ageing, penetration tests, Brookfield viscosity, kinematic viscosity and ductility.

The results of the investigations and analysis of the data showed the following:

1. The pavements were substantial in terms of overall thickness as measured by the DCP. While the actual measured thicknesses of the bases were commensurate with low volume roads (LVRs) design (150mm+/-) the structural contribution of the subgrade in terms of the CBR values was high and equivalent to that of the subbase and this was a result of long term consolidation under traffic action. The traffic loading capacity of these pavements was much higher than is expected of LVRs, some in excess of 30MESAs before the subgrade fails.
2. There was no significant structural failure on the test sections even though,
    a. Some of the bases were very weak (soaked CBR = 5.1% and PI =19.8)
    b. Traffic loading was high (0.3 – 2.7 MESAs). In general maximum traffic loading for LVRs is 1MESAs
3. Most of the samples of bitumen extracted from surfacing samples were too hard with penetration ranging from 1 to 5dmm (fresh bitumen should be 80 to 100dmm and over >150dmm for MC3000). This showed bitumen which had exceeded its service life and had become brittle. Bitumen obtained from Otta seals had penetration values of close to 10dmm and hot sand asphalt was ranging from 18 to 35dmm. The ages of the bitumen were relatively low (4-7 years).
4. The fresh bitumen from Mozambique had a penetration value of 58 instead of 80 – 100dmm. No volatiles were found in the fresh samples when the gas spectrograph mass spectrometry (GSMS) tests were carried. This is a serious problem as loss of volatile is synonymous to loss of service life. This showed that the bitumen had hardened before it was applied on the road or harder bitumen had been supplied by the manufacturer.
5. Comparison of the test parameters for the fresh bitumen after ageing simulation and the recovered bitumen from surfacing samples showed that the bitumen was ageing too quickly on the roads (more than twice the expected rate).

Recommendations

1. When upgrading gravel roads it is important to follow the existing alignments and take advantage of the in-situ strength from the consolidation that would have occurred over long periods of time. This greatly increases the traffic carrying capacity of the pavements.

2. The good performance of the marginal materials that were used with high plasticity and low CBRs while carrying traffic that is much higher than is expected of low volume roads shows that the specifications for LVRs can be relaxed allowing the effective use of locally available marginal materials (see Table 7-2).
3. There is need to develop and implement a quality assurance system to ensure that the materials (including bitumen) are as specified and best practice is adhered to during handling and construction. This will also require that works are assessed and approved properly to ensure good value for money.
4. The rapidly deteriorating bitumen requires that timely maintenance interventions are carried out including crack sealing before the start for the rainy season or fog spray is applied to rejuvenate the surfacing or resealing is carried out before the existing surfacing reaches the end of its service life or before the deterioration index exceeds 10 (see deterioration index DI in Chapter 6)

As part of the project, a one day workshop was held on 13th June to disseminate the findings of the Phase 2, 3 and Back Analysis Project. This workshop was attended by stakeholders that included representatives of AFCAP, ANE, TRL, the consultants and contractors and the academia.

Definitions and abbreviations

AADT	Annual Average Daily Traffic
ADT	Average Daily Traffic
AFCAP	Africa Community Access Programme
ANE	Administracao Nacional de Estradas
ACV	Aggregate Crushing Value
CBR	California Bearing Ratio
CI	Coarseness Index
CrI	Cracking Index
CTB	Cement Treated Base
DI	Deterioration Index
DN 150	Number of DCP blows required to achieve a penetration of 150 mm
DN 300	Number of DCP blows required to achieve a penetration of 300 mm
DN 800	Number of DCP blows required to achieve a penetration of 800 mm
DSD	Double Surface Dressing
EOD	Environmentally Optimised Design
ETB	Emulsion Treated Base
FI	Fineness Index
FMC	Field Moisture Content
FWD	Falling Weight Deflectometer
Gc	Grading Coefficient
GM	Grading Modulus
HSA	Hot Sand Asphalt
IR	Reject Index
IRI	International Roughness Index
LEM	Laboratorio de Enginharia de Mocambique
Ls	Linear Shrinkage
LWD	Light Weight Deflectometer
MESA	Million Equivalent Standard Axles
NTEC	Nottingham Transportation Engineering Centre
OMC	Optimum Moisture Content
ORN	Overseas Road Note
PI	Plasticity Index
PL	Plastic Limit
PPI	Pothole Patching Index
RdI	Rut Depth Index
RTFOT	Rolling Thin Film Oven Test
SADC	Southern African Development Community
SATCC	Southern African Transport and communication Commission
SCC	Surrey County Council
SN	Structural Number (assumes that subgrade does not contribute to pavement strength)
SNP	Structural Number Corrected for Subgrade Strength Contribution
TRL	Transport Research Laboratory
WL	Liquid Limit

1 Introduction

This is the final report for the Back Analysis Project. The report covers the work carried out during the execution of the project. This comprised:

1. Literature review
2. Field reconnaissance
3. Field investigations
4. Laboratory testing
5. Analysis
6. Preparation of documentation

The project involved the back analysis of the performance of road sections that were built as far back as ten years ago and beyond. The purpose of the project is to develop an understanding of the performance of these road sections, most of which were built using low cost techniques and marginal materials. The data and information will contribute to the development of specifications for low volume roads, which is the main output of the project.

The information for the preliminary and reconnaissance surveys, including the final selection of sections, is given in the Inception Report. . It contains the information about the sites and all the preliminary work. This report gives a brief overview of the selected sites but with more emphasis on the information obtained from the field surveys, laboratory testing and the analysis of the performance data.

The preliminary work included:

i) Awareness meetings with the ANE Directorate of Maintenance (DIMAN) and ANE Provincial Delegations.
ii) Selection of sections on the candidate roads.
iii) Elementary condition surveys.
iv) Marking of sections.
v) Documentation of the preliminary information.

The main component of the project was the field investigations. A considerable amount of information was collected and is contained in the main body of the report. However, some of the more detailed information is given in the Appendix to avoid making the report itself too detailed, especially concerning data that did not help much in meeting the objectives. The data includes:

1. Visual condition surveys (condition of the carriageway and surfacing, drainage and vegetation).
2. Rut depth measurements.
3. Pavement strength measurements namely DCP tests and deflections tests with the Prima deflectometer.
4. Test pits and materials sampling.
5. Data coding and recording.

The analysis stage was crucial and involved:

1. Data cleansing to correct errors and obvious anomalies.
2. Data categorisation and analysis.

3. Presentation of trends.
4. Derivation of deterioration indices such as cracking index.
5. Derivation of performance indices.

The outputs of the analysis are crucial for determining the effect of the designs and resultant pavement strengths, as-built, to the performance of the roads under given combinations of deterioration factors which include traffic, climate, drainage and maintenance.

The TRL team, with the assistance of DIMAN and Provincial Delegations, particularly representatives from the laboratories and technical departments, partook in the field investigations exercise. ANE, through the Provincial Delegations, was mandated to assist with the logistics and support during the fieldwork. ANE's roles were coordinated from DIMAN through the designated counterpart engineer.

The data and information that comes out of the detailed field investigations is used to develop performance trends which give indications of the lower limits for the specifications for the materials and pavement structures for low volume roads. It is difficult to work out upper limits of specifications decisively because most of the roads have not reached the end of their service lives but upper limits are less important because they have no ramifications on the performance other than the cost of construction.

The main focus of the assessment of performance is to find out how much lower the specifications could be reduced without seriously compromising the life of the road. The principle was to build up data and information that can form the basis for developing appropriate specifications for low volume roads in consideration of the lower levels of traffic loading associated with these roads. However, general observations based on axle load surveys on some of the sites showed that overloading was prevalent and quite significant. This may have been one of the consequences of the lack of enforcement of axle load limits. Bearing in mind the huge impact of overloading on the performance of low-volume roads and the whole life costs of the road network, there is an urgent need to manage overload control, especially through enforcement using permanent and portable weighbridges.

2 Background

ANE, through the RRIP and support from AFCAP, undertook a programme of targeted interventions on some pilot projects on low-volume roads in Mozambique and a comprehensive research programme was incorporated into Phases 1, 2 and 3 of the AFCAP/RRIP.

The projects included design, construction and monitoring with the main output being recommendations on specifications and work norms for low volume roads. The performance of sections which were constructed under AFCAP/RRIP was monitored for more than two years. However, sealed roads require much longer periods of monitoring in order to develop evidence of significant differences in performance between different designs or design techniques. It was not possible to wait for the results of long term monitoring to obtain the necessary evidence hence it was decided to investigate the performance of existing low volume sealed roads which were built a decade or more ago. This involved determining their design, construction and in-service performance. The critical parameter that was missing from the previous project was 'Age'. The back analysis of older low-volume roads provided the opportunity to incorporate age into the analysis.

The overall objective of the assignment was to undertake a review of the performance of existing Low-Volume Sealed Roads (LVSR) that were built more than a decade or so ago, and the standards and specifications used for their construction.

The results were intended to provide additional research data to feed into the development of specifications, work norms and guidelines for provision of LVSRs in Mozambique.

The objective would be achieved through:

1. Site visits and desk study of historical projects.
2. Field Investigations, including collection of samples for testing in the laboratory.
3. Laboratory testing of materials.
4. Analysis of test results and reporting

The preliminary activities, and particularly the reconnaissance visits to the selected roads, provided vital information for the subsequent activities. The selection of test sections was key to the success of the project. The selection of the roads was carried out by ANE and it was later refined by TRL in order to meet the requirements of the assignment. It was important to cover a minimum set of parameters in order to produce a reasonable assessment of the performance of the roads. This is referred to as the research matrix.

1. Materials – the wide variety of materials found in Mozambique needed to be covered reasonably.
2. Climate – the general effect of climate on roads is well known but the effect is more pronounced on low-volume roads.
3. Traffic – traffic is a major causative agent of deterioration therefore the project needed to cover the range of traffic levels.
4. Surfacing types – surfacing type is a major element in the performance of low-volume roads. Better surfacing leads to higher durability and better performance but higher costs, however, the marginal materials that are often used for roadbases for low volume roads need good protection from the weather to achieve good long term performance.
5. Age – this was the main element of focus for the evaluation of performance. The age is critical in the development of specifications in order to determine the lower limits for incorporation in the documentation.

3 Preliminary Activities

Important information was collected during the early stages of the project through a desk study and reconnaissance surveys.

3.1 The Desk Study

This was carried out with the assistance ANE and some practitioners who were involved during the execution of the works. This entailed reviewing project reports which were prepared at the time. The information was particularly important because it gave the team some insight into what transpired during construction.

Not all information was readily available because some of the roads were built a long time ago, as long ago as colonial times in the early 70s. The information from that time was not readily available in ANE and this meant that the field investigations had to be tailor-made to decipher information relating to the design and construction of the sections.

A literature review was carried out on available reports relating to the project. These reports included the following:

- Estudo de Uma Argamassa Asalfatica-A Study of Asphatic Concrete produced by ANE in 2007. This was based on the trials carried out in the use of local sand to design and construct hot sand asphalt (HAS) mixes in Marracuene, Maxixe, Massinga and particularly on the Pambarra-Rio Save road. A grading envelope was produced, showing recommended ranges of gradings that could be suitable for use in the construction of hot sand asphalt.
- SISTEMA DE GESTÃO DA REDE VIÁRIA-RECENSEAMENTO DE TRÁFEGO 2010 –ANE (Traffic Report 2010)
- SISTEMA DE GESTÃO DA REDE VIÁRIA-RECENSEAMENTO DE TRÁFEGO 2011 – ANE (Traffic Report 2011)
- Rural Road Rehabilitation of N104 (EN239) Between Nametil and Angoche in Nampula Province, Mozambique: Project Completion Report. June 2007: ANE/Black and Veatch Africa
- Tender Document: Construction of Labour Based ETB Research Sections, Maputo: 2003: ANE/Kubu Consultancy
- Tender Document: Construction of Machine Based ETB Research Sections, Maputo: 2003: ANE/Kubu Consultancy
- A Guide to the Use of Otta Seals: Charles Overby, Directorate of Public Roads, Oslo, Norway
- Heavy Vehicle Overloading Control Study (March 2007) ANE/Africon

3.2 Reconnaissance Surveys

The reconnaissance surveys were carried out on all candidate roads included in Table 3-1. The main activities of these surveys were:

1. Meeting the provincial delegations of ANE to create awareness of the project and, in particular, the obligations of the ANE delegations in the execution of the project.
2. Assessing the capacities of the provincial laboratories in carrying out the necessary laboratory field work and, more importantly, the laboratory tests. This therefore included

the assessment of the equipment and personnel in the laboratories. This information was important because most of the testing of soil samples was going to be carried out at the ANE provincial laboratories.

3. Assessing the proposed roads and selecting possible representative sections of 300m length. The selected possible sites were marked with paint showing the chainages of the beginning and the end of each section. The selection criteria included:
    a. The level of deterioration of the section based on visual assessment, e.g. cracking and potholes.
    b. Types of materials used, particularly for the base.
    c. The type of surfacing used.
    d. Terrain and gradient.
    e. Drainage
4. Prioritising sections in order to include sections in different states of deterioration. The ideal situation was to select a section in good condition, one in fair condition and another in bad condition. The reason for this is that it is much easier to identify reasons for differential performance if as many variables as possible are the same (e.g. traffic, climate, subgrade).
5. Coding of the information collected during the reconnaissance surveys.

Altogether, 10 sites were selected in 6 provinces and the information is given in Table 3-1.

Table 3-1 Preliminary list of test sites

Province	Road No.	Road Name	Length (km)	Year of Construction	Contractor	Consultant	Type of Surface	Type of Base	Design / Construction documents	Maintenance records	Traffic (2011 AADT)	Annual Average Rainfall (mm) (1996-2011)	
													Station
MAPUTO	Unclassified	Marracuene - Ferroviario	2	2005	CETA	Africon	Sand Seal	Emulsion Treated Sand Base	Method statement/ specifications	No Information		806	Marracuene
MAPUTO	R400	Boane-Pequeno Limbombos-Goba	3	No Information			Double Surface Dressing	Crushed rock/cement treated base	No Information	No Information	1,197	No Information	
INHAMBANE	N1	Pambarra - Rio Save	13	2009	CETA	Stange	Hot Sand Asphalt	Patched Double Surface Dressing on cement treated base	Design Charts	No Information	538	532	Vilanculos
ZAMBEZIA	N1	Nicoadala - Rio Zambezi	1.8				Hot Sand Asphalt	Crushed stone	No Information	No Information	512 (Rio Zambezi) 548 (Rio Luala) 864 (Nocoadala)	894	Caia
ZAMBEZIA	N10,N11	Quelimane - Namacurra	5.2				Hot Sand Asphalt	Macadam	No Information	No Information	1975 (Quelimane) 1185 (Namacurra)	1,288	Quelimane
NAMPULA	N104	Nametil - Angoche	7.2	2007	CETA/CMC JV	Black & Veatch	Sand cover on Otta Seal	Laterite	Completion Report	No Information	229	1,089	Nametil
CABO DELGADO	N380	Oasse - Mocimboa da Praia	3.1	2001	OPCE/MOTA	CPG/CPP	Sand Seal	Emulsion Treated Sand Base	No Information	No Information	218	1,279	Mocimboa da Praia
NIASSA	N13	Lichinga - Mandimba	13	2005	CMC Africa Austral	ANE	Otta Seal	Natural Gravel	Completion Report	No Information	290 (Lichinga) 80 (Mandimba)	1,279	Lichinga
NIASSA	N361	Maniamba - Metangula	12	2008	CMC Africa Austral	ANE	Otta Seal	Natural Gravel	Project Progress Report	No Information	55	1,279	Lichinga

4 Field Surveys

The general approach to the field surveys involved a sequential and incremental assessment of the 300m sections to determine the weakest areas or points within each test section. During the reconnaissance surveys, sections of varying lengths were selected and marked. However, comparison would be difficult should different lengths be considered during investigations. Standardisation of test sections to a uniform length of 300m was considered to be appropriate in order to meet several assumptions.

1. 300m length is short and it can be assumed that there would only be small variations in materials for the base, sub-base and subgrade taking into account that each 300m section exhibited uniform visual surface condition.
2. It could also be assumed that similar or uniform construction must have been achieved through the 300m length.
3. The section is long enough to be representative of the longer sections with similar surface conditions. In other words the 300m sections were considered to be a representative sample of the selected sections which were considered to be good or fair or poor during the reconnaissance surveys.
4. It was assumed that any uniformity or lack of it would also be representative of the longer sections selected during the reconnaissance surveys.

Each 300m test section was divided into 50m segments. Chainage marks were placed at 50m intervals and the main test sections were located at these sections either at the centerline or centre of each lane. The criterion for determining whether to locate the tests at the centerline or centre of the lane was mainly based on the road width. Most low volume roads are narrow i.e. 5m to 6m width and mostly without the centerline marking. In this case traffic tends to ride in the middle of the road but in general terms there is no defined wheel track. Wider road carriageways tend to encourage separation of traffic travelling in either direction. In this case there are defined wheel paths with distinct inner and outer wheel tracks. Where there was a difference in condition between the lanes then the tendency was to place test points at the centre of each lane.

4.1 Visual Condition Surveys

Surface defects are a good indication of the soundness of the road and the pavement in particular. Such defects include:

1. Cracking – the appearance of cracks shows problems in either the base or surfacing or both. It may be caused by normal fatigue in the pavement or surfacing due to repeated loading or deterioration of the binder through loss of volatiles and oxidation of the bitumen. Poor construction can also be a contributing factor through under-application or overheating of the binder.
2. Potholes and patching – this is a more advanced stage of deterioration where the cracks deteriorate further allowing more water to enter and disrupt the road structure. This could have occurred because of a poor surfacing, a lack of timely maintenance or a consequence of a weak roadbase. It often occurs more rapidly on low volume roads because the bases of such roads are more likely to be constructed using marginal materials which tend to be moisture sensitive.
3. Rutting and Deformation – this is an indication of a structural failure in one or more of the road layers or a slip failure within the subgrade. It normally requires an in-depth assessment of the materials and layer strengths to identify the probable primary cause. For example, it

could be a result of consolidation following poor compaction during construction or the effect of heavily loaded traffic. In the least severe cases it may simply be some additional (secondary) compaction in the wheel paths which stabilises after a short time and does not progress to shear failures.

4. Drainage – water is roads' worst enemy, thus any evaluation of performance is incomplete without an assessment of the drainage, both surface and sub-surface drainage. Most materials that are used for the construction of low volume roads are moisture sensitive and ingress of moisture can have significant influence on the strength of the pavement.

5. Crown height above drain level. The current specifications based on environmentally optimised design (EOD) specify a minimum crown height of 750mm. It is assumed that this creates a drier environment for the pavement layers and thus enhances their performance. The crown heights of the different sections were measured in order to determine whether crown height had a significant effect on the performance of the test sections.

6. Camber – the magnitude of the cross-fall determines how effectively water is dispersed from the carriageway. This helps in preventing or reducing moisture ingress from the surface.

7. Road width – this is actually an issue on low volume roads in terms of the distribution of the traffic loading over the carriageway. On narrower roads the wheel tracks are less defined as vehicles tend to ride over the whole width of the carriageway. On wider roads vehicles tend to ride on defined wheel tracks with an inner and outer wheel track. Rutting is easier to measure in this situation

4.2 Axle Load Surveys

Axle load surveys were carried out on some of the selected sites. The purpose of this exercise was to develop a general understanding of the loading characteristics on low volume roads. It was anticipated that while there could be some medium to heavy trucks plying these low volume roads it is possible that the axle loads were lower than on the main roads. The cargo may generally be lighter. However, there could be a possibility of overloading on the few trucks that use these roads, particularly the rural roads because there is no overload control on them.

It is possible that such overloading, and the resulting high wheel loads, could have devastating effects on the performance of the low-volume roads where low strength or marginal materials and thin surfacings may have been used. The damage by a single truck can be significant leading to premature failure even at low traffic volumes. Figure 4-1 shows an overloaded truck on a low-volume road with single tandem axles.

Figure 4-1 Axle load survey – heavily loaded truck with single tandem axles

4.3 Deflection Tests

Deflection tests were carried out using a light weight deflectometer (LWD) which was temporarily imported from TRL in the UK, Figure 4-2. The principle of testing pavement stiffness is similar to that of the falling weight deflectometer (FWD) except that only one sensor or geophone is used instead of the 7 geophones used on the FWD. Nonetheless the LWD provides information on the deflection and elastic modulus of the pavement at the centre of the load (it is possible to use two more geophones on the LWD but these were not provided for this project).

Figure 4-2 Deflection test using a light weight deflectometer (LWD)

Deflection tests are a rapid assessment of the pavement's elastic or load-spreading properties. They are not directly related to strength per se although strength and elastic properties tend to be loosely correlated.

The tests were carried out at 50m intervals through the 300m test sections. The data was also recorded manually in case something went wrong with the hand held PDAs. Using the data from the LWD it was possible to determine the weakest points out of the sample of test data. Out of these data the two weakest points and one strongest point were considered for further tests using the dynamic cone penetrometer (DCP).

4.4 DCP Tests

While the deflection tests provide information on pavement stiffness, the DCP provides information on the strength and thicknesses of pavement layers. The in-situ CBRs obtained through the DCP tests (Figure 4-3) were particularly important in defining the weakest points of the test sections.

Following the DCP tests, the results were analysed using the UK DCP software to determine some strength parameters.

1. The in-situ CBRs of the base, the subgrade and other layers in-between.
2. The layer thicknesses.
3. The structure numbers (SN, SNP)

Using these data it was possible to assess the strength of the pavement and to compare with the stiffness tests. This approach was used to determine the weakest of the three points. This was the location of the test pit.

Figure 4-3 DCP test carried out on the sub-base and subgrade under the cement treated base

4.5 Test Pits

While DCP tests provide information about the layer thicknesses and their strengths, they do not provide all the information needed for a full evaluation of the pavement. They do not, for example, give moisture contents, densities, plasticity, or particle size distribution, information that is vital for a full evaluation.

Test pits measuring 1m x 1m on the surface were dug to the level of the subgrade, Figure 4-4. It was important to sample all pavement layers including the subgrade material. Care was taken to prevent contamination of the sample of one layer with material from another layer.

1. The position of the test pit was determined and marked with paint. It was necessary to ensure the DCP test point was inside the marked area for the test pit. This would make it easier to correlate the DCP information to the visual assessment of the layers as revealed through the test pit.
2. The edges of the test pit were cut with pickaxes.
3. The surfacing was removed and bagged for laboratory testing for bitumen content and bitumen quality tests.
4. Once the roadbase was reached the upper part of the base which was contaminated with bitumen was removed in order to ensure that uncontaminated material was retained for sampling
5. Once the homogeneous roadbase was reached, a small sample was immediately collected for moisture content measurements to minimize the effects of drying out whilst sampling. The soil was placed in small plastic bags and sealed off and the bags labeled.
6. 50kgs were collected of the base material and subsequent layers down to the subgrade, all bagged and labeled.
7. The edge of the test pits was shaped to produce a vertical face so that the layer thicknesses could be measured and identified distinctively. Photos of the materials from the individual layers and the vertical profile were collected and will form part of the analysis and database.
8. The soil samples were given to the provincial laboratories and the surfacing samples were sent to Maputo for testing at LEM and onward delivery to the TRL laboratory in UK.

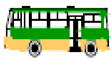
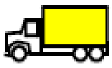
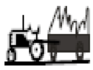
The provincial delegations organised the maintenance contractors to reinstate the test pits and this was done.

Other relevant data was also collected. The primary source of traffic data was ANE and the data is given in Table 4-1 and traffic categories used by ANE are given in Figure 4-5. The rainfall data was obtained from the Meteorological Office and is given in Table 4-2.

