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*Performance of Low Volume
Sealed Roads:
Results and Recommendations from
Studies in Southern Africa*



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Results and Recommendations from Studies
in Southern Africa**

by C S Gourley and P A K Greening

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PERFORMANCE OF LOW VOLUME SEALED ROADS:

Results and Recommendations from Studies in Southern Africa

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ABBREVIATIONS

AADT	Annual Average Daily Traffic
AASHO	American Association of State Highway Officials
AASHTO	American Association of State Highway and Transportation Officials
ARRB	Australian Road Research Board
BRDM	Botswana Road Design Manual
BS	British Standard
CBR	California Bearing Ratio
CIRIA	Construction Industry Research and Information Association
CL	Centre-Line
CS	Calcified Sand
CSIR	Council for Scientific and Industrial Research
DCP	Dynamic Cone Penetrometer
DFID	Department for International Development
esa	Equivalent standard axles
FWD	Falling Weight Deflectometer
GM	Grading Modulus
HC	Hardpan Calcrete
HDM	Highway Design and Maintenance Standards Model
IWT	Inner Wheel-Track
MoTE	Ministry of Transport and Energy
NC	Nodular Calcrete
NITRR	National Institute for Transport and Road Research
NRRL	Norwegian Road Research Laboratory
OMC	Optimum Moisture Content
ORN	Overseas Road Note
OWT	Outer Wheel-Track
PC	Powder Calcrete
PI	Plasticity Index
RMI	Road Maintenance Initiative
SADC	Southern African Development Community
SATCC	Southern African Transport and Communications Commission
SFRDP	Secondary and Feeder Road Development Programme
SG3,5,9,15	Subgrade design CBR class
SIDA	Swedish International Development Agency
SN	Structural Number
SNC	Modified Structural Number
SPP	Sulphonated Petroleum Products
TRB	Transportation Research Board
TRL	Transport Research Laboratory

Executive Summary

Background and aims

The overall aim of the research programme has been to investigate the use of natural gravels for roadbases and to recommend innovative approaches for their use in a way that is cost-effective and environmentally sensitive. Sections of road were selected on the existing road networks in Botswana, Malawi and Zimbabwe, and these were tested and monitored to enable designs to be evaluated. The research focused on measuring how road pavements performed with time and traffic, and in different climatic conditions. It also identified features which need to be included in the road design to minimise risk.

Conclusions and Recommendations

Summary of conclusions

The main conclusions and recommendations from this project are:

- The minimum standard of 80 per cent soaked CBR for natural gravel roadbases is inappropriately high for many low volume sealed roads, which form the majority of new surfaced road projects in the region. New limits are recommended depending on traffic, materials and climate and these are presented as a series of design charts.
- Field/optimum moisture content ratios in the outer wheel-track in the wettest condition ranged for between 0.5 to 0.7 in the driest areas to between 1.0 to 1.2 in the wettest areas. Roadbases constructed with materials of high PI (plasticity index) were less sensitive to moisture ingress than was envisaged at the outset of the project. Moisture ingress was greatest on bases constructed with non-plastic materials.
- The grading envelopes for natural gravel roadbases are too narrow. Alternative (wider) envelopes are recommended for relatively lightly trafficked roads.
- New pavement design tables have been produced which enable the strong subgrades prevalent in many areas in the region to be exploited.
- Traffic below 300,000 to 500,000 esa was not a significant factor on pavement deterioration. Many sections, especially those on the trunk road network, had been subjected to a high degree of overloading but deformation (rutting) was low even on roadbases with PI of 18. New limits for PI are recommended.
- Drainage was a significant factor on performance even in dry areas. A minimum crown height of 0.75 metre is recommended.
- Sealed shoulders provide a structural and maintenance benefit and should be considered even on low volume roads if this enables local materials to be exploited and there is an overall whole-life benefit. A method is suggested for determining the optimum width for sealing shoulders but the evidence from this study suggests a minimum width of one metre.

- Included in this study were sections on a road with base materials of gravel wearing course standard and subsequently sealed. There are other such examples in the region and these roads have generally performed exceptionally well. (This is further evidence of the need for a relaxation in design standards). In recognition of this practice, a design class for sealing an existing gravel road is included in the design chart for traffic up to 10,000 esa.

Classification of engineering properties of natural gravels

Roadbase

- 1) The major groups of natural gravel roadbase materials used in Zimbabwe, Malawi and Botswana are quartzitic gravels, weathered rocks, lateritic gravels, and calcareous gravels and sands. The study has shown that all of these can be successfully used in the upper pavement layers of low volume roads.
- 2) A common feature of the specifications in the region are for natural gravel roadbase materials to meet strict compliance criteria on particle size distribution, to have plasticity indices less than 6, and soaked CBRs of greater than 80 per cent at 98 per cent BS 4.5kg rammer or modified AASHTO compaction. In many parts of the region one of the biggest problems for the engineer is the location of materials which meet these specifications. Many natural gravels are often excluded from use because they fail to meet at least one of these criteria.
- 3) Where materials meeting the specification are not available locally, the alternatives are to:
 - Import suitable materials over long distances
 - Improve the materials by addition of stabilising agents such as lime or cement
 - Utilise sources of crushed stone if these are available
- 4) Most of the materials failed to meet the grading specifications for materials used in roadbases. Plasticity requirements were less of a problem, except in the N<2 climatic areas.

Subgrades

- 5) The subgrade materials encountered in the project were generally S5 and S6 soils, which were very good for road construction purposes.
- 6) Where poor subgrades are prevalent, judicious and selective stabilisation with lime may be warranted by the savings that can be made in pavement material thickness and quality.

Performance of natural gravel roadbases

Strength

- 7) Roadbase and sub-base strengths were generally above their design values. In the few cases where the in situ strengths were below the design value, the performance of the road was not adversely affected.
- 8) In situ structural numbers at the wettest time of the year, when the pavement is in its weakest condition, were in most cases higher than the design structural number.
- 9) The strengths measured in situ were very dependent on the compacted density. The soaked CBR was taken at the same compaction level as determined by the in situ density measurements to allow comparisons to be made to the design values.

Moisture in the roadbase

- 10) The seasonal effects of edge wetting of the roadbase, sub-base and subgrade were obvious in most situations. However, it was discovered that the ingress of moisture was less serious where more plastic materials were used in the roadbase. Where low plasticity materials were used in the roadbase, ingress of moisture could extend over substantial widths of the carriageway.
- 11) Where materials were poorly compacted, these exhibited a higher risk of wetting.
- 12) The field/optimum moisture ratios in the outer wheel-track and at the wettest time of year for the roadbases were different in the different climatic zones. In $N < 2$ areas, the ratio ranged from 1.0 to 1.2; in $N = 2-4$ areas, the ratio was 0.6 to 0.8; and, in $N > 4$ zones, it ranged between 0.5 to 0.7.

Moisture in the subgrade

- 13) The field/optimum moisture ratios in the outer wheel-track at the wettest time of year for the subgrade were different in the different climatic zones. In $N < 2$ areas, the ratio ranged from 1 to 1.5; in $N = 2-4$ areas, the ratio was 0.75 to 1.25; and, in $N > 4$ zones, it ranged between 0.5 to 0.7. The drainage of the road had an equally strong influence on the moisture condition and the strength, with better drained sites exhibiting higher strengths.
- 14) It proved difficult to track and explain moisture changes on roads less than about four years old. This was probably an effect of the road settling down to an equilibrium after construction.

Road performance parameters

- 15) Performance data indicated that traffic-induced permanent deformation is negligible until the traffic level gets to about 300,000 to 500,000 esa and, even at this level of traffic, rutting was generally less than 12mm. This was consistent with the magnitude of the structural numbers. There was also little evidence of a relationship between the level of roughness or the development of pavement cracking to the level of traffic.
- 16) Cracking and roughness could be explained better by the drainage conditions at the site, as measured by the height of the crown of the road above the invert of the ditch, and the distance of the outer wheel-track from the edge of the sealed area. Longitudinal cracks were prevalent where the drainage was poor. It was also evident that cracking increased with the age of the bituminous seal. In most cases, the severity of the cracking was less than 10 per cent, and crack sealing or re-sealing would arrest the problem. Where a higher degree of cracking was present, some improvement would also be needed to the drainage.
- 17) The central deflection was much lower than expected given the nature of the pavement materials. High deflections were again explained by poor drainage conditions or low strength poorly compacted materials at depth in the structure.
- 18) The road environment can be considered to encompass both local climate (rainfall, temperature range and evaporation), drainage (effectiveness of drains, carriageway cross fall and the crown height) and topographic and sub-soil conditions. This was of major importance in determining the degree and severity of the cracking and roughness on these low volume roads.
- 19) Another important variable controlling the performance of roads with marginal bases is the durability of the bituminous seal. A number of thin sealing technologies have been highlighted which are appropriate to these types of roads. All thin seals are subject to the effect of ageing and embrittlement of the binder. They are also susceptible to the movements in the pavement caused by wetting and shrinkage of the soils in the lower parts of the structure. Crack sealing is usually adequate to protect the structure when double seals have been used. If single seals are used at construction, it is important to re-seal once the road environment has stabilised the structure.

Sealed shoulders

- 20) The study has shown that the use of sealed shoulders gives a structural benefit by maintaining a drier environment under the running surface. The provision of a sealed shoulder decreases the risk of using weaker materials in the upper pavement layers.
- 21) The addition of narrow (<750mm) sealed shoulders had only a marginal affect on the strength of the roadbase strengths in the outer wheel-track. Their use had no impact on subgrade strengths in the wheel-tracks.
- 22) The outer wheel-track moisture and strength conditions will remain fairly stable provided that the shoulders of the road are sealed to a sufficient width such that the outer wheel-track is more than 1.5 metres from the edge of the sealed area, and the drainage is ensured by maintaining the crown height greater than one metre above the ditch. A one metre

sealed shoulders is therefore the minimum effective width. In some cases moisture can still ingress to this level, and wider shoulders are then required.

- 23) The need for sealed shoulders on existing roads can be determined by measuring the strength variation across the pavement in the wet and dry seasons, using a DCP, to assess the effective design width. In this way an estimate can be made of the likely ingress and the shoulder width can be designed accordingly. Shoulder sealing is now being carried out on many trunk roads in the region and the lessons should be learned for secondary road design.

Revisions to specifications and design criteria

Structural design charts

- 24) The results from this study have been used to develop a series of structural design charts for low volume roads. Guidelines for the selection of natural gravel roadbases have also been developed. The charts and recommendations are for lightly trafficked roads in Southern Africa. As with all guides, the design charts are not prescriptive and, if other local evidence and experience are available to the engineer, they should be modified or adjusted accordingly.
- 25) The design charts are applicable to two climatic zones: $N < 4$ and $N > 4$. The charts allow modifications to the materials selection which depend on the width of cross-section selected. The options provided in a climate where $N > 4$, and with wide cross-sections enables good use to be made of marginal quality materials.
- 26) Separate guidelines for the selection of lateritic and calcrete roadbases have been included, as these two groups of materials consistently exhibit better than expected performance.
- 27) Standards need not be relaxed over the whole road length, but there may be sections where such changes in approach are justified. Generally, the same design standard is applied to the whole length (or very long sections) of project roads. Although it would be impractical to apply too many designs within a project, considerable cost savings could be made from adopting a more flexible approach.

Mitigating risks

- 28) The design charts include an option for sealing an existing gravel road. This approach has been used successfully in some countries in the region as a low-cost option for upgrading gravel roads and analysis using HDM3 has shown that the optimum traffic level is less than 40 vehicles per day.
- 29) The risk of premature failure can be expected to increase as the number of the main factors influencing road performance (materials quality, construction standard, environment, maintenance, overloading) are relaxed together. In this study, many of the roads investigated performed well despite overloading, poor maintenance and the use of materials considered to be marginal or sub-standard. This can be interpreted as indicating a considerable degree of over design even in road pavements constructed with

sub-standard materials, Control of these factors will ensure that the maximum benefits are obtained from the design standard adopted.

- 30) Careful consideration needs to be given to determining the traffic growth rate. Projecting traffic growth and assigning accurate equivalence factors to the traffic is crucial if economic designs are to be achieved. Using unrealistically high growth rates or equivalence factors reduces the level of risk for the engineer but results in conservative pavement designs which can ultimately negate the feasibility of projects.
- 31) Where the road carries a larger proportion of heavy traffic, or where poor drainage conditions are unavoidable, the risk can be reduced by adjusting the design class upwards by one traffic class.

Concluding remarks and outstanding issues

- The actual savings in construction costs from implementation of the results will depend on the design adopted but substitution of crushed gravel or crushed rock by natural gravel will, itself reduce the costs of the roadbase by a factor between 3 and 8.
- Greater awareness of the importance and funding for maintenance has improved the climate for the application of more appropriate designs.
- At workshops held to discuss the results of the research, participants acknowledged the need for regionally derived standards and endorsed the recommendations.
- Recommendations from the workshops included a strong message, particularly by consultants and contractors, that the revised specifications and standards should be incorporated into country documents and that manuals should be treated as guides only. The research results need to be disseminated more widely in the region.
- The materials testing in this study was generally carried out to British Standards(BS) test methods. Various test methods are used throughout the region and test results need to be interpreted appropriately when applying the recommendations of this and other regional documents.
- The results of this study indicate the importance of drainage. There is a need to extend the work of this study to sections in cut to determine the most appropriate and cost-effective measures to prevent moisture ingress in moisture-sensitive materials.
- The cost savings from this study will accrue from a relaxation of pavement design standards. Similar and possibly much greater savings in some circumstances could be obtained from a relaxation in geometric standards for sealed roads carrying low volumes of traffic. Little data is currently available and additional research is required to quantify the impact of adopting reduced geometric standards for rural roads in the SADC region.

- There is also a need to improve methods of identifying and quantifying the benefits from the provision of low-cost sealed roads including those from a reduction in dust emissions and improved wet season passability.

1. Introduction

1.1 Background

Between 1988 and 1991, TRL held discussions with road authorities in the countries of the Southern African Development Community (SADC) to identify subject areas which would benefit from research and technology development. The outcome of these discussions led to proposals for a regional programme of research on highway engineering materials being submitted to the Department for International Development (DFID) and the Swedish International Development Agency (SIDA). As a result of these proposals, DFID agreed to fund a three-year programme of work which was also supported by a generous contribution from SIDA. A two man TRL team was established in Zimbabwe to work in collaboration with government departments in the region, other regional organisations, academic institutions and consulting engineers.

In the period since the 1960s, there has been a concerted effort to provide a national network of sealed roads in countries of the region. The design criteria for these trunk roads were based generally on those adopted in Europe or the USA, although some modifications were often incorporated based on the results of research in the region carried out, for example, by the Transport Research Laboratory, from the United Kingdom (TRL 1993) and the South African National Institute for Transport and Road Research (NITRR 1980). The emphasis of work in the region is now towards the maintenance and rehabilitation of this trunk road network, but the provision of a secondary and feeder network to develop the rural areas has also been given high priority. Although the design methods adopted may remain appropriate for trunk roads, a more cost-effective approach is needed for low volume sealed roads if these are to be economically feasible.

There is now an increasing amount of evidence to suggest that more use can be made of natural gravels for the construction of low volume roads which can reduce the cost of their provision, rehabilitation and maintenance. The overall aim of the research programme has therefore been to investigate the use of natural gravels for roadbases and to recommend innovative approaches for their use in a way that is cost-effective and environmentally sensitive.

1.2 Objectives

The research applies to 'low volume' roads. These are roads typically carrying less than 200 vehicles per day and which over a 20 year period, even with high growth rates, are unlikely to reach design traffic loadings exceeding one million cumulative equivalent standard axles (esa). Current designs produced by TRL (1993) and NITRR (1980), and the draft SATCC regional pavement design guide (SATCC 1998) do not cater for design traffic loadings of less than 300,000 esa. The aim was therefore to provide design guidance for a range of traffic levels not currently covered by these guides.

Specific objectives set out at the project inception were to:

- a) Recommend a revised approach to the classification of the engineering properties of natural gravels
- b) Relate the engineering properties of natural gravels to their performance as roadbases, giving due consideration to the level of traffic and the influence of climate
- c) Identify revisions to current materials specifications and materials design criteria to achieve improved utilisation of natural gravel in roadbases

1.3 Regional emphasis

The research programme was also supported by the Southern African Transport and Communications Commission (SATCC). The need for the work to have a regional impact was recognised. There is also a reticence by many national engineering organisations, both in government and the private sector, to accept and apply the results of research conducted outside their borders. The involvement of SATCC raised awareness of the research, and increased opportunities for the transfer of the technology developed. This also gave member countries in the region an opportunity to influence the project objectives and methodology. Project progress and results were disseminated to representatives of the Southern Africa Development Community (SADC) countries at SATCC meetings, regional conferences and workshops.

There are also technical benefits of operating regionally. Collaboration with different road administrations enabled the project team to achieve a better appreciation of the range of traditional methods as well as gaining an understanding of the benefits and constraints of the local working environment. By operating in the different countries it has been possible to collate experience from the region and thus extend the conditions to which the results of the research can be applied. Although regional application was not a project objective, the importance of sharing and using the available information and knowledge should be an achievable future aim for the region.

1.4 General approach

Research on the performance of non-standard materials in road pavements normally requires the design of test sections on which construction and performance can be closely monitored. However, this approach requires access to road construction projects and a long monitoring period to collect time-series performance data. The time frame available for this project was relatively short in the context of road performance, so an alternative strategy had to be adopted for the experimental design. Sections of road were selected on the existing road networks in the region and these were tested and monitored to enable designs to be evaluated.

The research focused on measuring how road pavements performed with time and traffic, and in different climatic conditions. It also aimed to identify those features which need to be included in the road design to minimise any additional risk.

1.5 Report structure

This report presents a brief description of the test sections and the monitoring techniques used to collect pavement performance data. Local and other pavement design procedures are presented and discussed. The data collected from the field have been summarised and is supplemented by laboratory investigations of the pavement materials. The methodology for the data analysis is given and comments and recommendations are made on the appropriate use of marginal pavement materials and the introduction of a new structural and materials design procedure for low volume roads in the region. An analysis of the performance of the test sections to date is given and the influence of seasonal moisture variations on the strength of the pavements has been assessed.

2. Methodology

2.1 Site selection

2.1.1 Basis of selection

Base specifications for natural gravels usually set limits on grading, plasticity index, and strength determined by the soaked California Bearing Ratio (CBR) test. The selection of test sections involved a desk study in Zimbabwe, Malawi, and Botswana to identify sites for monitoring. Advice was taken from roads departments staff in the countries, and project documents were consulted to determine which sections of road had been constructed using roadbase materials which did not meet the accepted specifications. This selection process placed considerable reliance on the accuracy of the as-built records and, in some cases, subsequent sampling and laboratory testing of the materials revealed significant differences to results recorded at the time of construction. However, using the test results, it was possible to select sections which were outside the current specifications for roadbase materials. Some sections with crushed stone roadbase, or a natural gravel meeting specifications, were selected as control sections. Existing trial and experimental sections from other programmes were incorporated where available.

Natural gravel materials occurring in the region, which can be used for roadbase construction, fall essentially into four main groups: quartzitic and lateritic gravels; gravels resulting from weathering of rocks, including weathered granitic and basaltic gravels; calcretes and other pedogenic materials; and sands. These have different engineering properties, and most were represented in the study.

It was possible to select sites with a reasonably wide regional spread covering a range of the climatic conditions found in Southern Africa. The region has been mapped using the Weinert (1980) climatic N-value and this information was used, in addition to other local climatic records, to categorise and classify the sites on this basis.

Other information was all also considered during the final site selection process. This included the pavement structural design, geometry, age of the road and surfacing, condition of the surfacing, prevailing drainage, standard of maintenance, construction quality, type and volume of traffic.

2.1.2 Sites selected

Test sections on the regional trunk and secondary road networks were then established in Malawi, Zimbabwe and Botswana. Site characteristics are listed in Appendix A. Details are given in Table A-1 to Table A-3, which also note the roadbase material type, design traffic, grading, plasticity index, soaked CBR value, and climatic area. Sites were referenced using the first and last letters of the road name, and the section number along the road in the direction of increasing chainage. For example the second section on the Nyanga to Ruangwe road has the reference 'NARE2'. Sections were marked out physically on each of the sites.

During site selection, care was taken to select sites which were representative of the longer sections constructed to the same design and with similar materials. (This was confirmed by a survey of sites in Malawi carried out by a senior member of TRL staff.) Where possible,

sections with unrepaired potholes were avoided because part of the study involved investigating moisture ingress from sources other than through permeable surfacings. However, even this could not be avoided completely as some of the oldest roads in the study had not been re-surfaced since construction and the surfacings were cracked due to age hardening of the bitumen.

The road sections selected in Malawi were exposed to the wettest climatic and drainage conditions. They also had some of the highest traffic levels and poorest quality roadbase materials. The driest climatic area was in Botswana, where it was possible to assess a range of poor quality roadbase materials, including sands. The materials in Zimbabwe were generally good, although a number of roads were incorporated where pavement standards and specifications had been relaxed. Most of the sites in Zimbabwe were on sealed secondary roads, which were often less than five years old. Here the traffic was much lighter than on the sections selected in the other countries. No sites were located on trunk roads in Zimbabwe because, in nearly all cases, the gravels used had been modified with about two per cent cement.

A comparison of the properties of the roadbase materials with the guidelines set out in *Overseas Road Note 31 (ORN31)* (TRL 1993) are given in Table A-4. The grading curve specification refers to recommendations in Table 6.5 of *ORN31*, except for the crushed stone bases which have been compared with Table 6.2. The plasticity index (Ip) should be less than 6, and the soaked CBR at 98 per cent BS heavy 4.5kg rammer or equivalent compaction should be greater than 80 per cent. It can be seen from the table that, of the 55 sections with natural gravel roadbases, only six meet all of the *ORN31* recommendations. Most sections are outside the grading requirements, and half were also outside requirements for plasticity and strength.

2.2 Site characteristics

General site information

Table A-5 to Table A-7 give the general characteristics of the Zimbabwe, Malawi and Botswana sites. Information on the date of construction, the surfacing type and re-sealing history was obtained from as-built and maintenance records. Widths of lanes, width of any sealed shoulders, longitudinal gradient and carriageway cross-fall were measured on site. The height of the crown of the road above the invert of the drainage ditch was also measured on site and recorded as 'crown height'. Where no drainage ditch exists because the land falls away from the road, this has been designated as 'free draining'.

Traffic

A summary of the annual average daily traffic, cumulative number of equivalent standard axles, and the percentages of heavy traffic carried by the road to March 1997 is given in Table A-8. These data were derived from roads department records, supplemented by further site collection.

Sampling and testing

A programme of sampling was carried out to confirm the site properties of materials. Test pits were located just outside the test sections, and were also used to confirm the thicknesses of the pavement layers. The materials were tested in the laboratories of the participating road departments. The following properties were recorded for the roadbase and sub-base materials:

- Grading (BS 1377 1990)

- Plasticity Index (Ip) (BS 1377 1990)
- Soaked CBR at 98 per cent BS 4.5kg rammer compaction for roadbase and 95 per cent BS 4.5kg rammer compaction for sub-base (BS 1377 1990)

Samples were also taken for testing subgrade design CBR on samples compacted at 100 per cent BS 2.5kg rammer compaction and optimum moisture content.

Roadbase

The gradings, plasticity and soaked CBR design values for the roadbase materials are summarised in Table A-9 to Table A-11 for the test sections in Zimbabwe, Malawi and Botswana, respectively.

Sub-base

The gradings, plasticity and soaked CBR design values for the sub-base materials are summarised in Table A-12 for the sections in Zimbabwe and Malawi.

Subgrade

The gradings, plasticity and soaked CBR design values for the sub-grade materials are summarised in Table A-13 to Table A-15 for the sections in Zimbabwe, Malawi and Botswana, respectively.

2.3 Monitoring programme

2.3.1 Basis of the design

The monitoring programme was designed to provide data on a range of materials and conditions being investigated by the study. The main variables were the material properties, moisture profile, strength profile, and shoulder width and type (sealed or unsealed). The duration and degree of moisture change, and the effect of this on pavement strength, requires frequent monitoring. Moisture and strength information was also required to investigate the impact of sealed shoulders. It was particularly important to monitor the test sections at the end of each of the dry seasons, in the period October-November, and at the end of the wet season in the period March-April.