Table 4-1 Traffic data for selected sites

PROVINCE	ROAD NUMBER	ROAD NAME	STATION	CODE	DESCRIPTION	TRAFFIC											
							A	B	C	D	E	F	G	H	LIGHT	HEAVY	TOTAL
MAPUTO	N200	Boane-Libombos	104	T1131	Boane: Junction with R407	Daily Traffic	205	75	36	4	77	42	41	6	**315**	**171**	**486**
						Percentage %	42	15	7	1	16	9	8	1	**65**	**35**	**100**
INHAMBANE	N1	Pambarra-Rio Save	321	T3019	Vilankulo	Daily Traffic	198	125	127	26	66	48	80	2	**450**	**222**	**672**
						Percentage %	29	19	19	4	10	7	12	0.1	**67**	**33**	**100**
CABO DELGADO	N380	Oasse-Mocimboa da Praia	932	T9108		Daily Traffic	329	202	177	9	155	70	44	5	**709**	**284**	**993**
						Percentage %	33	20	18	1	16	7	4	0.1	**71**	**29**	**100**
NIASSA	N361	Metangula-Maniamba	1021	T1074	Mbembe-Maniamba		32	39	37	1	32	11	10	5	**108**	**58**	**166**
	N361	Maniamba-Lichinga	1021	T1074	Mbembe-Maniamba	Daily Traffic	32	39	37	1	32	11	10	5	**108**	**58**	**166**
			1021	T1074	Mbembe-Maniamba	Percentage %	32	39	37	1	32	11	10	5	**108**	**58**	**166**
	N13	Lichinga-Mandimba	1015	T1067	Metonia	Daily Traffic	71	79	41	4	59	21	14	2	**191**	**100**	**191**
						Percentage %	24	27	14	2	20	7	5	1	**66**	**34**	**100**
NAMPULA	N104	Nametil-Angoche	807	T8138	Moquito-Boila	Daily Traffic	73	43	8	3	89	6	4	3	**125**	**105**	**229**
						Percentage %	32	19	4	1	39	2	2	1	**54**	**46**	**100**
ZAMBEZIA	N1	Rio Zambezi-Nicoadala	706	T7756	R641	Daily Traffic	289	62	94	43	259	39	77	1	**445**	**419**	**864**
						Percentage %	33	7	11	5	30	4	9	0.1	**52**	**48**	**100**

Para a classificação do tráfego para o recenseamento de 2011 da ANE, foram adoptadas 8 classes de veículos conforme a seguinte descrição:

Classe		Descrição
A		Veículos ligeiros simples e de tracção às 4 rodas
B		Veículos ligeiros de mercadorias (eixo traseiro com 2 rodas)
C		Veículos ligeiros de passageiros (minibus: capacidade inferior a 20 pessoas)
D		Veículos pesados de passageiros (autocarros)
E		Veículos pesados de mercadorias com 2 eixos (rodados duplos no eixo traseiro)
F		Veículos pesados de mercadorias com 3 ou 4 eixos
G		Veículos pesados de mercadorias com mais de 4 eixos
H		Tractores agrícolas com ou sem reboque

Quadro 2 – Classes de Veículos Recenseadas

Figure 4-5 ANE's vehicle classification chart

Table 4-2 Rainfall data

Road Name	Rainfall (mm/yr)
Boane Libombos	806
Pambara Rio Save	532
Rio Zambezi Nicoadala	894
Nametil Angoche	1089
Oasse Mocimboa da Praia	1279
Lichinga Mandimba	1279
Metangula Maniamba	1279
Maniamba Lichinga	1279

5 The Test Roads

5.1 Boane Libombos

5.1.1 General description

This road is in Maputo Province and it was classified as a low volume road at the time of rehabilitation. The current traffic volume (AADT) is 486, of which are medium and heavy vehicles. Records from ANE show that the road was last upgraded in the early 1990s. The average rainfall is 806 mm. The road was built using natural gravel which was predominantly basaltic gravel and sealed with a double surface dressing.

5.1.2 Visual observations

Six sections were selected on this road. These are summarised Table 5-1 and their condition is illustrated in Figure 5-1.

Table 5-1 Sections on Boane Libombos Road

Section	Chainage		Brief description	Figure No
	From	To		
1	4+700	5+100	Very little cracking Rutting rarely exceeded 10mm. There is no patching and the crown height is generally good (379mm on gentle slope). Considering the traffic over 13 years this probably indicates that the road base and overall structure are satisfactory. The DCP and test pit data shown in Table 5-2 confirm this.	5.2 and 5.3
2	5+300	5+700	Very extensive cracking and a few potholes. The rutting is not serious but Figs 5.2 and 5.3 indicate that rutting is probably beginning to follow on from the cracking. The cracking was attributed primarily to aging of the surface.	5.2 and 5.3
3	8+000	8+400	Some cracking and no significant rutting; cracking clearly preceding rutting indicating adequate base strengths for the traffic. Some minor deformation was noticed in the wheel track on the right hand side possibly caused by poor subsurface drainage.	5.4 and 5.5
4	8+400	8+700	Extensive cracking, no ruts and minor deformation. Cracking preceding rutting indicating adequate base strengths for the traffic.	5.4 and 5.5
5	8+700	9+000	Extensive cracking, no ruts and minor deformation. Cracking clearly preceding rutting indicating adequate base strengths for the traffic.	5.4 and 5.5
6	9+050	9+350	This section is in good condition and it is very smooth. No cracking or potholes were noticed. It appeared as though there had been some recent rehabilitation works carried out on it.	5.4 and 5.5

Section 1

Section 2

Section 3

Section 4

Section 5

Section 6

Figure 5-1 Sections on the Boane Libombos road

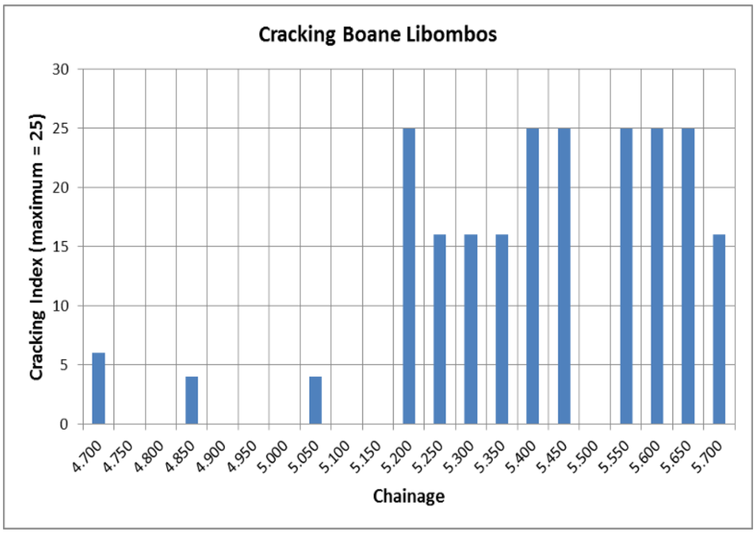


Figure 5-2 Cracking Sections 1 and 2 of the Boane Libombos road

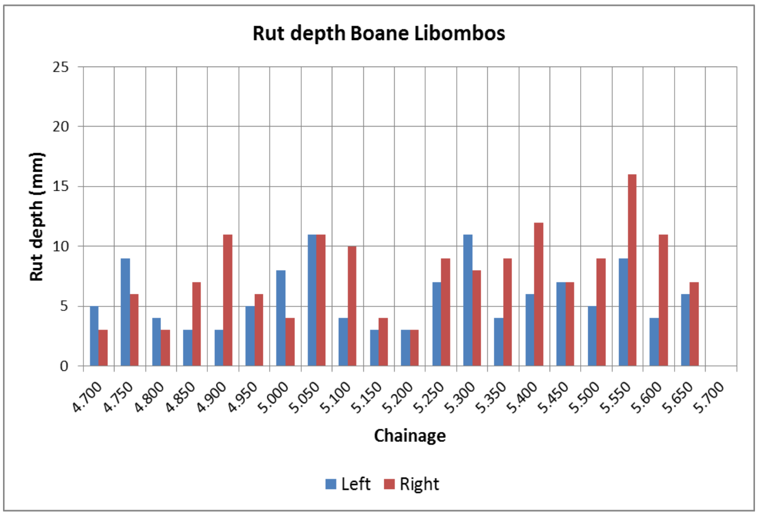


Figure 5-3 Rut Depth Section 1 and 2 of the Boane Libombos road

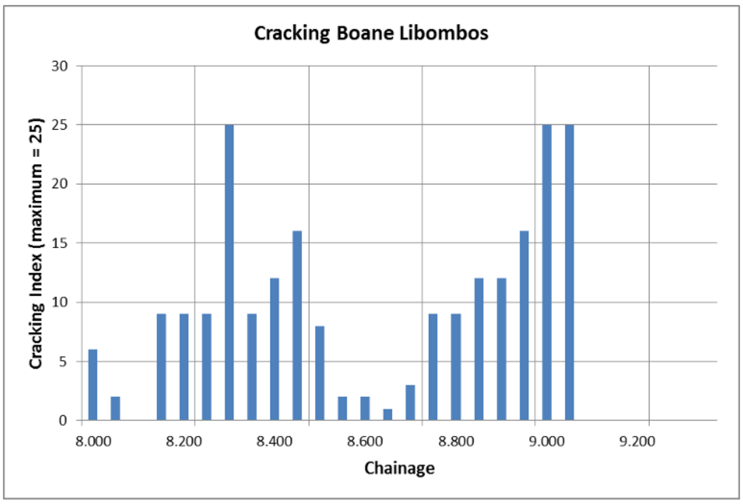


Figure 5-4 Cracking Section 3, 4 and 5 of the Boane Libombos road

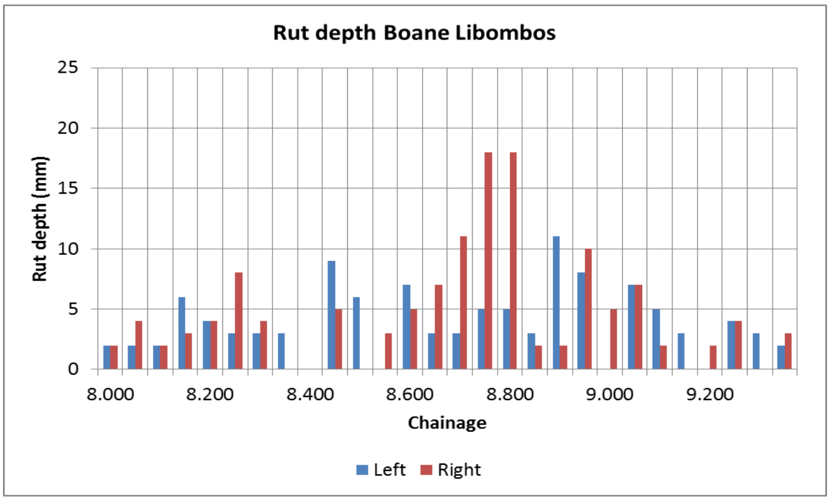


Figure 5-5 Rut Depth Section 3, 4 and 5 of the Boane Libombos road

5.1.3 DCP and test pit data

Table 5-2 shows a summary of the DCP and Prima deflection data.

Table 5-2 Summary of DCP and Prima deflection data on the Boane Libombos road

Chainage	In situ base CBR	Subgrade CBR	Depth of base	Depth of sub-base	SNP	DN150	DN300	DN800	Prima Modulus (MPa)
4.700	100+	100	140	660	4.19	105	160	380	525
5.050	95	50	144	656	4.05	50	85	190	165
5.100	100+	100	175	395	4.33	90	220	500	574-
5.350	100+	100	175	625	4.37	130	210	500	362
5.500	110	100	200	427	4.28	40	90	195	232
5.650	100+	100	200	444	4.44	40	110	350+	164
8.100	60	50	200	490	4.20	25	65	260	267
8.200	60	75	200	513	4.15	25	65	160	182
8.350	80	50	176	518	4.22	35	75	225	239
8.450	40-65	50	196	387	4.03	17	45	180	203
8.550	65-75	100	228	508	4.54	40	85	270	235
8.650	65-100	75	157	576	4.19	45	110	240	310
8.900	65-90	100	200	515	4.49	40	85	500	295
9.100	100+	100	212	521	4.51	100	155	285	333
9.200	100+	50	175	501	4.26	60	150	311	286
9.300	100+	50	160	474	4.16	65	120	250	316

The pavements were all deep with high strengths at subgrade level. Since the pavements are thick the corrected structural number (SNP) has been used to indicate overall strength and the values for all test points show that failure in the sub-base or subgrade is very unlikely indeed.

The DN 150 value shows the number of DCP blows to reach a depth of 150mm (similarly DN 300 and DN 800 show the number of blows to reach 300mm and 800mm respectively). All test points show high values for DN 800 indicating overall strength; some are exceptionally strong.

The SNP value also indicates overall strength. SNP includes the subgrade and hence, since the subgrades are strong, SNP is not sensitive to weaknesses in the road bases; DN 150 should be used for that.

As indicated by DN 150, some test points show relatively weak layers near to the surface. The weakest sections are highlighted in red, mainly in Section 3, but rut depths are actually very low on this section indicating that structural failures have not occurred.

Table 5-3 is similar to Table 5-2 except the data are more specific and includes in-situ roadbase CBRs and the results of laboratory tests on the materials. The roadbase materials in section 3 and 4 are outside normal specifications

Table 5-3 Summary of Test Pit data on the Boane Libombos road

Chainage	In situ road base CBR %	In situ moisture content %	Laboratory soaked CBR at Mod AASHTO %	Optimum moisture content %	Laboratory OMC CBR at Mod AASHTO %	Ratio FMC/OMC	PM	PI
5.050	95	6.0	90	9.3	NA	0.64	330	13
5.500	110	0.4(1)	86	8.2	NA	NA	0	0
8.200	60	6.5	69	8.1	NA	0.80	465	14
8.450	40-65	7.5	41	8.3	NA	0.90	0	0
9.100	100++	3.8	130	7.5	NA	0.5	157	9

5.2 Pambarra Rio Save

5.2.1 General description

This is a very old section on the N1 built more than 30 years ago and there are no as built records available. This study section is part of a national highway, and, carries traffic of approximately 500 vpd, of which a considerable number consists of heavy vehicles. The test section lies in an area of moderate rainfall (532 mm).

The pavement structure consisted of a single base layer 150mm thick overlying a red silt subgrade. Red silt was also used for the base and stabilised with cement. This is a typical low volume road design where material is poor and cannot be used untreated. The content of cement used for stabilisation is unknown. The condition of the section is illustrated in
Figure 5-6 and described in Table 5-4.

Figure 5-6 Hot sand asphalt surfacing over cement treated base on the Pambara Rio Save road

Table 5-4 Sections on the Pambarra Rio Save road

Section	**Chainage**		**Brief description**
	From	To	
1	22+600	22+900	The section was originally surface dressed but this had deteriorated over many years. A recent reseal of hot sand asphalt was still in good condition with no visible cracking, deformation or rutting hence no cracking or rutting graphs have been drawn.

5.2.2 Visual observations

Once the surfacing was removed it was observed that the base had not cracked, as is usually the case with cement-stabilised sand bases. Usually 5-7% cement is used in the stabilisation process and, at this high cement content, block cracking is common. This section has lasted a very long time without failing with the exception of the double surface dressing whose failure is attributed to aging.

The plan was to determine the cement content of the base and to understand why the CTB did not crack. A sample was collected for cement content tests. A natural sample was also collected from the adjacent in-situ material to carry out control cement content tests. These tests were carried out in the UK.

5.2.3 DCP and test pit data

DCP tests could not be carried out because the base was too strong to be penetrated. DCP tests were carried out on the sub-base and subgrade. The results are shown in Table 5-5.

Table 5-5 Summary of DCP and Prima deflection data on the Pambarra Rio Save road

Chainage	Subgrade CBR	Depth of base	Depth of subbase	Total depth (mm)	SNP	Prima Modulus (MPa)
22+650	21	150	652	837	4.01	653

5.3 Oasse Mucimboa da Praia

5.3.1 General description

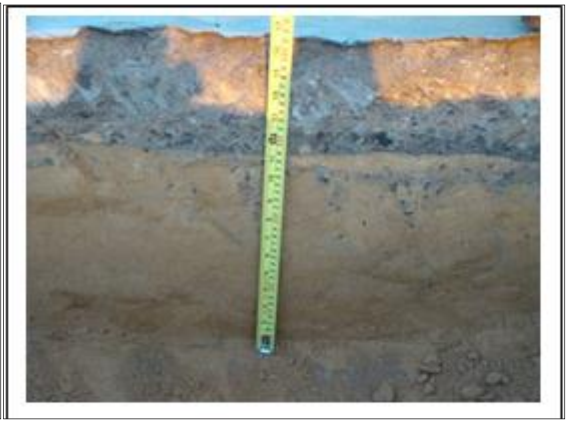
This road is a low volume road situated in the sandy coastal area of Cabo Delgado Province. The road is in a is in a moderate to dry and very hot climate. The road has performed relatively well compared to the Macomia Oassie Road with similar design which has failed. The roadbase was built with sand stabilised with stabilised with emulsion and surfaced with sand seal. The condition of the sections is shown in Figure 5-7 and described in

Table 5-6.

Section 1

Section 2

Section 3



Section 3 test pit

Figure 5-7 Sections on the Oase Mocimboa da Praia road

Table 5-6 Sections on the Oassie Mucimboa road

Section	Chainage		Brief description	Figure no.
	From	To		
1	0+000	0+300	Good condition. Some cracking. Rutting did not exceed 10mm but weakly correlated with the cracking.	Fig 5.8 and Fig 5.9
2	2+800	3+100	Some cracking. The rut depths are slightly higher than expected but not obviously correlated with the cracking	Fig 5.10 and Fig 5.11
3	19+800	20+125	Failed	Not plotted

5.3.2 Visual observations

Three sections were selected in good, fair and poor condition. Figure 5-8 and Figure 5-9 show that Section 1 had some cracking and that rutting did not exceed 10mm. This probably indicates that the road base and overall structure are satisfactory for the traffic but the DCP and test pit data shown in Table 5-7 indicate a relatively weak roadbase and no obvious difference between the three sections. In fact there is an exact correspondence between the in-situ roadbase CBR by DCP and the DN 150 value, as would be expected (DN 150 < 40 indicates insitu CBRs < 80%).

The data shows wide structural variability along all sections and any structural failures would be expected to reflect this with no single section being better or worse than another. Some of the *overall* structures, for example, are also weak as indicated by the DN 800 values.

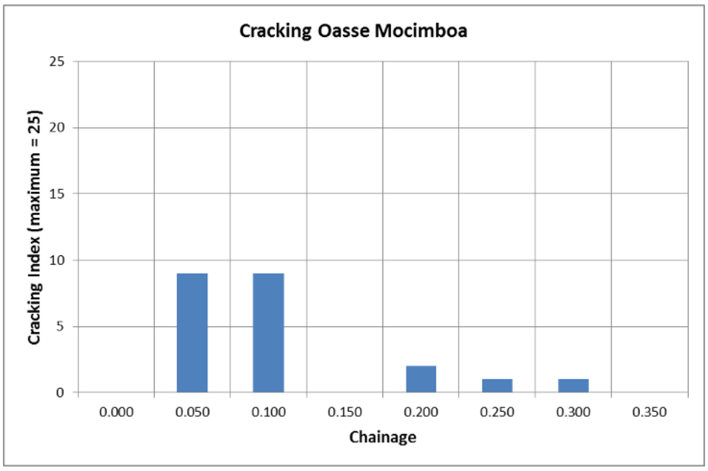


Figure 5-8 Cracking on Section 1 of the Oassie Mucimboa road

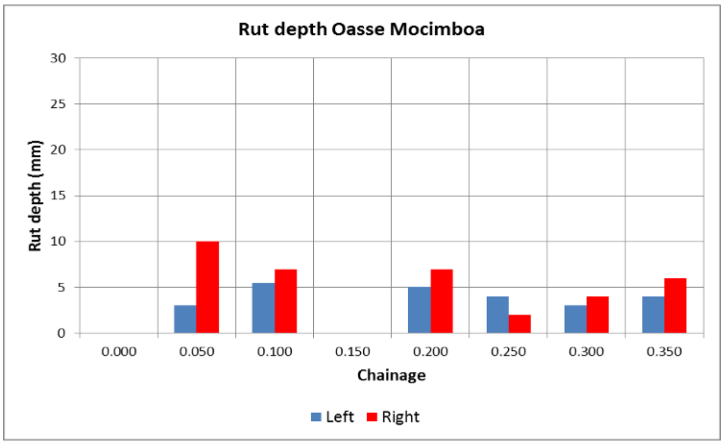


Figure 5-9 Rut depth on Section 1 of the Oassie Mucimboa road

Section 2 also shows some cracking (Figure 5-10) but the rut depths, Figure 5-11, are slightly higher than expected and could indicate a weakness The DCP data in Table 5-7 shows that the road is strong.

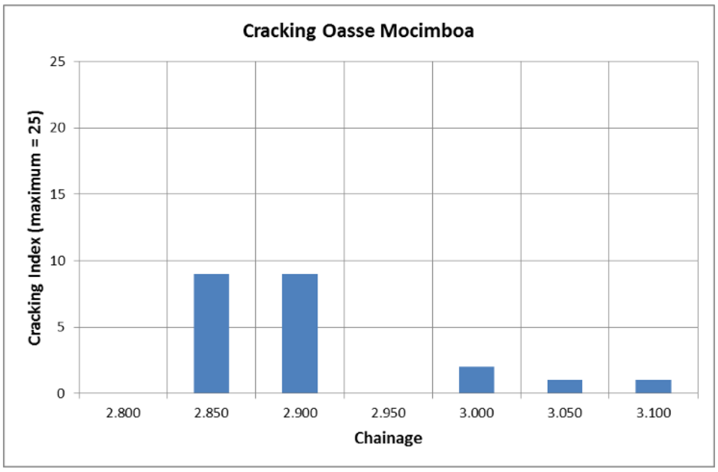


Figure 5-10 Cracking on Section 2 of the Oassie Mucimboa road

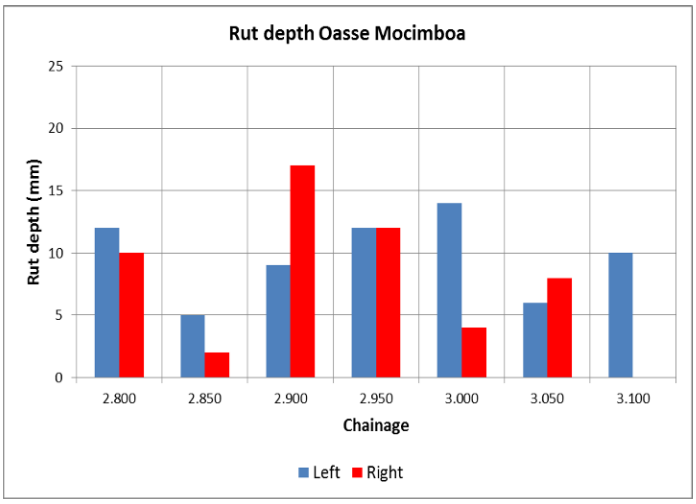


Figure 5-11 Rut depth on Section 2 of the Oassie Mucimboa road

The third section is a very poor section that has failed and is heavily potholed. However it appears to be no weaker than Sections 1 and 2. There could be several possible reasons for this failure.

1) Poor construction of the ETB could have led to its failure or perhaps the failure of the interface between the ETB and the sand seal surfacing. This could have caused the surfacing to delaminate and peel off exposing the ETB.

2) The failure could have been caused by failure of the sand seal. This would also have led to the exposure of the ETB.

However, identifying the cause when the road is in such a poor state is difficult, if not impossible. The main focus is on the sections that performed well because this will help in the development of specifications defining what has worked and why.

5.3.3 DCP and test pit data

Table 5-7 summarises the DCP data. The red text indicates the weaker materials.

While the DCP penetrated relatively easily through the upper layer, there was a very hard layer at approximately 700mm depth.

During the excavation of the test pits it was discovered that the hard layer was consolidated greyish sand which was apparently also quite dry. This was a surprise because the layer above it was wet and soft and there was no particular reason why the layer would have remained relatively drier. It could be that the sand had consolidated so much that it prevented moisture ingress into the layer.

Table 5-7 Summary of DCP and Prima deflection data on the Oassie Mucimboa road

Chainage	In situ base CBR	Subgrade CBR	Depth of base	Depth of sub-base	SNP	DN150	DN300	DN800	Prima Modulus (MPa)
0.050	65	100	140	506	3.72	35	60	200	717
0.150	60	35	190	662	3.87	35	58	175	1371
0.200	80	20	123	652	3.66	40	38	240	1443
0.250	70	18	171	683	3.81	35	65	165	862
2.850	90	45	200	588	4.29	45	95	240	2467
2.900	50	9	200	528	3.31	30	50	95	866
3.100	60-80	25	115	687	3.56	40	70	135	3008
19.900	50	30	182	604	3.46	30	45	85	735
20.000	90	20	126	697	3.53	45	65	125	783
20.150	60-110	75	179	428	4.25	40	105	425	-

Table 5-8 Summary of Test Pit data of the Oassie Mucimboa road

Chainage	In situ road base (ETB) CBR %	
0.050	65	No laboratory CBR tests can be carried out on ETB
0.150	60	
0.200	80	
0.250	70	

The main focus of the investigation is now on determining the bitumen content of the ETB and that of the of the sand surfacing because no laboratory CBR tests can be carried out on ETB,

Table 5-8.

5.4 Mandimba- Lichinga Road

5.4.1 General description

Changes were made to the previous selection of test sections made during the reconnaissance survey because of ongoing work on this road by the contractor. The sections finally chosen were older and fitted better with the main objective of the project.