Some of the selected roads were relatively old in terms of their design life. It was therefore anticipated that, after such a long period of trafficking, it would be possible to draw firm conclusions on the performance of the pavement materials, and particularly on that of the roadbases. The main variables investigated were traffic, pavement age, material properties and pavement condition. This approach also meant that the performance data consisted essentially of just two points: the measurements made, or implied, at the time of construction and the current measurements. Some small changes were observed during the monitoring period mainly on the older sections with poor surfacings.

Pavement condition was monitored in terms of moisture, strength (in situ CBR measured with a dynamic cone penetrometer), density, riding quality (roughness), deformation (rutting) and deflection at the end of each wet and dry seasons as described above. Visual inspections were also carried out to assess surface condition. The layout of a typical test section with measurement positions is shown in Figure 2-1.

2.3.2 In situ strength

The strength of the pavement layers on the test sections was assessed using a dynamic cone penetrometer (DCP). The DCP is an instrument designed for the rapid in situ measurement of the strength of road pavements constructed with unbound materials. It consists of a small steel cone mounted on a rod connected to an anvil. The cone is driven vertically into the road using the constant force provided by a weight falling through a fixed distance onto the anvil. The weight is guided by a rod connected above the anvil. The distance penetrated by the cone for each blow is recorded. Continuous assessments can be made to a depth of 800mm. Where pavement layers have different strengths, the boundaries can be identified and the strengths of the individual layers can be found.

Correlations between measurements with the DCP and CBR have been established by several authors, so that results can be interpreted for pavement design purposes in terms of CBR. A typical test takes only a few minutes, and the instrument provides a very efficient method of obtaining sub-surface information.

Cross-sections within each test section were tested with the DCP. The number of measurement positions chosen depended on the width of the road, but always included the outer and inner wheel-tracks (OWT and IWT, respectively) and the centre-line (CL). Further measurement positions were concentrated between the outer wheel-track and the shoulder. The longitudinal measurement position was relocated about one metre further along the road in each successive survey. Thus, over time, there were only relatively small variations in the measurement position, and results from successive surveys are comparable. The transverse measurements at these locations were always made at the same offset positions.

The data from the DCP surveys were analysed using the computer program developed by TRL and described in *Overseas Road Note 8* (TRRL 1990). The data were summarised to establish the strengths within the roadbase, sub-base and the subgrade.

2.3.3 Moisture and density

Moisture ingress is the main factor affecting the strength and performance of many road building materials, but naturally occurring gravels are likely to be particularly moisture-susceptible because of their clay content. The rate and degree of moisture penetration are, therefore, important measurements to define the limits of use of these materials. Sealed shoulders provide a drier environment in the pavement structure, so may also be particularly

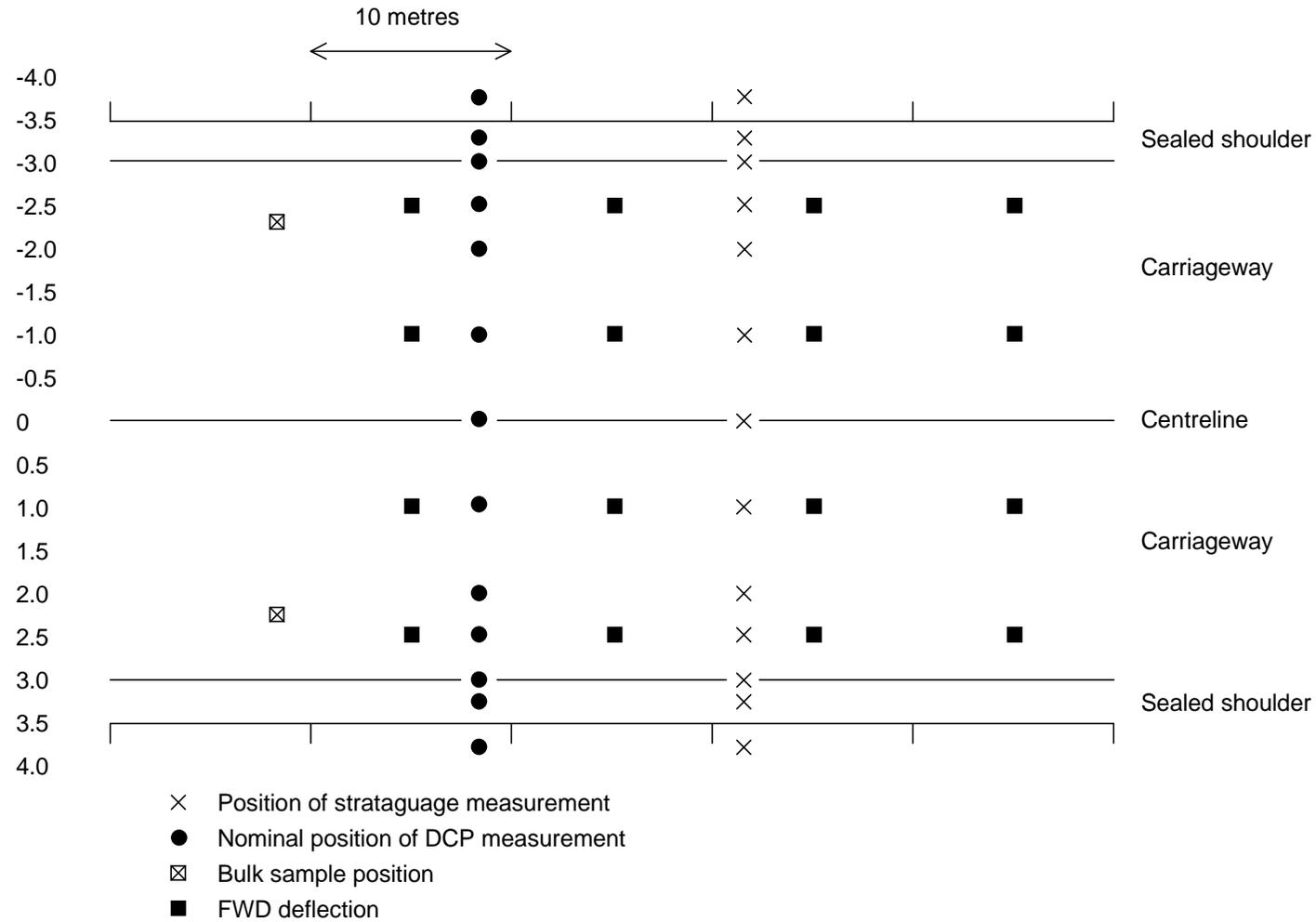


Figure 2-1 Typical measurement positions for a road section with 3m carriageway and 0.5m sealed shoulders

influential in the performance of natural materials. Additional sections with different widths of sealed shoulders were included so that recommendations could be made on the effects of shoulder sealing on the pavement structure.

Moisture and density conditions within the road structure were determined using a nuclear gauge. The gauge type was a CPN Stratagauge which measures the soil moisture and density horizontally between two probes. The probes are inserted vertically into the road via two pre-formed holes, lined with thin aluminium access tubes. This enables return to the same measurement position. The gauge can measure at 50mm intervals to a depth of 600mm. Measurement offsets were the same as those selected for the DCP.

The gauge uses two types of radiation source. Gamma radiation measures wet density and neutron radiation measures soil moisture density. The radiation emitted from these sources passes through the soil to their individual detectors and is counted. The detectors count the radiation received within the pre-set count period and adjust it to counts per minute, which are the units of data collected on site. These data were entered into a spreadsheet. Each stratagauge has a unique calibration based on a standard count taken regularly on a calibration block. This calibration enables counts per minute to be converted to wet density or moisture density. Moisture content (determined from ratio of weight of water to the weight of soil solids given as a percentage) and the dry density are calculated using:

$$\text{Wet density} - \text{moisture density} = \text{dry density (kg/m}^3\text{)}$$

$$\begin{aligned} & [\text{Moisture density}/(\text{wet density} - \text{moisture density})] \times 100 \\ & = \text{moisture content (per cent)} \end{aligned}$$

Calibration checks on each of the test sections were carried out using standard sand replacement techniques to increase the reliability of the density and moisture data collected. In some cases, it was necessary to apply corrections to the dry density and moisture content data.

The measurements were taken twice a year at the same time as the DCP tests. The test positions were at the same transverse position as the DCP test, but were offset longitudinally by approximately 10 metres.

2.3.4 Deflection

Deflection tests were carried out using a KUAB 150 falling weight deflectometer (FWD). The FWD operates by dropping a weight from a known height onto a circular plate through a spring system. This applies a load to the pavement which is measured using a load cell. The deflection of the pavement in response to the load is recorded through a series of six transducers placed along a radius from the load centre. The 50kN load used by the FWD is roughly equivalent to the loading applied during a Benkelman beam deflection test. A series of both Benkelman beam and FWD tests were carried out to determine the relationship between them. The resulting equation developed to relate the central deflections is:-

$$\text{Benkelman beam central deflection} = 0.71 \times (\text{FWD central deflection})$$

A high deflection is indicative of weakening of one or more pavement layers or the subgrade or the presence of a weak layer at construction, so deflection data were used to assess the overall

stiffness or strength of the pavement structure. The tests were carried out at 10 metre intervals along the test sections in the outer and inner wheel-tracks. The FWD tests were undertaken twice a year at the end of the wet and dry seasons. This frequency of testing was adopted in order to monitor changes in deflection with moisture movements within the pavement structure. The central deflection was used to give an overall measurement of strength of the pavement structure.

2.3.5 Rutting

Deformation in terms of rutting was measured in each of the wheel-tracks using a 2-metre straight edge and wedge. The maximum rut depth in the outer and inner wheel-tracks was recorded longitudinally at one metre intervals for 30 to 40 metres along the test sections. The median, mean, and 80th and 90th percentile rut depths were then calculated.

2.3.6 Roughness

Roughness measurements were recorded using a MERLIN device. The instrument, which is cheap, simple to operate and reliable is described by Cundill (1991). The roughness measured by the instrument, can be directly related to the International Roughness Index (IRI) through calibration equations. Measurements were taken in each of the wheel-tracks across the carriageway. At least two surveys were carried out on each test section.

2.3.7 Visual condition

Visual surveys were carried out to record changes in the pavement condition which might not otherwise be detected. Visual survey data can be used to diagnose mechanisms of pavement deterioration and, for example, whether deformation precedes the onset of cracking or vice versa. This enables better evaluation of the deterioration mechanisms to be made.

Each lane of the test section was divided up into 10 metre long blocks, marked on the road, to facilitate collection of the visual condition data. The visual data on type, width and extent of cracking, was collected using a standard proforma for each block in both carriageways. Each individual block was divided into three sectors covering the outer and inner wheel-track area, and the area between the wheel-tracks. The blocks and sectors made up a grid on the road surface, enabling data collected to be referenced in terms of its location on the carriageway. Other defects, such as pot-holing, edge wear, and patching were also logged. The percentage of cracked area was calculated for each block, but excluded the sections where stratagauge access holes were located or DCP measurements were made.

2.3.8 Traffic and axle loading

Cumulative equivalent standard axle loading was derived for each test section, based on 12-hour and 24-hour classified counts of traffic and axle loading data collected from the roads departments during the project. These data were supplemented and verified by collection of further data at the sites.

The impact of traffic loading on an individual road will be influenced by the road width. If the carriageway is less than about six metres wide, then larger vehicles tend to drift towards the centre of the road rather than driving in their lane. These phenomenon needs to be taken into account when analysing the data.

2.3.9 Rainfall and climate

Data on rainfall and climate were collected from various sources, including meteorological offices and agricultural stations that were close to the research sites. Existing climatological maps for the region using climatic indices, such as the Thornthwaite 'Im' value and the Weinert N-value, were also used. Both of these indices have been found to relate well to the moisture conditions in pavements in the region, and relate well to the location and performance of materials. The wide geographic distribution of the sites covered a range of N-values, from less than 2 to greater than 5. Typically, N-values of less than 4 imply a climate that is seasonally wet; whereas N-values of greater than 4 imply a climate that is arid, semi-arid, or dry.

The rainfall in the region for the 1995/96 season was relatively poor, with many areas experiencing drought. However, above average rainfall was recorded throughout most of the region in 1996/97.

3. Results of monitoring programme

3.1 Roadbase strength, moisture condition and density

3.1.1 Structural numbers

Basic theory

The analysis of different pavements is facilitated if an index of pavement strengths is used to enable performance to be compared. Such an index for pavement materials was developed from the AASHO Road Test, and is known as the *structural number* (SN). The structural number is derived from an empirical relationship in which the thickness and strength of each pavement layer are combined using the following relationship:

$$SN = \sum a_i d_i$$

where

- a_i = AASHTO strength coefficient, which is a different function of layer CBR for bases and sub-bases,
- i = 1 for surfacing
2 for roadbase
3 for sub-base, and
- d_i = thickness of each pavement layer in inches

The strength coefficient for sub-base materials is calculated using:

$$a_3 = 0.01 + 0.065 (\log_{10} CBR)$$

where

CBR = CBR of the sub-base

The strength coefficient for granular roadbase materials is calculated using:

$$a_2 = (29.14 CBR - 0.1977 CBR^2 + 0.00045 CBR^3) \times 10^{-4}$$

where

CBR = CBR of the granular roadbase

Any combination of strength coefficient and thickness may satisfy the AASHTO structural number relationship. However, it is important that a material of adequate strength is provided in an upper layer to resist the traffic stresses imposed upon it. This ensures that the layer itself does not fail, and that its thickness is sufficient to reduce the maximum stress that can be resisted by the next lower layer. Use of marginal quality roadbase materials, as are being investigated here, must comply with the requirements for determining structural number in this manner.

A *modified structural number* (SNC) can also be used, which takes into account the contribution to pavement strength of the subgrade. This is calculated from the following relationship:

$$\text{SNC} = \text{SN} + 3.51 \log_{10} \text{CBR} - 0.85 \log_{10} \text{CBR}^2 - 1.43$$

where

CBR = subgrade CBR

Determination of structural number

Test pits were dug to measure the thicknesses of individual layers, and these were used in the determination of the structural numbers of the pavement on the test sections. A constant thickness of 25mm and a structural coefficient of 0.1 were assumed for the road surfacing.

The results of the performance measurements are shown in Appendix B. The structural number and modified structural number for the test sections is shown in Table B-1 to Table B-3 for Zimbabwe, Malawi and Botswana, respectively. This indicates a high traffic-carrying capacity for all of the test sections which, in many cases, is higher than the designed traffic loading. The subgrade contribution to the modified structural number is high for all sections. It represents between 50 and 75 per cent of the modified structural number in most cases. However, even at these high values of subgrade CBR, the modified structural number is not sensitive to even large changes in subgrade CBR. Thus, for the purposes of the analysis of the performance of the roadbases, the contribution of the subgrade CBR can be ignored.

3.1.2 CBR

The in situ roadbase, sub-base and subgrade CBRs for the Zimbabwe, Malawi and Botswana test sections are shown in Table B-1 to Table B-3. The measurements shown were made in centre-line, inner and outer wheel-tracks. The tables show the variation from the end of the dry season (1996) to wettest season (1997).

3.1.3 Moisture condition

Table B4 to Table B6 show the roadbase moisture conditions of test sections within the three climatic areas ($N < 2$, $N = 2-4$ and $N > 4$). The highest measured ratios of the in situ moisture content to the optimum moisture content are shown for the centre-line, and the inner and outer wheel-tracks.

3.1.4 Relative density

Table B to Table B-9 summarise the in situ relative densities measured using the CPN Stratagauge on the test sections in Zimbabwe, Malawi and Botswana.

3.1.5 Deflection

The results of the FWD deflection surveys, given as central deflections, are shown in Table B-10.

3.2 Deterioration

3.2.1 Cracking

The percentage of the area of cracking affecting the outer and inner wheel-tracks, and the area between the wheel tracks, was calculated. The results summarised in Table B-11 to Table B-13 for the sections in Zimbabwe, Malawi and Botswana respectively. Around half of the sections showed some cracking, although only a quarter of these had cracking of more than ten per cent.

3.2.2 Roughness

The results of the roughness surveys are given in Table B-14 and Table B-15 for the test sections in Zimbabwe and Malawi.

3.2.3 Rutting

The results of the rut depth surveys are given in Table B-16 to Table B-18 for the test sections in Zimbabwe, Malawi and Botswana respectively.

3.3 Traffic

Traffic levels varied from 20 to 400 vehicles per day in Zimbabwe, 230 to 270 vehicles per day in Botswana, and 40 to 1 100 vehicles per day in Malawi. The proportion and type of heavy vehicles using these roads varied greatly, and this underlines the importance of carrying out accurate traffic surveys for design on the secondary roads.

4. Performance of low volume roads

4.1 Strength

4.1.1 Roadbase

Pavement design strengths are determined on the basis of the CBR test. Samples are normally compacted to at least 98 per cent BS 4.5kg rammer or equivalent maximum dry density at optimum moisture content, and samples are soaked in water for four days after compaction and before testing for CBR. A minimum CBR of 80 per cent is the target for pavement design purposes. Field conditions should be considerably drier than 'soaked' for most of the time, so the in situ strength should normally be substantially higher than the soaked values. It is also important to recognise that the densities of the roadbases in the field will vary from the 98 per cent compaction value. Comparisons of soaked to in situ CBR values should be made on the basis of a laboratory test where material is compacted to the equivalent field density.

Figure 4-1 to Figure 4-3 show a comparison of the outer wheel-track in situ CBR (weakest values recorded) plotted against the soaked CBR values at an equivalent field density, for each of the three climatic zones. These show that the field CBRs in the outer wheel-tracks remain at least as high as the design CBR value, irrespective of the climatic zone. For those cases where the design CBR is less than 80 per cent, the in situ CBR values are not necessarily as high as 80 per cent. The increases in strength for the majority of the sites is between one and two times the soaked value. It is apparent that, in the region, even in the drier areas, design strength testing at a moisture condition below the soaked value could be risky. The spread of data above the line of equality in the figures results from the moisture-strength interaction with other important variables, such as the material type, design characteristics, site drainage, and the integrity of the surfacing seal.

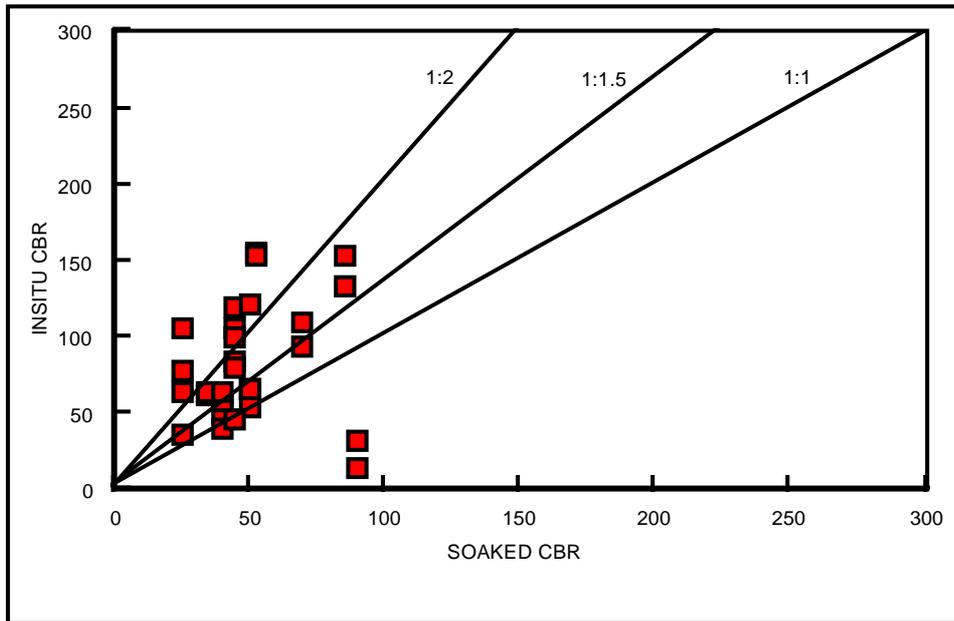


Figure 4-1 Comparison of in situ and soaked CBR for roadbase in Climatic Zones N<2

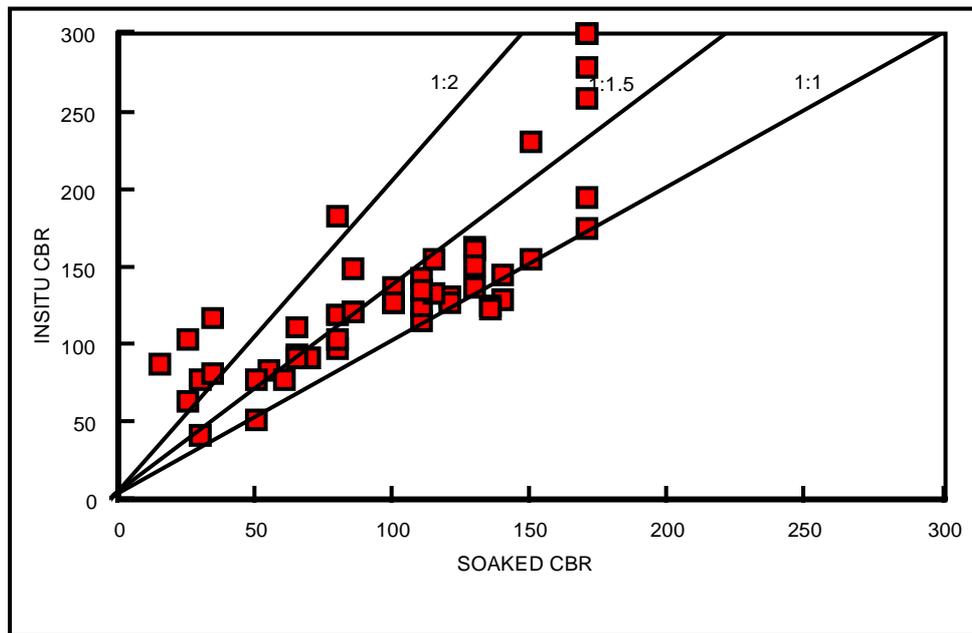


Figure 4-2 Comparison of in situ and soaked CBR for roadbase in Climatic Zones N=2-4

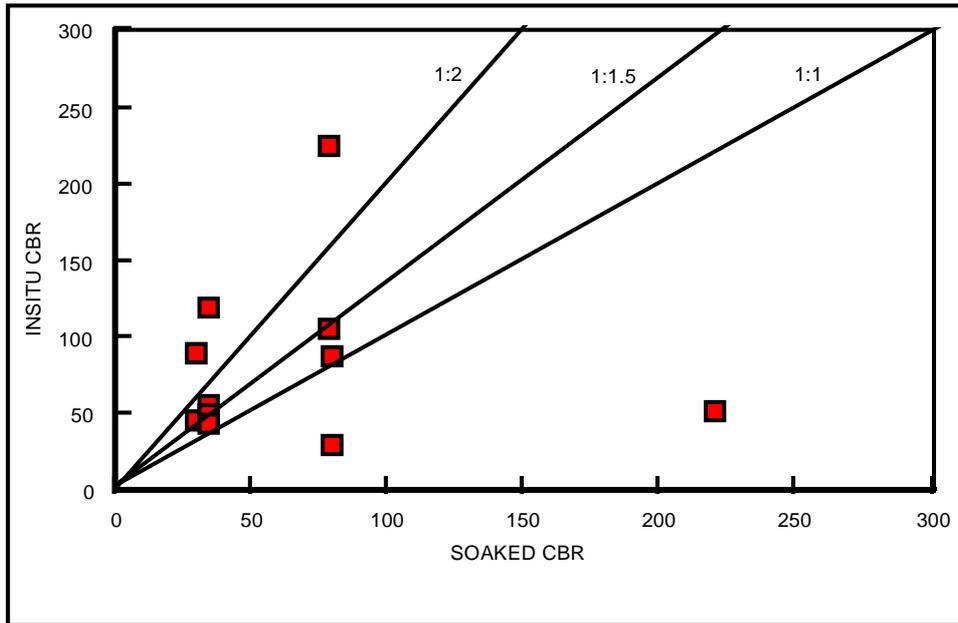


Figure 4-3 Comparison of in situ and soaked CBR for roadbase in Climatic Zones N>4

Figure 4-4 to Figure 4-6 show the CBR values at the carriageway centre, and inner and outer wheel-tracks, for each of the climatic zones. The minimum strengths in the central part of the carriageway, from the inside wheel-track to the centre-line, are, as expected, generally higher than those at the outer tracks.

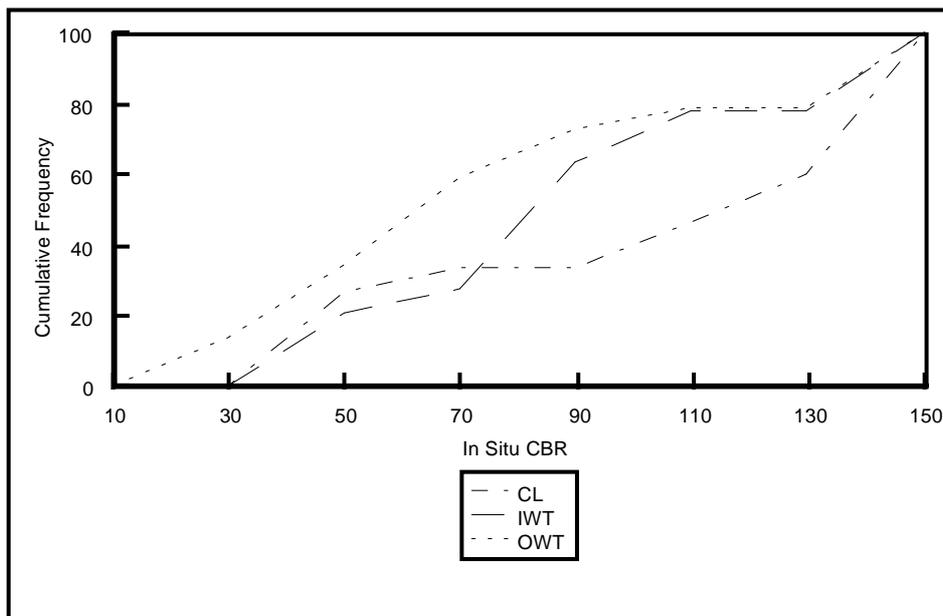


Figure 4-4 Cumulative frequency of CBR for roadbase in Climatic Zones N<2

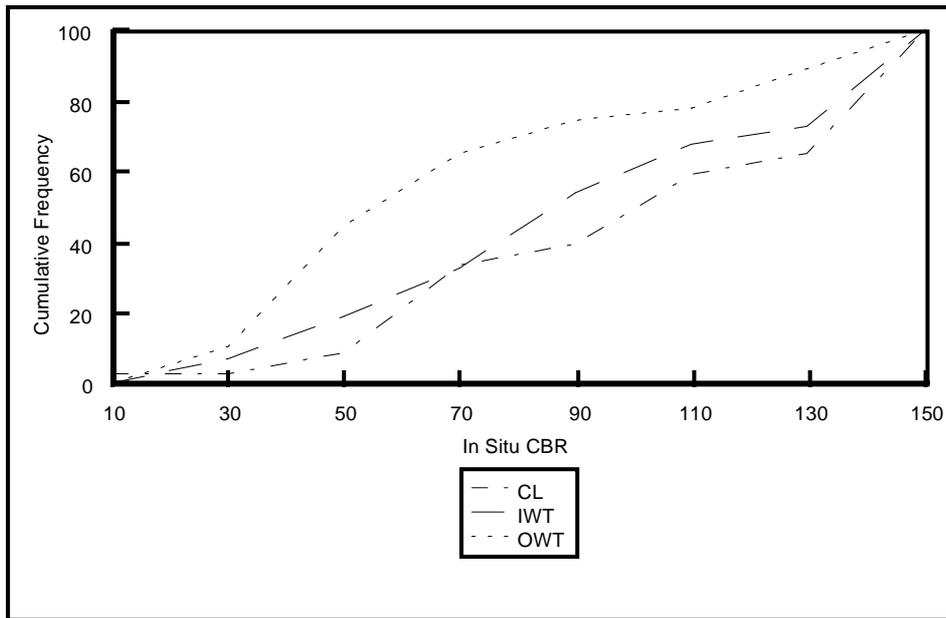


Figure 4-5 Cumulative frequency of CBR for roadbase in Climatic Zones N=2-4

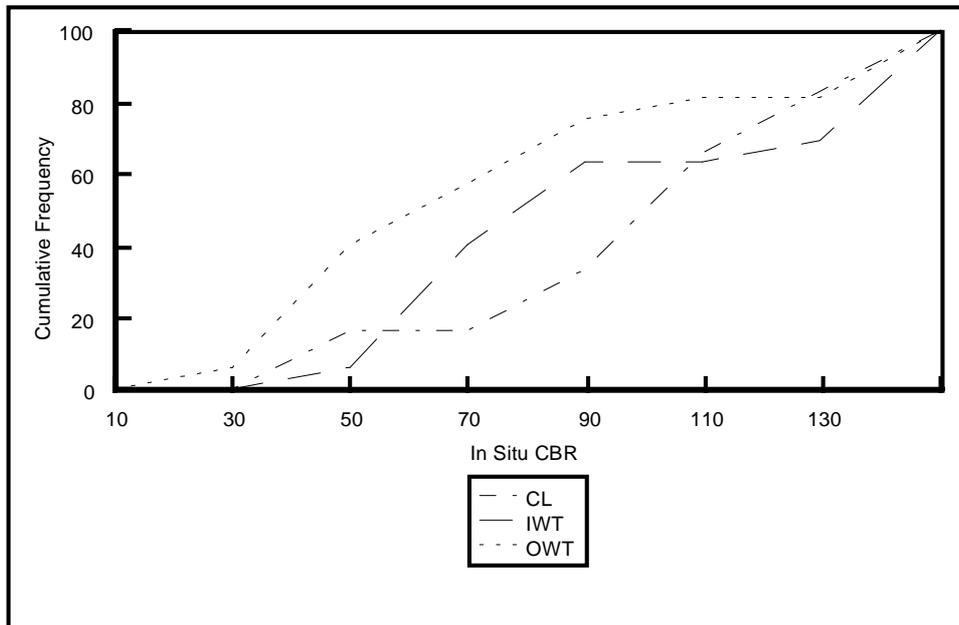


Figure 4-6 Cumulative frequency of CBR for roadbase in Climatic Zones N>4

In the wettest of the climatic zones ($N < 2$), the sites at MIMY, GINA, KUMA, LEZA1 and LEMI3 all had a CBR value in the outer wheel-tracks of less than 70 per cent. Nevertheless, the performance of these roads have been satisfactory, particularly considering the very high plasticities of the roadbase materials used. The sections at KUMA, LEMI3 and LEZA1 had in situ relative densities much lower than 98 per cent of the BS 4.5kg rammer compaction requirement. The sections at MIMY and GINA were poorly drained. The relationship between

the field moisture content and the in situ CBR is not well defined for any of the sites studied. This is probably due to the influence and variability of field density in climatic area ($N < 2$). The ratio between field and optimum moisture content for the roadbases lies generally between 0.8 and 1.0. However, where sites are well drained, such as on high embankments, roadbases can dry to about 60 to 70 per cent of optimum. In general, the ratio of field to optimum moisture content is similar across the whole width of the road. This is particularly noticeable where the surface condition of the road is poor, such as on LEZA1 and GINA.

The test sections at KIBA, HSMO, and MAMI were constructed using a 'low cost' approach unique to Zimbabwe. Construction and sealing of the road is carried out by the maintenance units. The sections are in the $N=2-4$ climatic area, and had noticeably low strengths in the outside wheel-tracks, although all were performing reasonably well. These sites were all constructed using low cost techniques, and the density data reveals that all sites had relatively poor levels of compaction. These were often below 95 per cent in the outer wheel-tracks. Increasing the level of compaction, to at least the 98 per cent of the BS 4.5kg rammer compaction would increase the strength, reduce the moisture susceptibility, and increase the probability of an improved the level of service. This suggests that better control of site compaction on low cost projects could result in significant benefits.

The drainage regime at both KIBA and HSMO was also relatively poor and would have an impact on the moisture regime in the pavement and, consequently, the strength. The ratio of field to optimum moisture content of the roadbases lies generally between 0.6 and 0.8 in this drier climatic area. However, where sites are well drained and with good crown height, the roadbases can dry to close to 50 per cent of optimum. The field/optimum moisture ratio can be higher in the central portion of newly constructed roads where a stable regime has still to develop. Examples of this include NARE, GECY, MAMI and MIAS. Older sections such as KIBA, HSMO, KAKE and WAMI, were noticeably drier in the central portion of the road.

In the dry climatic zone ($N > 4$), the minimum in situ CBR measured on the Kalahari and calcareous sand bases were low when compared to a target in situ CBR value of 80 per cent although, in both cases, the soaked CBR values were also low. Although the strengths of the sections at OASE were relatively low, this section had performed well considering the nature of the material. The calcareous sand section at TUNG also had low in situ strength, reflecting the low relative compaction of the material, which was between 90 and 95 per cent of the mean dry density. Again, this section had performed well, but higher compaction levels achieved during construction would have resulted in a more durable pavement. However, achieving high levels of compaction in sandy materials is difficult, because of the risk of shearing the material, and this may be a limiting factor. Poor compaction may also explain the low in situ strength of the calcrete base on NAMN. The section at TOSA was also built using low cost methods, and the earlier comments may also apply. The field/optimum moisture content ratio for the roadbases in this dry area lie generally between 0.5 and 0.7, although the outer wheel-tracks at SATU and TOSA were slightly wetter, even though well drained. The field/optimum moisture ratio is lower in the central portion of the road, although seldom below 0.5.

Where marginal materials have been used, the performance of the pavement does not seem to be affected adversely by the low in situ strengths and the seasonal reduction in strength in the outer wheel-track.

4.1.2 Influence of shoulder width

A number of sections of road in Zimbabwe were constructed with sealed shoulders. These had different widths so that the effect on the moisture ingress and strength of the pavement could be determined. Nine sections were constructed at NARE in three groups, with unsealed, 0.5 metre and 1.0 metre sealed shoulders. Two sections were constructed at GECY with unsealed and one metre sealed shoulders. Figure 4-7 to Figure 4-9 show the variation across the road in base component of the structural number, the variation in structural number, and the variation in modified structural number for Sections NARE7 to NARE9, respectively.

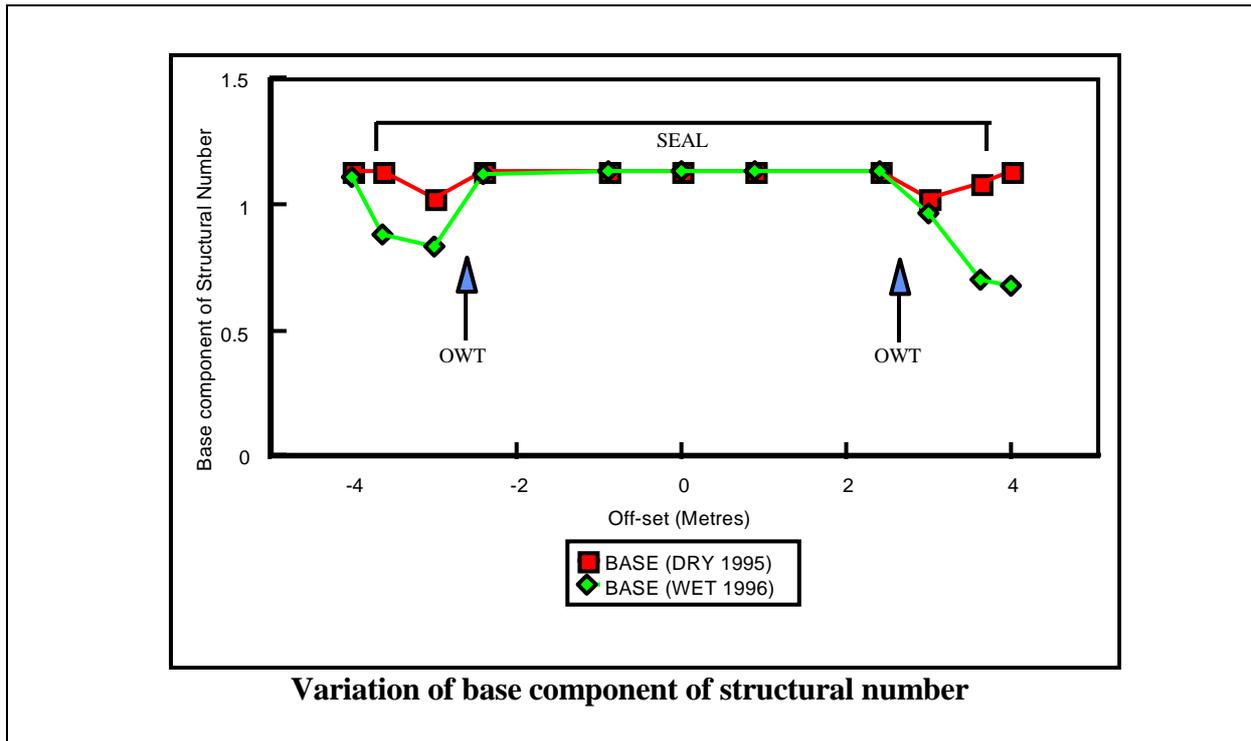
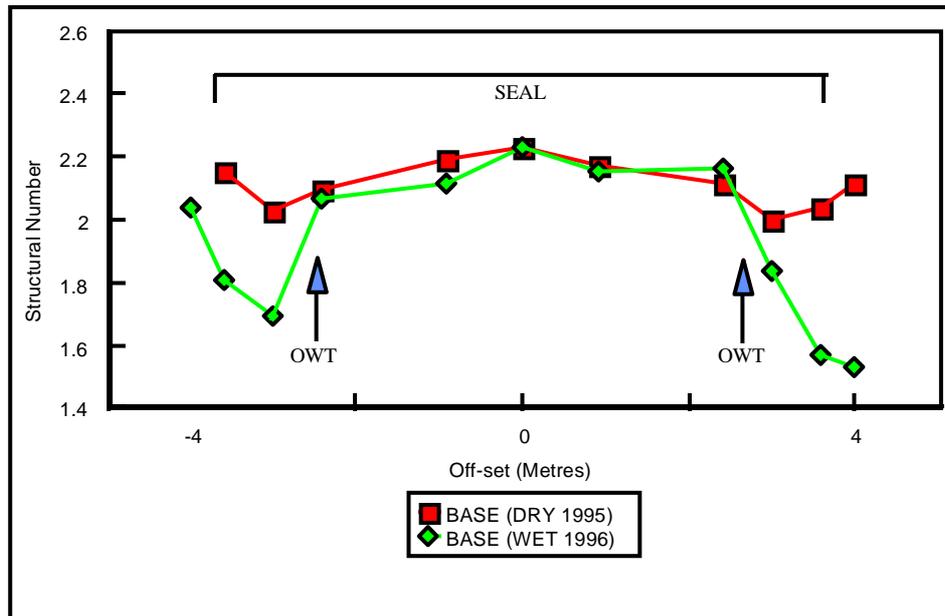
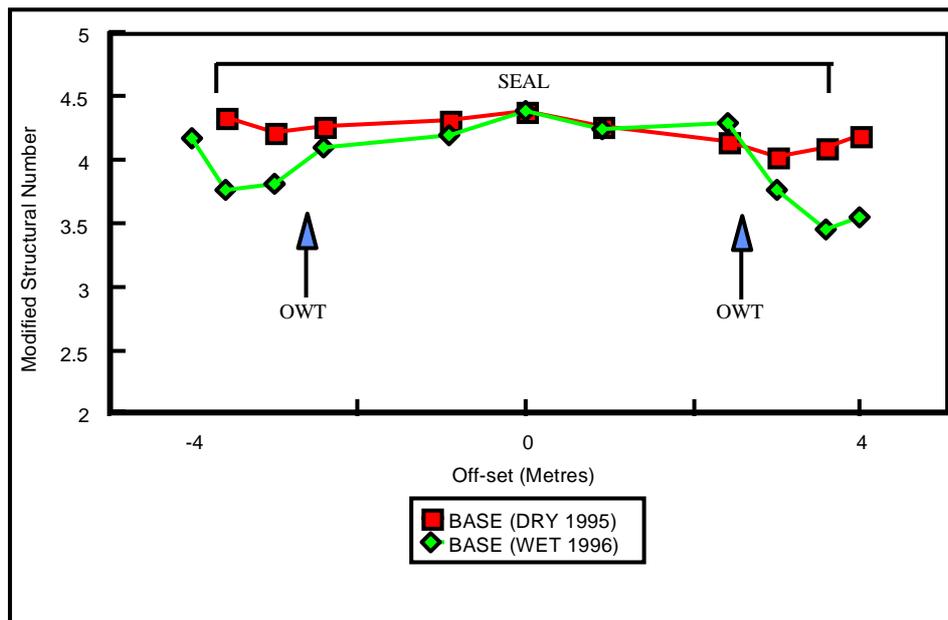


Figure 4-7 Impact of sealed shoulders on pavement strength on Section NARE7

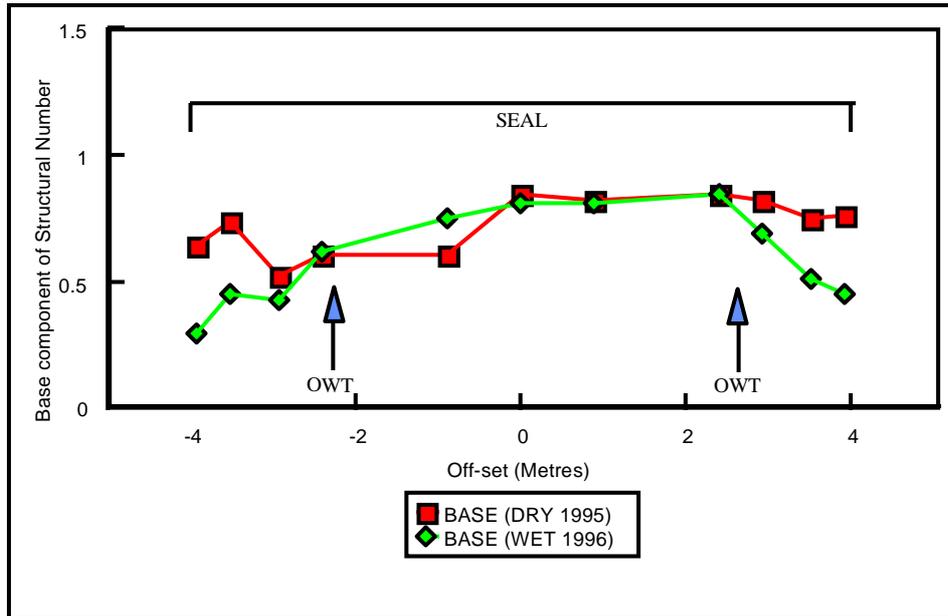


Variation in structural number

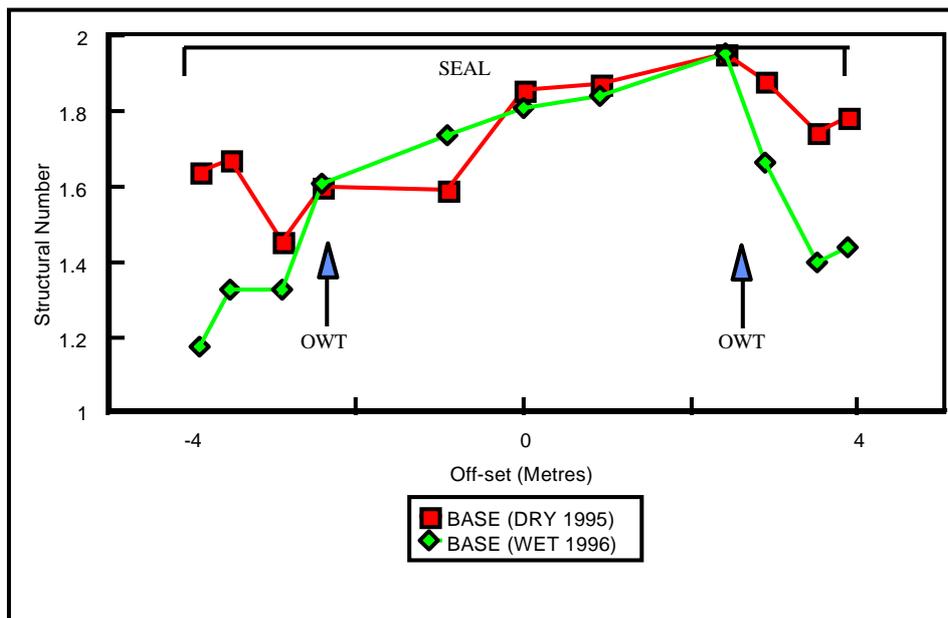


Variation in modified structural number

Figure 4-7 (continued)

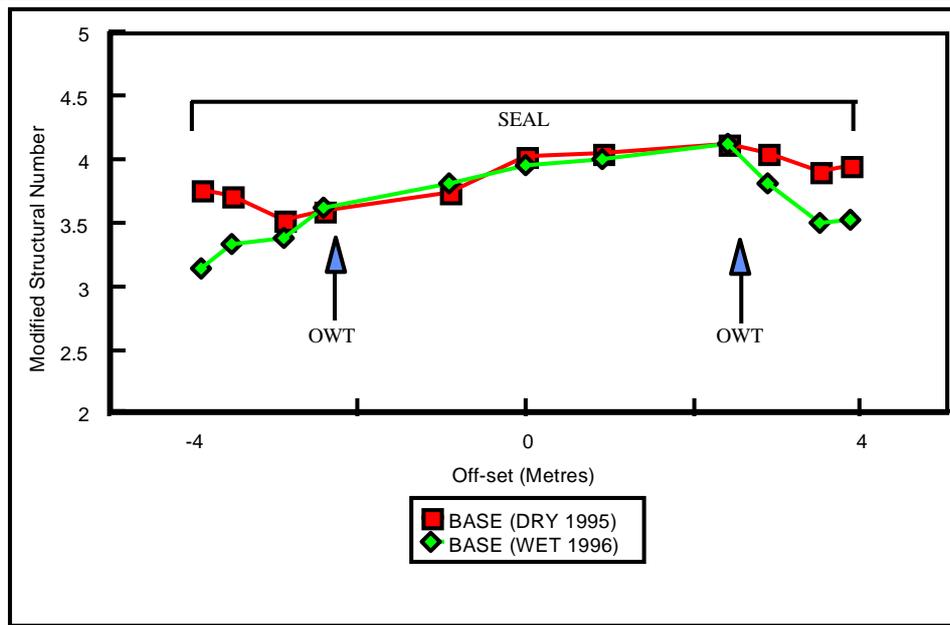


Variation of base component of structural number



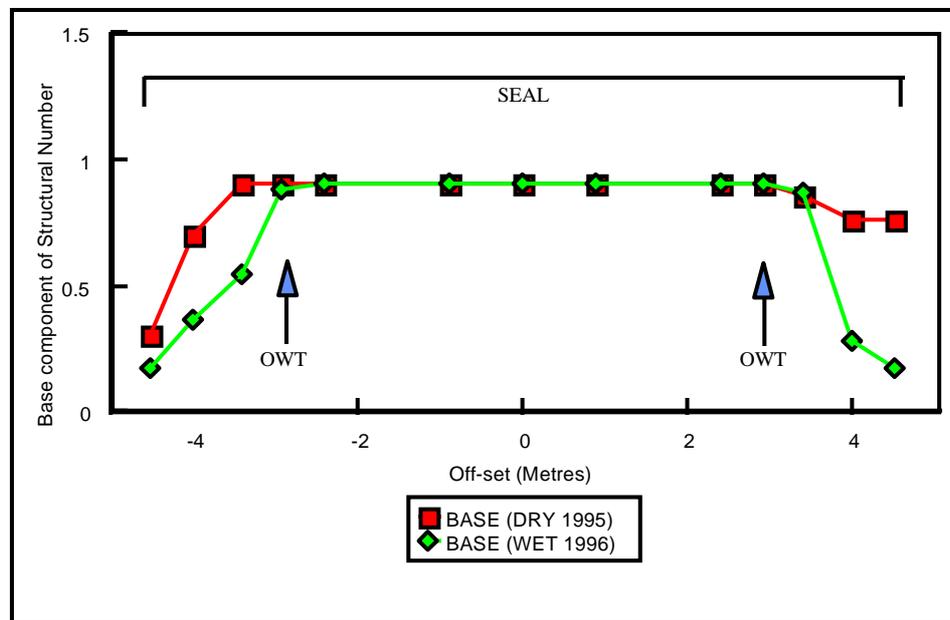
Variation in structural number

Figure 4-8 Impact of sealed shoulders on pavement strength on Section NARE8



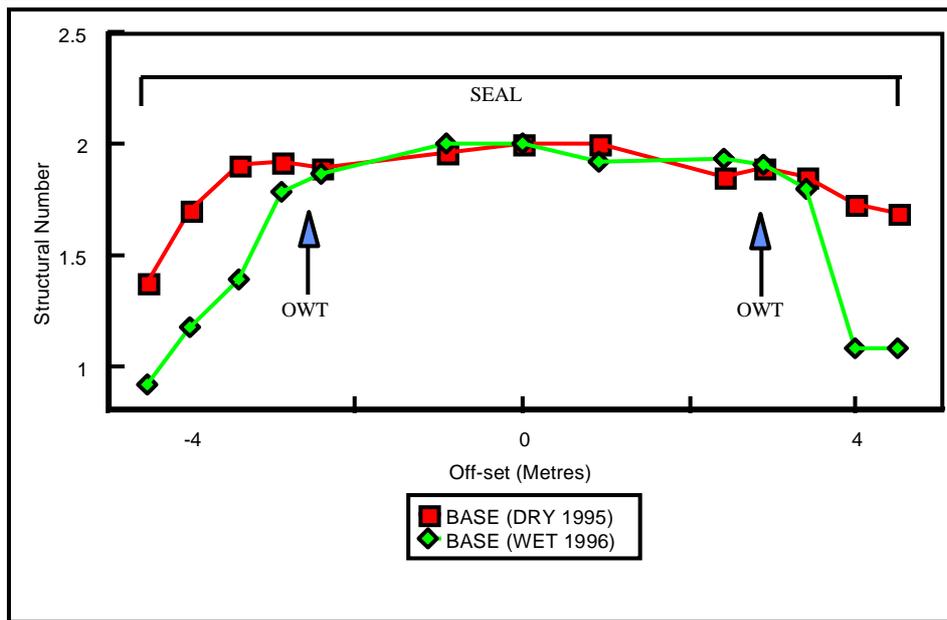
Variation in modified structural number

Figure 4-8 (continued)