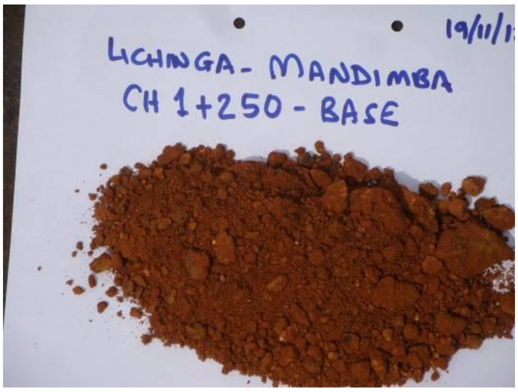
The interventions on this road were of a spot improvement nature carried out in 2005. There were also some sections on this road which were upgraded to sealed road standards. The materials that are available within economic distances are generally plastic and this makes the greater part of the 150km road slippery. The sealed sections on the Mandimba side were much longer and more recent, 2007. Table 5-9 describes the Sections and Figure 5-12 gives illustration of the condition of the sites.

Section 1

Section 2

Figure 5-12 Conditions of sections on the Lichinga Mandimba road

Heavy trucks on Lichinga Mandimba



Laterite base on Lichinga Mandimba Section 1

Figure 5-13 Overloaded trucks and Laterite base on Section 1 of the Lichinga Mandimba road

Some of the trucks using this road were heavily loaded possibly because there are no weighbridges for overload control.

Table 5-9 Sections on the Mandimba Lichinga road

Section	Chainage		Brief description	Figure no
	From	To		
1	1.150	1.450	This section has a very steep gradient. It is in reasonable condition. There is some cracking but no significant rutting. The roadbase material was fine laterite. The surfacing was an Otta seal constructed using fine laterite aggregate.	Fig 5.12, Fig 5.14 and Fig 5.15
2	19.550	19.850	The section had failed. Serious cracking and potholing had occurred and rut depths appeared to be increasing. There was also a lot of patching. The section was on a gentle to flat gradient and drainage was good	Fig 5.12, Fig 5.14 and Fig 5.15

5.4.2 Visual observations

There exist deposits of laterite, which are good for road construction in this area in contrast to other areas in most of Mozambique where good road building materials are scarce. The section from 1+150 to 1+450 has performed well with no serious defects (Figure 5-14 and Figure 5-15).

The DCP results show that the base is weak (Table 5-10 and Table 5-11) but the overall pavement is deep. There were several other layers of gravel including a quartzitic gravel layer before the subgrade was reached.

The Otta seal on section 0.150 to 0.450 was constructed on a very steep section. The performance of the Otta seal at this gradient was something to note and investigate because it is difficult to construct Otta seal surfacing properly on very steep slopes.

The second section from 19+550 to 19+850 had failed with excessive potholing and patching (Figure 5-17) and a deep rut at the beginning, otherwise deformation was slight and in the outer wheel tracks.

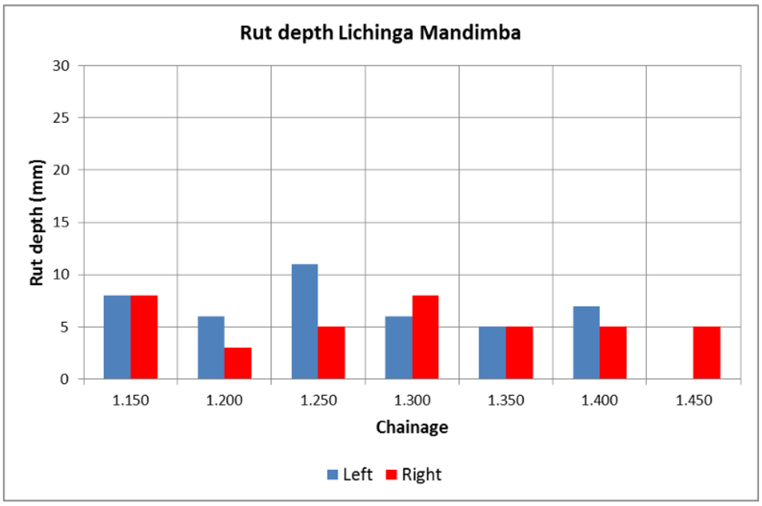


Figure 5-14 Rut depth on Section 1 of the Lichinga Mandimba road

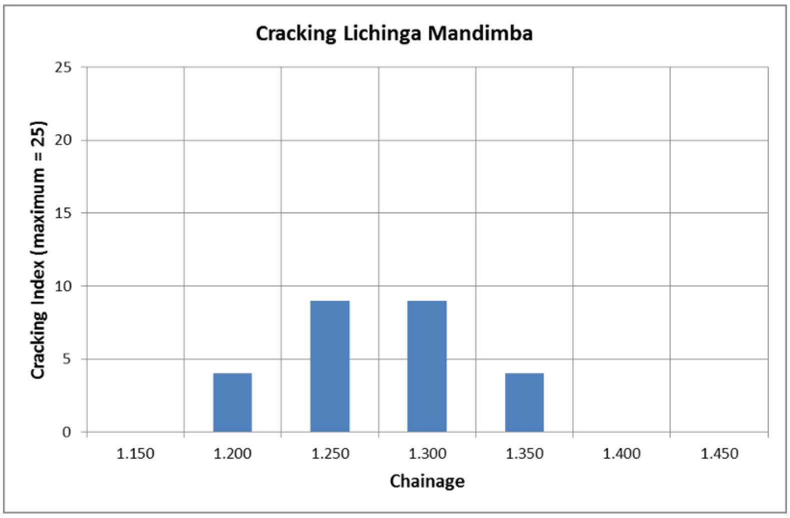


Figure 5-15 Cracking on Section 1 of the Lichinga Mandimba road

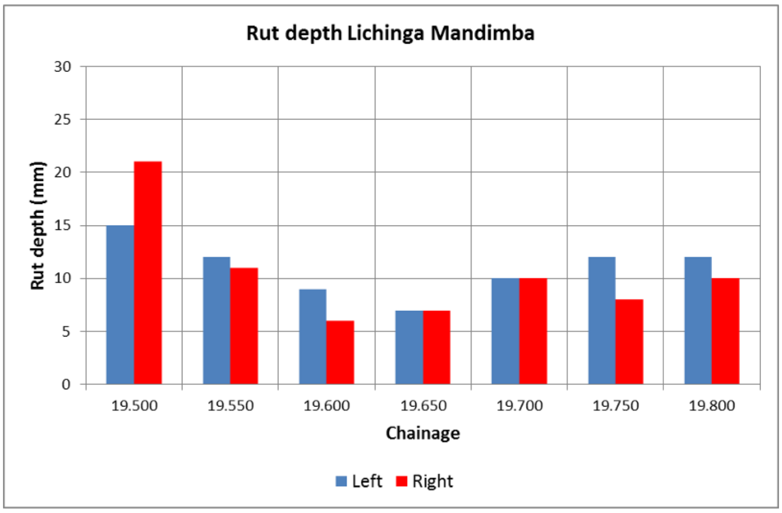


Figure 5-16 Rut depth on Section 2 of the Lichinga Mandimba road

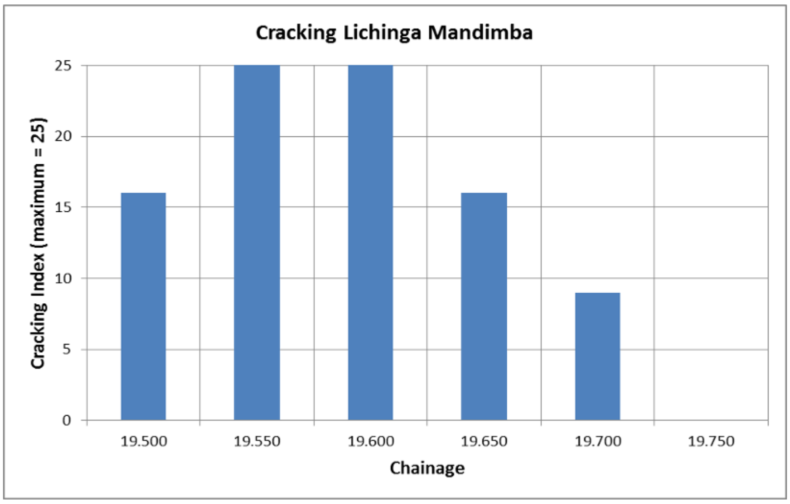


Figure 5-17 Cracking on Section 2 of the Lichinga Mandimba road

5.4.3 DCP and test pit data

The in-situ DCP CBRs indicate low values for the time of year and the DN 150 values confirm the weakness of the roadbase (red values in Table 5.10 indicate weak material). However, Section 1 has performed satisfactorily for the traffic carried (166 vpd). In Section 2 two test positions show very strong subgrades but the sub-bases above are not as strong as they should be with in-situ values similar to the required soaked values using conventional specifications.

The failures were thought to have been caused by failure of the Otta seal surfacing but the single laboratory CBR value also suggests that the base is of very low quality although, at the time of year that the surveys were carried out, the in situ strengths do not appear to differ from Section 1. This section has a flat to a gentle slope and the Otta seal should have performed better under these circumstances.

Table 5-10 Summary of DCP and Prima deflection data on the Lichinga Mandimba road

Chainage	In situ base CBR	Subgrade CBR	Depth of base	Depth of sub-base	SNP	DN150	DN300	DN800	Prima Modulus (MPa)
1.150	100	15	175	665	3.86	50	95	185	510
1.250	55	11	184	594	3.46	30	55	135	375
1.400	60	18	123	680	3.52	40	85	175	485
19.500	70-90	100	191	477	4.18	40	90	*500*	*750*
19.550	50-65	100	194	472	4.02	30	65	*400*	*485*
19.650	60	31	175	625	3.81	30	65	165	450

Table 5-11 Summary of Test Pit data on the Lichinga Mandimba road

Chainage	In situ road base CBR	In situ moisture content	Laboratory soaked CBR at Mod AASHTO %	Optimum moisture content %	Laboratory OMC CBR at Mod AASHTO %	Ratio FMC/OMC	PM	PI
1.250	55	NA	65	NA	107	NA	445	8
19.550	50-65	9.2	20	12.1	49	0.76	530	9.5

5.5 Maniamba-Metangula Road

5.5.1 General description

The Maniamba Metangula Road had been excluded from the final selection because of the low traffic volume (approximately 50 vpd). The road was constructed using some shale gravel and surfaced with an Otta seal. The road is still in very good condition. However the section that was selected was showing the beginning of surfacing failure and distress. It was also in a low lying area with poor drainage.

Section of Metangula Maniamba

Pavement profile

Figure 5-18 Condition of the section and pavement profile on the Metangula Maniamba road

Table 5-12 Sections on the Maniamba Metangula road

Section	Chainage		Brief description
	From	To	
1	2.550	2.800	The section is in fair condition with no potholes or cracking and only minor loss of surfacing. It was apparent that there was overtopping of this section from the runoff coming down from the hills close by. The deterioration was caused by this drainage problem. The Otta seal was considered to be robust because it has a sand capping.

5.5.2 Visual observations

There was slight loss of surfacing and some cracking. The section was in a fair condition. The subgrade was granitic and very coarse with some large boulders which affected the DCP tests. There were no significant structural failures and there was no cracking noticed on this section although the DN 150 values are low.

5.5.3 DCP and test pit data

The results of the DCP and deflection tests are shown in Table 5-13 and the test pit data is shown in Table 5-14.

Table 5-13 Summary of DCP and Prima deflection data on the Maniamba Metangula road

Chainage	In situ base CBR	Subgrade CBR	Depth of base	Depth of sub-base	SNP	DN150	DN300	DN800	Prima Modulus (MPa)
2.55	70	25	175	625	3.80	35	80	150	370
2.60	75	20	175	630	3.86	37	80	170	390
2.80	100+	100(1)	188	512	4.25	95	185	500	985

Note 1 Very strong subgrade

Table 5-14 Summary of Test Pit data on the Maniamba Metangula road

Chainage	In situ road base CBR	In situ moisture content	Laboratory soaked CBR at Mod AASHTO %	Optimum moisture content %	Laboratory OMC CBR at Mod AASHTO %	Ratio FMC/OMC	PM	PI
2.55	70	4.6	67	7.2	160	0.64	242	5.5

5.6 Maniamba Lichinga Road

5.6.1 General description

The section was built many years ago by the military. It is assumed that it was built during the 70s. The base consists of laterite material which was assumed to be cement-stabilised. The road has no sealed shoulders and was initially surface dressed. There are sections which were constructed recently and these are a 6m wide mat. The section that was selected is a narrow mat, 4.2m wide. Narrow mats are not very common nowadays.

5.6.2 Visual observations

There were no cracks or ruts on the Section (Table 5-15). The crown height is approximately 500mm on average and the road is on a watershed so no ingress of moisture from the side-slopes could occur.

Table 5-15 Sections on the Maniamba Lichinga road

Section	Chainage		Brief description
	From	To	
	32.650	32.850	The section was resealed in 2007 and it was in good condition with no indication of any structural failures. The section was on a high embankment and no drainage problems were noted.

Figure 5-19 Condition of section and pavement profile on the Maniamba Lichinga road

5.6.3 DCP and test pit data

The base was very hard to excavate and very dry too, synonymous with cement stabilisation, but it could also be a highly compacted and dried-back laterite gravel. A sample was collected for verification and cement content tests. The hard base was approximately 100mm thick.

Below the hard dry base is a moist laterite base of 100mm thickness and similar to the hard base. It was not immediately clear as to why this layer was moist and soft.

Below the soft moist lateritic base or sub-base was a very hard dry layer of red silt approximately 100mm thick. It could not be ascertained whether this hard layer was cement or lime-stabilised or just dried back. A sample was collected for cement content tests.

Below the hard red silt layer was a moist and soft red silt subgrade.

The moisture variations in the vertical profile were perplexing.

Table 5-16 Summary of DCP and Prima deflection data on the Maniamba Lichinga road

Chainage	In situ base CBR	Subgrade CBR	Depth of base	Depth of sub-base	SNP	DN150	DN300	DN800	Prima Modulus (MPa)
32.650	70-100	9	87	496	2.87	55	125	150	685
32.800	100+	15	135	653	3.39	75	125	155	1525
32.850	100+	10	182	456	3.31	60	130	155	705

Table 5-17 Summary of Test Pit data on the Maniamba Lichinga road

Chainage	In situ road base CBR	In situ moisture content	Laboratory soaked CBR at Mod AASHTO %	Optimum moisture content %	Laboratory OMC CBR at Mod AASHTO %	Ratio FMC/OMC	PM	PI
32.650	70-100+	5.4	62	14.4	76	0.38	271	6.6

5.7 Nametil-Angoche Road

5.7.1 General description

This road is in Nampula Province. It was constructed in 2007 and has a lateritic roadbase and Otta seal surfacing with laterite aggregate. There were some experimental sections incorporated in the design of the project; a section with unmodified laterite bases; section with a cement-stabilised laterite base; section with a single Otta seal surfacing; and section with a single Otta seal and sand seal capping. There was also a section with no surfacing that was only primed. Four test sections were selected and details are given in Table 5-18.

5.7.2 Visual observations

The condition of the sections is illustrated in Figure 5-20.

The cracking and rut depth on Section 2 are shown in Figure 5-21 and Figure 5-22. There was only very minor cracking and the rut depths did not indicate any structural failures.

Table 5-18 Sections on the Nametil Angoche Road

Section	Chainage		Brief description	Figure no.
	From	To		
1	14+550	14+750	This experimental section without surfacing has completely deteriorated and there is more of the exposed gravel base than the primed surface.	None
2	14+900	15+200	The section was in good condition. There was no cracking but the rut depths were slightly higher than desired. The base, which was said to be cement stabilised, exhibited low strength (CBR < 80%). This is unusual.	Fig 5.21 and Fig 5.22
3	27+100	27+400	The section with natural base and Otta seal with sand seal capping was in good condition with no cracking and acceptable levels of rutting.	Fig 5.23
4	27+400	27+500	The surfacing was a single Otta seal and has deteriorated, though not badly. There was some cracking but the rutting indicates complete structural failure at one point and near failure at another.	Fig 5.24 and Fig 5.25

Section 1

Section 2

Section 3

Section 4

Figure 5-20 Condition of sections on the Nametil Angoche road

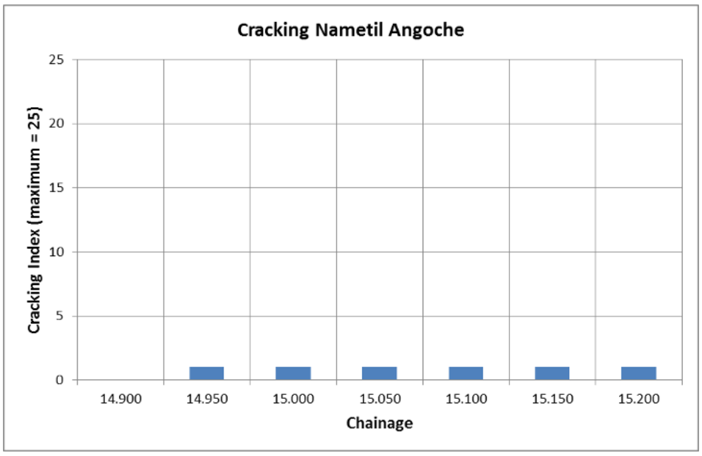


Figure 5-21 Cracking on Section 2 of the Nametil Angoche road

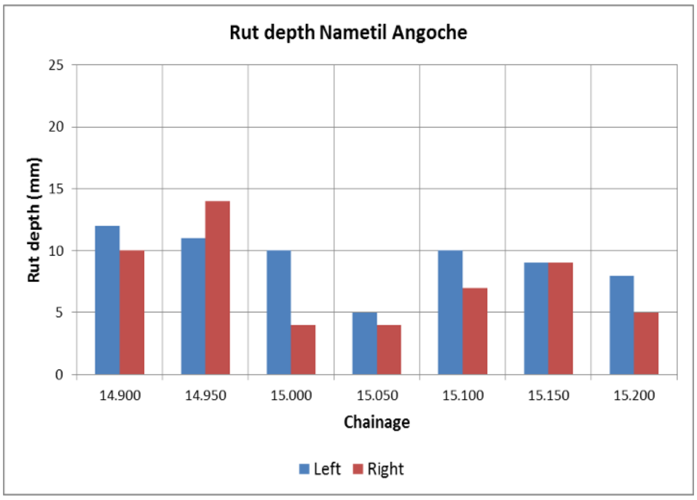


Figure 5-22 Rut depth on Section 2 of the Nametil Angoche road

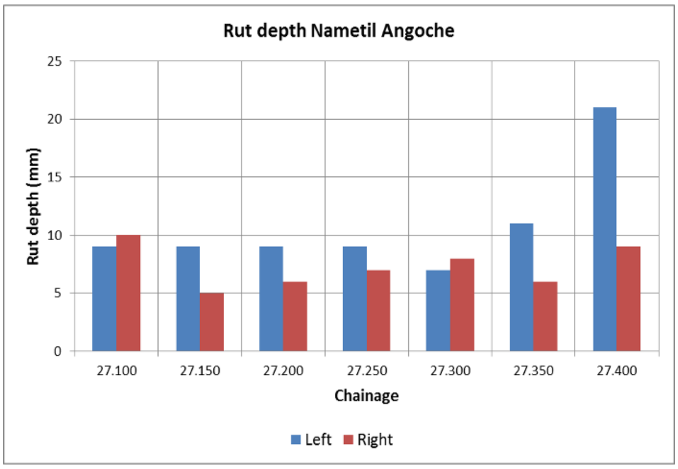


Figure 5-23 Rut depth on Section 3 of the Nametil Angoche road

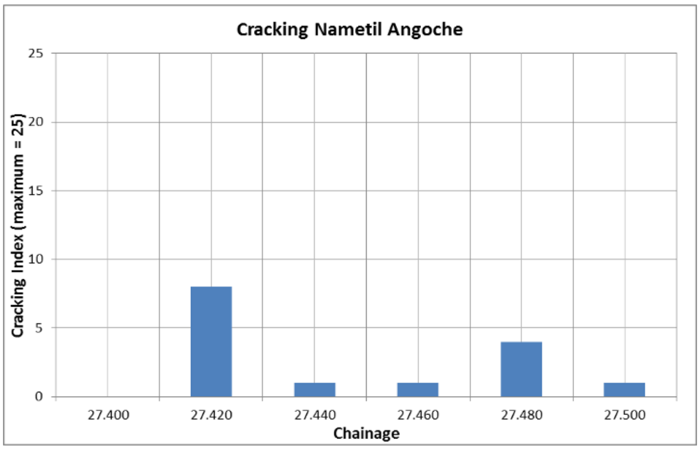


Figure 5-24 Cracking on Section 4 of the Nametil Angoche road

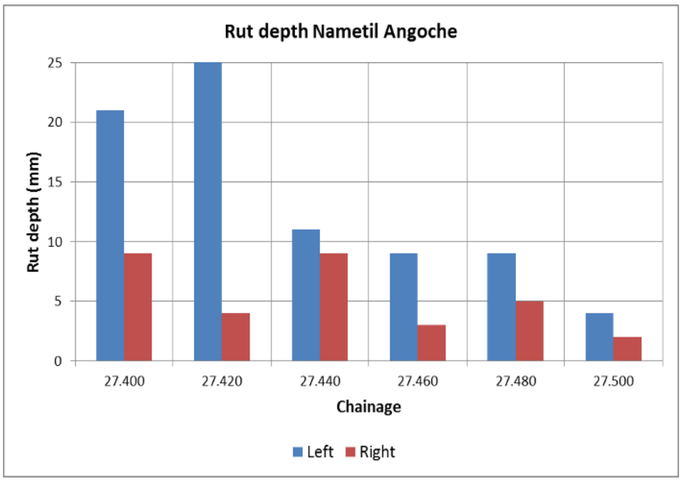


Figure 5-25 Rut depths on Section 4 of the Nametil Angoche road

5.7.3 DCP and test pit data

In Sections 2 and 3 the roadbase was weak as indicated by the in-situ DCP/CBR values and the DN 150 values of 35 or less. The CBR values for the base course were generally much lower than the CBR values of the underlying layers.

On Section 4 there were two chainages showing deep ruts as shown in Figure 5-23. On excavation of the test pit at chainage 27+400, it was found out that the surfacing had delaminated from the base. The surfacing ($1m^2$) could be lifted from the base intact. The base was very weak (see the DN 150 values) and appeared uncompacted and loose. The in-situ CBR from DCP tests was 39% which is very low. It is surprising how the surfacing over this base did not fail over the years.

Under most of the test points there was an impenetrable subgrade layer of highly dense grey sand giving zero penetration for 20 blows of the DCP. This layer was not stabilised but it could have been heavily compacted or consolidated under traffic. The strength at depth is illustrated by the DN 300 and the DN 800 values.

Table 5-19 Summary of DCP data on the Nametil Angoche road

Chainage	In situ base CBR	Subgrade CBR	Depth of base	Depth of sub-base	SNP[1]	DN150	DN300	DN800	Prima Modulus (MPa
14.765	100+	100	80	236	3.66	75	-	-	826
14.766	100+	100	150	300	4.05	200+	-	-	826
15.000	75-80	100	122	171	3.69	35	155	350+	608
15.050	100+	100	135	300	3.97	75	-	-	1112
15.100	65	45	155	644	3.95	35	97	280	304
27.300	40-60	100	121	135	3.47	35	300+	-	580
27.301	30-45	100	128	414	3.61	20	105	450+	580
27.350	45	25	143	616	3.66	30	315	490	685
27.400	35-55	100	111	150	3.49	66	250+	250+	1220
27.401	100+	100	93	200	3.65	70	200+	200+	580
27.460	60	100	124	200	3.70	60	200+	200+	1620
27.500	45-55	35	98	691	3.52	35	94	185	810

Note 1 The SNP values are indicative only because full penetration was not achieved in most of the DCP tests

Table 5-20 Summary of Test Pit data on the Nametil Angoche road

Chainage	In situ road base CBR	In situ moisture content	Laboratory soaked CBR at Mod AASHTO %	Optimum moisture content %	Laboratory OMC CBR at Mod AASHTO %	Ratio FMC/OMC	PM	PI
14.765	100+	0.6	86	7.8	NA	0.08	30	1.3
15.100	65	6.7	65	6.8	NA	0.99	7	0
27.300	40-60	3.7	55	7.8	NA	0.47	27	1
27.400	35-55	3.6	49	8.3	NA	0.43	82	3.7

5.8 Rio Zambezi Nicoadala Road

5.8.1 General description

This is a segment of the national road, N1. The traffic on this road is relatively low. Four sections were selected because the left and right lanes appeared to differ. The base is natural. The surfacing is hot sand asphalt. Figure 5-26 illustrates the surface condition and Table 5-21 is a summary.