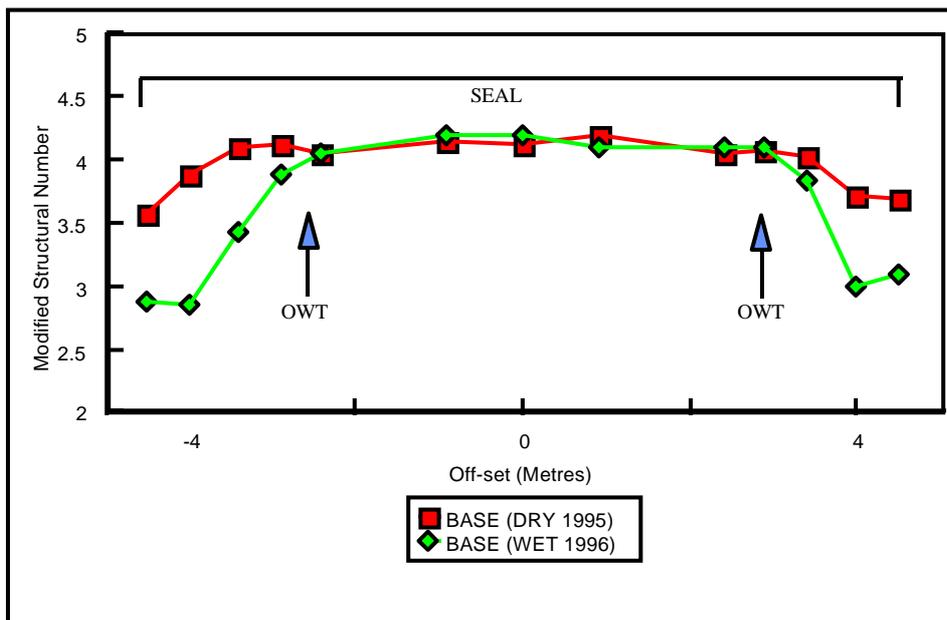


Variation of base component of structural number

Figure 4-9 Impact of sealed shoulders on pavement strength on Section NARE9



Variation in structural number



Variation in modified structural number

Figure 4-9 (continued)

The figures show that the additional width of seal moderates the seasonal weakening at the vulnerable outer wheel-track. However, the pattern of variation of the modified structural number indicates that the influence of the sealed shoulder on the subgrade is somewhat less. The one metre sealed shoulder has much greater influence on strength than the 0.5 metre shoulder.

The maximum ingress of moisture into the roadbase, causing a reduction in the strength, was measured on all of the sections. These data have been plotted against the distance of the outer wheel-track to the edge of the bituminous seal in Figure 4-10. The figure shows that over half of the sections are being effected by the seasonal wetting. The effect of increasing the distance of the outer wheel-track to the edge of the seal by 1.0 metre is shown in Figure 4-11. This reduces dramatically the proportion of sections. Figure 4-12 shows that the addition of 1.5 metre shoulders reduces the proportion further. There are two possible explanations for why some sections remain wet across the full width of the road. Either the drainage is so poor that complete wetting is unavoidable without improving the crown height, or the materials have high permeability because of poor compaction or low plasticity.

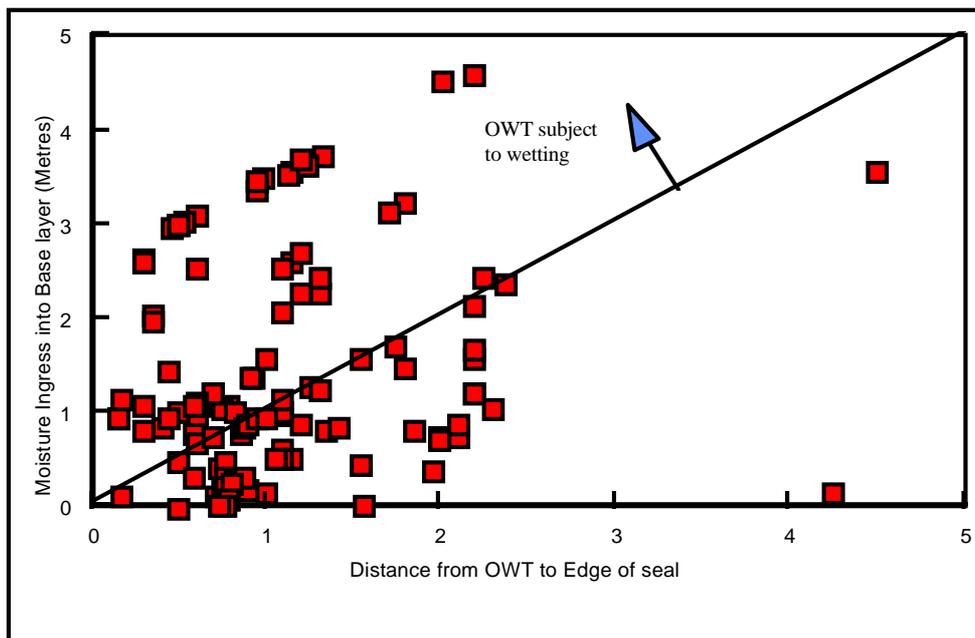


Figure 4-10 Moisture ingress into the roadbase on all sections

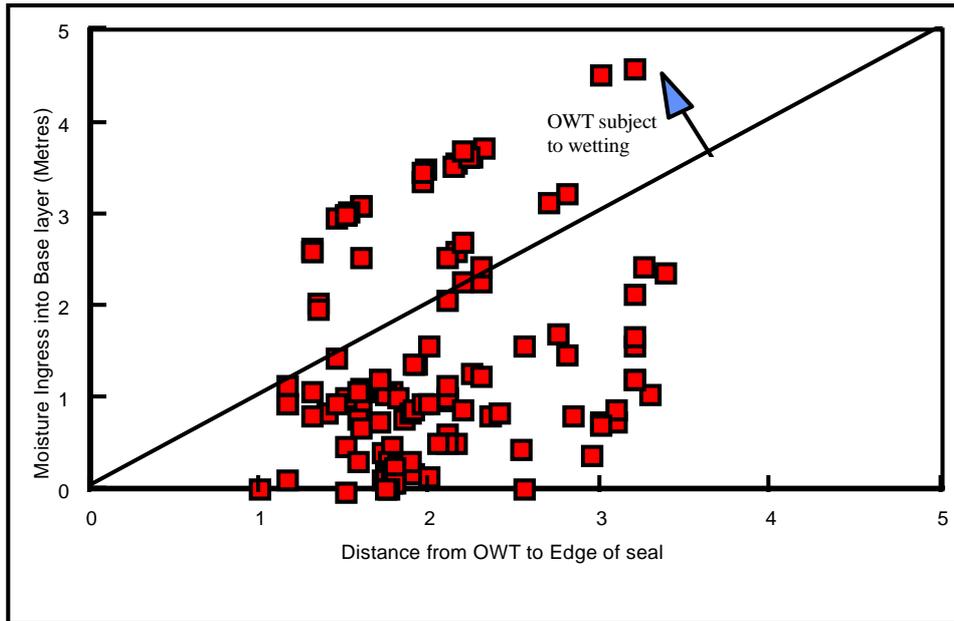


Figure 4-11 Moisture ingress into the roadbase on sections with one metre sealed shoulders

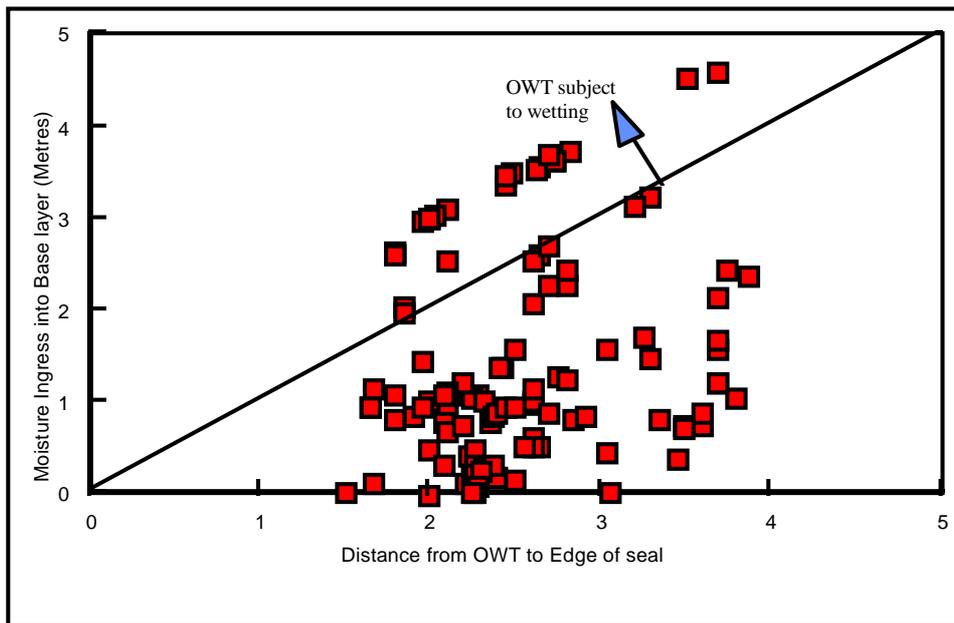


Figure 4-12 Moisture ingress into the roadbase on sections with 1.5 metre sealed shoulders

The influence of the characteristics of the roadbase materials on the susceptibility to moisture ingress is shown in Figure 4-13. The figure shows that, as the plasticity of the material increases, the ingress of moisture reduces significantly. Conversely, materials of low plasticity, such as those that meet the normal base material specifications, appear to be highly permeable and capable of allowing water to penetrate across and into the pavement. Unless the road is

constructed with a free draining sub-base and free-draining subgrade, any ingress of water from the bases can lead to the risk of sub-base or subgrade wetting. Under conditions of heavy traffic, this could result in deformation of the subgrade. It can therefore be concluded that the inclusion of more plastic materials in the roadbase of the low volume roads is beneficial to the longer term durability of the pavement. A sealed shoulder of at least one metre width in conjunction with plastic roadbases will give significant serviceability benefits. If materials that are more plastic are to be used, then it is crucial that impermeable surfacings are maintained.

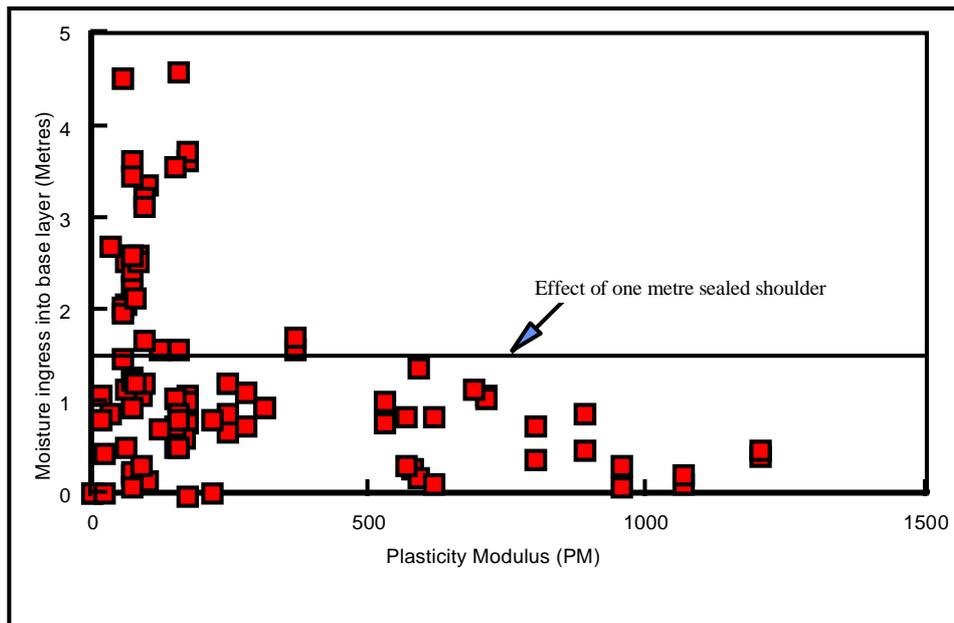


Figure 4-13 Moisture ingress into the roadbase for different plasticity moduli

A simple sealed shoulder design procedure could be based on the results of DCP tests across the width of the road during the dry and wet seasons, by considering the structural weakening caused by the ingress of moisture. This can be used as the basis of the design of sealed shoulders of an appropriate width to protect the vulnerable outer wheel-track.

4.1.3 Sub-base

Pavement structural design normally assumes that the sub-base should have minimum soaked CBR of 30 per cent. Samples are normally compacted to at least 95 per cent BS 4.5kg rammer compaction at optimum moisture content. Field conditions are likely to be considerably drier than 'soaked' for most of the time, so in situ strength is normally substantially higher than the soaked values.

It will be seen from Table B-1 to Table B-3 that the average wet and dry sub-base strengths are generally high in both the inner and outer wheel-tracks. However, the outer wheel-track is generally weaker than the central portion of the road. Only two sections, CABA and LEZA1, showed in situ sub-base strengths less than the design value. The crown height and drainage conditions at these sites were poor and probably contributed to the weakening of the sub-bases.

The in situ strengths of the sub-bases in all climatic areas were generally higher than the soaked values, as shown in Figure 4-14.

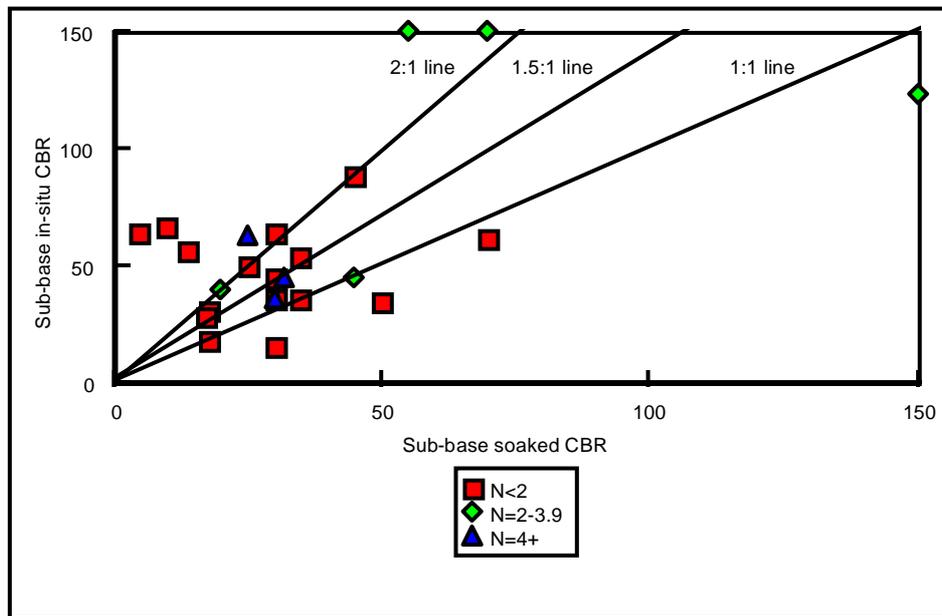


Figure 4-14 In situ and soaked sub-base CBRs for all climatic areas

4.1.4 Subgrade

Subgrade soils in the region are generally good, and this was reflected across many of the test sections. Subgrade soils retained high CBR values, even in the wet season. However, as shown in Table B-1 to Table B-3, most of the subgrade soils were subject to the influence of seasonal wetting and weakening in the outer wheel area.

The maximum subgrade moisture content values varied during the wet season, as expected, with the amount of rainfall affecting the site. The highest field/optimum moisture content ratios were during the heavy rains of the 1996/97 season. The field/optimum moisture content ratios of the subgrade under the outer wheel-tracks are shown in Figure 4-15 to Figure 4-17, as a function of the plasticity index for sites in each of the three climatic zones. In the wettest climatic area (N<2), it can be seen from Figure 4-15 that the in situ subgrade moistures are generally in range of 100 to 150 per cent of the optimum, with a maximum of 200 per cent. In the intermediate climatic area (N=2-4), the range is generally between 75 and 125 per cent of optimum, with a maximum 150 per cent. In the driest areas (N>4), the range is generally around 50 per cent of optimum. One site, TOSA1, is above this level, but this is at the margin of the N>4 climatic zone. It is also less than five years old, so the subgrade may not yet have reached an equilibrium with the local environmental conditions.

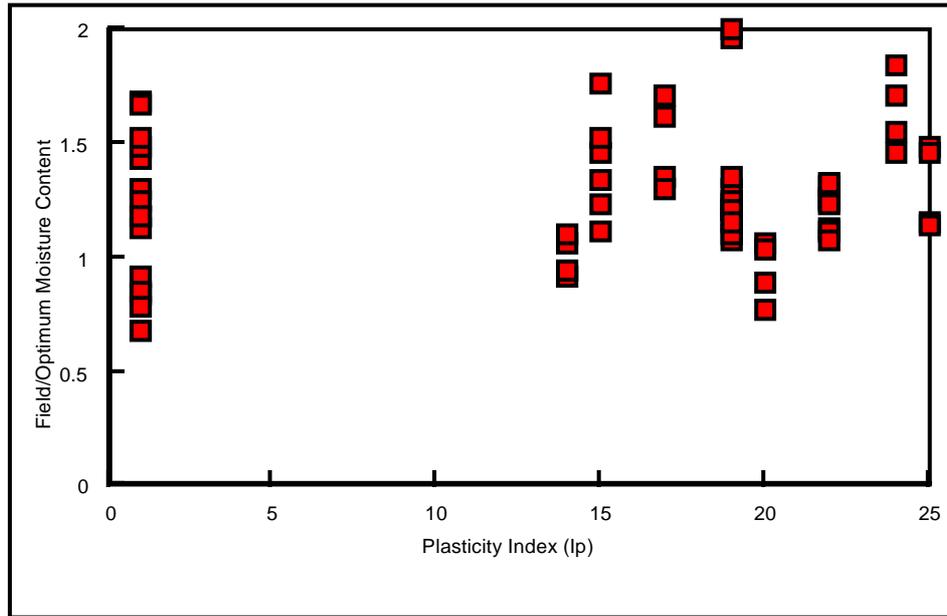


Figure 4-15 Field/optimum moisture content and plasticity for subgrade in Climatic Zones N<2

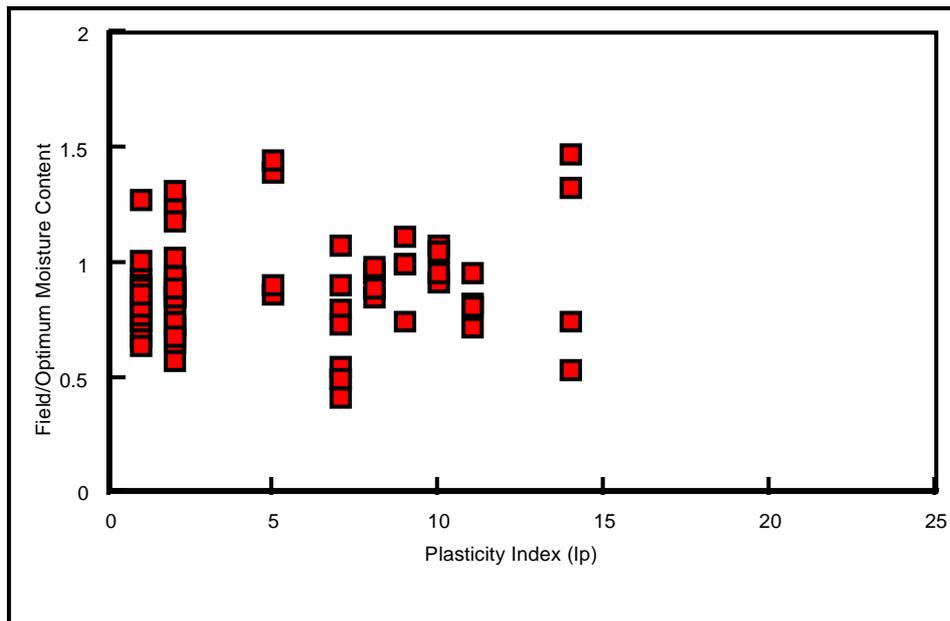


Figure 4-16 Field/optimum moisture content and plasticity for subgrade in Climatic Zones N=2-4

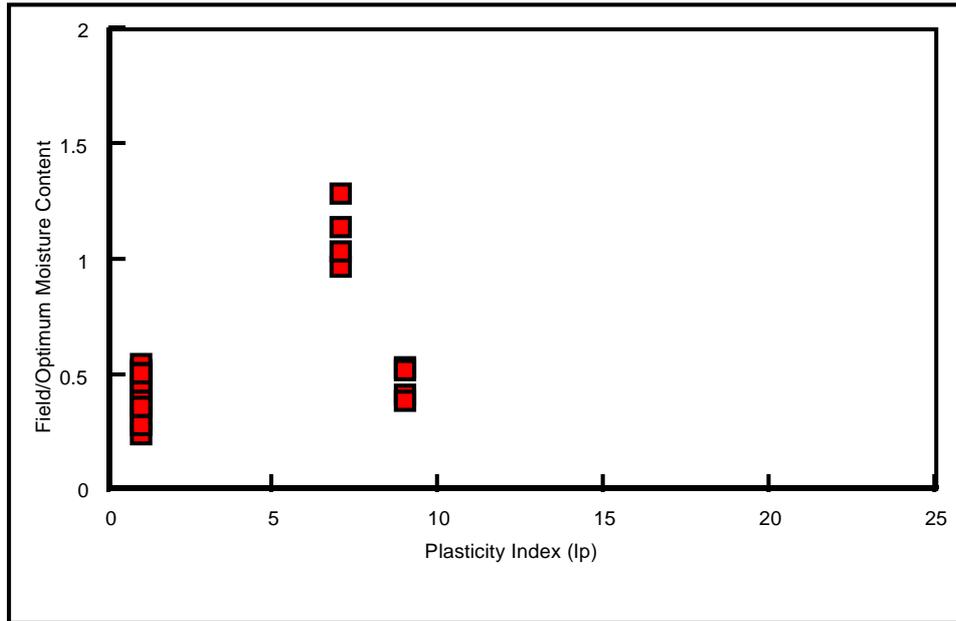


Figure 4-17 Field/optimum moisture content and plasticity for subgrade in Climatic Zones N>4

Comparisons were made between the weakest in situ subgrade CBRs and the soaked CBR values corrected to the field density. These are shown for all climatic regions in Figure 4-18 and Figure 4-19, separated according to the prevailing drainage. Irrespective of climatic region, if the site has effective drains and adequate crown height, then the subgrade strength stays above the design value. Where drainage is poor, subgrade strengths can drop below the design value, as shown in Figure 4-19, although drier climates still exert some drying influence.