Table 5-21 Sections on the Rio Zambezi Nicoadala road

Section	**Chainage**		**Brief description**	**Figure no.**
	From	To		
1 (Left Lane)	140+700	141+000	This was divided into two sections the left hand lane and right hand lane. The right hand lane looked pale and the left hand lane looked darker. There was a moderate amount of cracking on Section 1 (left lane) but virtually no rutting.	Fig 5.27 and Fig 5.28
2 (Left Lane)	141+050	141+350	There were indications of sub-surface drainage problems but there was no rutting and cracking was small.	Fig 5.29 and Fig 5.30
3 (Right Lane)	140+700	141+000	The surface was lighter in colour than the left lane. The hot sand asphalt showed signs of distress, (more pronounced cracking than the left lane) and minimal deformation	Fig 5.27 and Fig 5.28
4 (Right Lane)	141+050	141+350	More pronounced cracking on hot sand asphalt surfacing on the right lane than on the left lane. Slight deformation and possible perched water table	Fig 5.29 and Fig 5.30

Section 1

Section 2

Section 3 (same as section 4) Test pit excavation

Figure 5-26 Sections on the Zambezi Nicoadala road

5.8.2 *Visual observations*

Figure 5-27 and Figure 5-28 shows that there was a moderate amount of cracking on Section 1 but virtually no rutting that could not be attributed to slight densification under traffic.

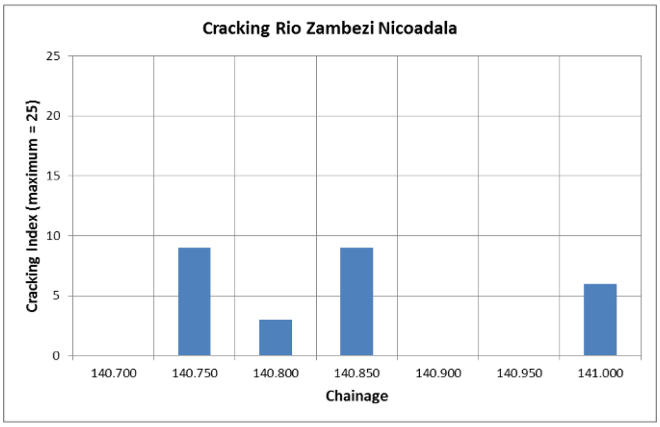


Figure 5-27 Cracking on Section 1 of the Rio Zambezi Nicoadala road

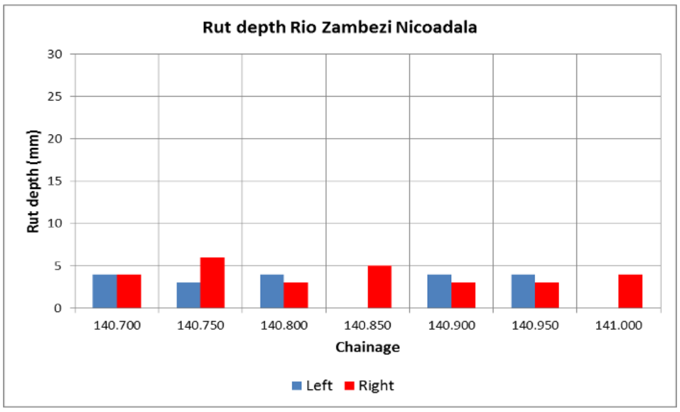


Figure 5-28 Rut depth on Section 1 of the Rio Zambezi Nicoadala road

Figure 5-29 shows some cracking on Section 2 (slightly less than on Section 1 but no real difference). Similarly there was no significant rutting, Figure 5-30.

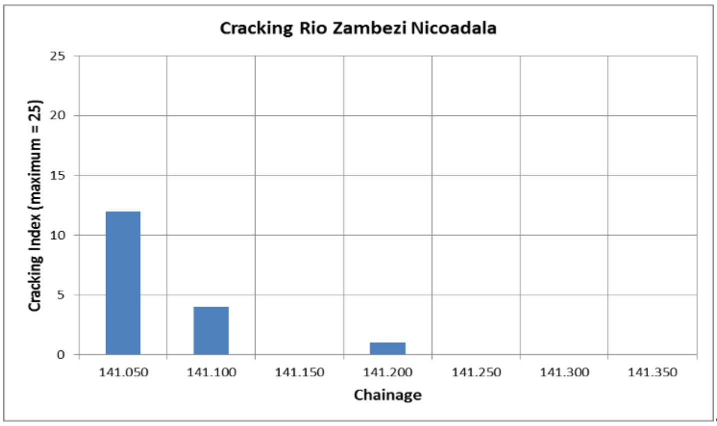


Figure 5-29 Cracking on Section 2 of the Rio Zambezi Nicoadala road

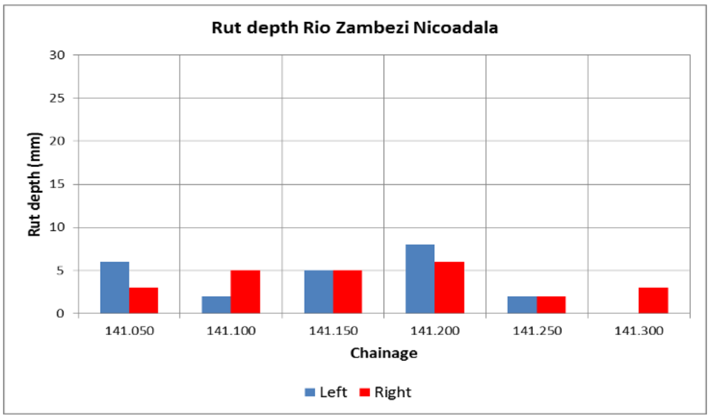


Figure 5-30 Rut depth on Section 2 of the Rio Zambezi Nicoadala road

5.8.3 DCP and test pit data

Apart from three test points where the in-situ strength of the roadbase fell below CBR 80%, all the roadbases were strong. Test pits showed that the top 40mm layer of the base was clayey and moist in these weak areas. The underlying layer was much harder.

The sub-base on the second section was weak. Test pits showed that the sub-base was clayey and moist and also stony.

The sub-bases were generally strong and deep hence it was quite difficult to decide what comprised the roadbase for analysis purposes and what comprised the sub-base. The very high strength with depth is illustrated by the very high values of DN 800. An investigation into the few weak areas is required and details of the test pit data, especially the in-situ moisture conditions.

Unfortunately there were no PRIMA data for this site because the PRIMA Light Weight Deflectometer had broken down.

The hot sand asphalt on the left lane look much richer in bitumen and it is expected that the bitumen content should be higher.

The hot sand asphalt on the left hand side had a lot of clean sand cavities which means that the mixing was poor.

There was much more deterioration on the right hand side than the left hand side and this could have been attributed to the weak base and subgrade, and insufficient bitumen in the hot sand asphalt surfacing.

Table 5-22 Summary of DCP data on the Rio Zambezi Nicoadala road

Chainage	In situ base CBR	Subgrade CBR	Depth of base	Depth of sub-base	SNP	DN150	DN300	DN800
Left Side								
140.700	125	18	377	593	4.45	45	125	535
140.701	120	38	351	581	4.62	50	155	465
140.751	150	50	369	607	4.58	72	230	565
140.800	85-150	50	158	472	4.50	65	135	655
140.801	55-105	50	129	659	4.10	37	125	415
140.900	40-90	75	295	459	4.28	35	70	400
140.901	100	50	310	618	4.35	45	155	490
141.000	100	100	268	591	4.36	48	145	545
Right Side								
141.001	130	50	241	588	4.74	45	110	495
141.101	100	100	290	613	4.41	53	125	370
141.200	120	100	178	499	4.39	40	90	800+
141.201	150	50	175	551	4.30	80	165	405
141.300	150	100	301	418	4.36	75	172	900+
141.301	72	100	175	637	4.21	40	75	470
141.350	150	100	175	585	4.36	70	120	1000+
141.351	150	100	175	683	4.35	85	180	475

Table 5-23 Summary of Test Pit data on the Rio Zambezi Nicoadala road

Chainage	In situ road base CBR	In situ moisture content	Laboratory soaked CBR at Mod AASHTO %	Optimum moisture content %	Laboratory OMC CBR at Mod AASHTO %	Ratio FMC/OMC	PM	PI
140.700	125	5.1	19	5.6	NA	0.91	315	12.8
140.900	40-90	9.3	5	8.7	NA	1.07	664	19.8
141.300	150	5.1	54	4.7	NA	1.09	142	5.9
141.350	150	6.7	46	5.9	NA	1.14	308	12.9

6 Detailed Analysis

This section covers the detailed analysis of the performance of the test sections based on the interpretation of the field investigations and laboratory tests on samples that were collected. The aspects of performance evaluation covered include:

1. Visual condition rating
2. Pavement strength evaluation
3. Traffic analysis
4. Materials investigation
    a. Roadbase materials
        i. Natural bases
        ii. Stabilised (Emulsion and cement stabilised bases)
    b. Surfacing
        i. Binder content, quality, condition and in-service performance
        ii. Aggregate grading and petrography

The purpose of the detailed analysis is to develop the thresholds of performance and limits parameters of material properties.

6.1 Overall Performance of Test Sections

It is important at this stage to look at the overall performance of the various sections under study. This can be assessed by determining the performance indices or deterioration indices in order to develop a picture of how the pavements responded to agents of deterioration.

The deterioration parameters which were investigated and documented during the field surveys are given in Table 6-1.

1. The average rut depth is the average of the readings in millimetres at the fixed chainages and is equal to the Rut Depth Index (RdI)
2. The Cracking Index (CrI) = intensity x extent

    The intensity is a measure of the density of cracks in a given area measured on a scale from 1 to 5 where 1 is a single crack, 2 is unconnected multiple cracks, 3 is connected multiple cracks, 4 is crocodile cracks and 5 is severe crocodile cracks with rocking movements. Extent is a measure of how much the cracking covered the length of road and is quantified on a linear scale of 1 to 5 with 5 representing 100% of the road length. The maximum CrI is therefore 25.

3. The Pothole/Patching Index (PPI) = (pothole area + patched area) x the extent.

    Pothole/Patching Index takes into account the area of potholes and also assumes that the patches were once potholes that were repaired. The extent of the potholes gives a measure of how the potholes are distributed in a given area i.e. whether they are localised or wide spread. Extent is measured on a scale from 0 to 5 where 5 is 100%. Widespread potholing signifies major problems whereas localised potholes signify spot weaknesses which can be easily resolved.

The overall deterioration index is defined as

DI = RdI + CrI + PPI

A value of 40 was derived based on what was perceived to be reasonable condition.

The levels of deterioration and performance variations are given in Table 6-1 and Figure 6-1, Figure 6-2, Figure 6-3 and Figure 6-4. In Table 6-1, the performance variation is defined as the deviation from the reasonable condition and is given by the equation:

DI Variance = 40 - DI

The results show the following:

1. Only 6 out of the 22 sections which were investigated had failed and the rest were in good to fair condition in accordance with the DI Variance.
2. There was very little structural failure judging by the rut depth index (RdI)
3. Cracking and potholing were the major contributors to the deterioration index (DI)

6.2 Pavement Strength Analysis

The information on the results of strength tests for all the individual sites and sections is given in the Tables in chapter 5. For convenience the data have been accumulated into Table 6-2. The strength is represented in the form of CBRs, the structural number (SN and SNP) and the DN values (DN150, DN300 and DN800).

Table 6-1 Performance assessment for all test sections

Road Name	Chainages	Section	Average rut depth (RdI)	Average cracking index (CrI)	Average pothole/ patching index (PPI)	Average Roughness (IRI m/km)	Average crown height mm	Overall Deterioration index (DI)	DI Variance (40-DI)	IRI Variance (5-IRI)
Boane Libombos	4+700 - 5+100	1	6	2	0	4	379	12	28	1
	5+300 - 5+700	2	8	19	16	6	204	49	-9	-1
	8+000 - 8+400	3	3	9	15	2	901	29	11	3
	8+400 - 8+700	4	5	5	17	3	548	30	10	2
	8+700 - 9+000	5	7	14	49	3	891	73	-33	2
	9+050 - 9+350	6	2	0	0	2	557	4	36	3
Pambara Rio Save	22+600 - 22+900	1	0	0	0	3	132	3	37	2
Rio Zambezi Nicoadala	140+700 - 141+000	1	3	5	0	3	174	11	29	2
	141+050 - 141+350	2	4	3	0	3	226	10	30	2
	140+700 - 141+000	3	3	5	0	3	174	11	29	2
	141+050 - 141+350	4	4	3	0	3	226	10	30	2
Nametil Angoche	14+550 - 14+750	1	-	25	496	13	143	534	-494	-8
	14+900 - 15+100	2	8	1	0	3	185	12	28	2
	27+100 - 27+400	3	9	0	0	3	109	12	28	2
	27+400 - 27+500	4	10	3	109	6	72	128	-88	-1
Oasse Mocimboa da Praia	0+000 - 0+300	1	5	4	24	4	295	37	3	1
	2+800 - 3+100	2	9	4	0	4	542	17	23	1
	19+800 - 20+100	3	9	5	120	5	51.5	139	-99	0
Lichinga Mandimba	0+150 - 0+450	1	7	4	0	4	401	15	25	1
	19+500 - 19+800	2	10	15	262	9	342	296	-256	-4
Metangula Maniamba	2+500 - 2+800	1	3	0	0	4	128	7	33	1
Maniamba Lichinga	32+600 - 32+900	1	5	0	0	4	474	9	31	1

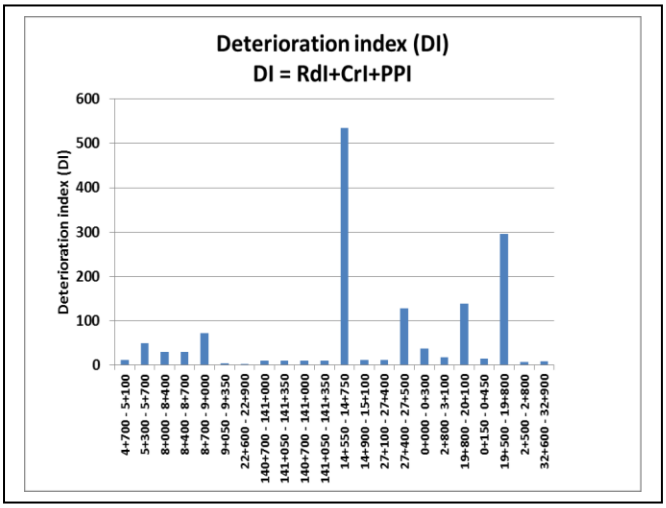


Figure 6-1 Deterioration Index (DI)

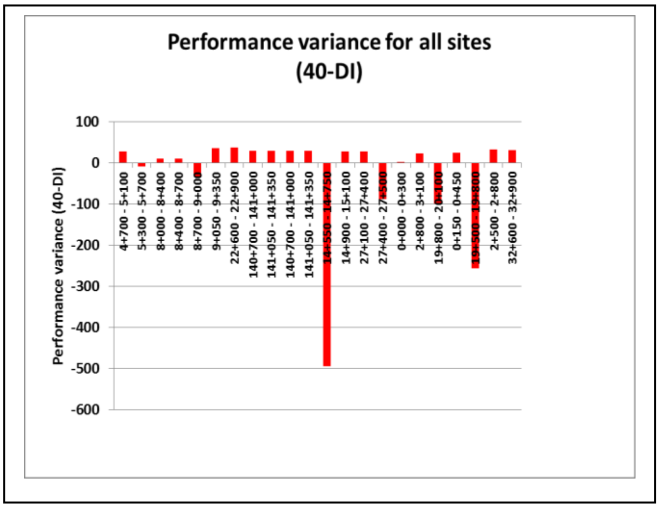


Figure 6-2 Performance variance for all sites (40-DI)

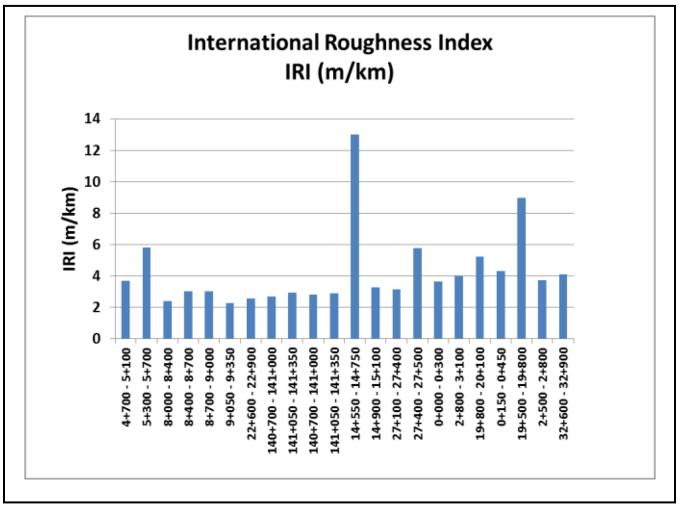


Figure 6-3 International Roughness Index (IRI m/km)

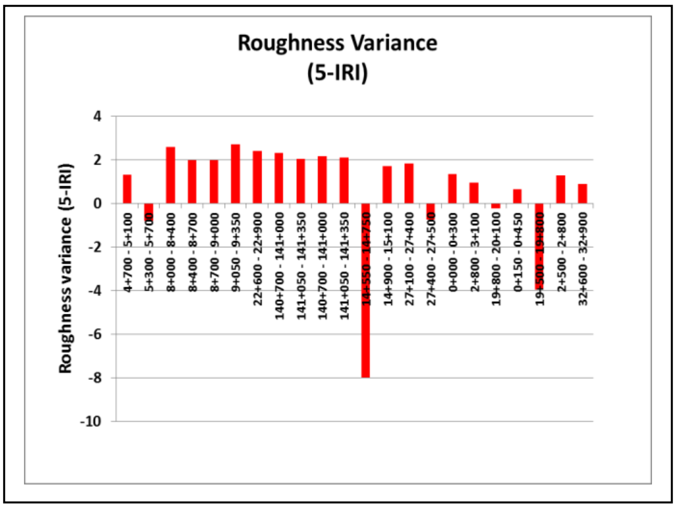


Figure 6-4 Roughness variance for all site (5-IRI)

Table 6-2 Structural strength parameters and traffic capacity of pavements for all sites

Road	Chainage	In situ base CBR	Subgrade CBR	Depth of base	Depth of sub-base	SNP	DN150	DN300	DN800	SNP	SN	Traffic Capacity (mesa)	Prima Modulus
Oasse Musimboa da Praia 1	0.050	65	100	140	506	3.72	35	61	200	3.72	2.73	> 30	717
	0.150	60	35	190	662	3.87	35	58	175	3.87	3.71		1371
	0.200	80	20	123	652	3.66	42	65	238	3.66	3.56		1443
	0.250	70	18	171	683	3.81	37	66	166	3.81	3.84		862
Oasse Musimboa da Praia 2	2.850	90	45	200	588	4.29	46	94	241	4.29	3.79		2467
	2.900	50	9	200	528	3.31	28	51	96	3.31	2.95	>30	866
	3.100	60-80	24	115	687	3.56	38	71	135	3.56	3.51		3008
Oasse Musimboa da Praia 3	19.900	50	30	182	604	3.46	28	45	87	3.46	2.98	>30	735
	20.000	90	21	126	697	3.53	45	63	127	3.53	3.48		783
	20.150	60-110	75	179	428	4.25	40	105	425	4.25	2.98		-
Boane Pequenos Libombos 1	4.700	100+	100	140	660	4.19	103	159	380	4.19	3.95		526
	5.050	95	50	144	656	4.05	48	83	190	4.05	3.80		163
	5.100	100+	100	175	395	4.33	90	220	500	4.33	2.91	>30	
Boane Pequenos Libombos 2	5.350	100+	100	175	625	4.37	131	208	500	4.37	3.96		
	5.500	110	100	200	427	4.28	42	92	195	4.28	2.97		
	5.650	100+	100	200	444	4.44	42	110	350	4.44	3.23		
Boane Pequenos Libombos 3	8.100	60	50	200	490	4.20	25	63	261	4.20	3.26		
	8.200	60	75	200	513	4.15	24	64	158	4.15	3.24		
	8.350	80	48	176	518	4.22	33	77	224	4.22	3.40		
Boane Pequenos Libombos 4	8.450	40-65	50	196	387	4.03	17	47	180	4.03	2.65	>30	
	8.550	65-75	100	228	508	4.54	38	86	270	4.54	3.62		237

	8.650	65-100	75	157	576	4.19	45	110	240	4.19	3.58		310
Boane Pequenos Libombos 5	8.900	65-90	100	200	515	4.49	42	87	500	4.49	3.60		297
Boane Pequenos Libombos 6	9.100	100+	100	212	521	4.51	100	155	285	4.51	3.63		
	9.200	100+	50	175	501	4.26	58	151	311	4.26	3.36		
	9.300	100+	50	160	474	4.16	65	121	250	4.16	3.15		
Metangula Maniamba 1	2.55	70	25	175	625	3.80	35	82	152	3.80	3.49		372
	2.60	75	20	175	630	3.86	37	81	172	3.86	3.61		391
	2.80	100+	100	188	512	4.25	95	185	500	4.25	3.33	>30	986
Metangula Maniamba Lichinga 1	32.650	70-100	9	87	496	2.87	53	127	150	2.87	2.37	8.0	685
	32.800	100+	15	135	653	3.39	73	125	157	3.39	3.21		1523
	32.850	100+	10	182	456	3.31	60	132	155	3.31	2.61		704
Lichinga Mandimba 1	1.150	100	15	175	665	3.86	51	94	187	3.86	3.83		512
	1.250	55	11	184	594	3.46	30	53	137	3.46	3.26	>30	373
	1.400	60	18	123	680	3.52	39	86	176	3.52	3.51		484
Lichinga Mandimba 2	19.500	70-90	100	191	477	4.18	38	88	500	4.18	3.11		*750*
	19.550	50-65	100	194	472	4.02	30	64	400	4.02	2.91	>30	*485*
	19.650	60	30	175	625	3.81	30	63	163	3.81	3.47		451
Nametil Angoche 1	14.765	100+	100	80	236	3.66	75	-	-	3.66	1.60		
	14.766	100+	100	150	300	4.05	200+	-	-	4.05	2.23		
Nametil Angoche 2	15.000	75-80	100	122	171	3.69	35	155	350	3.69	1.42	8.0	
	15.050	100+	100	135	300	3.97	75	-	-	3.97	2.14		
	15.100	65	45	155	644	3.95	34	97	280	3.95	3.69		
Nametil Angoche 3	27.300	40-60	100	121	135	3.47	35	300+	-	3.47	1.12	1.5	
	27.301	30-45	100	128	414	3.61	20	105	450	3.61	2.27		
	27.350	45	25	143	616	3.66	29	315	490	3.66	3.38		

Nametil Angoche 4	27.400	35-55	100	111	150	3.49	66	250+	250	3.49	1.17		
	27.401	100+	100	93	200	3.65	70	200+	200	3.65	1.46		
	27.460	60	100	124	200	3.70	60	200+	200	3.70	1.51		
	27.500	45-55	35	98	691	3.52	35	94	185	3.52	3.45		
Rio Zambezi Nicoadala Left Side	140.700	125	18	200	665	4.19	45	125	535	4.19	4.17		
	140.701	120	38	200	613	4.45	50	155	465	4.45	4.07		
	140.751	150	50	200	626	4.47	72	230	565	4.47	4.13		
	140.800	85-150	50	213	472	4.50	65	135	655	4.50	3.48		
	140.801	55-105	50	144	659	4.10	37	125	415	4.10	3.90		
	140.900	40-90	75	204	459	4.28	35	70	400	4.28	3.16		
	140.901	100	50	173	618	4.35	45	155	490	4.35	3.97		
	141.000	100	100	177	591	4.36	48	145	545	4.36	3.81		
Rio Zambezi Nicoadala Right Side	141.001	130	50	200	639	4.47	45	110	495	4.47	4.18		
	141.101	100	100	185	613	4.41	53	125	370	4.41	3.96		
	141.200	120	100	178	499	4.39	40	90	800	4.39	3.42		
	141.201	150	50	175	551	4.30	80	165	405	4.30	3.62		
	141.300	150	100	178	418	4.36	75	172	900	4.36	3.04	> 30	
	141.301	72	100	175	637	4.21	40	75	470	4.21	3.86		
	141.350	150	100	175	585	4.36	70	120	1000	4.36	3.78		
	141.351	150	100	175	683	4.35	85	180	475	4.35	4.22		

Note The red values in the DN150 column are low values below or equal to 35 for essentially roadbase strength.

The 'Traffic Capacity' values are for the weakest test point within each section in terms of overall strength (not merely roadbase strength)

The red values in the 'In situ Base CBR' column indicate values that are well below typical specifications.

6.3 Analysis of Traffic

It is important to determine the traffic loading for each road in order to quantify the adequacy of the pavement structures for the intended traffic.

Consistent traffic count data from the time of rehabilitation to date could not be obtained, understandably, for the much older sections which were built during colonial times. The data obtained were from the traffic count exercises of 2010 and 2011 (ANE 2010. *SISTEMA DE GESTÃO DA REDE VIÁRIA-RECENSEAMENTO DE TRÁFEGO* (Management System of Road Network-Census of Traffic 2010). For each study section, the closest traffic count station along the same link was selected.

These data were used to estimate the traffic (ADT) after construction by multiplying the base traffic by the compounded growth rate and from these figures the cumulative traffic was calculated to date. The data are shown in Table 6-3.

The traffic loading was developed from the data collected during the axle load surveys which were carried out during the field work and from experience of typical axle loadings from other countries. The expected traffic loading for each traffic category was determined and these estimates were used to calculate the cumulative traffic loading on the test sites. A growth rate of 5% was assumed and this is consistent with what was used for the 2007 Heavy Vehicle Overloading Control Study, the Millennium Challenge Corporation Roads Project report and the African Development Fund Nacala road Project Phase III Project appraisal Report (September, 2012).

Table 6-3 Traffic engineering data

ROAD NAME	Age of Road	Age of Surface		TRAFFIC CLASS										
				A	B	C	D	E	F	G	H	LIGHT	HEAVY	TOTAL
Boane-Libombos	13	13	Daily Traffic	205	75	36	4	77	42	41	6	**315**	**171**	**486**
			Percentage %	42	15	7	1	16	9	8	1	**65**	**35**	**100**
			a0	41,665	15,243	7,317	813	15,650	8,536	8,333	1,219			
			Cum Total	738,018	270,007	129,603	14,400	277,207	151,204	147,604	21,601			
			Esa/vehicle	0	0	1	1	2	3	5	0			
			Cum esa	0	0	64,802	14,400	415,810	453,611	738,018	0			**1,686,641**
Pambarra-Rio Save	30	4	Daily Traffic	198	125	127	26	66	48	80	2	**450**	**222**	**672**
			Percentage %	29	19	19	4	10	7	12	0	**67**	**33**	**100**
			a0	62,430	39,413	40,043	8,198	20,810	15,134	25,224	631			
			Cum Total	269,079	169,873	172,591	35,334	89,693	65,231	108,719	2,718			
			Esa/vehicle	0	0	1	1	2	3	5	0			
			Cum esa	0	0	86,296	35,334	134,540	195,694	543,594	0			**995,457**
Oasse-Mocimboa da Praia	13	13	Daily Traffic	329	202	177	9	155	70	44	5	**709**	**284**	**993**
			Percentage %	33	20	18	1	16	7	4	0	**71**	**29**	**100**
			a0	66,868	41,056	35,974	1,829	31,503	14,227	8,943	1,016			
			Cum Total	1,184,429	727,218	637,215	32,401	558,013	252,006	158,404	18,000			
			Esa/vehicle	0	0	1	1	2	3	5	0			
			Cum esa	0	0	318,608	32,401	837,020	756,018	792,019	0			**2,736,066**
Metangula-Maniamba	5	5		32	39	37	1	32	11	10	5	**108**	**58**	**166**
			a0	9,609	11,711	11,111	300	9,609	3,303	3,003	1,501			
			Cum Total	53,097	64,712	61,393	1,659	53,097	18,252	16,593	8,296			
			Esa/vehicle	0	0	1	1	2	3	5	0			
			Cum esa	0	0	30,697	1,659	79,645	54,756	82,964	0			**249,720**

Maniamba-Lichinga	41	4	Daily Traffic	32	39	37	1	32	11	10	5	**108**	**58**	**166**
			Percentage %	32	39	37	1	32	11	10	5	**108**	**58**	**166**
			a0	10,090	12,297	11,666	315	10,090	3,468	3,153	1,577			
			Cum Total	43,488	53,000	50,282	1,359	43,488	14,949	13,590	6,795			
			Esa/vehicle	0	0	1	1	2	3	5	0			
			Cum esa	0	0	25,141	1,359	65,231	44,847	67,949	0			**204,527**
Lichinga-Mandimba	8	8	Daily Traffic	71	79	41	4	59	21	14	2	**191**	**100**	**191**
			Percentage %	24	27	14	2	20	7	5	1	**66**	**34**	**100**
			a0	18,417	20,492	10,635	1,038	15,305	5,447	3,632	519			
			Cum Total	175,869	195,685	101,558	9,908	146,145	52,018	34,678	4,954			
			Esa/vehicle	0	0	1	1	2	3	5	0			
			Cum esa	0	0	50,779	9,908	219,217	156,053	173,392	0			**609,349**
Nametil-Angoche	5	5	Daily Traffic	73	43	8	3	89	6	4	3	**125**	**105**	**229**
			Percentage %	32	19	4	1	39	2	2	1	**54**	**46**	**100**
			a0	21,921	12,912	2,402	901	26,725	1,802	1,201	901			
			Cum Total	121,127	71,349	13,274	4,978	147,675	9,956	6,637	4,978			
			esa	0	0	1	1	2	3	5	0			
			Cum esa	0	0	6,637	4,978	221,513	29,867	33,185	0			**296,180**
Rio Zambezi-Nicoadala	30	4	Daily Traffic	289	62	94	43	259	39	77	1	**445**	**419**	**864**
			Percentage %	33	7	11	5	30	4	9	0	**52**	**48**	**100**
			a0	91,122	19,549	29,638	13,558	81,663	12,297	24,278	315			
			Cum Total	392,747	84,257	127,745	58,436	351,977	53,000	104,642	1,359			
			Esa/vehicle	0	0	1	1	2	3	5	0			
			Cum esa	0	0	63,872	58,436	527,966	159,001	523,209	0			**1,332,485**

It is generally accepted that low volume roads are defined as those carrying not more than 300 vehicles per day or less than or 1 million standard axles (mesas) depending on which one the authority chooses to use. The analysis of the traffic loading given in Table 6-3 shows that many of the test sites have almost reached or have surpassed the upper limit of traffic loading expected for low volume roads.

It is also important to take note of the age of the roads. For example the Nametil Angoche and Metangula Maniamba roads are only 5 years old. This means that they will also probably carry more traffic through their design life than is expected for low volume roads.

There are also roads like Maniamba Lichinga that have been in existence for many years (>30 years) which have surpassed the conventional design life of 15 or 20 years but have not carried much traffic and can still be categorised as low volume roads in terms of their cumulative traffic loading.

Roads such as Rio Zambeze Nicoadala, Oase Mocimboa da Praia and Boane Libombos have surpassed 1 million esas. Oase Mocimboa da Praia, for example, has reached 2.7M at the age of 13 years. This pavement is therefore a 3M standard or more. 3M standard refers to the traffic loading in million equivalent standard axles that the road pavement is designed to carry.

It is also noticeable from Table 6-2 regarding traffic capacity and the overall pavement strengths for preventing failure in the subgrade (second to last column in Table 6-2 and Table 6-3 regarding traffic loading, that, although the traffic loading seems high, it is nowhere near the capacity of the pavements. It could be a long while before the pavements succumb to fatigue failure in the subgrade.

However, looking at the condition survey data, which is the measure of performance of the different sections, significant failures were noted on some sections. For the study, sections were deliberately selected to include some which performed well and others which had failed or were showing signs of significant distress. The reason for this was to find out why some sections were performing well and others were performing poorly, sometimes at the same level of traffic loading, taking into account local conditions such as drainage.

It is apparent that the traffic was not high enough to cause significant pavement failures in the study sections except only in localised places. This is shown through results of the rut depth measurements which were relatively low. There is little or no deformation that was recorded during the field surveys.

The major mode of failure that was noticed on most of the distressed or failed sections was mainly extensive potholing (and remedial patching), Table 6-1. This is generally an indication of failure of the upper part of the base or surfacing or both. Investigations were therefore focused on the road base and the surfacing.

Factors affecting the performance of road base in service include:

1. The properties of the base material
    a. Plasticity, which affects the materials' sensitivity to moisture.
    b. Grading which determines the bearing strength of the material.
    c. Variability of material properties which is usually caused by poor handling of the materials during construction.
2. In-situ strength. This is dependent on:
    a. The level of compaction achieved during construction.
    b. The level of in-situ moisture. For sensitive materials any increase in the in-situ moisture can have a significant effect on the bearing strength of the material. Both surface and subsurface drainage play a key role.

c. Stability/durability of the material. Some materials degrade in-service and thus their properties change significantly. This can seriously affect the performance of the pavements and the surfacings. For example some basalt gravels show poor durability and is indicated by an increase in plasticity with time as the gravel degrades in-service. Other materials strengthen in-service through consolidation and/or cementation resulting in significant increase in performance.

3. The integrity of the surfacing. Failure of the surfacing tends to be followed by failure of the base. The cracking that takes place in the surfacing breaks the seal and water seeps through the cracks weakening the base. Once the top part of the base is wetted the CBR decreases significantly and failures of the base occur causing the surfacing to peel off, thus creating potholes under the action of traffic.

The surfacing is crucial to the performance of the whole pavement and particularly the base course. The performance and durability of the surfacing itself is dependent on a number of factors.

1. The design. The design of the surfacing is paramount to its performance and durability. A poorly designed surfacing can be either too rich or deficient in binder. The determination of appropriate binder content for the aggregate is a very important engineering factor.

   As a result of the ever increasing cost of binder, there is a possible tendency to reduce the binder content and, at times, this has devastating effects on the surfacings.

2. The quality of binder. Specific tests and test procedures have been developed for the industry to enable the quality of binder that is appropriate for different types of surfacing to be measured and approved. These include the Rolling Thin Film Oven Test (RTFOT) that is carried out in the laboratory to simulate the in service deterioration of binder. Failure to adhere to the prescribed specifications can lead to serious problems relating to the performance of the binder and the surfacing.

   Sometimes the wrong type of binder is prescribed for the surfacing.

3. Quality of the aggregate. There are also prescribed specifications for the aggregate that are to be used for surfacing. Often there are challenges in meeting these specifications due to scarcity of good aggregate within economic haul distances. For example, there is no aggregate which can meet standard specifications in Inhambane Province of Mozambique and crushed stone aggregate can only be obtained from Maputo which is more than 500km away. Generally, Mozambique has very weak aggregate and competent aggregate, including Otta seals, can only be obtained in very few and isolated places. The tendency is therefore to use the best that is available and this could affect the performance and durability of the surfacings.

4. Workmanship during construction. In most cases this is where things can go wrong because some contractors do not follow the correct procedures for the application of surfacings. For some, it is lack of knowledge and experience and for others it is mere intransigency. Poor construction practice can lead to serious consequences. The most common problems are:

   a. Some contractors tend to overheat the binder. The binder then loses a significant content of the volatiles which are essential for long term durability. This means that the binder can be approved for surfacing, having met the quality criteria, but then loses those good properties through mishandling and incorrect preparatory and construction processes.

   b. The suppliers could also supply binder which is substandard and does not meet specifications or criteria for approval. Thus even if the contractor follows the procedures properly, the performance and durability of the binder and surfacing can be greatly affected. Such problems arise when there is no constant quality control of the bitumen that is being applied on site.

   c. The control of application rate of binder is a major problem on sites. Previous experience has shown very high variability of the application rates, some through intransigency but mostly through lack of knowledge and experience on the part of the operatives and inadequate supervision by the supervising consultants.

5. Environment. This is a major contributing factor to the performance and durability of surfacings.

   a. Bituminous binders deteriorate through ageing. Ageing is caused by oxidation and loss of volatiles which make the binder hard and stiff thus making the surfacing brittle. Thermal stresses and stresses caused by movement in the subgrade also cause cracks to initiate within the surfacing, which will eventually develop into potholes and more extensive failures.

   b. High intensity ultraviolet radiation from the sun accelerates oxidation of the binder in the surfacing, thus affecting performance and durability of surfacings.

   c. High temperatures also accelerate the rate of loss of volatiles thus affecting the composition of the bitumen and its ability to flex and dissipate stresses.

   d. High moisture environments are detrimental to the performance of surfacings through seepage of water through micro and macro cracks in the surfacing. This accelerates the disintegration of surfacing. Capillary moisture has the same effect, particularly in areas of high water tables.

6. Loading – heavy loads can cause a surfacing to fail. There are two aspects to this:

   a. Fatigue failure. This is gradual and is a result of repeated loading over a long period of time. This is not a major problem on low volume roads because the traffic is generally light, both in weight and volume (ADT).

   b. Damage caused by super-heavies such as logging trucks. The extreme surface stresses generated by such vehicles causes shear, shoving and raveling and tend to rip off surfacings such as single surface dressings and single Otta seals.

7. Maintenance. This is a critical component of the evaluation of any performance of surfacings. Timely crack sealing and reseals make a huge difference. This may also include rejuvenation of the binder through application of fog-sprays and stone replacement.

In consideration of the factors given above it was decided to focus the research on more detailed investigations of the surfacings and the road bases.

6.4 Unbound Roadbases

Most of the pavement structures which were investigated involved thin surfacings. In engineering terms these surfacings are mainly to protect the bases from direct contact with vehicle wheels and also from the environment mainly waterproofing. They contribute little or no bearing strength to the pavement. Thus the road bases are critical to the performance of the road. The critical factors include the quality of the base and the in-situ strength. The properties of the base include the grading and plasticity. It is therefore important to find out how the properties of the bases compare with the standard specifications.

6.4.1 Grading

The Mozambique manual i.e. the ANE Normas de Execucao does not specify grading envelopes for the base course therefore the grading envelopes for bases in TRL's Road Note 31 were used for comparison, Figure 6-5.

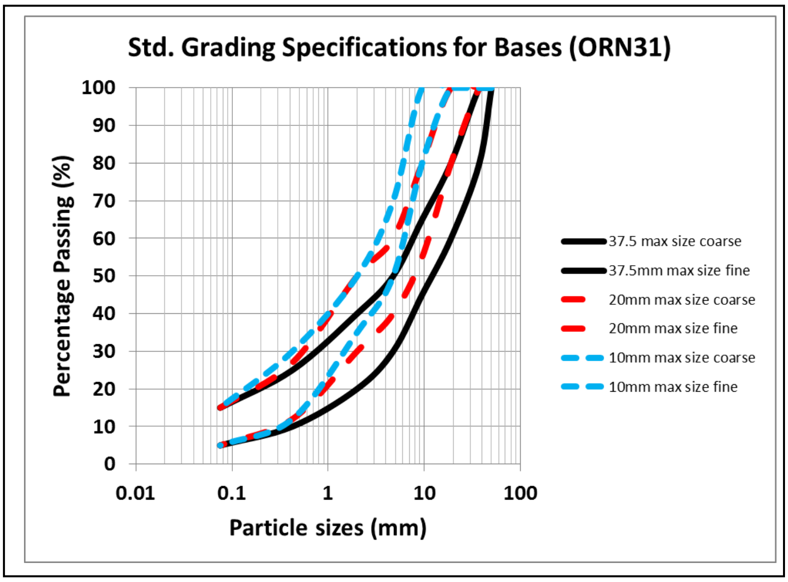


Figure 6-5 Grading of base course (TRL ORN 31)

The grading specification that is most appropriate for the materials in Mozambique particularly those that were encountered during the investigations is the envelope with maximum size aggregate of 20mm. Figure 6-6**,** Figure 6-7, Figure 6-8, Figure 6-9, Figure 6-10 and Figure 6-11 show the comparison between the grading of the base and the specifications.

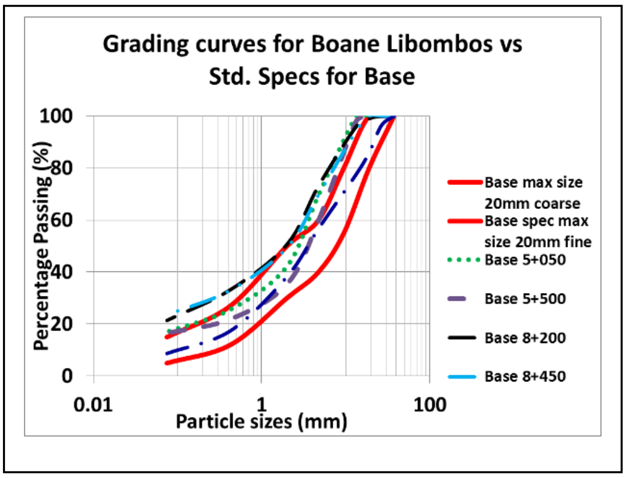


Figure 6-6 Grading – Roadbase Boane Libombos

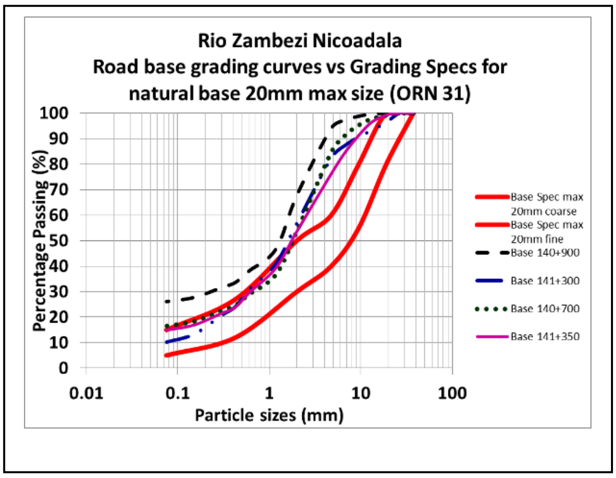
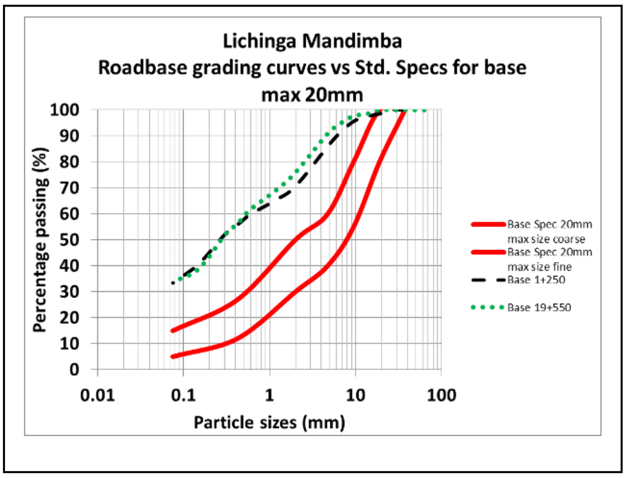


Figure 6-7 Grading – Roadbase Rio Zambezi Nicoadala



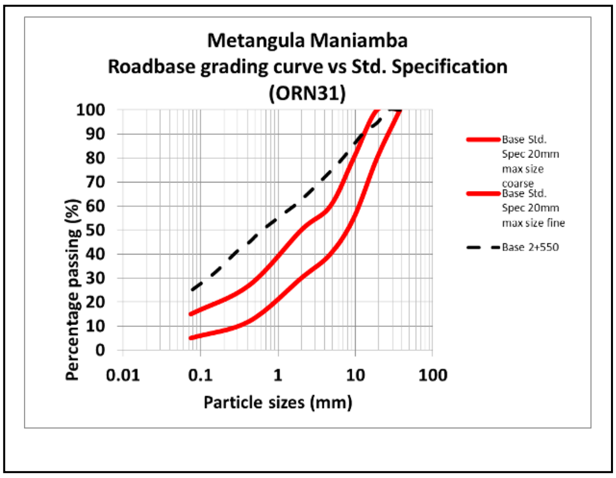
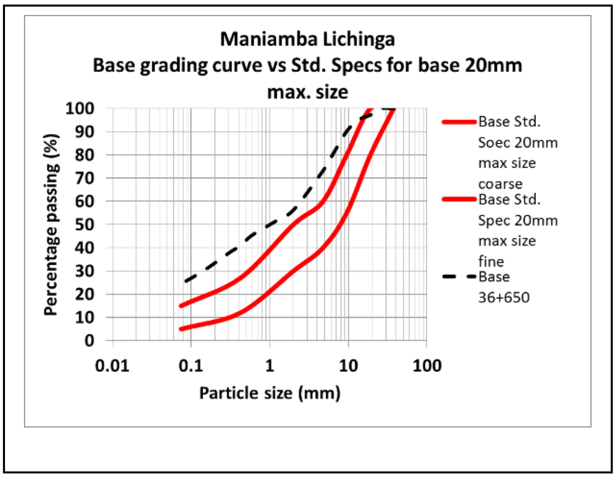


Figure 6-8 Grading – Roadbase Lichinga Mandimba

Figure 6-9 Grading – Metangula Maniamba



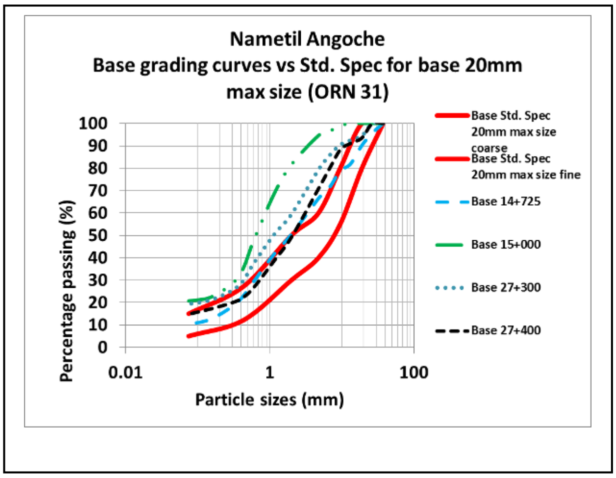


Figure 6-10 Grading –Base Maniamba Lichinga

Figure 6-11 Grading Nametil Angoche

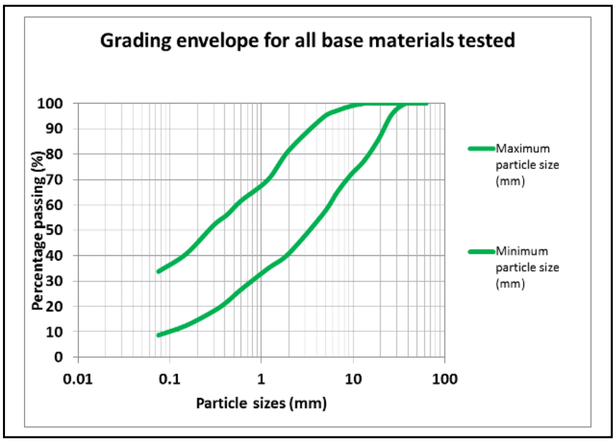


Figure 6-12 Grading envelope for all bases tested

Figure 6-12 shows the grading envelope of all base materials that were tested. This will be useful in developing grading specifications for base materials for low volume roads. Other grading parameters are given in Table 6-4.

Table 6-4 Other grading parameters for all bases tested

Road Name	Test Pits	Pavement Layers	Material description	Reject Index	Coarseness Index	Grading Modulus	Grading Coefficient
				IR	CI	GM	Gc
Boane Libombos	5+050	Base	Basaltic plastic gravel	0	57.9	2.2	14.8
	5+500	Base	Sandy gravel	0	64.7	2.3	14.2
	8+200	Base	Plastic Basaltic gravel	0	49.4	1.9	16.4
	8+450	Base	Fine sand basalt gravel	0	50.7	1.9	16.7
	9+100	Base	Silty basaltic gravel	0	61.8	2.4	10.8
Rio Zambezi Nicoadala	140+900	Base	Fine Quartzitic Clay	0	32.2	1.7	10.8
	141+300	Base	Fine sandy laterite	0	43.2	2.1	10.4
	140+700	Base	Fine Quartzitic Clay	0	46.3	2.1	11.4
	141+350	Base	Fine clayey laterite	0	46.4	2.1	11.1
Nametil Angoche	14+725	Base	Fine laterite gravel	0	49.6	2.2	11.3
	15+000	Base	Fine Laterite (CTB?)	0	18.4	1.6	6.4
	27+300	Base	Fine Laterite (loose)	0	40.6	1.9	12.0
	27+400	Base	Fine Laterite	0	50.8	2.1	11.3
Lichinga Mandimba	1+250	Base	Plastic laterite	0	29.4	1.4	16.4
	19+550	Base	Fine plastic laterite	0	24.1	1.3	13.6
Metangula Maniamba	2+550	Base	Shale gravel	0	37.4	1.7	16.8
Maniamba Lichinga	32+650	Base	Fine laterite	0	43.7	1.8	17.9
			Minimum Values	0	18.4	1.3	6.4
			Maximum Values	0	64.7	2.4	17.9
			Range	**0-0**	**18.4-64.7**	**1.3-2.4**	**6.4-17.9**

Note The red values show the extreme 'out of specification' values

The parameters used to assess grading (i.e. the particle size distribution of a material) are

a. Reject index (I_R), which is the percentage of oversize material. The oversize material may need screening if excessive.