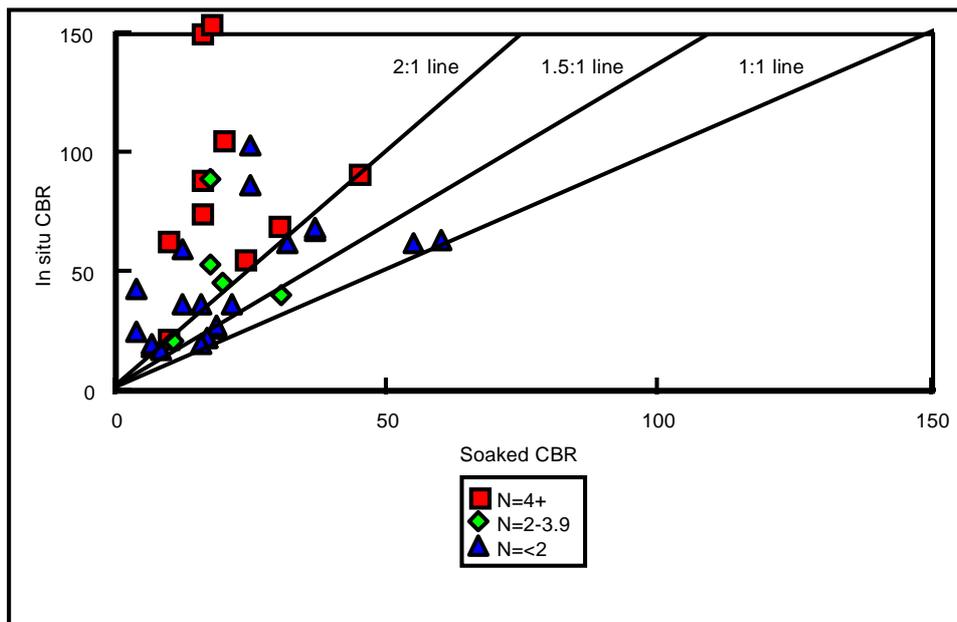


Figure 4-18 In situ and soaked CBRs for well drained subgrade soils

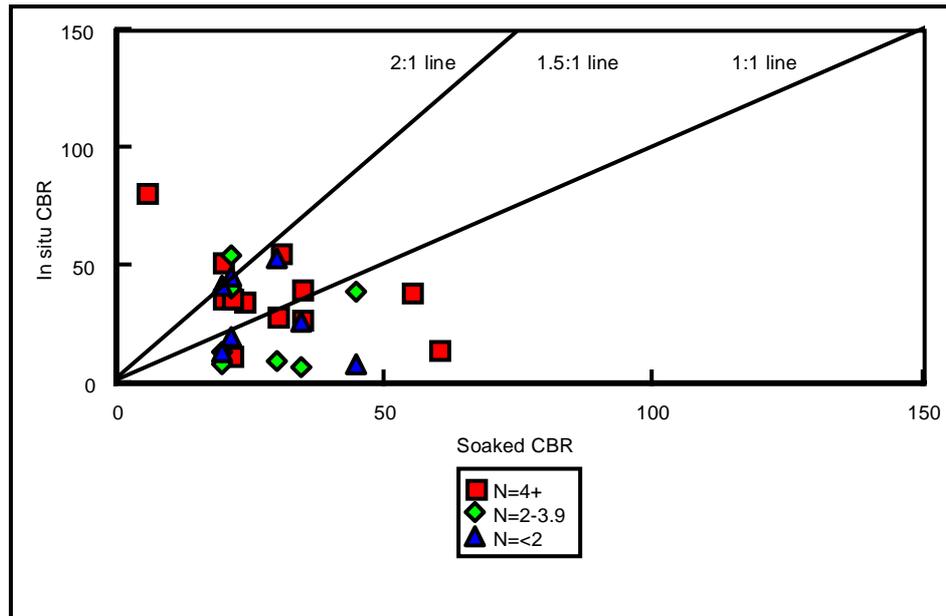


Figure 4-19 In situ and soaked CBRs for poorly drained subgrade soils

A more detailed representation of the influence of both climate and the drainage are shown in Figure 4-20 to Figure 4-25 for poorly and well drained soils, and for each of the climatic zones. In wet climatic areas ($N < 2$), and where the drainage is poor, the subgrade strength lies close to or below the design value, as shown in Figure 4-20. Where the drainage conditions are better, the in situ subgrade strengths are at least those of the design value. Where the climate is slightly drier ($N = 2-4$), and the drainage is poor, the subgrade strength lies close to the design value, as shown in Figure 4-22. Where the drainage conditions are better, the in situ subgrade strengths increase to about twice the design value, as shown in Figure 4-23. In dry areas ($N > 4$), the drainage again influences the subgrade strengths in a similar fashion. In situ subgrade strengths increase to about twice the design value where the drainage is good, as in Figure 4-25. If the drainage is poor, the in situ strengths can drop to around the design soaked values.

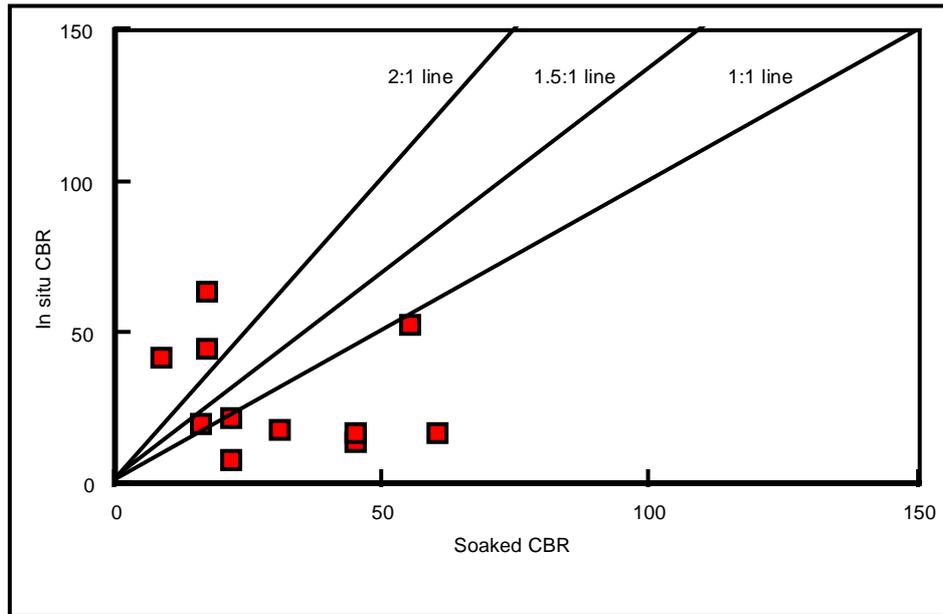


Figure 4-20 In situ and soaked CBRs for poorly drained subgrade soils in Climatic Zone N<2

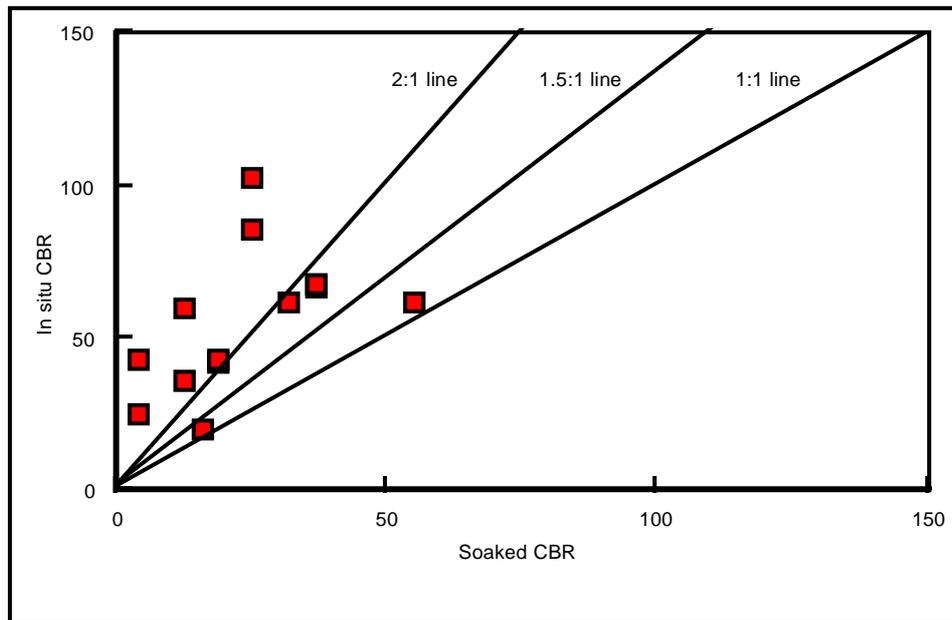


Figure 4-21 In situ and soaked CBRs for well drained subgrade soils in Climatic Zone N<2

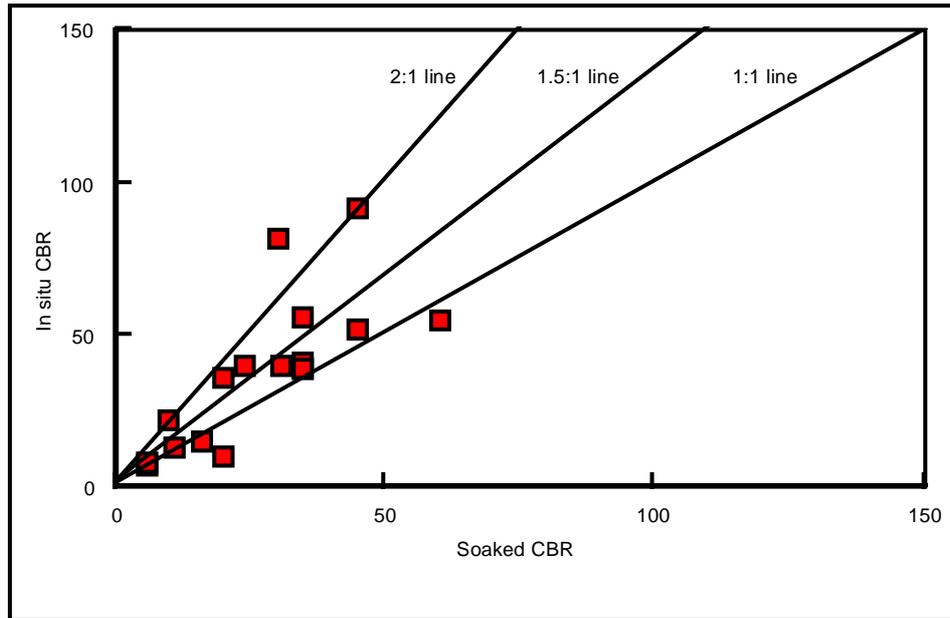


Figure 4-22 In situ and soaked CBRs for poorly drained subgrade soils in Climatic Zone N=2-4

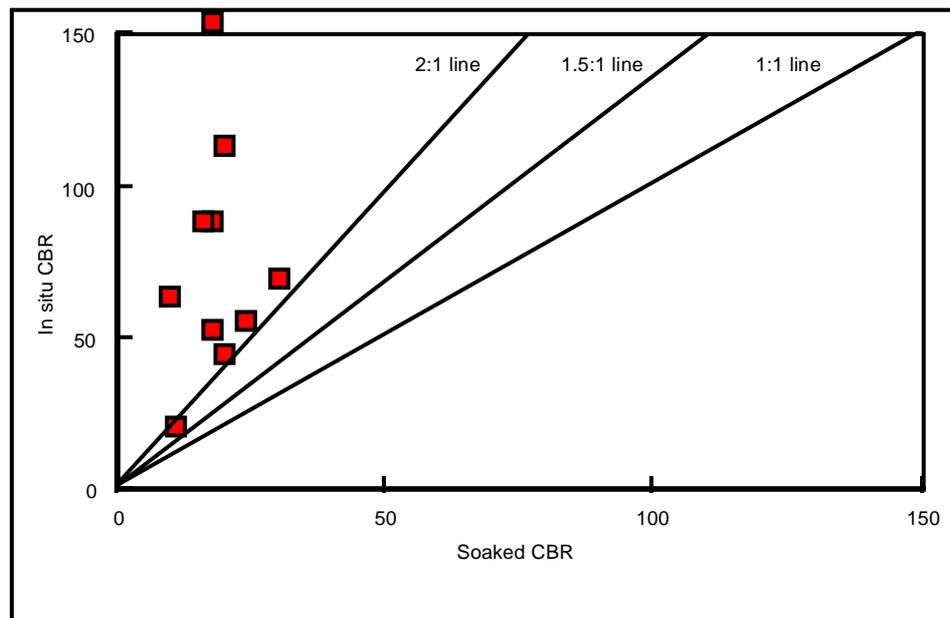


Figure 4-23 In situ and soaked CBRs for well drained subgrade soils in Climatic Zone N=2-4

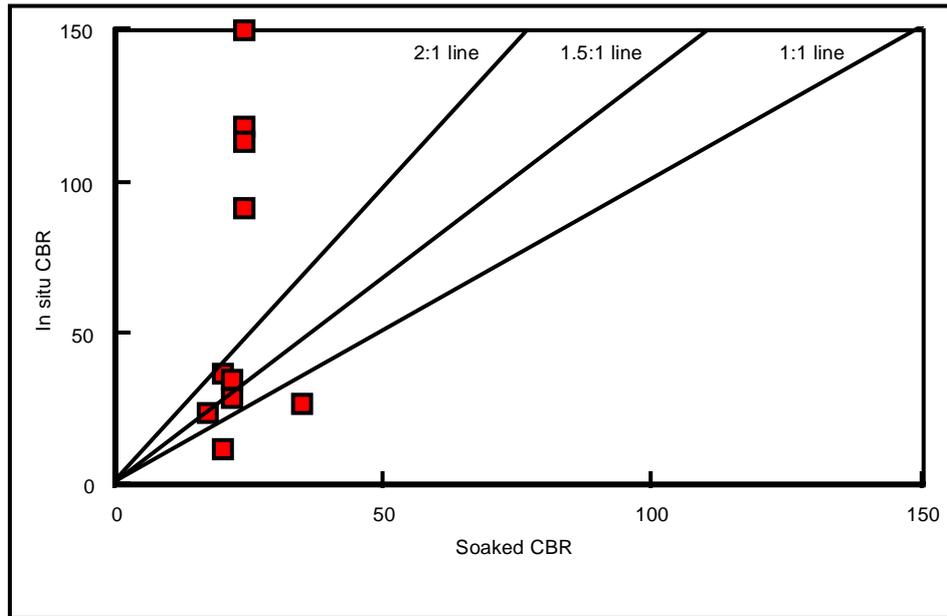


Figure 4-24 In situ and soaked CBRs for poorly drained subgrade soils in Climatic Zone N>4

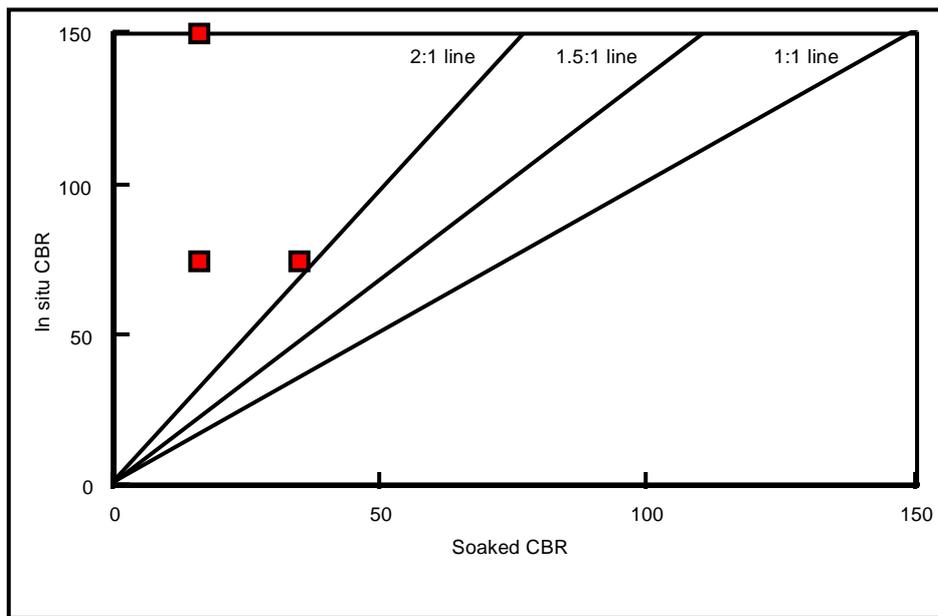


Figure 4-25 In situ and soaked CBRs for well drained subgrade soils in Climatic Zone N>4

These are important observations, and are used as the basis of the proposed pavement design method presented in the next chapter. The results of the strength analysis also support a case for introducing further subgrade classes beyond the limiting SG9 used by the Department of Roads in Zimbabwe.

4.1.5 Deflection

Most of the test sections showed an increase in deflection in the outer wheel-track from the dry to wet seasons, as shown in Figure 4-26. This again demonstrated the vulnerability of the outer wheel-track to wetting. The deflections at KIBA3, NAMN1, CAKA1, LEZA1, and MIMY1 were high, being in excess of one millimetre. With the exception of the NAMN1 section, these roads were over 10 years old. The NAMN1 section was less than five years old, and poor fill densities at construction may have contributed to the substantial wetting at depth in the pavement. The design strength of the roadbase was low on all of the sites, but the in situ roadbase strength was still reasonable. The in situ strengths of the sub-bases and sub-grades were generally low in the wet season, contributing to the high deflection.

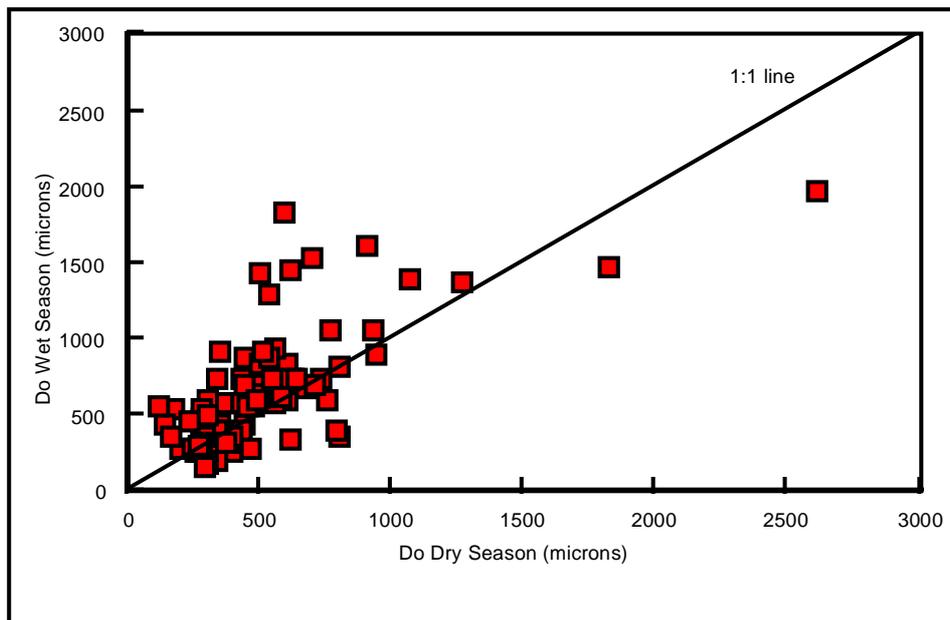


Figure 4-26 Outer wheel-track deflections in wet and dry seasons

4.2 Surface condition

4.2.1 Cracking

Cracking is likely to have complex causes, and Figure 4-27 shows little correlation between pavement strength and cracking. A number of the sections with high deflection did show extensive cracking, with crocodile cracking being observed on Sections CAKA1 and LEZA1. There was also little correlation between cracking and the level of traffic.

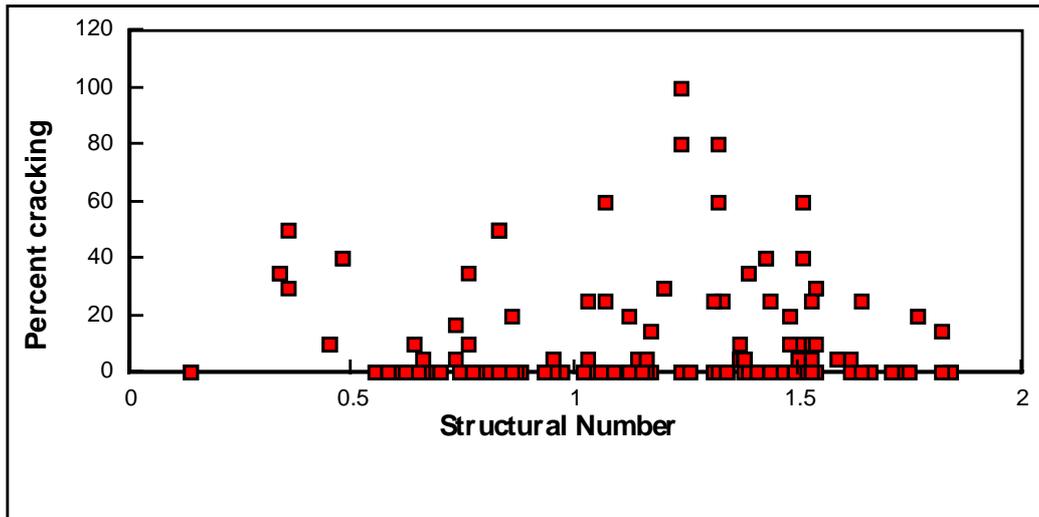


Figure 4-27 Cracking and structural number

The cracking observed on Sections CABA1, CAKA1, CAJE1 and LEZA1 was attributed to the age of the bituminous seals, which were well over 20 years old and embrittled. The cracks observed on Section HSMO were attributed to poor surfacing quality.

Figure 4-28 to Figure 4-30 show the percentage of cracking as a function of age of the sealed surface for three ranges of heavy vehicles using the road. Lower volume roads, with less than 100 heavy vehicles per day, show the most extensive cracking, but this is generally associated with poorly drained sites.