   I_R = percentage retained on 37.5 mm sieve

b. Particle size distribution is given in the form of a grading curve or simply Grading Modulus. Grading Modulus (GM) is given by:

$$GM = 3 - \left(\frac{P_{2.36} + P_{0.425} + P_{0.075}}{100} \right)$$

where: $P_{2.36}$ = percentage passing 2.36 mm sieve

$P_{0.425}$ = percentage passing 0.425 mm sieve

$P_{0.075}$ = percentage passing 0.075 mm sieve

The reject index (IR) is the percentage of particles larger than 37.5mm. In this case IR = 0 and this indicates fine gravels without oversizes. The standard specifications require a grading modulus (GM) of between 2.05 and 2.65 for bases. The range of GM of between 1.3 and 2.4 shows that the grading is finer than the specification range. Material at the lower end of the GM scale could experience some stability problems at higher traffic volumes but should suffice for low volume roads.

It is important to have an idea of the size of the deviation of the grading of the bases from the standard specifications. This can be shown through calculation of the variance in GM values. The variance is given by:

GM Variance = (GM of base) – (minimum GM specification)

The variance of the GM values compared with minimum GM specification for roadbases are shown in Figure 6-13. The chart shows that more than half of the bases did not meet the minimum specifications of GM for roadbase material and that 5 bases differed considerably and yet they performed well.

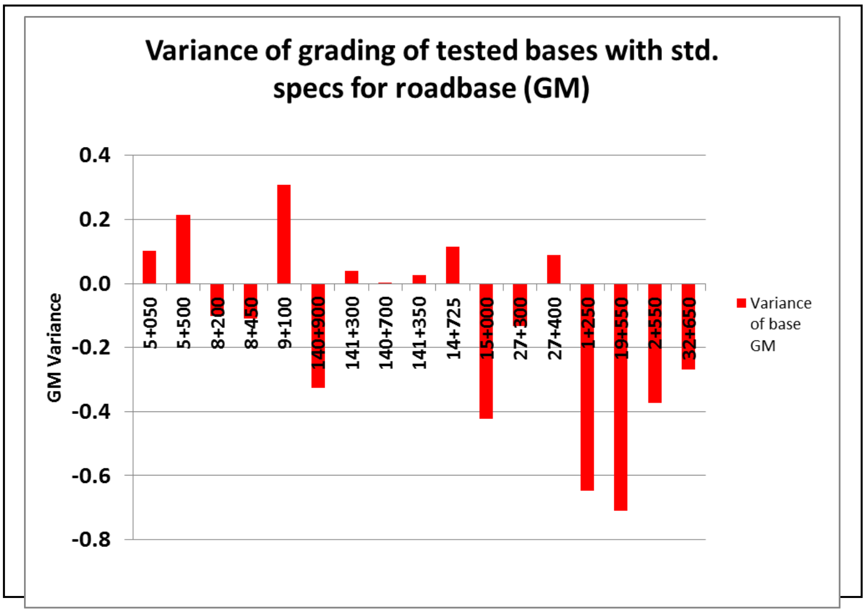


Figure 6-13 Variance of grading modulus (GM) for the bases tested

6.4.2 Plasticity

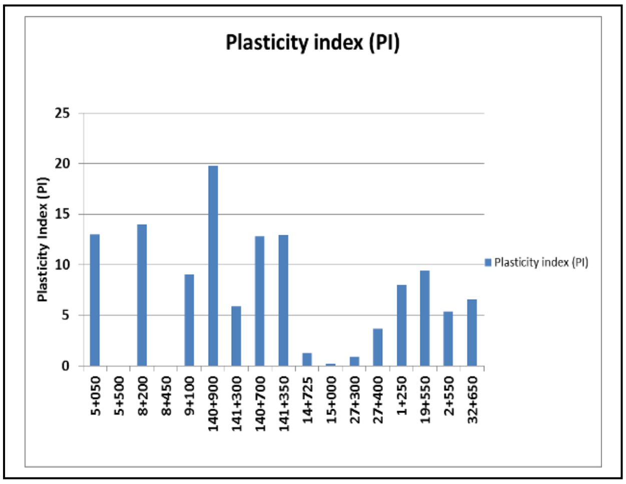
Plasticity plays a major part in the performance of road bases and indeed other layers within the pavement. However, recent research has shown that specifying plasticity of a material using the plasticity index (PI) alone is incomplete and therefore not accurate enough. PI values provide information on the quality of the clay in the gravel matrix but miss out on the quantity of the clayey material. It is fairly obvious that a small amount of clay will have a smaller effect than a larger quantity. PI alone has worked previously because the specifications only allowed very low values for the percentage passing the 0.075mm sieve (the fineness index, FI) which is basically the clay and active silt fraction ($P_{0.075}$). Standard specifications allowed a maximum $P_{0.075}$ of 15%.

However, with the ever increasing use of marginal materials, $P_{0.075}$ values are much higher than 15% and, at times, over 30%. In this case the specifications should be more elaborate by specifying the quality and the quantity of the plastic material. This is expressed through the plasticity product (PP) for which standard specifications require a value of less than

$$PP = PI \times P_{0.075}$$

Typical draft specifications for PP are that it should be less than 60 for high traffic conditions and less than 90 for lower traffic levels but these values have not been underwritten at this time.

The plasticities of the bases are shown in Table 6-5 and the variances from standard specifications are shown in Figure 6-14, Figure 6-15 and Figure 6-16 .



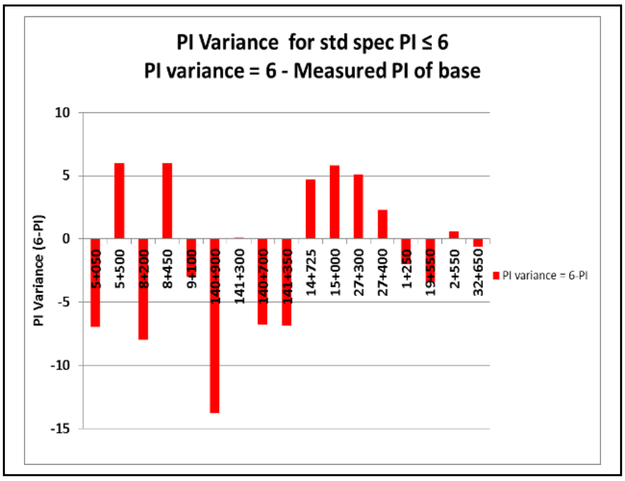


Figure 6-14 Plasticity Indices (PI) for all bases tested Figure 6-15 PI variance from std. spec PI<6

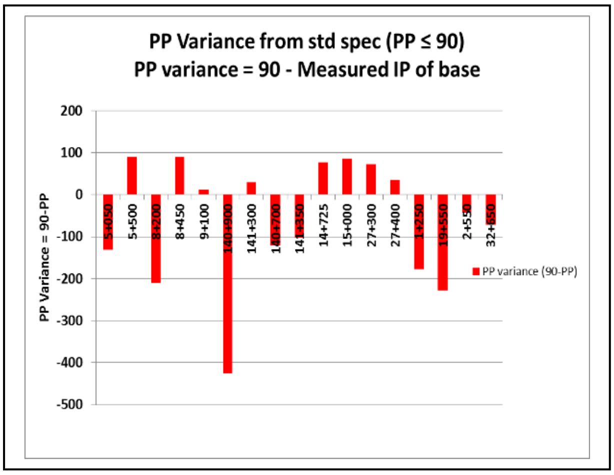


Figure 6-16 PP Variance for all bases tested

The results show that 9 bases out of 17 failed to meet the standard plasticity specifications but still performed well in-service. The highest plasticity values recorded were on the section on the Rio Zambezi Nicoadala Road which is part of the N1 North-South Highway. This section of road carries heavy trucks and the performance has been very satisfactory. The effect of plasticity on performance identified here will contribute to the review of specifications.

Table 6-5 Plasticity of the road bases

Road Name	Test Pits	Material description	Liquid Limit WL	Plastic Limit PL	Linear Shrinkage LS	Plasticity Index Ip	Shrinkage Product SP	Plastic Modulus PM	Plastic Product PP	Variances	
										Ip	PP
Boane Libombos	5+050	Basaltic plastic gravel	31	18	6.5	13	6.5	333	221	-7	-131
	5+500	Sandy gravel	0	0	0	0	0	0	0	6	90
	8+200	Plastic Basaltic gravel	31	17	7	14	7	465	300	-8	-210
	8+450	Fine sand basalt gravel	0	0	0	0	0	0	0	6	90
	9+100	Silty basaltic gravel	28	19	4.5	9	4.5	157	78	-3	12
Rio Zambezi Nicoadala	140+900	Fine Quartzitic Clay	43.4	23.6	9.3	19.8	294	664	516	-14	-426
	141+300	Fine sandy laterite	18.6	12.7	2.1	5.9	43	142	60	0	30
	140+700	Fine Quartzitic Clay	35.6	22.8	6.4	12.8	144	315	211	-7	-121
	141+350	Fine clayey laterite	32.6	19.7	5.7	12.9	123	308	192	-7	-102
Nametil Angoche	14+725	Fine laterite gravel	19.3	18	0.1	1.3	1.8	30	13	5	77
	15+000	Fine Laterite (CTB?)	26	25.8	0.6	0.2	17	7	4	6	86
	27+300	Fine Laterite (loose)	33.3	32.4	0.1	0.9	2.5	27	17	5	73
	27+400	Fine Laterite	33.8	30.1	0.1	3.7	2.0	82	55	2	35
Lichinga Mandimba	1+250	Plastic laterite	30.9	22.9	4	8	208	446	267	-2	-177
	19+550	Fine plastic laterite	32	22.6	4.7	9.4	243	529	318	-3	-228
Metangula Maniamba	2+550	Shale gravel	24.1	18.7	2.7	5.4	112	242	133	1	-43
Maniamba Lichinga	32+650	Fine laterite	31.4	24.8	3.3	6.6	124	271	162	-1	-72
		Minimum Values	0	0	0	0	0	0	0		
		Maximum Values	43	32	9.3	20	294	664	516		
		Range	0 - 43	0-32	0-9.3	0-20	0-294	0-664	0-516		

Note The red values for PI are outside standard specifications of 6.

The red values for PP are much higher than draft specifications of 90.

6.5 Stabilised bases

6.5.1 Cement-stabilised bases

Mozambique has very poor road construction materials and it is highly likely that cases will always arise where stabilisation is the only viable design option. These cases included:

1. Materials which are too plastic and exhibit very low CBRs. Stabilisation may be required to modify the materials for both strength and stability purposes.
2. Materials which are too loose such as coastal sands and single-sized or cohesionless soils, which may loosen up under traffic action.
3. Where traffic loading is expected to change or increase or where high wheel loads are anticipated even at low traffic volumes.

Generally, if the materials do not meet minimum specifications for low volume roads some modification of the material may be necessary.

1. Materials can be stabilised mechanically through compaction. High compactive effort can yield high bearing strength even on materials which may be considered to be weak.
2. Blending is another viable option where opportunities arise. Two or more unsuitable materials can be blended together to form a good material. The deficiency in the materials could be caused by high plasticity or lack of cohesion. Blending such materials could result in a material that meets the minimum specification for road base, especially for low volume roads.
3. Cement stabilisation is the most common form of stabilisation of road bases in Mozambique. Most of the main roads and some of the secondary roads have cement-stabilised bases (CTBs). The design and construction of cement-stabilised bases is critical to the performance of the bases and the pavement structures as a whole. The most common challenge is the cracking of the CTB. There are various causes of this and include:
    a. High cement content in the base material. This is very common in Mozambique because the materials are mostly sandy and consequently require high cement contents for stabilisation (5-7%).
    b. Over-application of compaction moisture. This causes excessive shrinkage and thus intensifies shrinkage cracking.
    c. Poor curing. This is perhaps the major cause. Usually, the curing procedures are not properly followed because, in most places in Mozambique, water is scarce and a lot of water is required to keep the sand blanket wet.
4. Emulsion-stabilisation. The advantage of emulsion treatment of bases is that they do not crack as do CTBs. Emulsion treated bases (ETBs) have been built in Mozambique in several places and are becoming more common. The quality of ETB is dependent on a number of aspects.
    a. The quality of the soil. The grading and PI of the soils are critical for the design of ETB. It is important to make reference to the design specifications of ETB when evaluating the performance of ETB in-service.
    b. The emulsion content. Higher emulsion content translates into higher content of residual bitumen and higher strength. However, a balance needs to be worked out between producing high quality ETB and the costs.

c. The quality of construction. The biggest challenge with the construction of ETB is the mixing. There is a tendency to form lumps when emulsion is mixed with soil, especially the fine soils. This results in uneven distribution of emulsion and consequently in poor and weaker ETB.

As part of this investigation, a number of sections with cement-stabilised bases, or where there was information that cement had been used, were selected. The sites included:

1. Pambara Rio Save Road was built during colonial time and is very old (> 30 years). Interestingly, on excavation of the test pit at the site it was observed that there were no cracks at all in the CTB which is curious. Extensive cracking is generally observed nowadays. It appeared as though a very high cement content was used on this site and it became necessary to find out what this cement content actually was.
2. Nametil Angoche Road, which was built 5 years ago, has sections which were stabilised with cement. The project document indicates that 3% cement was added to the fine laterite gravel. On excavation of the test pit the base appeared too soft to have been stabilised but nonetheless it is possible at this low cement content. It also became necessary to determine the cement content of the base.
3. Maniamba Lichinga Road was also built during colonial times and is very old. On excavation of the test pit, the road base of fine laterite was very hard. It was assumed that it had been stabilised but it was not clear whether it was cement or lime. Some old literature that was found towards the end of the project indicated that a lot of roads including parts of Maniamba Lichinga were built using laterite stabilised with lime but nothing specific to the test site was found. At the time of sampling it was assumed to have been cement and samples were collected for cement content tests. Below the hard base was a soft laterite sub-base which, unlike the base which was very dry, was wet. Below the wet base was a very hard and very dry red silt layer or subgrade and below this was a layer of wet red silt. This configuration and the moisture regime was puzzling. It was assumed that the hard red silt could have been stabilised too and samples were collected for cement content tests.
4. The cement content tests and the results are given in the Appendix. Table 6-6 summarises the results. It is important to note that the results do not show any cement in the base sample from Pambara Rio Save which was said to be cement stabilised. This was unexpected because the material in the shoulder which was not mixed properly during construction showed that the material had been stabilised and there were pockets of red silt which had not been coated. The section was built more than 30 years ago and it is possible that the cement could have completely carbonated over the years and thus no trace of it was found in the base. However, if this is the case it also follows that the material has reverted to its original natural state though with a higher fines content from the cement constituency. It also follows that the natural material is capable of carrying the current traffic, which is higher than at the time of construction, without stabilisation except for the aided compaction and modification of material properties. If this material was never stabilised in the first place then it opens the opportunity for expanded use of marginal materials.

Table 6-6 Cement content test results

Location	Material Type	Test Carried out	Ex silica (%)	Ex lime (%)	Cement content Preferred/ mean value (%)
Pambarra – Rio Save CH 22+650	Cement treated base	Cement content	0	0	0
Pambarra – Rio Save CH 22+650 LHS	Red silt	Soluble silica and calcium oxide content	0	0	0
Pambarra – Rio Save, CH 93+000 extract from Pit at Pande	Natural soil	Soluble silica and calcium oxide content	0	0	0
Maniamba – Lichinga, CH 32+650	Cement treated base	Cement content	7.3	7.3	7.3
Maniamba – Lichinga, CH 44+500	Laterite base	Soluble silica and calcium oxide content	0	0	0
Maniamba – Lichinga, CH 44+600	Red silt sub-grade	Soluble silica and calcium oxide content	0	0	0
Maniamba – Lichinga, CH 44+500 RHS	Sub-grade	Soluble silica and calcium oxide content	0	0	0
Maniamba – Lichinga, CH 32+650	Cement treated base	Cement content (use samples TRL 6A to C for soluble silica and calcium oxide content)	13.4	7.5	7.5
Nametil – Angoche, CH 15+100	Cement treated base	Cement content	5.6	1.9	1.9
Nametil – Angoche CH 15+100	Cement treated base	Cement content	0.9	0	0.5
Nametil – Angoche, CH 21+000 Borrow Pit	Natural soil	Soluble silica and calcium oxide content	0	0	0
Nametil – Angoche CH 36+000 Gravel Pit	Natural soil	Soluble silica and calcium oxide content	0	0	0
Rio Zambeze – Nichoadala, CH 140+700 RHS	Cement treated base	Cement content	0	0	0
Rio Zambeze – Nichoadala, CH 10+000	Clayey laterite Base	Soluble silica and calcium oxide content	0	0	0

6.5.2 Emulsion treated bases

Emulsion treated bases have been in development in Mozambique for some time. Major construction works were carried out on the Oasse Mocimboa da Praia and Macomia Oasse Roads approximately 13 years ago. At the time of our investigations the Macomia Oasse road had failed and had been earmarked for rehabilitation or reconstruction. It was important to investigate the performance and the parameters related to the performance of the ETB.

1. Oasse Mocimboa da Praia Road. Most sections of the road were in good to fair conditions but there were also some substantial failures in a limited number of areas along the road. Three test sections were selected. The first two sections were in fair condition but the third section at km 19 had lots of failures.
2. Macomia Oasse Road. Even though this road was not part of the selection of sites for the study it was important to understand why it had failed while the Oasse Mocimboa Road had performed relatively well.

Figure 6-17 shows the particle size distribution of the sand used for the ETB. It shows that the sand was unsuitable in terms of the particle size distribution compared with recommended specifications for ETB for low volume roads i.e. less than 1 million standard axles (< 1M). The sand is single-sized and needs high bitumen content if the mix is to achieve the desired characteristics.

The residual soluble bitumen content obtained through recovery of bitumen from ETB samples is shown in Table 6-7.

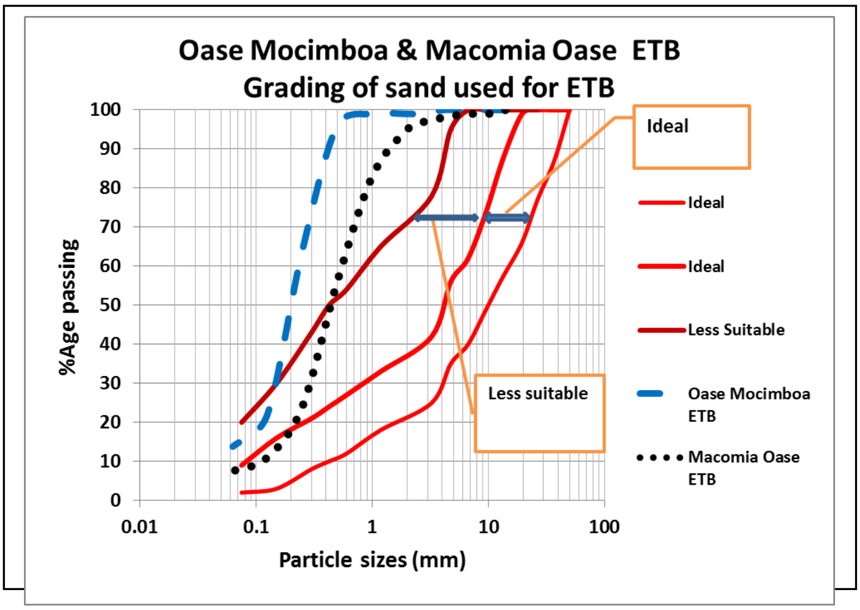


Figure 6-17 Grading of sand used for ETB on Oasse Mocimboa da Praia Road

Table 6-7 Bitumen content in ETB on the Macomia Oasse and Oasse Mocimboa roads

Road	Soluble binder content/residual bitumen	Stabilisation Emulsion content in ETB
Oase Mocimboa da Praia Road	6.9	=6.9 x (1/0.6) = 11.5%
Macomia Oase Road	2.0	= 2.0 x (1/0.6) = 3.3%

The difference in the emulsion content used during construction is significant and could explain the difference in performance. The emulsion content on the Macomia Oasse road was too low to provide adequate strength in the sand base. In contrast the emulsion content on Oase Mocimboa da Praia was on the high side. Usually the emulsion content should be in the range of approximately 5 to 7% subject to design. Although 11.5% was probably good for the strength of the ETB, it was much more expensive.

Too much bitumen might also have been a problem if the voids in the sand had become too full, that is why the design procedure is so critical. Single-sized sands have high voids but low stability hence, to achieve the correct characteristics in the mix, the bitumen content needs to be high.

6.6 Surfacings

The surfacing on the road has several important functions:

1. Waterproofing. Bituminous surfacings, in particular, provide a seal coat on the surface of the pavement thereby maintaining the base materials at or below the optimum moisture content. This helps to maintain the strength of the base course and other pavement layers below it.
2. Wearing course. The bituminous surfacing is resistant to wear and tear. Opening of the surfacing usually results in rapid deterioration of the pavement and the road as a whole.
3. Structural strength. Thick surfacings (greater than 40 mm) contribute to the overall strength of the pavement. However, the strength contribution of thin surfacings is usually negligible.

6.6.1 Bitumen tests

In order to investigate the surfacings on the test sites samples were collected and delivered to laboratories for testing. The laboratory tests included the basic bitumen tests at the initial stages.

1. Composition tests. This involved bitumen extraction to determine the bitumen content and other constituents of surfacing samples.
2. Sieve analysis of the aggregate to determine the particle size distribution.
3. Penetration and softening points of the bitumen. These are important parameters that define the properties of the bitumen and its quality.
4. Viscosity. When bitumen ages and oxidises its stiffness increases to such an extent that the standard penetration test becomes too insensitive to be of much use. The usual procedure to characterize the bitumen is to measure its viscosity and durability at an elevated temperature. This is directly related to the ageing of the bitumen.

The tests were carried out in the UK under the responsibility of the TRL Laboratory. The results for the bitumen extractions are given in Table 6-8.

Table 6-8 Bitumen content, penetration and softening point for all samples

Site	Chainage	Sample	Bitumen Content %	Penetration	Softening point °C	Comments
Boane Libombos	5+050	DSD	6.0	2	78.2	Very little cracking
Boane Libombos	8+200	DSD	5.7	3	84	Some cracking
Boane Libombos	5+050	DSD	8.9	3	76.2	Minor cracking
Pambara Rio Save	22+650	Hot sand asphalt	9.0	18	61.2	Seal is recent. No distress
Oasse Mocimboa da Praia	0+050	ETB	2.0	2	93	Fair condition, minor cracking
Oasse Mocimboa da Praia	0+050	Sand seal	6.9	3	84.3	
Maniamba Lichinga	32+650	DSD	5.4	3	77.8	Resealed in 2007. No deterioration
Nametil Angoche	27+300	Single Otta seal + sand seal	5.3	10	64.8	Good condition
Nametil Angoche	27+400	Single Otta seal	5.8	12	63.9	Some cracking
Rio Zambezi Nicoadala	140+700	Hot sand asphalt	6.0	*35*	*51.4*	Minor cracking
Rio Zambezi Nicoadala	140+900 Left Lane	Hot sand asphalt	8.7	35	51.4	Minor cracking
Fresh bitumen from Mozambique (80/100 pen)				58		

The important aspects to note regarding the results are:

1. The very low penetration values obtained on some the samples (<10) are an indication that the bitumen had hardened and is very stiff. Loss of ductility results in the development of cracks under traffic loading.
2. Only bitumen extracted from the hot sand asphalt still had slightly higher penetration values. This is a result of reduced oxidation in the lower part of the hot sand asphalt layers.
3. The penetration value (58) of the fresh bitumen sample that was supplied as 80/100 penetration grade was from Mozambique and is cause for concern. A penetration value of 58dmm at 25˚C instead of 80-100dmm implies that either the wrong bitumen was supplied or that the bitumen is old and already degraded. 80/100 pen bitumen is used as a base binder in the production of MC3000 and emulsions which are also commonly used in Mozambique. If the base bitumen is harder, the addition of 10% kerosene will not produce MC3000 but one that is stiffer than MC3000. This has lots of implications too.
    a. The construction of Otta seals which require the use of soft binder like MC3000, 150/200 pen and MC800 can be seriously affected because the binder could harden too quickly and prevent curing of the Otta seal. The same applies to the construction of sand seals.
    b. The use of such hard binder also affects the proper construction of surface dressings because the binder may be too hard at the road temperature for the aggregate to adhere to it properly. Bitumen penetration should always be checked but it is unlikely that this is happening on all sites.
4. The Otta seal on Nametil Angoche Road was constructed 5 years ago and MC3000 was used. To obtain MC3000, 80/100 pen bitumen was mixed with kerosene to give a penetration values of over 150dmm. The results show that the bitumen has already hardened to a penetration of 10dmm within 5 years. The bitumen is therefore almost brittle and if traffic loading were to increase significantly the surfacing could start to crack rapidly and fail.
5. The softening point test of the samples show that most of the values were above 60˚C and therefore too hard for some standard viscosity tests. Brookfield and kinematic viscosity tests were carried out instead. The Brookfield viscosity is carried out at 120˚C, 150˚C and 180˚C. At these temperatures the bitumen is fluid enough for viscosity tests to be carried out effectively. This is not a common test and there are limited laboratories that have capacity to conduct it.
6. Bitumen content is another important parameter because it gives information on the adequacy of the binder in the surfacing. For example, the residual bitumen content for ETB should be approximately 3.6% minimum. The bitumen content for double surface dressing should be approximately 2.5L/m^2, which is approximately 6-8%.