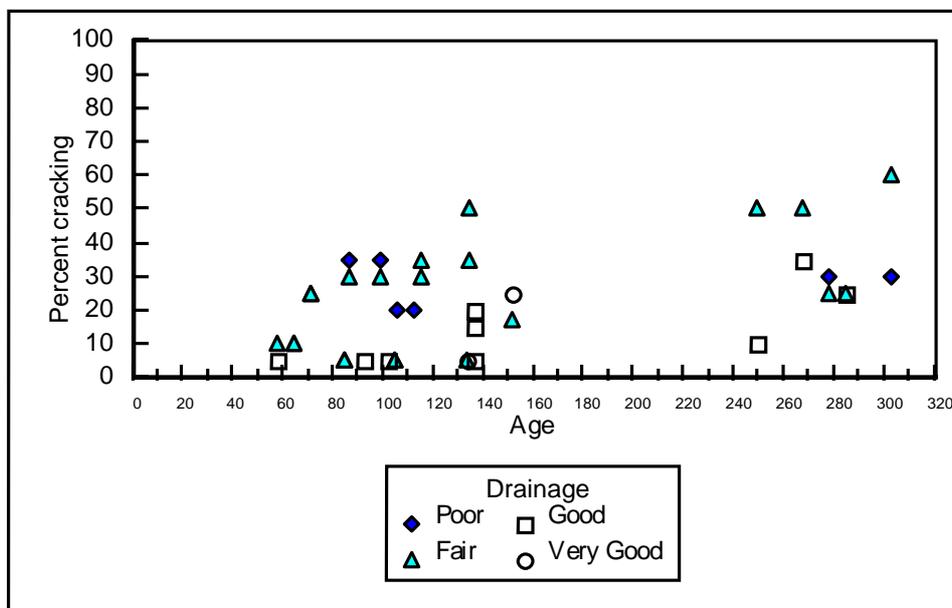


Figure 4-28 Cracking and age of seal for less than 50 vehicles/day

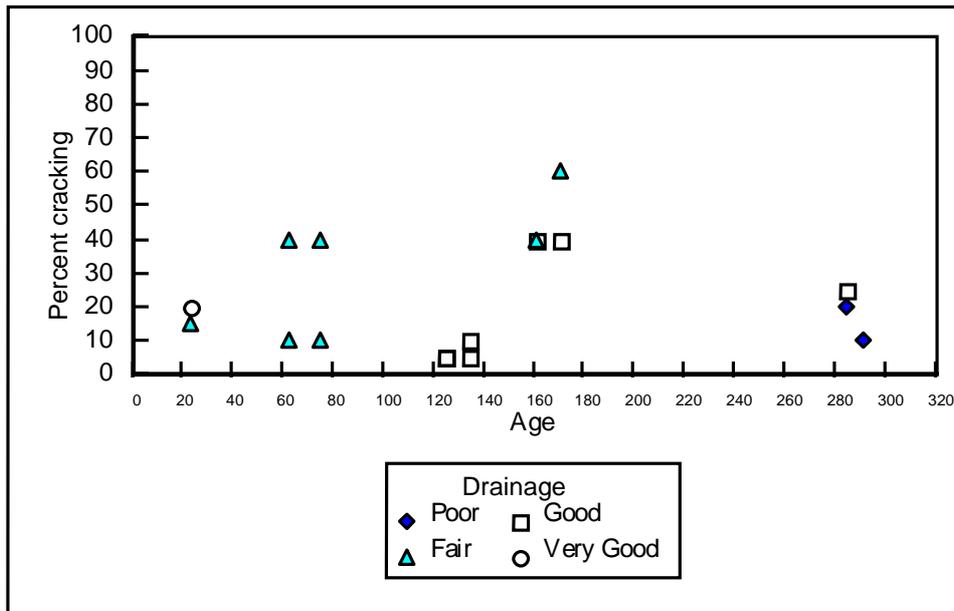


Figure 4-29 Cracking and age of seal for 50-100 vehicles/day



Figure 4-30 Cracking and age of seal for greater than 100 vehicles/day

Longitudinal and transverse cracking were the predominant modes. Transverse cracks were very often located at the edge of the seal, extending less than half a metre into the pavement. These cracks did not penetrate beyond the thickness of the roadbase and did not progress during the period of observation. It is likely that these cracks occur early in the pavement life as a result of shrinkage in the roadbase through drying. Longitudinal cracks tend to be confined to the outer wheel-track area, and are often quite long. They tend to progress over time. Longitudinal cracking is most likely to be caused by shrinkage and expansion of the subgrade materials as a result of seasonal wetting and drying, or poor drainage.

The performance observed suggests that the roads have adequate structural capacity for the volumes of traffic. Cracking is caused mainly by the road environment, including the complex inter-relationship of climate and drainage, and deterioration and embrittlement of the thin seals with age.

4.2.2 General surface condition

The thin bituminous surfacings have performed exceptionally well, often lasting more than 20 years with little maintenance and no re-sealing. The generally younger seals in Zimbabwe and Botswana were well constructed and have also performed well. However, the low cost single seals constructed initially on MAMI, TOSA1, HSMO2 and, to some extent also at KIBA, were less durable. All of the double seals performed well.

The quality of maintenance in Zimbabwe was impressive. In particular, the rapid response and effective use of crack sealing enhanced the durability of the weaker pavement structures, in particular at KIBA. It was also noticeable that the early provision of the second seal on HSMO1 had arrested the surface deterioration, particularly when compared with the section at HSMO2 which had only a single seal. Timely provision of a maintenance re-seal also arrested deterioration on the sections constructed with sand bases at OASE and TUNG1. Pot-holes were eventually observed on LEZA1, as the surfacing deteriorated, but none occurred on any of the other sites.

Loss of the shoulder material was commonly observed on roads constructed with unsealed shoulders. The absence of re-gravelling caused the edge of the sealed surface to break up. In the longer term, this will increase the level of moisture ingress into the pavement.

4.2.3 Roughness

Roughness values for new surfaced dressed roads are typically in the range 2.5 to 3.0 IRI. The values on some of the newer test sections are slightly higher than this, but may be caused by a 'noise' effect because of the large stone sizes which have been used in the surfacing. Examples of this are on sections at GECY, NARE and MAMI. The roughness levels in Malawi are remarkably low, with the exception of CABA1, particularly when considering the age of roads and the standards of materials used in their construction. In Zimbabwe, higher roughness levels were observed, in particular at KIBA, HSMO2, MIAS, SSMA and TOSA.

Little relationship exists between the observed levels of roughness and heavy traffic, as shown in Figure 4-31.

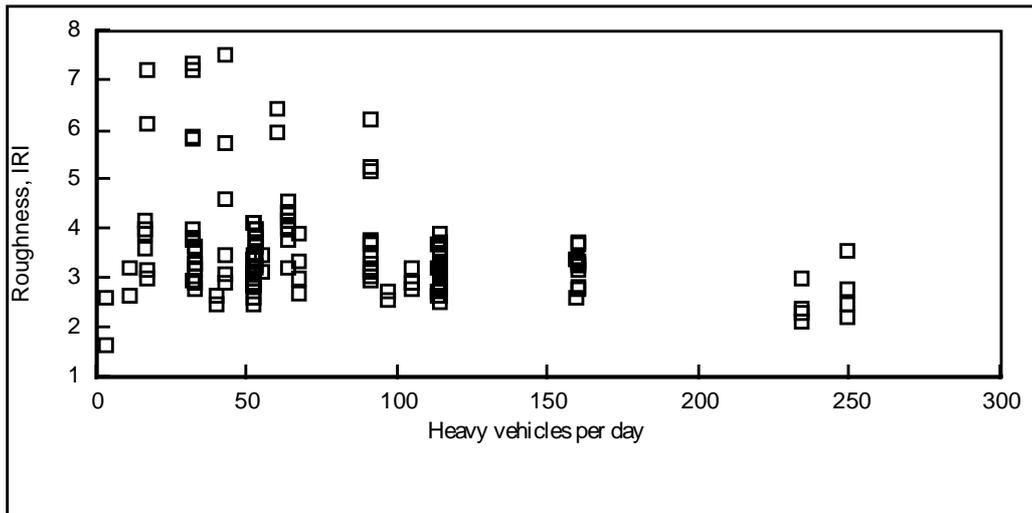


Figure 4-31 Roughness related to heavy traffic

Sections exhibiting higher roughness in Zimbabwe were generally those constructed using either ‘design-by-eye’ or low cost construction methods. In these cases, it is likely that roughness was either ‘built in’ at construction, or was caused by poor drainage. The standard of materials in the roadbase layers had little impact on the development of the roughness.

4.2.4 Rutting

The performance data show that the 90th percentile rut depths values are low, indicating that little deformation has taken place under the traffic loading. This would be expected from the magnitude of the modified structural number values. The sites showing the highest rutting are at KIBA3-5, MAMI1-2, CABA1, KUMA, LEZA and LEMI, although values are generally below 12mm. High deflection was only associated with that at KIBA3.

Figure 4-32 to Figure 4-33 indicate that relationships between rutting and either the volume of heavy traffic or the age of the road are very weak. The data indicate that the pavements, including those with very marginal roadbases materials, have been adequate to carry the type and volume of traffic to date. The performance of the roads in Malawi has been particularly impressive. The bases of these roads have been exposed to high levels of traffic loading, wetter climates, and a lower level of maintenance than in many of the other sites.

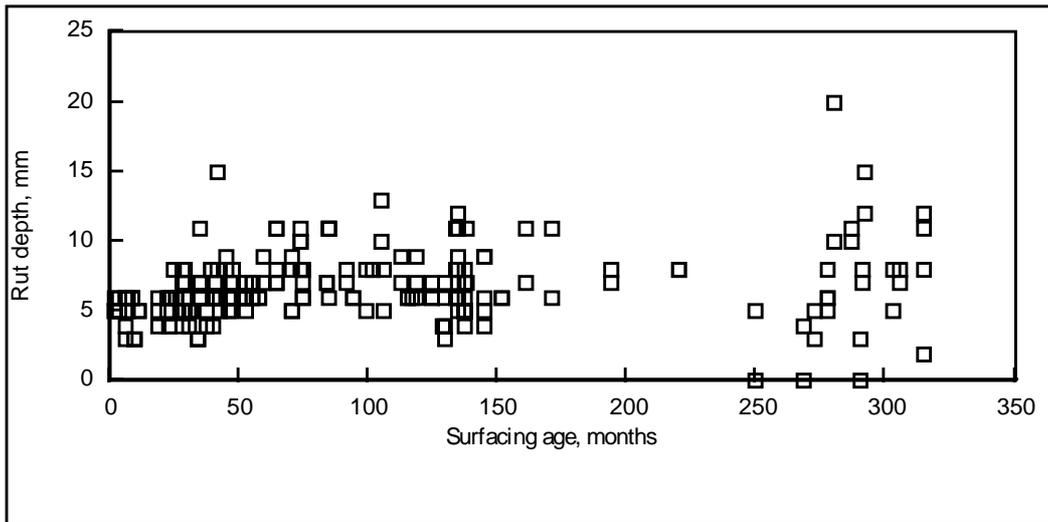


Figure 4-32 Rutting and surfacing age

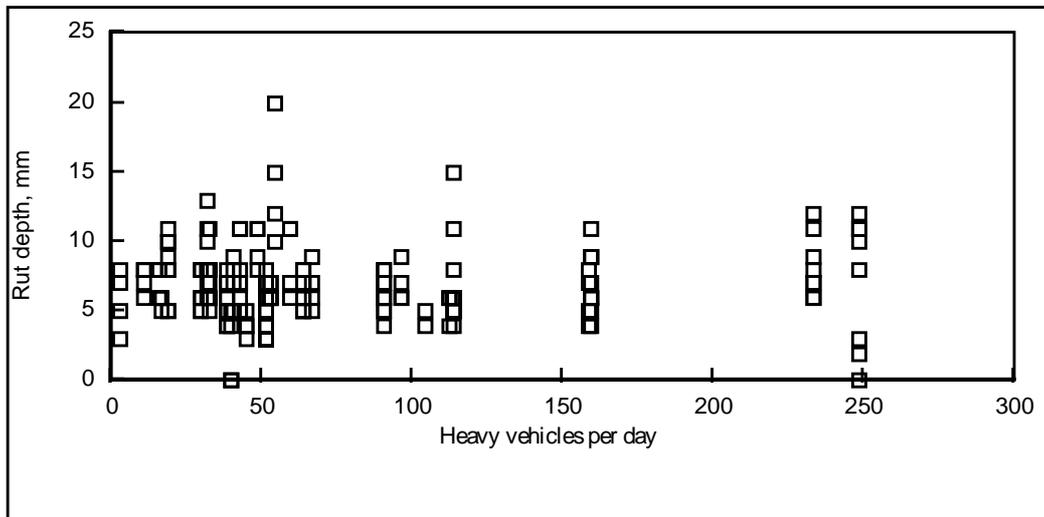


Figure 4-33 Rutting and heavy vehicles per day

No catastrophic shear failures attributed to one-off very heavy axle loading were observed on any of the test sections. It is considered that these failures are rare on the secondary network in the region.

4.3 Discussion of performance

The performance of many of the test sections was impressive, and a number of general conclusions could be drawn about the performance.

Seasonal effects

The strength of many of the pavements changed with the seasonal moisture conditions. These changes affected all layers, but the extent of ingress of moisture into the roadbase appeared to

be related to the plasticity of the roadbase material. Plastic roadbases with lower permeability wetted up less than other more permeable and non-plastic materials. The provision of sealed shoulders was effective at moving the zone of seasonal wetting towards the edge of the pavement and away from the vulnerable outer wheel-track. This resulted in drier conditions, giving the pavement additional structural capacity. Sealed shoulders less than one metre wide had little effect.

Structural strength

The modified structural numbers were very high on most of the roads investigated. Even in those cases where the structural numbers were lower than the design values, the roads had performed adequately. The performance of the roads constructed using plastic lateritic materials was particularly good, even though the in situ CBR of the roadbase layers were around 50 per cent. Many of these roads had carried well over half a million cumulative equivalent standard axles. Other roads, also constructed with very marginal roadbase materials, had also performed extremely well, and the observed deflections were much lower than what would normally be expected with such poor materials in the roadbase. Strong subgrade soils are prevalent over much of the region, and may have contributed significantly to the low deflection and to the good performance of many of the roads.

Traffic-induced deterioration

Traffic appeared to have little influence on the deterioration of the pavements until:

- Daily flows were over about 250 vehicle per day
- Heavy trucks and buses comprised more than about 25 to 30 per cent of the flow
- Roads were old enough to have carried over 0.5 million esa

Surface deterioration

Cracking and relatively high roughnesses were observed on a number of sections where traffic was light. The road environment was the main factor contributing to these defects in terms of:

- Climate (rainfall, temperature and evaporation)
- Drainage (effectiveness of drains, carriageway cross fall and crown height)
- Topographic and sub-soil conditions

Durability of surface seals

The durability of the bituminous surfacing seal has also proved to be important in determining the performance of these roads. Many of the double seals are protecting very marginal roadbases. These have performed exceptionally well, and some on trunk roads have lasted for 25 years. Evidence from many other roads in the region suggests that frequent re-sealing is crucial to performance. The single seals performed less well. These seals are known to be less durable and are particularly susceptible to environmental ageing and other factors which influence deterioration. Single seals are not recommended for use on roadbases constructed with marginal materials.

5. Development of structural design charts

5.1 Objectives

The original objective of the project was to determine whether marginal quality roadbase materials could be used on low volume roads. However, the data collected were considered sufficiently extensive to enable the recommendations to go beyond a revised selection procedure for the base layer materials. Structural pavement and materials design charts were therefore developed, and are described in the following chapter. The charts are based primarily on the performance data collected from the low volume roads monitored during the research programme.

5.2 Background to the development of the design method

By definition, traffic levels on low volume roads are small, and it follows that benefits accruing from savings in vehicle operating costs will also be small. The return on investment can be improved by reducing construction costs. The approach in the design presented here is to utilise locally available natural gravels in the upper pavement layers rather than the more commonly adopted, and expensive option, of using stabilisation or crushed rock.

Standards for design, construction and maintenance, and the specifications of materials used in roads needs to be set at an appropriate level to support the function that the road is providing as well as recognising the important influences of the deterioration mechanisms such as local environment and traffic. Adoption of inappropriate design standards, both too high and too low, will incur unnecessary expenditure. Recognition of the function of the road will therefore determine the standard of road geometry, pavement structure and quality of materials that needs to be provided.

However, it is worth re-iterating here that traffic levels will influence the choice of road width, cross-sectional standard and gradients. These in turn have an important influence on the performance of light pavement structures and, as such, need to be discussed as part of the design procedure presented. The volume and composition of current traffic with sensible forecasts of future traffic demand also need to be known if the design procedure is to be implemented effectively. On low volume roads, movement of construction traffic can be a significant component of overall traffic loading, and the design recommendations and work methodology should take this into account.

For low volume roads there is often a choice between providing a gravel or sealed surface. The use of sealed surfaces may be justified at relatively low levels of traffic where, for example, materials resources used for re-gravelling are scarce and long haul distances are needed. A life cycle costing approach is needed for taking these decisions, which recognise increasing costs of gravel resources over time.

5.3 Basic principles of the approach

The data obtained from the roads in Botswana, Malawi and Zimbabwe covers a reasonably wide range of climates, classified broadly as arid to semi-arid, seasonally wet, and wet. Other

performance data available for roads, both within and outside the region, were also consulted to verify and support the findings.

Two approaches can be followed when developing structural design charts:

- a) *A single design chart developed on the basis of actual field experience*
The chart is based on in situ subgrade strength, with the onus on the user to predict the in situ strength based on soil type, climate, drainage conditions and any other risk factors.
- b) *Several design charts based on a subgrade classification test*
Different charts are provided to suit the different climate, drainage conditions and so on.

Option a)

The behaviour of road pavements is controlled very largely by the most adverse conditions that prevail even if such conditions occur for a relatively short time. This approach to developing design charts therefore requires that performance of individual sections of road are determined in the most adverse conditions likely to be experienced. In practice, this means the weakest conditions experienced by the subgrade and, by implication, all the other pavement layers. During the period of this study, the Southern African region experienced both very dry and very wet seasons. The performance monitoring programme included measurements of moisture conditions and strengths in all pavement layers and subgrades throughout this period. It is therefore likely that the full range of conditions experienced through the life of the roads were encountered. However, use of this approach requires that users predict the most adverse conditions likely to be encountered by the new roads being designed. This is likely to pose difficulties, especially for low volume roads in development areas where little sealed road building has taken place before.

Option b)

The results of classification tests may have little direct relationship with the field conditions experienced by each road. Therefore, a problem with this approach is that if a design chart is based on the classification test then, for a particular traffic level and subgrade level, all roads on that particular subgrade will be assigned the same design irrespective of the drainage conditions or climate. This problem can be overcome by developing separate design charts for each climate category and drainage category.

In this case, a large enough data set is available to enable Option b) to be used as the basis of development of the design charts, provided that in situ road conditions can be related to the results of the classification tests. The laboratory test procedure for design used by most authorities in the region is the soaked CBR test. This is carried out on subgrade samples prepared under a standard set of conditions. Thus, there was a need to determine the relationship between the soaked values and the in situ CBR of the subgrades. This relationship is dependent primarily on climatic and drainage conditions, but it will also depend quite strongly on in situ densities. As a result, the relationship is subject to a great deal of variability. Risks can be reduced if the soaked CBR values used in the development of the chart reflect those at the same in situ density, rather than an arbitrary standard. This approach was used in the development of the charts.

5.4 Predicting in situ conditions

Sufficient examples were found in the region to enable the relationship between laboratory and in situ conditions to be determined. Some of the weaker roads investigated had deteriorated to a poor condition, but the majority of the roads were still performing well with little signs of normal traffic related deterioration.

Examination of the data shows that, in wet climates with poor drainage, the most adverse site conditions gave in situ CBR values equal to or stronger than the laboratory soaked values when tested at the same density.

In arid and semi-arid areas, the in situ CBR was found to be at least twice the value in wet areas, as shown in Figure 4-18 to Figure 4-25. Where exceptions occurred, these could be explained by the quality of the drainage or by the construction standards. Some subgrades were surprisingly weak. These tended to occur under pavements of roads that had previously been gravel, such as KIBA, and where soil densities were very low. The analysis underlined the value of preparing and compacting the subgrade properly when upgrading from an unpaved to a paved road to take full advantage of the cost savings possible in arid and semi-arid areas.

Drainage conditions also influence road performance. This can be measured in terms of the height of the crown of the road above the invert of the drainage ditch, referred to as the 'crown height', and the distance of the outer wheel-track from the edge of the sealed area. The results show that the provision of a sealed shoulder at least one metre wide increases subgrade strength under the wheel-tracks to about twice that of the worst case value in wet and poorly drained conditions, at least in arid and moderately wet climatic areas. However, strengths are affected only marginally by the addition of sealed shoulders less than one metre in width.

The results show that pavement structures which work well under wet and poorly drained conditions have an in situ subgrade strength, at the seasonally worst condition, similar to that obtained in the standard laboratory soaked CBR test. These structures behave in a similar way on a subgrade of half this strength in arid and semi-arid conditions provided the subgrade has been prepared to the density standard used in the design test. Therefore, as a first approximation, the same design chart can be used in arid and semi arid climates as in wet climates, except that the subgrade strength values in the standard soaked CBR classification test will be halved. This is equivalent to a shift of one subgrade category in the chart because each category represents a CBR range where the highest value in the range is twice the lowest. For example: S2 covers a CBR range of 3 to 4 per cent; S3 covers a range of 5 to 7 per cent; S5 covers a range of 8 to 14 per cent, etc.

Further shifts in subgrade class are possible in situations where a particularly dry environment can be assured. This may require that the shoulders of the road are sealed to a minimum width of one metre, that the outer wheel-track is more than 1.5 metres from the edge of the sealed area, and that the drainage is ensured by maintaining the crown height greater than one metre above the ditch.

5.5 Traffic induced deterioration

Traditional design principles for the traffic factor rely on two assumptions:

- The thickness design is sufficient to protect the subgrade from a ‘fatigue’ type of failure brought about by repetitive loads; this implies that higher levels of traffic will require thicker structures.
- The strength of the roadbase is sufficient to prevent failures of any sort; this implies that the roadbase specification is a ‘zero risk’ design.

The evidence from this and other studies in the region indicates that roadbase materials, which would be considered of ‘marginal’ quality using traditional specifications, can give satisfactory performance on low volume rural roads carrying typical rural traffic. In general, this does not include vehicles with excessive axle loads. As traffic levels increase, the specification for roadbases should approach those of the traditional design charts. The experience gained during the study indicates that this change of function occurs at traffic levels around 500,000 esa. *ORN 31* (TRL 1993), or other relevant design guides, can be used at higher traffic levels. In the proposed design charts, the transition between the new designs and those given in *ORN 31* have been smoothed to provide an appropriate transition. The studies have shown that roads built with laterites and calcretes can carry particularly heavy traffic loadings, and these provide important exceptions to the above principle.

Low volume roads serving functions where particularly heavy flows result need to be considered differently. For example, roads serving a specific ‘heavy’ industry, such as a mining operation, may require that the roadbase specifications are tightened, or that the next higher traffic category can be used for design to reduce risks.

It was noted earlier that deterioration on low volume sealed roads is controlled mainly by the environmental factors rather than traffic. Thus, the thickness designs and material specifications have been devised to mitigate this. A standard sub-base layer has been provided for all designs to protect the weaker subgrades from environmental deterioration, even for low traffic levels. A gradual increase in pavement thickness has been used to provide a transition to the thickness required at the higher traffic levels.

5.6 Specific materials issues

5.6.1 Laterites

Lateritic gravels are the product of intensive tropical weathering of the parent rock and continued leaching of the initial weathering products of the rock (clays). This continual weathering and leaching results in the solution, residual accumulation and precipitation of iron and aluminium rich weathering products in distinct horizons. To develop a concretionary (hardened) deposit of laterite, chemical precipitation and loss of water of crystallisation is required. These conditions generally arise in areas of fluctuating groundwater level. Lowering of the groundwater leads to oxidising conditions, whereby both precipitation of the hydrated oxides and dehydration occur.

Lateritic gravels are widespread throughout the northern reaches of the region. True laterites self-harden irreversibly on exposure to air, but the vast majority of the lateritic gravels, and certainly all of those occurring in Southern Africa, do not possess this capability. They do however seem to perform extremely well as roadbase materials, even though the majority of the deposits fail to meet at least one of the normal design criteria required in the specifications.

Only sporadic use has been made of lateritic gravels as roadbase materials for sealed roads in the region. This is mainly because they exhibit tremendous variability in their engineering characteristics, both between deposits and within the same deposit. The lateritic gravels found in the region commonly exhibit gaps in the grading curve, such as in the sand fraction. They also tend to have high plasticity, with plasticity indices greater than 15, and soaked CBR values lower than the minimum of 80 per cent normally specified. Most lateritic gravels are therefore considered sub-standard, and are generally precluded from use as roadbase materials even for low volume roads. Other more expensive options are normally used to provide roadbase layers in these areas. These options include hauling other natural gravels, which meet the specifications, over long distances; stabilising the lateritic gravels with cement and lime, which is the preferred option in Zambia and Zimbabwe; or using crushed stone for the base, which is the preferred option in Malawi. All of these options can be prohibitively expensive, particularly for low volume roads.

However, the study has shown that some of these 'sub-standard' lateritic gravels can be successfully used as roadbase materials for low to medium traffic levels, and guidelines for their selection are provided.

5.6.2 Calcretes

The results of the joint research project set up between TRL and the Roads Department of the Ministry of Works and Communications of Botswana led to the development of revised specifications for calcrete materials in dry climatic areas. The research is summarised in Appendix C.

5.6.3 Basalts

Gravels derived from basalts and dolerites are often weathered and may release additional plastic fines during construction or in service. The problems worsen if water gains entry into the pavement and this can lead to rapid and premature failure. Although traffic can exacerbate the problem, low volume roads can also suffer the adverse effects because of the release of expansive clay fines into the pavement. Even large, apparently sound, particles may contain minerals that are prone to rapid decomposition and volume change. The weathering environment in the road can lead to continued decomposition of minerals which can eventually invalidate the design tests (Weinert 1980). Care is needed in the selection of these materials, particularly where they appear fresh or slightly weathered, as these are often more susceptible to rapid weathering than the more weathered gravels. The state of decomposition also affects the long term durability when stabilised with lime or cement.