These tests were carried out at Surrey County Council in conjunction with the TRL laboratory in UK. However, Surrey County Council had no capacity to carry out the Brookfield viscosity tests and samples were transferred to Nottingham Technical Laboratory (NTEC). The tests that were carried out at NTEC included:

1. Penetration tests at 45˚C.
2. Ductility tests
3. Brookfield viscosity tests at 120˚C, 150˚C and 180˚C.
4. Kinematic viscosity

The penetration, Brookfield and kinematic viscosity and ductility tests results for the recovered bitumen are given in Table 6-9. The results of similar tests carried on fresh bitumen from Mozambique and the UK are given in Table 6-10. These can be compared with the bitumen test specifications given in Table 6-11.

The fresh bitumen samples were subjected to aging in the laboratory using the Rolling Thin Film Oven Test and the pre and post aging tests results are compared in Table 6-12. Both samples were not compliant with specifications on the difference between the penetration values, which according to specifications in Table 6-11 should not exceed 20.However, the rest of the parameters are compliant.

All samples showed compliance with minimum specifications on ductility.

Table 6-9 Bitumen test results: penetration, Brookfield and kinematic viscosity and ductility (recovered samples)

Location	Penetration	Test	Remarks	Results	Comments
Pambarra - Rio Save CH 22+650	18	Brookfield Viscosity	At 120, 150 & 180°C	2.4, 0.4, 0.11 Pa.s	Recent reseal in good condition
		Ductility @25°C	2 samples	373mm	
Nametil - Angoche CH 27+300	10	Brookfield Viscosity	At 120, 150 & 180°C	2.1, 0.3, 0.1 Pa.s	Surface is an Otta seal with sand seal capping in good condition
		Ductility @25°C	2 samples	481mm	
Nametil - Angoche CH 27+400	12	Brookfield Viscosity	At 120, 150 & 180°C	2.4, 0.42, 0.11 Pa.s	A single Otta seal with no sand capping. Some cracks
		Ductility @25°C	2 samples	475mm	
Rio Zambeze - Nicoadala CH 140+700	21	Brookfield Viscosity	At 120, 150 & 180°C	1.0, 0.2, 0.06 Pa.s	Sand asphalt. Some cracking but not at this chainage
		Ductility @25°C	2 samples	>1000mm	
Rio Zambeze - Nicoadala CH 140+900	35	Brookfield Viscosity	At 120, 150 & 180°C	1.2, 0.2, 0.07 Pa.s	Sand Asphalt but no cracking at this chainage
		Ductility @25°C	2 samples	>1000mm	

Table 6-10 Bitumen test results: penetration, Brookfield viscosity, ductility and kinematic viscosity (fresh bitumen samples)

Location	Test	Remarks	Results
Mozambique Stock 80/100 Pen Bitumen	Penetration	Before ageing	72dmm
	Softening Point	Before ageing	46.4 °C
	Solubility of bitumen	Before ageing	100%
	Ductility of Bitumen	Before ageing (@ 25°C)	>1000mm
	Brookfield Viscosity	Before ageing @ 120, 150 & 180°C	0.74, 0.17, 0.05 Pa.s
	RTFOT Resistance to hardening	Mass loss/gain	-0.17%
	Penetration	After ageing	44 dmm
	Softening Point	After ageing	51.0°C
	Brookfield Viscosity	After ageing @ 120, 150 & 180°C	1.15, 0.22 & 0.07 Pa.s
	Ductility of Bitumen	After ageing (Test at 25°C)	>1000 mm
	Kinematic viscosity at 135°C		359 mm^2/s
Mozambique Stock 80/100 Pen Bitumen	Penetration	Before ageing	71 dmm
	Softening Point	Before ageing	47.6°C
	Solubility of bitumen	Before ageing	100%
	Ductility of Bitumen	Before ageing (@ 25°C)	>1000 dmm
	Brookfield Viscosity	Before ageing @ 120, 150 & 180°C	0.79, 0.17, 0.06 Pa.s
	RTFOT Resistance to hardening	Mass loss/gain	-0.15%
	Penetration	After ageing	42 dmm
	Softening Point	After ageing	50.6°C
	Brookfield Viscosity	After ageing @ 120, 150 & 180°C	1.04, 0.21 & 0.06 Pa.s
	Ductility of Bitumen	After ageing (@ 25°C)	>1000mm
	Kinematic viscosity at 135°C		356 mm^2/s
UK Stock 70/100 Pen Bitumen	Penetration	Before ageing	81 dmm
	Softening Point	Before ageing	46 °C
	Solubility of bitumen	Before ageing	100%
	Ductility of Bitumen	Before ageing (Test at 25°C)	>1000mm
	Brookfield Viscosity	Before ageing at 120, 150 & 180°C	0.76, 0.17, 0.05 Pa.s
	RTFOT Resistance to hardening	Mass loss/gain	-0.07%
	Penetration & Softening Point	After ageing	54dmm & 50.4°C
	Softening Point	After ageing	50.4°C

	Brookfield Viscosity	After ageing at 120, 150 & 180°C	1.2, 0.24 & 0.08 Pa.s
	Ductility of Bitumen	After ageing (Test at 25°C)	>1000mm
	Kinematic viscosity at 135°C		380mm2/s
UK Stock 70/100 Pen Bitumen	Penetration	Before ageing	82 dmm
	Softening Point	Before ageing	45.6°C
	Solubility of bitumen	Before ageing	99.90%
	Ductility of Bitumen	Before ageing (Test at 25°C)	>1000mm
	Brookfield Viscosity	Before ageing at 120, 150 & 180°C	0.77, 0.18, 0.06 Pa.s
	RTFOT Resistance to hardening	mass loss/gain	-0.16%
	Penetration	After ageing	55 dmm & 50.4°C
	Softening Point	After ageing	50.4°C
	Brookfield Viscosity	After ageing at 120, 150 & 180°C	1.23, 0.24, & 0.07 Pa.s
	Ductility of Bitumen	After ageing (Test at 25°C)	>1000mm
	Kinematic viscosity at 135°C		361 mm2/s

Table 6-11 Bitumen test specifications

Bitumen Tests	Test Method	CEN
Softening Point Test (°C)	EN 12697-1	43-51
Penetration Test at 25°C (dmm)	EN 1426	80-100
Dynamic Viscosity (Pas) minimum		90
Kinematic Viscosity (mm^2/s) minimum		230
Ductility Tests at 25°C (mm) minimum	EN 13589	100
Loss on heating (wt)% maximum		0.5
Rolling Thin Film Oven Test (RTFOT): Drop in penetration after heating: Minimum		20

Table 6-12 Assessment of quality of bitumen through aging (RTFOT)

Source	Parameter	Test results		
		Before Aging	After Aging	Difference
Fresh bitumen from Mozambique	Penetration (dmm)	72	43	29
	Softening point	47	51	3.8
	Ductility	>1000	>1000	0
	Brookfiled viscosity (Pas) @120°C	0.77	1.09	0.33
	@150°C	0.17	0.22	0.045
	@180°C	0.06	0.07	0.01
	Kinematic viscosity @ 135°C (minimum = 230 mm^2/s)	358		
Fresh bitumen from UK	Penetration (dmm)	82	55	27
	Softening point	46	50	4
	Ductility	>1000	>1000	0
	Brookfiled viscosity (Pas) @120°C	0.77	1.22	0.45
	@150°C	0.18	0.24	0.06
	@180°C	0.06	0.08	0.02
	Kinematic viscosity @ 135°C (minimum = 230 mm^2/s)	371		

6.6.2 Evaluation performance and durability of binders in-service

The aging tests carried out on the fresh bitumen samples from Mozambique and the UK were an accelerated simulation of the long term aging of bitumen in-service, Table 6-12. The results of the Brookfield viscosity and ductility tests carried on the recovered bitumen from the samples obtained from the test sites were then compared with the results of the laboratory aged samples of fresh bitumen, Table 6-13. The comparison is given in the form of deterioration factor (DF) of the binders.

For Brookfield viscosity - deterioration Factor (DFv) is given by:

DFv = (Viscosity of field sample)/(Viscosity of aged fresh bitumen sample)

For ductility – deterioration ratio (DFd) is given by:

DFd = (Ductility of fresh sample)/(Ductility of field sample)

The results show that in general terms the deterioration of the binders from site is greater by a factor of 2 approximately. It is also important take note of the ages of the surfacings given in Table 6-13 which are not more than 5 years which is relatively a short period.

It can be concluded from this assessment that the bitumen is deteriorating for too rapidly under the Mozambique conditions and this translates to reduced service life of the surfacings.

Table 6-13 Assessment of in-service performance of bitumen from sites

Site	Parameter	Test results			
		Actual aging in-service	Simulated aging in laboratory	Deterioration factor	Age of surfacing
Pambara Rio Save	Ductility	373	>1000	> 2.7	4
	Brookfiled viscosity (Pas) @120˚C	2.4	1.22	2.0	
	@150˚C	0.4	0.24	1.7	
	@180˚C	0.11	0.08	1.4	
Nametil Angoche (27+300)	Ductility	481	>1000	> 2.1	5
	Brookfiled viscosity (Pas) @120˚C	2.1	1.22	1.7	
	@150˚C	0.3	0.24	1.3	
	@180˚C	0.1	0.08	1.3	
Nametil Angoche (27+400)	Ductility	475	>1000	> 2.1	5
	Brookfiled viscosity (Pas) @120˚C	2.4	1.22	2.0	
	@150˚C	0.42	0.24	1.8	
	@180˚C	0.11	0.08	1.4	
Rio Zambezi Nicoadala (140+700) Right Lane	Ductility	>1000	>1000	0	4
	Brookfiled viscosity (Pas) @120˚C	1	1.22	0.8	
	@150˚C	0.2	0.24	0.8	
	@180˚C	0.06	0.08	0.8	
Rio Zambezi Nicoadala (140+900) Right Lane	Ductility	>1000	>1000	0	4
	Brookfiled viscosity (Pas) @120˚C	1.2	1.22	1.0	
	@150˚C	0.2	0.24	0.8	
	@180˚C	0.07	0.08	0.9	

6.6.3 Petrography

Petrography is the study of the mineral composition and textural relationships within the rock. The aggregate obtained from the surfacing samples after the binder was extracted were sent for petrography tests. The main objective was to determine the mineral composition and the type of aggregate used in general terms. The mineralogy and texture contribute to the strength of the aggregate and also influence the performance of surfacings.

Usually the engineering properties take precedence in the evaluation process and such test as the aggregate crushing value (ACV), the 10% FACT and water absorption, bitumen affinity are carried out before the aggregate can be approved for use. These can be carried out locally in Mozambique. For this study petrography was prioritized in order to understand the typical mineralogy of the aggregate that was used. The results are given in Table 6-14.

It can be deduced from the results that rhyolite exists in the southern regions of Mozambique. Rhyolite tends to exhibit high flakiness and layered lattice structures. It can be used successfully in the construction of surfacings but care should be taken as the variability of rhyolite is very significant. There is a very wide range of aggregate strengths and types from chalky limestone look-a-like to one that looks like granite.

The same type of rock can be found in the central parts of Mozambique, Chiluvo in particular and it is generally weak and unsuitable for the construction of surfacings on high volume roads. For low volume roads such marginal aggregate can be used successfully.

The results also show the feldspar type of aggregate. Feldspar resembles quartz but it is much weaker. It can be used successfully in the construction of surfacings on low volume roads but tends to strip because it has a relatively loose texture.

Another significant factor is the clay coating on the minor minerals. Though minor they tend to affect the adhesion of bitumen to the surfacing of the aggregate, sometimes resulting in stripping even at adequate bitumen content. Single seals or single Otta seals can be affected.

Table 6-14 Petrography tests results

Road Name	Chainage	Layer	Description	Aggregate type	Constituents[1]			Grade[3]
					Major	Minor	Trace	
Boane-Libombos	5+050	Surfacing	Double surface dressing	Rhyolite gravel	Rhyolite			2
Boane-Libombos	5+050	Surfacing	Double surface dressing	Rhyolite gravel	Rhyolite			2
Boane-Libombos	8+200	Surfacing	Double surface dressing	Rhyolite gravel	Rhyolite			2
Pambarra-Rio Save	22+650	Surfacing	Hot sand asphalt	Quartzitic sand	Quartz	Clay	Muscovite mica	1
						Agglomeration[2]	Zircon	1
Oasse-Mocimboa da Praia	0+050	Base	Emulsion treated base	Quartzitic sand	Quartz	Clay	Muscovite mica	1
						Agglomeration[2]	Zircon	
Maniamba-Lichinga	32+650	Surfacing	Double surface dressing	Diorite gravel	Diorite			2
					Granite			2
							Agglomeration[2]	1
Nametil-Angoche	27+400	Surfacing	Otta seal with no sand cover	Quartzitic sand	Quartz	Alkali feldspar	Muscovite mica	1
					Agglomeration[2]	Clay		
						Plagioclase feldspar		
Rio Zambeze-Nicoadala RHS	140+700	Surfacing	Hot sand asphalt	Quartzitic sand	Quartz	Clay	Muscovite mica	1
					Agglomeration[2]		Zircon	
Rio Zambeze-Nocoadala LHS	140+900	Surfacing	Hot sand asphalt	Quartzitic sand	Quartz	Clay	Muscovite mica	1
						Agglomeration[2]		
Macomia-Oasse	39+000	Base	Emulsion treated base	Quartzitic sand	Quartz	Quartzite	Muscovite mica	1
						Agglomeration[2]	Clay	
						Zircon		

1 Major≥10%, minor 2-10%, trace <2%.
2 This is not a specific rock type and is an agglomeration of lithological components, compirsied of quartz grains loosely bound by iron rich-clays
3 Grade I (**Fresh**): Unchanged from original state; Grade II (**Slightly Weathered**): Slight discoloration, slight weakening; Grade III (**Moderately Weathered**): Considerably weakened, penetrative discoloration, large pieces cannot be broken by hand; Grade IV (**Highly Weathered**): large pieces can be broken by hand, does not readily disaggregate (slake) when dry sample immersed in water; Grade V (**Completely Weathered**): considerably weakened, slakes, original texture apparent; Grade VI (**Residual Soil**): soil derived by in situ weathering but retaining none of the original texture or fabric.

7 Proposed recommendations

These recommendations are based on the general assessment of performance and material properties determined through this project.

A. Low and highly variable in-situ CBRs in base courses

Observations:

On most of the sections that were upgraded to sealed road standards, the strength of the base layers in terms of the in-situ CBR values measured with the DCP were much lower than the minimum expected (40% soaked means a much higher in-situ value) and some in-situ CBRs were unacceptably too low (e.g. 30%) because the in-situ CBR is supposed to be much higher than the minimum soaked CBR.

Possible causes:

This could be an indication of poor construction practices

a. Contamination of the gravel at source through an excavate-and-load approach.

b. Sub-optimum or over application of moisture during mixing.

c. Inadequate rolling.

Recommendations:

1. *Stockpile materials in the gravel or borrow pits and carry out materials tests on the stockpiled materials before haulage.*
2. *It is recommended that field density tests should be verified with DCP tests within 1m of the sand replacement test position. This will help to check for accuracy of the field density results. In addition much more DCP tests can be carried out than the sand replacement density tests.*
3. *It is recommended to use the field density judgment chart Figure 7-8. It is important to determine the average of the field densities and the standard deviation. The next step is to plot the point in the chart and read out whether the test results should be acceptable or not. The advantage of using this chart is that the variability of the test results is taken into account using the standard deviation in approving compaction. This ensures better quality control of the compaction.*

B. High pavement strengths

Observation:

There was no significant structural failure on all of the test sections and it appeared that most of the failures were superficial i.e. on the surfacings.

Possible explanations:

1. The structural number (SN, SNP) and DN800 values indicated that the pavements were very strong and some had capacity to carry in excess of 1 million standard axles (1MESA) and others up to 30 MESAs before the subgrade is expected to fail, Table 6-2.
2. While most of pavements have a single base layer or single base and sub-base layers a greater part of the pavement strength is the result of consolidated subgrade which is now behaving as sub-base.

Recommendations:

1. *When upgrading unpaved roads to sealed road standard it is advisable to follow the existing alignment in order to take advantage of the in-situ strength resulting from long term consolidation in-service.*
2. *It is advisable during construction not to disturb consolidated existing layers unless it is absolutely necessary.*

C. Field moisture conditions

Observations:

The general trend is that the in-service moisture is generally lower than the OMC with the exception of Rio Zambezi Nicoadala sections. This aids the performance of the road pavements.

Possible explanations:

1. The subgrade and in-situ soils were generally well-drained and the significance of the minimum crown height of 750mm (Environmentally Optimised Design, EOD specification) was diminished. Most of the crown heights that were measured were much lower than 750mm, so the crown height did not seem to have much influence on the performance of the pavements.

2. Despite high rainfall in some areas, others received much less hence the localized nature of rainfall probably has some influence.
3. Most sections still had sound surfacing which provide an effective seal keeping the bases and pavement layers dry.

Recommendations:

1. *Where the soil is freely draining it may not be necessary to achieve the minimum requirements for crown height (750mm). Crown height can be designed to be as low as 300mm.*
2. *High water table or poor subsurface drainage should be dealt with at design stage. Sub-surface drainage can be incorporated in the design where required.*

D. Weak bases

Observations:

There were some cases where roadbase materials had high plasticity and low CBRs. The effect of the plasticity was apparent in inducing cracking of the surfacing. However the failures were not major; one section of high plasticity (PI=19.8) was on a test section of the N1 Rio Save Nicoadala but it was performing adequately.

Possible explanations

1. The in-situ strength of the base layer was much higher (~70%) than would be expected of a materials of soaked CBR 5.1%. This could be a result of long term consolidation of the clayey base when traffic was still very low. This material would most likely fail prematurely without the added strength from consolidation.

2. Some of the materials were simply too fine and with high plasticity but in the same manner consolidation could have enhanced the performance of the materials.

Recommendations:

1. *The recommendations are based on the good performance of the roads where the materials were adequate for purpose but did not meet standard specifications. The assessment was carried out using cumulative sums of CBR, PI, PP and GM. See Table 7-1.*
2. *Recommendations for strength in terms of soaked and in-situ CBRs are given in Figure 7-1 and Figure 7-2 and Table 7-2. The normal standard specification for roadbase material is a minimum soaked CBR of 80%. The Figure shows that 85% of the roadbases were below this value but have performed well, thus the specification appears to be very conservative. Relatively recent research elsewhere in southern Africa (TRL. Environmentally Optimised Design) has also shown that lower CBR values are acceptable. The minimum recommended soaked CBR for the roadbases of low volume roads is about 40% (and even lower for very low volume roads). This corresponds to the lower 25th percentile for the roads in this project indicating that a CBR of 40% might still be conservative. However, in view of the relatively small sample of roads in this project it is recommended that 40% is used.*
3. *However, recent research has shown that it is the in situ CBRs at the worst likely moisture content that are more important and that the moisture content is very rarely above OMC. Figure 7-2 shows that the lower quartile was about 60%. The 25% of the roads with lower values all performed well so 60% may be considered conservative but the in situ values were not all obtained during the wettest period of the year so, until more data are available, 60% is a reasonable recommendation.*
4. *Figure 7-3 shows the distribution of Grading Modulus for the roadbase materials. A recommended value of > 2.05 is sometimes quoted but values much lower than this have performed well. Recent research elsewhere is showing that provided the CBR criteria are met, the Grading Modulus is not as important as previously thought. From this research the recommended minimum grading modulus (GM) is 1.6. This may be conservative for very low volume roads but until more data are available, this is a prudent choice. When GM is between 1.3 and 1.6 the PP must be less than 200. This range of GM is for very fine gravel or fine to coarse sandy material.*
5. *Plasticity (Figure 7-4) is considered to be an important parameter in the performance of gravels. Based on the analysis, none of the test sections failed significantly. The upper limit of the plasticity in terms of the plasticity index (PI) and the plasticity product (PP) determines the quality of the base material. The 85th percentile PI was 13 and the 95 percentile PI was 15. Since no failures were obtained, 15% is acceptable and can be considered as the maximum limit.*

   Plasticity Product (Figure 7-5) is more representative of the plasticity of a material because it takes into account the amount of material that is plastic. The 85th percentile value for PP was 460 and the 95th percentile was 570. Recent research elsewhere is showing that provided the CBR criteria are met, the plasticity is not as important as previously thought. However, until more data are available it is prudent to recommend that the PP should not exceed 460.

The cumulative percentage graphs shown in Figure 7-1, Figure 7-2, Figure 7-3, Figure 7-4, Figure 7-5 are derived from plotting the cumulative frequency values in percentage against the values of items or group averages of item value ranges. The cumulative frequency is the sum of frequencies of each item in percentage. The frequency for each value is the number of times the value occurs out of the total number of items obtaining. For example, there are 17 values of plasticity index in Table 7-1 and each value occurs once so for each value the frequency is (1/17) expressed as (1/17)x100%. As the

PIs are presented in order of increasing values the cumulative percentage is the sum of the percentages up to the given value. This statistical method is used to determine the percentile values of the items and in this case the PIs. The same method was used for the rest of the parameters.

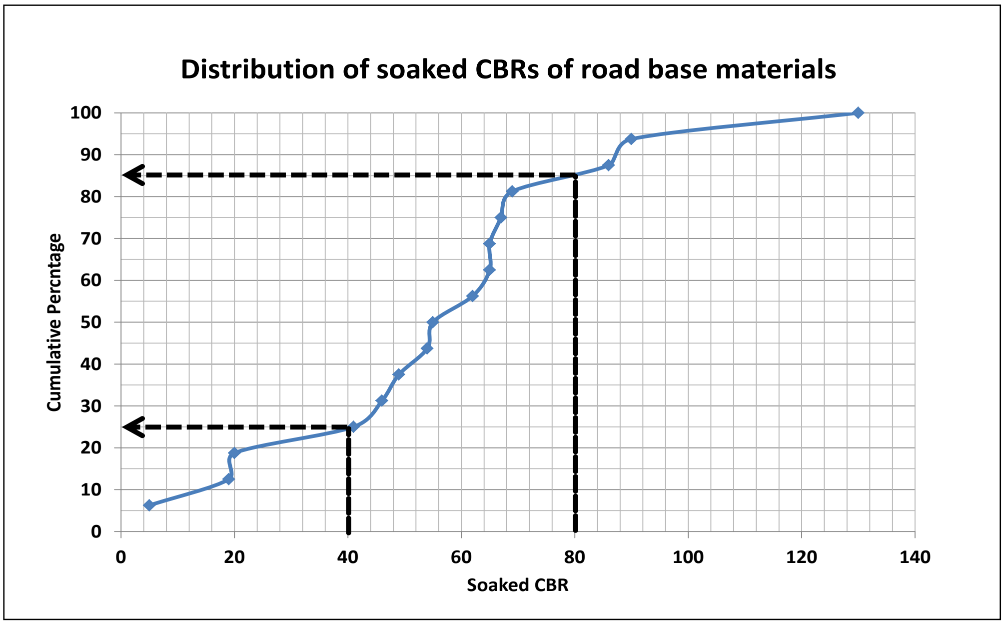


Figure 7-1 Limits of percentiles determined from known soaked CBR limits for LVRs

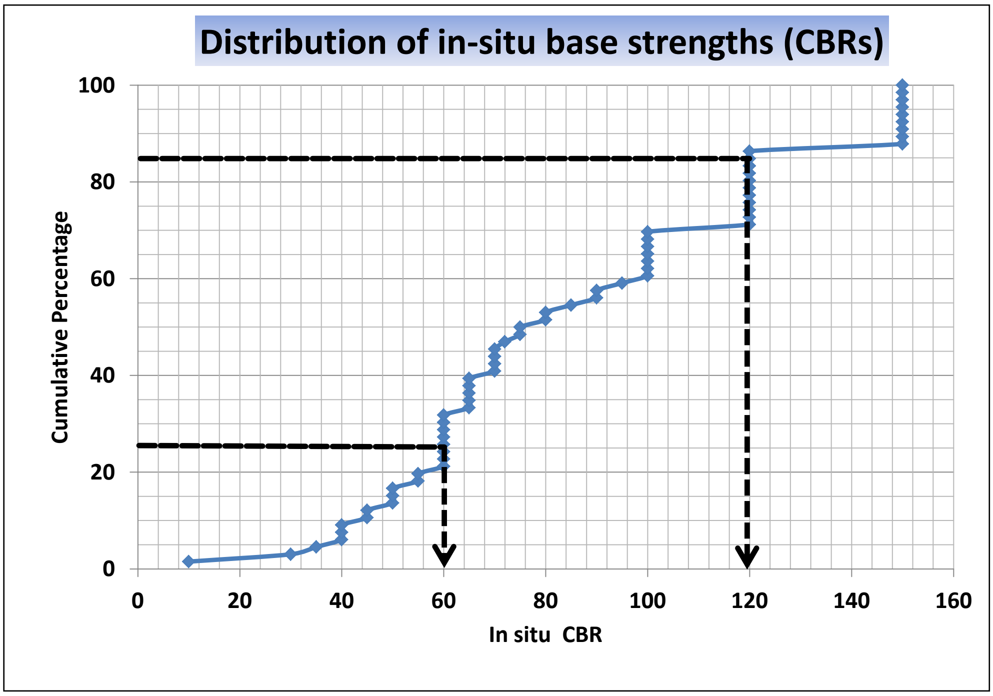


Figure 7-2 Determination of average in-situ CBR limits for bases on LVRs

Table 7-1 Calculation of cumulative sums

Coarseness Index	Coarseness Index	Grading Modulus	Grading Modulus	Grading Coefficient	Grading Coefficient	Plasticity Modulus	Plasticity Modulus	Plasticity Index	Plasticity Index	Plasticity Product	Plasticity Product
CI	**%**	**GM**	**%**	**Gc**	**%**	PM	%	PI	%	PP	%
18.4	5.3	1.3	5.3	6.4	5.3	0	5.9	0	5.9	0	6
18.4	10.5	1.3	10.5	6.4	10.5	0	11.8	0	11.8	0	12
24.1	15.8	1.4	15.8	10.4	15.8	7	17.6	0	17.6	4	18
29.4	21.1	1.6	21.1	10.8	21.1	27	23.5	1	23.5	13	24
32.2	26.3	1.7	26.3	10.8	26.3	30	29.4	1.3	29.4	17	29
37.4	31.6	1.7	31.6	11.1	31.6	82	35.3	3.7	35.3	55	35
40.6	36.8	1.8	36.8	11.3	36.8	142	41.2	5.5	41.2	60	41
43.2	42.1	1.9	42.1	11.3	42.1	157	47.1	5.9	47.1	78	47
43.7	47.4	1.9	47.4	11.4	47.4	242	52.9	6.6	52.9	133	53
46.3	52.6	1.9	52.6	12	52.6	271	58.8	8	58.8	162	59
46.4	57.9	2.1	57.9	13.6	57.9	308	64.7	9	64.7	192	65
49.4	63.2	2.1	63.2	14.2	63.2	315	70.6	9.5	70.6	211	71
49.6	68.4	2.1	68.4	14.8	68.4	330	76.5	12.8	76.5	221	76
50.7	73.7	2.1	73.7	16.4	73.7	445	82.4	12.9	82.4	267	82
50.8	78.9	2.2	78.9	16.4	78.9	465	88.2	13	88.2	300	88
57.9	84.2	2.2	84.2	16.7	84.2	530	94.1	14	94.1	318	94
61.8	89.5	2.3	89.5	16.8	89.5	664	100.0	19.8	100.0	516	100
64.7	94.7	2.4	94.7	17.9	94.7						
64.7	100.0	2.4	100.0	17.9	100.0						
19		18		18		17		17		17	

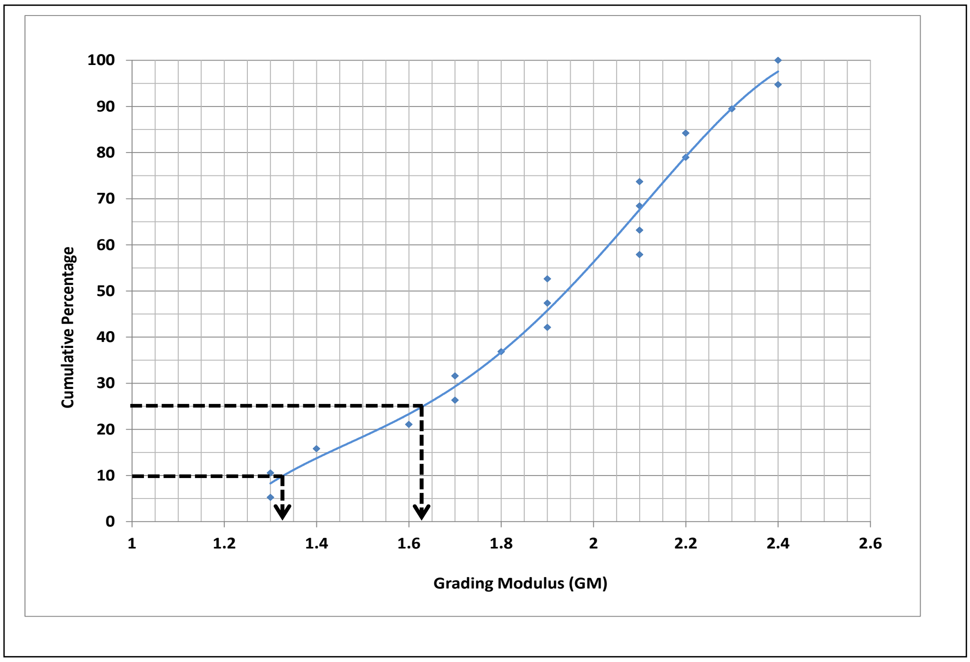


Figure 7-3 Determination of limits for GM for LVRs

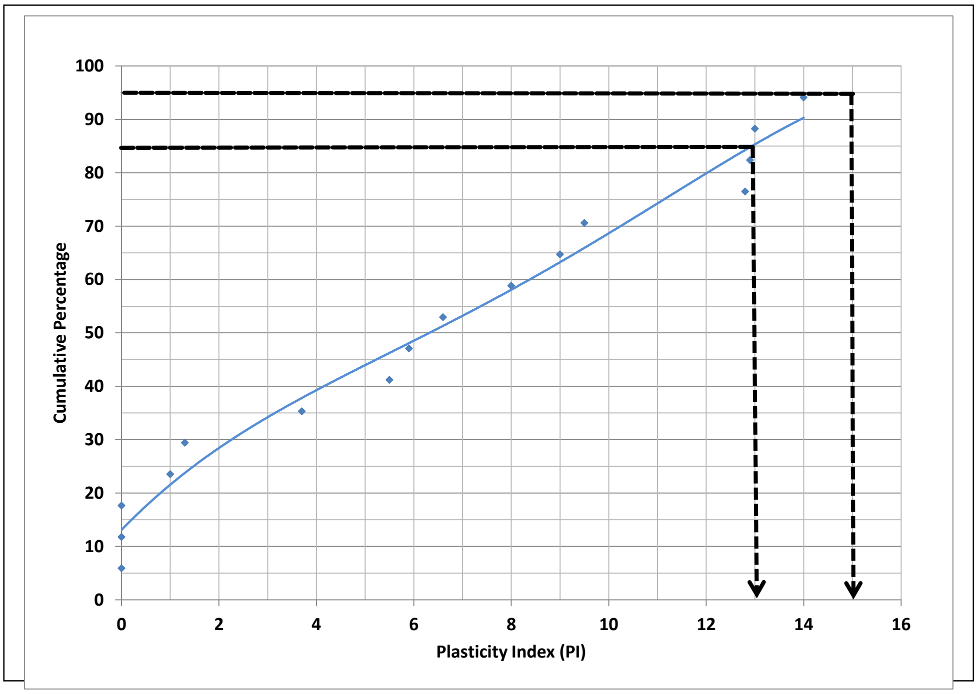


Figure 7-4 Determination of limits for PI for LVRs

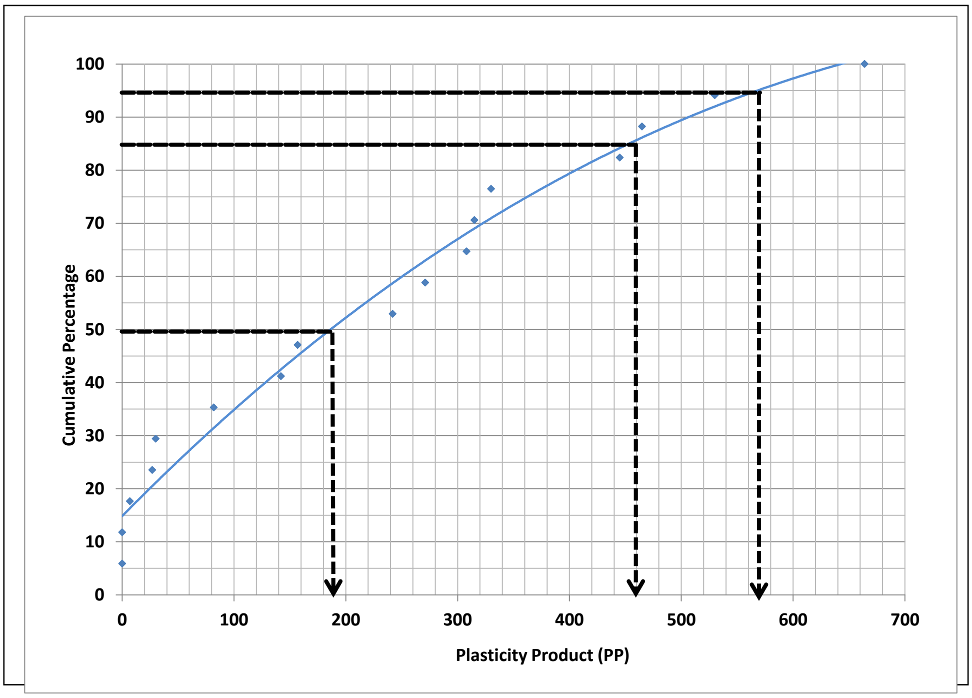


Figure 7-5 Determination of PP limits for LVRs

Table 7-2 Proposed change of plasticity ranges for low volume roads

Parameter	Standard Specifications	LVRs	Condition
Soaked CBR	≥ 80%	≥ 40%	Drainage conditions are good
In-situ CBR	≥ 120%	≥ 60%	Drainage condition are good
PI	0 - 6	0 - 14	Maximum PP = 460
		14 - 20	Maximum PP = 200
PP	0 - 90	0 - 460	
		460 - 570	Semi-arid to arid
GM	2.05 – 2.65	1.6 – 2.65	
		1.3 – 1.6	PI is less than 9 and PP is less than 200

The natural materials that were used for the construction of the bases were much finer than the standard specifications, (Figure 7-6), yet the sections still performed well. This shows that for low volume roads the grading specifications can be relaxed and envelope given in Figure 7-7 is recommended.

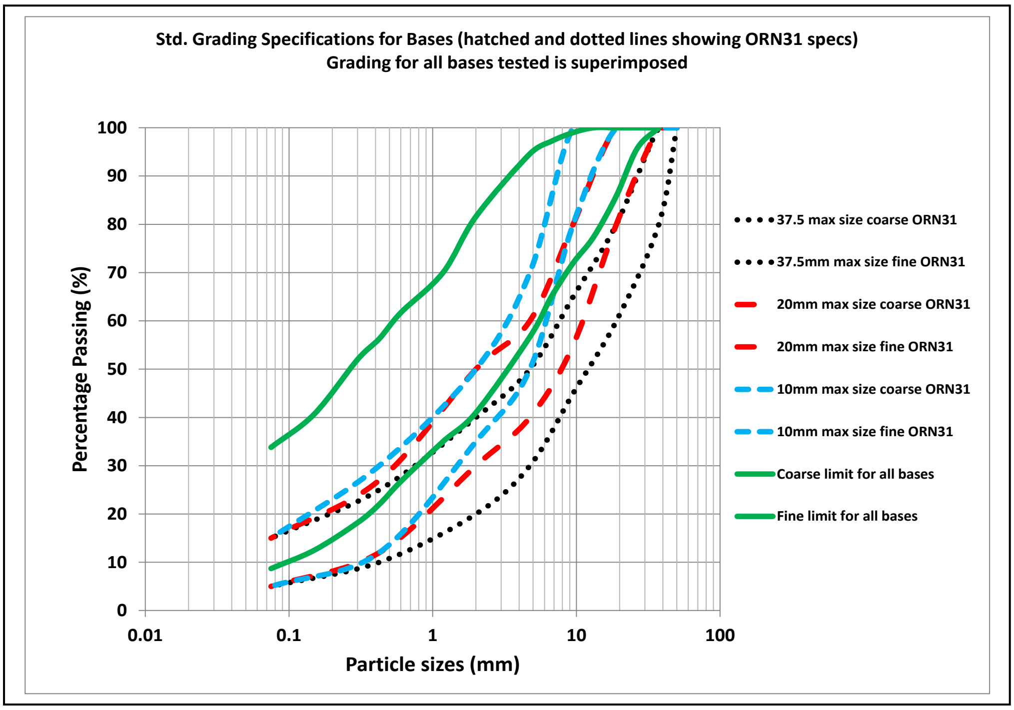


Figure 7-6 Grading of roadbases tested superimposed on TRL ORN31 specifications

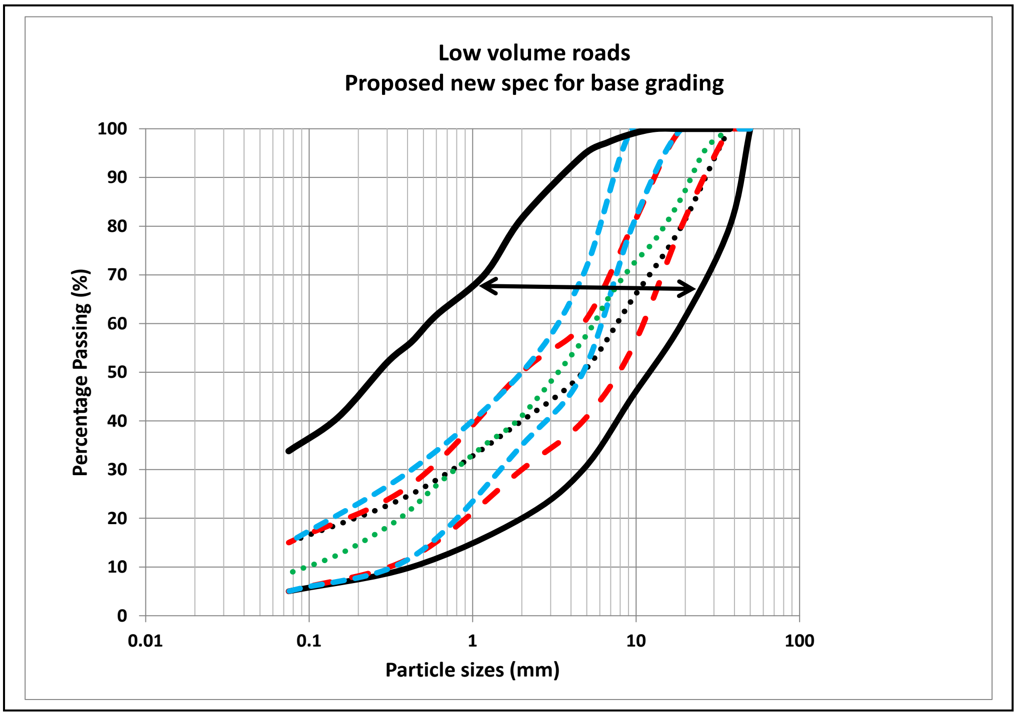


Figure 7-7 Proposed new grading specifications for bases for low volume roads

E. Extensive cracking on bituminous seals

Observations:

Cracking of the surfacing was evident on most sites but subsequent rutting caused by the weakening effect of water ingress appeared to be progressing quite slowly even when the roadbase was not of the highest quality. However the cracking progressed to potholing on several sites and then to full surface failure.

Possible explanations:

1. Surfacing had not been constructed properly. However this could not be substantiated through this study.
2. Surfacing had deteriorated due to oxidation and loss of volatiles from the binders causing the surfacing to be brittle. This in turn causes cracking and propagation resulting in failure of the surfacing. Results of bitumen tests have revealed bitumen is ageing too quickly and this is a major problem for thin bituminous surfacings in Mozambique.
3. Excessive traffic. There was no excessive traffic in terms of the expected capacities of most of the surfacings. With the exception of the sand seals, the traffic loading and the ages of the surfacing were within the expected ranges. Also, the capacities of the pavements to carry traffic were much higher that the cumulative traffic loading that they had experienced. Thus some of the surfacings had failed prematurely.

Recommendations

1. *It is important to maintain good surface and subsurface drainage. This helps to improve the durability of the surfacing. High moisture in the base causes de-bonding of the surfacing. Weaker base results in higher deflections which induce crack initiation and propagation.*
2. *Surfacings should be inspected before the start of the rainy season and all noticeable cracks should be sealed. This prevents moisture ingress and subsequent failure of the base and surfacing.*
3. *The application of a fog spray to rejuvenate the surfacing and to retard and to stop cracking. This is relatively cheap and could prolong the life of the surfacing by up to 4 years.*
4. *Timely resealing is of paramount importance. For low volume roads, resealing results in a significant increase in the life of the pavement and the surfacing (i.e. 50% or more). Reseals should be carried out before the existing surfacing reaches the end of its service life or before the cracking index exceeds 10 (the maximum cracking index is 25).*
5. *Potholes should be repaired within 48hrs in order to prevent ingress of water into the pavement, even under the nearby surfacing that has not failed. However, if the maintenance recommended above is carried out, potholes should be rare.*
6. *Single layer surfacing is discouraged because it exposes the thin bitumen layer to UV light and weather elements thus causing rapid deterioration of the bitumen leading to premature surfacing failure.*

F. Supply, handling and approval of binders

Observations:

The samples of fresh bitumen (80/100 pen) which were obtained from Mozambique and tested in UK showed that it was too hard. The penetration value was 58. The sample was obtained from a batch of bitumen which was destined for use on site. This is cause for concern in terms of the quality of the bitumen that is being supplied and the bitumen that is being used on sites.

Possible explanations:

1. It is possible that the wrong binder had been supplied by the manufacturer. This could be a problem of quality control during the manufacturing process.
2. There could be a problem with the procurement process that a harder binder could have been ordered or the order was unclear.
3. It could be the handling of the binder but this is highly unlikely. Binders could lose volatiles if they are overheated or subjected to repeated heating and cooling.

Recommendations

1. *Binders should be supplied with quality certificates produced by the supplier guaranteeing the quality of the product. This should be the same for all batches.*
2. *Binders should be tested before use:*
    a. *Penetration – the penetration values should be compared with the specifications for the particular binder. This applies to penetration grade bitumen.*
    b. *Softening point – This test distinguishes the hard from the soft binders. The higher the softening point the harder the bitumen.*
    c. *Viscosity – This is an important test and each binder has specifications and standards for comparison purposes.*
    d. *Ductility – This is a measure of how much the bitumen can stretch without breaking. It is a direct measurement of what actually transpires in-service where the thin film of binder is stretched as a result of wheel induced stresses.*
    e. *Durability – It is important to test the durability of the binder through the rolling thin film oven test (RTFOT). This simulates ageing. The results can be used to determine if the bitumen is good or bad in relation to its anticipated in-service performance.*
3. *Tests (a), (b) and (c) should be mandatory. If there is no capacity to carry out test (c) then samples should sent regularly to a laboratory that has the capacity.*
4. *Ductility test is not very common and could be left out but it is nonetheless important. It appears that the ductility test had been phased out in favour of the viscosity test but it has been reintroduced in a number of laboratories in the UK.*
5. *It is possible to carry out the rolling thin film oven test at LEM and regular tests should be carried out.*

G. HDM4 Calibration

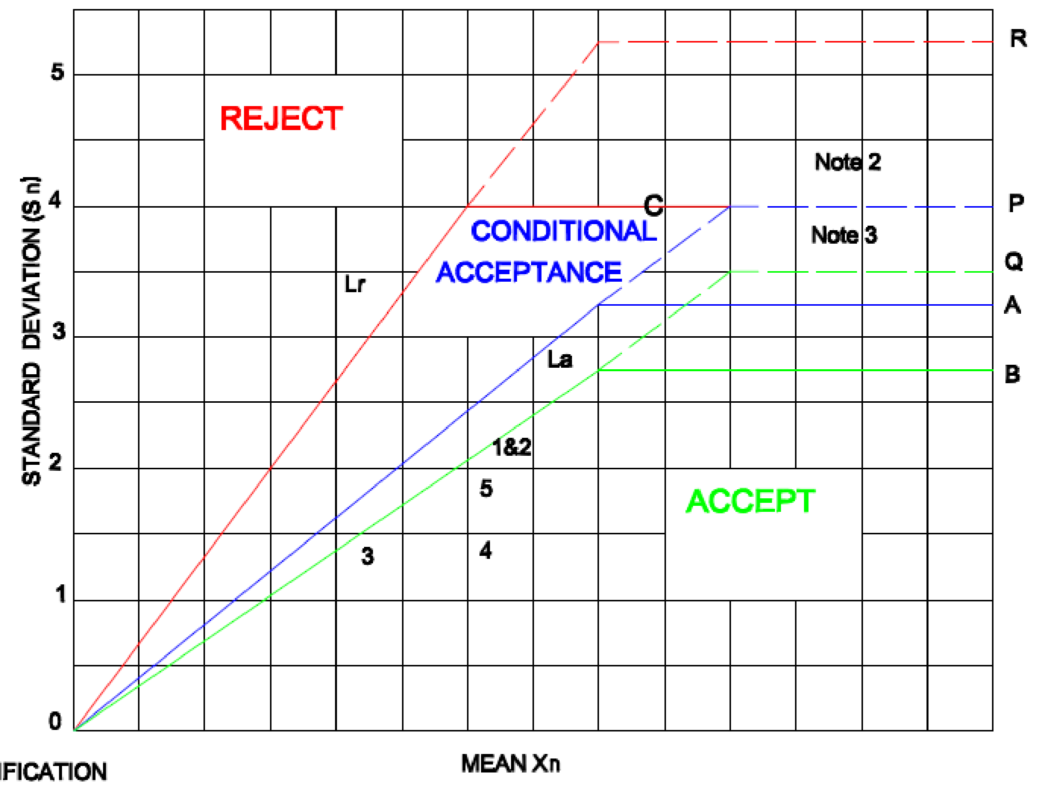
It was perceived during the planning and inception stages of the project that in would be useful to use the data and analysis results from the study to calibrate the HDM4 and this would be quite a milestone for low volume roads. However, after all the field and laboratory investigations the results were unexpected.

1. Technically some of the low quality materials used for the construction of the road bases judging by the laboratory and field test results showed that these materials should not have been used in the first place e.g. materials with high plasticity and low GM. The study showed that they actually performed well.
2. Some of the bases that were thought to be stabilised showed no signs of stabilisation based on the specialised tests and still the pavements performed well.
3. The traffic loading was high and in some cases too high for low volume road standards yet the pavements performed well.
4. The bitumen tests results produced answers but questions too which are subject to further investigations. The binders were too hard and in-service deterioration of the binders was notably too high and this was affecting the performance of the surfacings.

Recommendation:

The issues stated above were considered and after going through the data it was concluded that it would be premature and perhaps inappropriate to recalibrate the HDM4 at this stage because this would result in drastic changes. Such analysis would require more time and it is recommended that this exercise is reconsidered in future undertakings where there could be more time for in-depth analysis.

JUDGEMENT PLAN



SPECIFICATION LIMIT[(1)] Ls

	87	88	89	90	91	92	93	94
2&3	89	90	91	92	93	94	95	96
1	91	92	93	94	95	96	97	98
5	93	94	95	96	97	98	99	100
4	95	96	97	98	99	100	101	102
	97	98	99	100	101	102	103	104
	99	100	101	102	103	104	105	106

VARIABLE SCALE FOR RELEVANT L_S

A = Initial submission limit $L_s \geq 93$ P = Initial submission limit $L_s \leq 91$

B = Resubmission limit $L_s \geq 93$ Q = Resubmission limit $L_s \leq 91$

C = Rejection limit $L_s \geq 93$ R = Rejction limit $L_s \leq 91$

NOTE (1) Circle relevant L_s and put reference number of lot next to L_s

(2) Reject for L_s ≥ 93 Conditional for L_s ≤ 91

(3) Conditional for L_s ≥ 93 Accept for L_s ≤ 91

Figure 7-8 Compaction judgement chart

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