Normal aggregate tests are often unable to identify unsuitable materials in this group, and other methods are needed. These include:

- Petrographic analysis to detect secondary (clay) minerals
- Various chemical soundness tests, such as the sodium or magnesium sulphate (BSI 1990)
- Dye adsorption tests (Sameshima and Black 1979)
- Modified Texas ball mill test (Sampson and Netterberg 1989), which was developed in the region

Experience in the region with basaltic materials on low volume roads varies. In some cases, poor performance is reported where the materials have actually met the local specifications. It has been suggested that, in wet areas, a plasticity index of 6 may even be too high. In this study, the road between Kazungula and Kasane had a weathered basalt roadbase with a plasticity index of 11 and a soaked CBR of 55 per cent. This performed reasonably well and has carried around 100,000 esa. The area has been classified climatically as seasonally wet (N=2-4). Several factors are considered to have contributed to its good performance. These include good drainage, adequate crown height, the quality and integrity of surfacing, good compaction, and the moderate to complete weathering of the material. However, the road does not have sealed shoulders.

5.6.4 Problem subgrades

There is little data from this study covering very weak subgrades (soaked CBR[2]) and problem soils. No recommendations are made in this area. The design recommendations given in *ORN 31* or other relevant guides should be followed without any reduction in standards if expansive, dispersive or other geotechnically difficult soils are encountered. This principle has been adopted in the development of the design charts, such that the transition from the 'old' designs for weaker S2 (CBR of 3-4 per cent) soils to the 'new' designs has been smoothed.

5.7 Pavement materials specifications

5.7.1 Information sources

The materials design characteristics recommended for use with the design chart have been developed using a number of information sources in addition to the data from the road sections investigated in this study. These include:

- TRL's *ORN 31*
- AASHTO specifications for natural gravels
- Experimental pavements investigated by TRL and others in the region
- Other sources of information such as the CIRIA (1988) report on laterites
- Many other world wide specifications, most importantly from Australia, South Africa and Brazil

Material properties used in the design charts are assigned based on traffic level and climate.

5.7.2 Roadbase specifications

The requirements for roadbase have been developed using the natural gravel groups which are most predominant in the region. These include quartzitic gravels, weathered rocks, lateritic gravels, sands and calcrete. Some materials, such as calcretes and laterites, are identified as special cases where special recommendations can be made. Weathered basalt materials are also subject to special treatment.

Roadbase properties have been set at values which are more conservative than those observed during the study. This is because there were insufficient combinations of subgrade and traffic in the test sections to enable an approach based on percentile values to be used. The design values therefore offer a low risk approach. As further data become available, it may be possible to relax these guidelines further in the future.

The principles of the roadbase selection are based on the following:

- Traffic and climate
- Roadbase strength
- Grading envelopes
- Plasticity

Traffic and climate

The strength, plasticity and grading requirement varies depending on the traffic level and climate.

Roadbase strength

The soaked CBR test has been used to specify the minimum base material strength. This has assumed a test compaction requirement of 98 per cent BS 4.5kg rammer compaction, or equivalent, with a minimum soaking time of four days or until zero swell is recorded.

Grading envelopes

Four grading envelopes (A, B, C and D) are used which depend on the traffic and subgrade design class.

Envelope A varies depending on the nominal maximum particle sizes of 37.5, 20 and 10mm. It has been derived using *ORN 31*, *AASHTO M147-65* and the MoTE (Zimbabwe) recommended grading envelopes.

The lower limit for Envelope B is the same as the lower limit of the Envelope A with a 37.5mm maximum particle size, and the upper limit corresponds to the upper boundary of the Envelope A with a 10mm maximum particle size. This wider envelope allows use of a much wider range of natural gravels including the more commonly gap-graded materials such as laterites and ferricretes. A requirement for 5 to 10 per cent retention on successive sieves may be specified to prevent excessive loss in stability.

Envelope C applies only to dry ($N > 4$) climates, and extends the upper limit of Envelope B to allow the use of calcareous and Kalahari sands.

Envelope D provides a basic gravel wearing course specification. This is specified in terms of grading modulus (GM) with a range of 1.5-2.5.

Plasticity

The maximum plasticity index of the roadbase also depends on the traffic and subgrade design class. A maximum plasticity index of 6 has been retained for higher traffic levels and where the road is to be constructed over a weak subgrade. For arid and semi-arid environments, the plasticity index can be increased by three units, and the plasticity modulus increased by 40 per cent.

The limit of the plasticity index for laterite and calcrete gravels may be increased by 40 per cent up to a limit of 18 for wet areas and 21 for arid and semi-arid areas.

5.7.3 Sub-base materials

Insufficient data were available from the study to confirm whether or not the sub-base requirements could be relaxed. There is a particular need to ensure that the subgrade has adequate protection because of the important impact on deterioration of the environment. The normal quality standards for sub-base were therefore retained. These are to use a soaked CBR of 30 per cent at 95 per cent BS 4.5kg rammer compaction, or equivalent maximum dry density, and the normal grading requirements.

5.7.4 Selected fill

The requirement for selected fill is a soaked CBR of 15 per cent, using 95 per cent BS 4.5kg rammer compaction, or equivalent maximum dry density, or a minimum CBR of 15 per cent at the highest anticipated field moisture condition at the specified field density.

5.8 Environment and maintenance

The purpose of a pavement is to protect the natural ground, or subgrade, from the high and concentrated load stresses applied to the subgrade by the wheels of vehicles. Layers of material are provided to reduce these stresses and to distribute them evenly throughout the pavement so that traffic can be supported for as long as required. The principal elements in the design process are the choice of materials and their thickness for each pavement layer. The design engineer also needs to understand all other external impacts on the design, and to recognise the influence exerted by these other parameters.

An aim of pavement design is, therefore, to limit the level of pavement distress caused by environment or traffic to predetermined values. These values are set with reference to a suitable remedial treatment being provided at the end of the design period. It is assumed that strengthening is carried out at this time. It is also assumed that adequate maintenance is carried out during the design period of the road.

The road environment will also influence this interaction, as discussed earlier, and the pavement design process must therefore recognise and deal with this in the context of any particular road design project. Experience in the region on low volume roads is that, where a timely re-seal has been carried out, this will arrest environmental deterioration. Also, as the surfacing becomes thicker as a result of re-sealing, it will start to act as a semi-structural layer, thus reducing stresses lower in the pavement. This can prolong the serviceability of the pavement well beyond its normal design life. Clearly, this observation requires more research, but it is considered at this stage that the benefits resulting could become an important component of the whole life costing of low volume roads.

6. The design of low volume roads

6.1 Design process for low volume roads

6.1.1 The road design process

The design method takes account of local conditions of climate, traffic and other environmental conditions to enable a relaxation from the traditional traffic dependent thickness and materials standards which would otherwise be used to achieve a more cost-effective approach for these low volume roads. The design charts are based on standard laboratory-based tests to classify subgrade materials. Subgrade strengths are related to different climate and drainage conditions. Natural gravel materials are used for roadbase and sub-base layers. The design process needs to take consideration of the series of steps outlined in Box 6-1.

Box 6-1 Summary of road design process

Feasibility study

Assumptions and results at this stage should detail:

- The road function
- Implications of the topographic, climatic and environmental area
- Application of an appropriate design standard (eg whether new road or upgrading existing gravel roads)
- Economic and cost parameters



Design

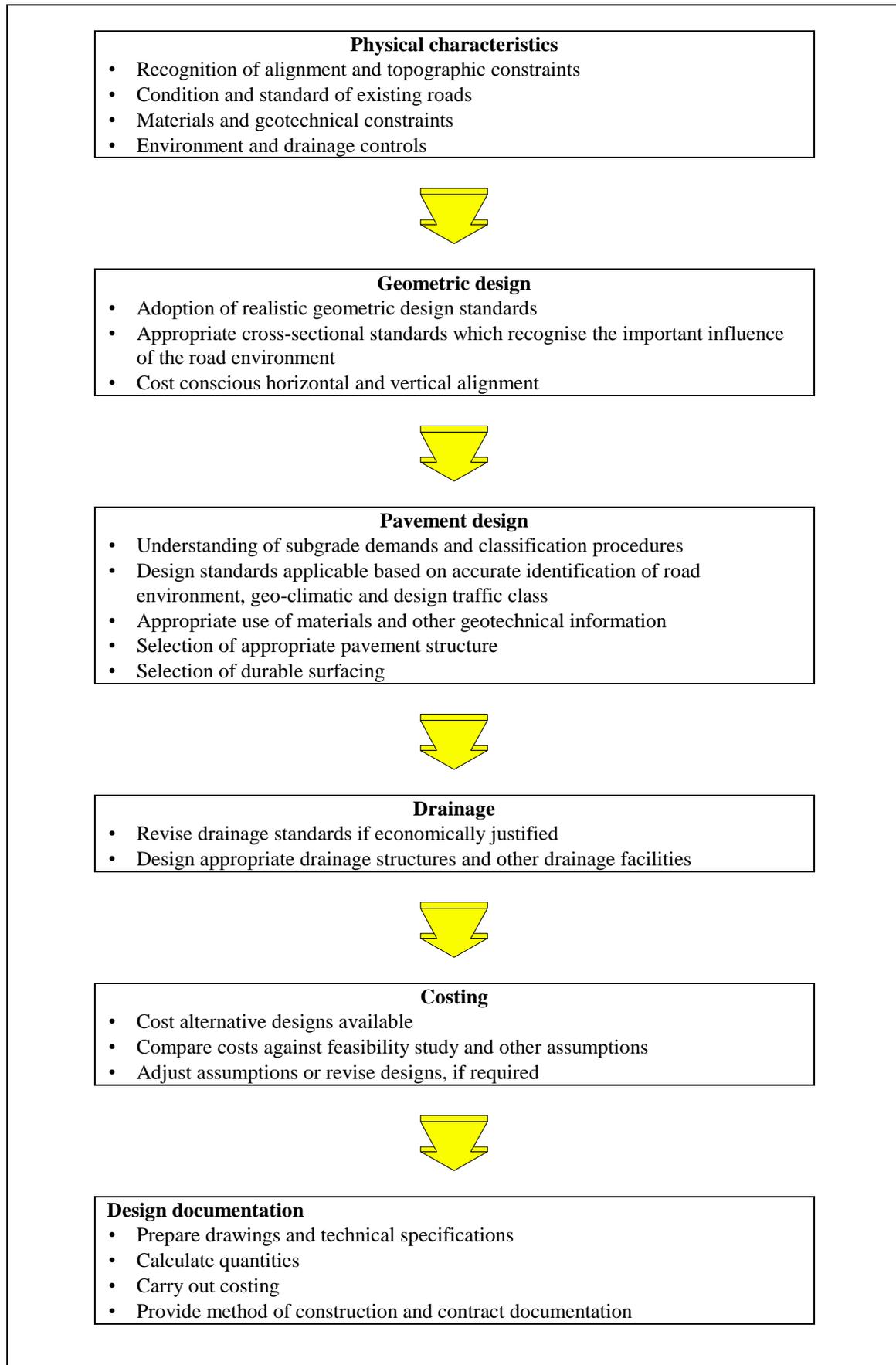


Traffic

- Reliable baseline traffic data
- Accurate and realistic forecasts of traffic and axle loading



/continued



The procedure developed is not intended as a prescriptive manual or specification, but has the purpose of expanding the options available to engineers, providing guidance on the approach to design, and encouraging a more rational and economically viable approach to the provision of low volume roads. It is intended that the procedures recommended should be flexible enough to be updated or revised as new techniques and developments become available or where local conditions or experience indicate that revisions, modifications or improvements can be made.

The judgement of the design and construction engineers are fundamental to the success of this approach and the procedures presented recognise the importance of the engineers input in the process. Engineering design procedures for high volume/trunk roads are fairly well defined and understood and the procedures for successful implementation are relatively straight forward. To build sealed low volume roads, the design engineer is required to balance costs against the design requirement. This requires that design engineers have suitable experience, and have an open minded and flexible attitude. Their approach needs to balance the risks and benefits of the design process within the economic and financial constraints.

6.1.2 Traffic and axle load estimates

Accurate data on the annual average daily traffic (AADT) using or likely to use the road is a prerequisite to the design. Vehicle types should be categorised and the proportion of heavy vehicles, including buses, determined. Seasonal variations in the traffic movements, as is often the case on low volume roads servicing rural areas, should be recognised in the count procedures. To reduce uncertainty in the traffic forecasting seven day, 24 hour classified traffic counts should be carried out several times during a year if possible. A full resume of traffic counting procedures and methods of analysis appropriate to these roads is given in *ORN 31*.

As the design procedure demands the determination of design life it is necessary to acquire accurate data on the configuration and equivalence factors of vehicles using, or likely to use the road. There is no reliable alternative to carrying out axle load surveys on or close to the route under consideration as any errors in axle loads are amplified by the fourth power law relationship when calculating cumulative equivalent standard axles. Documents such as *TRL Road Note 40* (Transport and Road Research Laboratory 1978) provide suitable guidance on carrying out and analysing data from these surveys.

Over-estimation of the traffic growth can lead to adoption of unnecessarily conservative and expensive road designs. Selection of realistic flow levels and growth rates will ultimately enable considerable cost savings to be made. Judicious and careful consideration should therefore be given to all the available information.

6.1.3 Identifying the characteristics of the road environment

Several factors influence how the local environment will interact with the proposed road structure. These include geometry, alignment, topography, materials, effectiveness of maintenance strategies, geotechnical information and, most importantly, drainage. These factors can have a significant impact on costs, either during construction, or during subsequent

operation because of the maintenance demands of the road. The type and characteristics of the pavement structure proposed by the design could alter the impact of these factors.

Many sources of information are available to investigate these factors, including maps, aerial photographs and other relevant recorded information. These should all be consulted by the design engineer. Information on drainage problems experienced on the existing road can, for instance, be obtained from maintenance records. These can identify inadequate sizes of culverts, weak pavement spots, and other deficiencies. In general, good meteorological information is available in the region, and the historical records from these should be consulted.

6.2 Subgrade design class

Subgrades are classified on the basis of laboratory soaked CBR tests on samples compacted to 100 per cent BS 2.5kg rammer, or equivalent, maximum dry density and optimum moisture content. Samples are soaked for four days or until zero swell is recorded. The design CBR, determined as above, is used to assign a design subgrade class, as in Table 6-1.

Table 6-1 Subgrade classes

Class	Design CBR	Notes
S6	30	May be used in all fills and as sub-base layer if the upper 150mm of the layer or the sub-base layer is compacted to 95% BS 4.5kg rammer compaction
S5	15-29	May be used in all fills and as selected fill layer; the selected fill is usually compacted to 95% BS 4.5kg rammer compaction
S4	9-14	May be used in all fills
S3	5-8	May be used in all fills
S2	3-4	May be used in fills not exceeding 2m in height

6.3 Traffic design class

The design method is based on the bands of cumulative esa shown in Table 6-2.

Table 6-2 Design traffic classes

Traffic class	Range of cumulative equivalent standard axles
0.01M	Less than 10,000
0.05M	10,000 to 50,000
0.1M	50,000 to 100,000
0.3M	100,000 to 300,000
0.5M	300,000 to 500,000
1M	500,000 to 1 million
3M ⁽¹⁾	1 to 3 million
<i>Note:</i>	
(1) This design class follows the same standards as <i>ORN 31</i> and is included in the design charts for completeness	

6.4 Pavement design strategy

The approach to pavement design is outlined in Figure 6-1. This shows the sequence of steps that are required to produce a pavement design that is appropriate and adequate for an individual road.

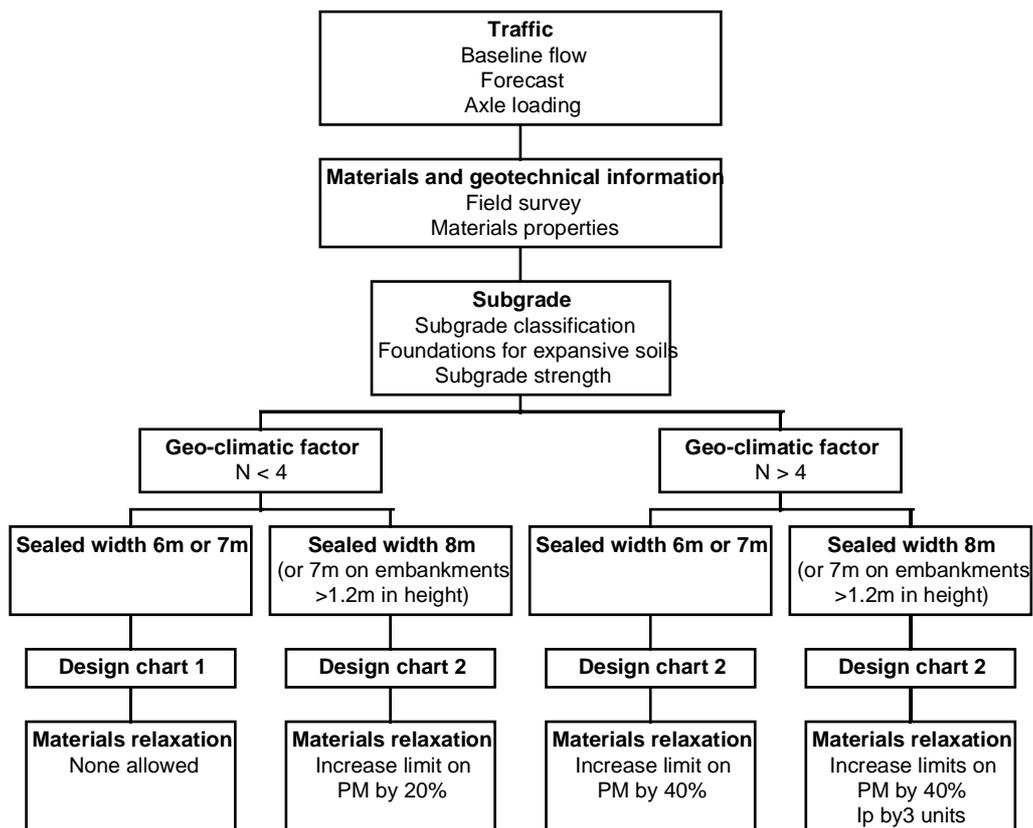


Figure 6-1 Flow chart for sealed road pavement design process

6.5 Subgrade

6.5.1 Subgrade class

It is essential that results from the materials and geotechnical investigations are evaluated in all cases, since these have important implications for the input parameters to the pavement design charts. It is also important to know the availability and quality of pavement materials, as well as haul distances, so that the options provided by the design charts are fully utilised.

The subgrade CBR will vary along the line of the road being designed. The actual subgrade values used for the design will depend on the design traffic class, as shown in Table 6-3.

Table 6-3 Dependence of design subgrade values on design traffic class

Design traffic class	Design CBR
< 0.3M	Mean CBR
0.3M - 0.5M	Lower quartile CBR
1M	Lower decile CBR

6.5.2 Treatment of subgrade

Subgrade materials will require different treatments to form a satisfactory foundation for the pavement. The treatments necessary for subgrade preparation are listed in Table 6-4. Codes have been assigned to the treatment groups for use in the pavement design and construction.

Where expansive soils exist, perhaps for only part of the route, then the foundation treatment required should follow the procedures given in the national design standards for all roads.

For the lowest design standard (less than 10,000 esa), the existing gravel wearing course can be utilised as roadbase, sub-base or selected fill. In this situation, the gravel wearing course should be removed and stockpiled, ensuring that there is no contamination from the lower layers. The underlying layers can then be treated as appropriate to the respective subgrade design class. The stockpiled gravel wearing course can be re-used as roadbase, sub-base or selected fill, as appropriate. Imported material of a suitable standard may have to be added and mixed into the gravel wearing course to form the roadbase.

Table 6-4 Subgrade preparation

Code	Subgrade treatment required
T5	S5 materials or better (selected fill and sub-base quality materials): (1) Remove topsoil, scarify and compact road bed to a minimum density of 100% BS 2.5kg rammer or equivalent compaction (2) Construct subgrade and compact to a minimum density of 100% BS 2.5kg rammer or equivalent compaction (3) Ensure that material in the top layer complies with the pavement design requirements and compact to a minimum density of 95% BS 4.5kg rammer or equivalent compaction at optimum moisture content
T3-4	S3 and S4 materials: (1) Remove topsoil, scarify and compact roadbed to a minimum density of 100% BS 2.5kg rammer or equivalent compaction (2) Construct subgrade and compact to a minimum density of 100% BS 2.5kg rammer or equivalent compaction (3) Ensure that material in the top layer complies with the pavement design requirements and compact to a minimum density of 100% BS 2.5kg rammer or equivalent compaction
T2	S2 materials: (1) Remove topsoil, scarify and compact roadbed to a minimum density of 100% BS 2.5kg rammer or equivalent compaction (2) Construct subgrade and compact to a minimum density of 100% BS 2.5kg rammer or equivalent compaction

6.6 Weinert N-value

The appropriate pavement design differs depending on the climatic zone in which the road is situated. These zones are characterised by Weinert N-values. Typically, N-values of less than four imply a climate that is seasonally tropical and wet; whereas N-values of greater than four imply a climate that is arid, semi-arid, or dry. Figure 6-2 provides a guide to these zones in Southern Africa. An important output from the materials investigation should be the establishment of the appropriate N-value.

6.7 Design charts

6.7.1 The design process

The pavement and materials design adopted, depends on the climatic zone and on the shoulder design, as illustrated in Figure 6-1. The process utilises two pavement design charts, which are shown with the key chart in Table 6-5 to Table 6-7.

Table 6-5 Key to structural catalogue

Traffic classes (10 ⁶ esa)	Subgrade strength classes (CBR%)
<0.01 = < 0.01	S2 = 3 , 4
0.05 = 0.01 - 0.05	S3 = 5 - 7
0.1 = 0.05 - 0.1	S4 = 8 -
0.3 = 0.1 - 0.3	S5 = 15 - 29
0.5 = 0.3 - 0.5	S6 = 30
1 = 0.5 - 1	
3 = 1 - 3	

Material Definitions

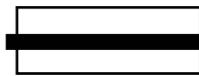
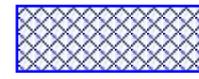
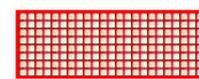
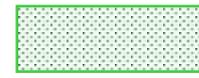
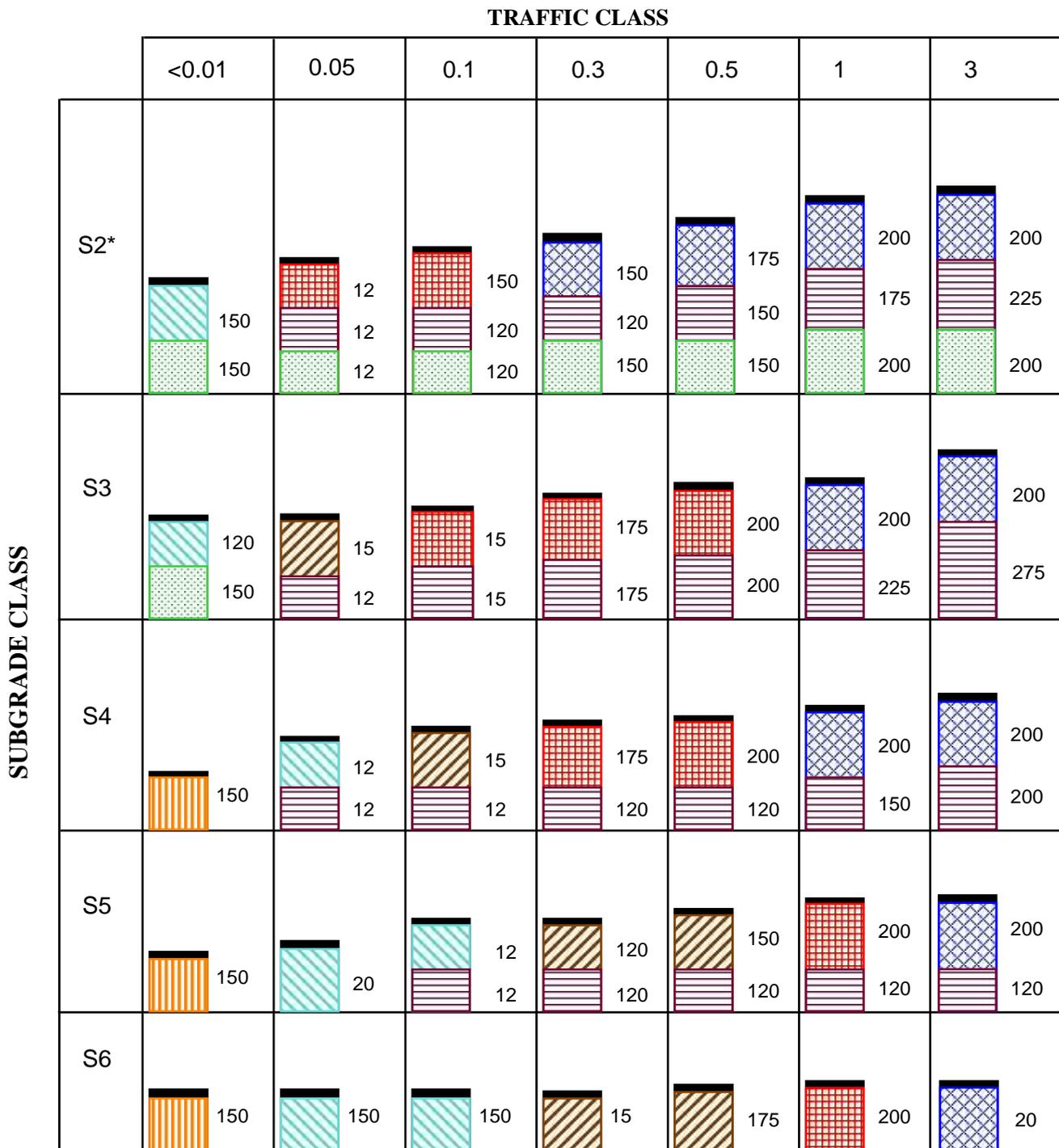
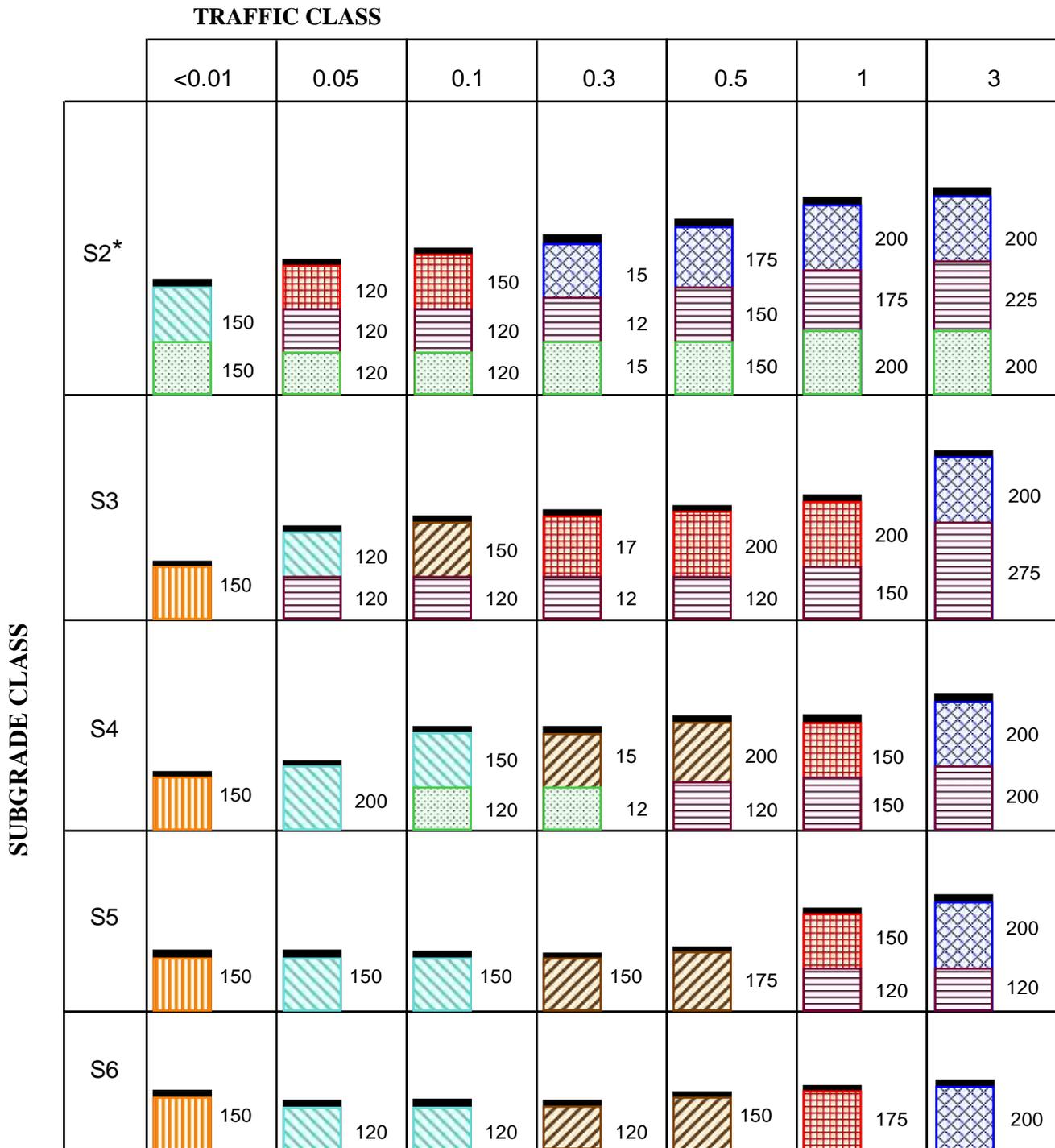
-  Bituminous surfacing
-  Base, CBR 80
-  Base, CBR 65
-  Base, CBR 55
-  Base, CBR 45
-  Gravel wearing course
-  Sub-base, CBR 30
-  Selected subgrade fill, CBR 15

Table 6-6 Pavement design Chart 1 (N<4)



Note: * Non-expansive subgrade

Table 6-7 Pavement design Chart 2 (N>4)



Note: * Non-expansive subgrade

On total sealed widths of seven metres or less, the outer wheel-track is within one metre of the edge of the seal. This affects pavement performance adversely, because of seasonal moisture ingress, so relatively stronger pavements are necessary in these situations. If the road width is sufficient for the outer wheel to be more than 1.5 metres from the pavement edge, and good drainage is ensured by maintaining the crown height at least 750mm above the ditch, an improvement in performance results. This is reflected in the charts, where different sealed surface widths are treated separately.

When a road is on an embankment of more than 1.2 metres in height, the material in the roadbase and sub-base stays relatively dry, even in the wet season. In this case, the design category can be relaxed, and a pavement with a seven metre total sealed width can be designed to the same criteria as an eight metre seal.

The use of a wider sealed cross-section in climatic zones where $N < 4$ (likely to be relatively wet environments) allows a shift from Chart 1 to 2. This allows the use of thinner pavement layers and a relaxation of the quality requirements for roadbase. In climatic zones where $N > 4$ (likely to be drier climates), it can be assumed that the subgrade strength requirement will approximately be halved, as described earlier.

The following design options are used.

1. *Climatic zones where $N < 4$:*

- Where the total sealed surface is 6 metres or 7 metres
 - use Chart 1
 - no roadbase materials relaxations are allowed
- Where the total sealed surface is 8 metres, or 7 metres when the pavement is on an embankment in excess of 1.2 metres in height
 - use Chart 2
 - the limit on the plasticity modulus of the roadbase may be relaxed and increased by 20 per cent
 - if the engineer deems that other risk factors (eg maintenance, construction quality, etc) are too high or uncertain, then Chart 1 should be used

2. *Climatic zones where $N > 4$:*

- Where the total sealed surface is less than 8 metres
 - use Chart 2
 - the limit on the plasticity modulus of the roadbase may be increased by 40 per cent
- Where the total sealed surface is over 8 metres and when the pavement is on an embankment in excess of 1.2 metres in height
 - use Chart 2
 - relaxations in roadbase quality are allowed by increasing the limit on the
 - plasticity modulus by 40 per cent
 - plasticity index by 3 units

The design flow chart in Figure 6-1 should be used iteratively depending on conditions on the individual project as in the following examples:

- Once the quality of the available materials and haul distances are known, the flow chart and the design charts can be used to review the most economical cross-section and pavement; this would involve assessment of design traffic class, design period, cross-section and other environmental and design considerations
- It may be more economical to use a wider cross-section in the seasonal tropical and wet climate zone, and then shift to Chart 2 than to design a narrow cross-section and a pavement using Chart 1

6.7.2 Reducing risks in special cases

When the project is located close to the border between the two climatic zones, the lower N-value should be used to reduce risks.

When close to the borderline between two traffic design classes, the higher traffic class should be used. In the absence of reliable data, the next highest design class should be considered. If the road is expected to carry unusually heavy loads, for example from industries such as sawmills, mines, and the like, it may be prudent to adjust the design class upwards, to reduce risks.

6.8 Selected fill

The requirement for wet climates is a soaked CBR of 15 per cent at 95 per cent BS 4.5kg rammer or equivalent compaction. For dry climates, the requirement is for a minimum CBR of 15 per cent at the highest anticipated field moisture condition, and at the specified field density.

6.9 Sub-base materials

The selection of natural gravel sub-base materials should follow the local design standards or those given in *ORN 31*.

6.10 Roadbase materials specifications

6.10.1 Particle size distribution

The grading envelopes to be used for the roadbases are shown in Table 6-8. Envelope A varies depending whether the nominal maximum particle size is 37.5mm, 20mm or 10mm. A requirement of five to ten per cent retained on successive sieves may be specified at higher traffic (>0.3M esa) to prevent excessive loss in stability. Envelope C extends the upper limit of envelope B to allow the use of sandy materials, but its use is not permitted in wet climates. Envelope D is similar to a gravel wearing course specification, and is used for very low traffic volumes. The grading is specified only in terms of the grading modulus (GM) and can be used in both wet and dry climates.

Table 6-8 Particle size distribution for natural gravel roadbases

Test sieve size	Per cent by mass of total road aggregate passing test sieve				
	<i>Envelope A</i>			<i>Envelope B</i>	<i>Envelope C</i>
	Nominal maximum particle size				
	37.5mm	20mm	10mm		
50mm	100	-	-	100	-
37.5mm	80-100	100	-	80-100	-
20mm	55-95	80-100	100	55-100	-
10mm	40-80	55-85	60-100	40-100	-
5mm	30-65	40-70	45-80	30-80	-
2.36mm	20-50	30-55	35-75	20-70	20-100
1.18mm	-	-	-	-	-
425µm	8-30	12-30	12-45	8-45	8-80
300µm	-	-	-	-	-
75µm	5-20	5-20	5-20	5-20	5-30
<i>Envelope D</i> 1.65 < GM < 2.65					

6.10.2 Strength and plasticity requirements

The strength and plasticity requirement varies depending on the traffic level and climate, as outlined in Table 6-9. The soaked CBR test is used to specify the minimum base material strength.

A maximum plasticity index of 6 has been retained for higher traffic levels, where the design chart merges to standard design documents, and also on weaker subgrades. For designs in dry environments, the index can be increased by a value of 3, and the plasticity modulus by 20 and 40 per cent, depending on the crown height and whether unsealed or sealed shoulders are to be used.

Table 6-9 Selection of natural gravel roadbase materials

Subgrade class	Property	Upper limit of design traffic class						
		0.01M	0.05M	0.1M	0.3M	0.5M	1M	3M
S2	I _p	(12	(12	(9	(6	(6	(6	(6
	PM	400	250	150	120	90	90	90
	Grading	B	B	B	A	A	A	A
S3	I _p	(15	(12	(12	(9	(6	(6	(6
	PM	550	320	250	180	90	90	90
	Grading	C ⁽¹⁾	B	B	B	A	A	A
S4	I _p	Note (2)	(15	(12	(12	(9	(9	(6
	PM	800	450	320	300	200	90	90
	Grading	D ⁽³⁾	B	B	B	B	A	A
S5	I _p	Note (2)	(15	(15	(12	(12	(9	(6
	PM	n/s	550	400	350	250	150	90
	Grading	D ⁽³⁾	C ⁽¹⁾	B	B	B	A	A
S6	I _p	Note (2)	(18	(15	(15	(12	(9	(6
	PM	n/s	650	550	500	300	180	90
	Grading	D ⁽³⁾	C ⁽¹⁾	C ⁽¹⁾	B	B	A	A

Road base CBR (%)	Max swell (%)
45	0.5
55	0.3
65-80	0.2

Notes:

(1) Grading 'C' is not permitted in wet environments or climates (N<4); grading 'B' is the minimum requirement

(2) Maximum I_p = 8 x GM

(3) Grading 'D' is based on the grading modulus 1.65 < GM < 2.65

- All base materials are natural gravels
- Subgrades are non-expansive
- Separate notes are provided covering the use of laterites, calcretes (N>4) and weathered basalts

I_p Plasticity index
PM Plasticity modulus
n/s Not specified

6.10.3 Lateritic roadbase gravels

The guidelines for selection and use of lateritic gravels for bases are slightly different to those given for other natural gravels. These are presented in Table 6-10. The maximum plasticity index of the lateritic roadbase is also relaxed in comparison to Table 6-9. A maximum plasticity index of 9 has been specified for higher traffic levels and weak subgrades. For design traffic levels greater than 300,000 esa, a requirement is set that the liquid limit should be less than 30. Below this traffic level, this requirement is relaxed to a liquid limit of less than 35. For traffic classes above one million esa, the selection properties are the same as for other natural gravels, as given in Table 6-9. Where sealed shoulders over one metre wide are specified in the design, the maximum plasticity modulus may be increased by 40 per cent. A minimum field compacted dry density of 2.0Mg/m³ is required for these materials.

Table 6-10 Guidelines for the selection of lateritic gravel roadbase materials

Subgrade CBR		Design traffic class					
		(0.01)	0.05	0.1	0.3	0.5	1.0
S2	I _p	(15	(15	(12	(9	(9	(6
	PM	(400	(250	(150	(150	(120	(90
	GE	B	B	B	A	A	A
S3	I _p	(18	(15	(15	(12	(9	(6
	PM	(550	(320	(250	(180	(120	(90
	GE	B	B	B	B	A	A
S4	I _p	(20 ⁽¹⁾	(18	(15	(15	(9	(9
	PM	(800	(450	(320	(300	(200	(90
	GE	GM 1.6-2.6	B	B	B	B	A
S5	I _p	(25 ⁽¹⁾	(20	(18	(15	(12	(9
	PM	n/s	(550	(400	(350	(250	(150
	GE	GM 1.6-2.6	B	B	B	B	B
S6	I _p	(25 ⁽¹⁾	(20	(20	(18	(15	(12
	PM	n/s	(650	(550	(400	(300	(180
	GE	GM 1.6-2.6	B	B	B	B	A
<i>Notes:</i>		I _p = plasticity index					
(1) I _p maximum = 8 x GM		PM = plasticity modulus					
n/s = not specified		GE = grading envelope					
Unsealed shoulders are assumed		GM = grading modulus					

The following should also be noted:

- All the roadbase materials from the test sections with lateritic materials were weaker than those specified in Table 6-10
- Further modification to the limits can be made if the shoulders are sealed
- The compaction requirement for the soaked CBR test is 100 per cent BS 4.5kg rammer or equivalent compaction with a minimum soaking time of four days or until zero swell is recorded; this is a relaxation of the soaked CBR requirement for natural gravel base materials given in Charts 1 and 2

6.10.4 Calcrete roadbase gravels

Specifications for the use of calcrete roadbases are given in Table 6-11. These are based on a combination of the projected traffic which will produce an 80th percentile value rut depth of 20mm, the engineering properties of the materials, and other environmental factors known to influence performance. Two intermediate traffic design classes have been introduced. It must be noted that these recommendations apply only to S6 subgrade design class and N>4 climatic regions. In other situations (N<4), or where weaker subgrades are encountered, the recommendations given in Table 6-6, Table 6-8 and Table 6-9 should be followed.

The compaction requirement for the soaked CBR test is 100 per cent BS 4.5kg rammer or equivalent compaction. This is a relaxation of the soaked CBR requirement for natural gravel base materials given in the Charts 1 and 2, where the CBR requirement is set at 98 per cent BS 4.5kg rammer. The minimum soaking time is four days or until zero swell is recorded.

Table 6-11 Recommended specifications for calcrete roadbases

	Maximum traffic (esa x 10 ⁶)				
	<0.3	0.5	0.7	1.0	1.5
Maximum particle size (mm)	75	75	75	75	75
Max % passing 425µm sieve	80	65	65	45	30
Max % passing 63µm sieve	30	30	25	20	15
Liquid limit (maximum)	60	55	50	40	30
Plasticity index (maximum)	25	20	15	12	10
Maximum linear shrinkage (LS) (%)	12	12	8	6	5
LS x % passing 425µm sieve (max)	800	700	550	400	200
LS x % passing 63µm sieve (max)	300	300	300	200	100
Minimum soaked CBR ⁽¹⁾	40	50	60	60	80

Notes:
 (1) At 100 per cent BS 4.5kg rammer or modified AASHTO compaction

6.10.5 Basaltic gravels

More work is required before these materials can be used with confidence. The following indicative limits can contribute to successful use of the material:

- Maximum secondary mineral content of 20 per cent (determined from petrographic analysis)
- Maximum loss of 12 or 20 per cent after 5 cycles in the sodium or magnesium sulphate soundness tests, respectively
- Clay index of less than 3 in the dye absorption test
- Durability mill index of less than 125

In drier climatic areas ($N > 4$), the materials can be used unmodified up to a maximum plasticity index of 10. However, it is suggested that the materials should not be used in wet areas unless chemically modified. The risk of using the material can be minimised if consideration is given to:

- The variability of the material deposit, with good selection and control procedures in place for the operation of the pit and on site
- The provision of good drainage conditions (these materials are particularly sensitive to moisture)
- The adequacy of the pavement design (the use of Chart 2 with sealed shoulders is suggested)
- The use of double surface treatments

Engineers need to use considerable judgement, and experience and information from other roads in the area to utilise these materials successfully. Risks must be identified and controlled.

6.11 Sealing gravel roads

There are a number of examples in the region where roads constructed to gravel standard have subsequently had a bituminous sealed applied and the gravel wearing course used as roadbase.

Some of the sections investigated in this study were situated on the Kamativi-Binga road, which is an example of this type of approach in Zimbabwe. These roads have generally performed well despite the roadbase being constructed to gravel wearing course standards.

An option has been included in the designs for this type of approach for traffic up to 10,000 esa (0.01M). There is anecdotal evidence that other roads constructed by this method in the region have carried higher traffic but this limit has been set in accordance with the quantitative evidence available in this study.

If the existing wearing course material conforms to subgrade class S5 or better, it is specified as roadbase. Many gravel road wearing courses easily meet this specification although the specified minimum thickness of 150mm may need the importation of additional material. It is important that adequate drainage is provided and a minimum crown height of 750mm is recommended. Material made available from clearing and deepening drains can be tested and, if suitable, used to make up the thickness of the base. The construction process involves scarrifying and placing the existing wearing course materials to one side, compacting the subgrade, replacing the gravel, compacting this layer and applying the bituminous surfacing.

This approach provides a sealed road from an existing gravel road at very low cost. In the SFRDP study in Zimbabwe, an analysis with HDM to determine the optimum traffic level for upgrading a gravel road to a paved road by this method indicated that it was less than 40 vehicles per day.

6.12 Sealed surfacings

The results of the study emphasised the importance of the surfacing in ensuring the durability and successful use of lower standard materials in other pavement layers. There are a number of surfacing options available which can be successfully used for low volume roads, and a comparison of the durability of commonly used seals in the region, in terms of expected service lives, is given in Table 6-12. The main factor influencing durability appears to be texture. The importance of this is attributed to the greater exposure to solar radiation in open textured seals, which increases the ageing process and hardening of the bitumen, resulting in reduced service life. The coarse graded Otta seals are generally less open-textured than equivalent conventional surface dressings and these are receiving increasing attention for low volume roads.

Table 6-12 Expected service lives from some surface seals (Department of State Roads/SweRoad, 1995)

Type of seal	Expected service life (years)
Single graded seal	7-8
Single graded seal plus sand seal	8-10
Double graded seal	10-12
Double graded seal plus sand seal	12-14
Single conventional seal	5-6
Single conventional seal plus sand seal	7-8

Double conventional seal	8-10
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Single applications of surface dressings or Otta seals have often been considered in the past for lightly trafficked roads. However, single seals in general are not considered to be durable and single surface dressings are particularly prone to environmental deterioration. Single graded or Otta seals are more durable because they contain more binder and produce a thicker closer textured carpet. Nevertheless, it is difficult to produce a good finish with single seals, they do not cover small surface defects well and any stone loss can lead to the accelerated development of potholes. This suggests that a double seal may be the most economic long term solution even for very low volume roads. Should a single seal be used, a reseal should be applied early in the design life, and certainly within five years of construction. On more heavily trafficked roads, with traffic loadings greater than about 100,000 esa, a double seal should always be used. Prime coats should be used in all cases.

Some surfacings perform better than others under different conditions. Guidance on selection is given in Table 6-13. Whichever surfacing type is selected, the specification should follow the recognised local or international standards.

Table 6-13 Selection criteria for different sealed surfacings

Surfacing type	Situations of use			
	Limited maintenance capability	Steep gradients (>10%)	Wet climate or poor drainage	Turning trucks (eg intersections)
Single conventional seal	No	No	Yes	No
Double conventional seal	Yes	Yes	Yes	Yes
Single graded seal	Yes	No	Yes	No
Double graded seal	Yes	Yes	Yes	Yes
Slurry seal	Yes ⁽¹⁾	No	Yes	Yes ⁽¹⁾
Cape seal	Yes	Yes	Yes	Yes
Sand seal	No	No	No	No
<i>Notes:</i> (1) Use only if slurry is thick				

Normally, mechanised construction methods will be used for sealed surfacing works. However, there are situations where the use of labour based or intermediate technology methods may be appropriate. The feasibility of using these technologies is related to the size of the project and to the speed required for construction. Labour-based surfacing methods can be considered for traffic loadings up to about 50,000 esa. At higher traffic levels, labour-based methods are generally considered to be impractical.

7. Regional experience of low volume roads

7.1 Information sharing

This chapter highlights some examples of techniques and other innovations which have been developed in the region in recent years. Not all of these will be appropriate to all countries, nor will they be appropriate in every project situation. There may also be other innovative developments that are not mentioned here. Regional staff should be encouraged share experiences in the use of innovative techniques and practices. Regional conferences, seminars and workshops, provide a particularly useful opportunity for the interchange of experience.

7.2 Regional research

7.2.1 Range of activities

In recent years, a number of organisations have been involved in investigations in Southern Africa with the aim of devising improved mechanisms of providing and operating roads at lower whole-life costs. This has involved studies of the appraisal, geometric and structural design, and the maintenance of low-volume roads. A number of external organisations have collaborated closely with Roads Departments in the region and have an active and strong interest in research. Innovative trials of new designs, practices, methods and materials have been implemented. It is unfortunate that many of the results of these trials remain within the country borders and they have not been documented fully or the information disseminated.

7.2.2 TRL

The work of the TRL in Southern Africa began in Botswana in the early 1980s with a DFID-funded project which demonstrated that large savings could be made from the use of local materials. Kalahari sand and calcareous gravels were investigated for their suitability as road construction materials, where conventional approaches prevented their use in the pavement structure (Greening and Rolt 1996). The results of this work have been incorporated, where appropriate, into the present study (Appendix C).

7.2.3 SweRoad

Recently, the Swedish International Development Agency (SIDA) funded the construction of 176km of secondary and feeder roads in Zimbabwe. TRL were involved with the consultants SweRoad to investigate the impact of alternative approaches to geometric design, the use of 'non-standard' materials in the pavement structure, and alternative bituminous surfacings, including graded seals. The results indicated that, amongst other things, it was economic to seal the project roads at traffic levels of between 30 and 40 vehicles a day. (SweRoad 1996). The results of this work have underpinned the conclusions developed in the current study.

7.2.4 NRRL

The Norwegian Road Research Laboratory (NRRL) pioneered the work on transferring the technology of using graded seals for the surfacing of low-volume roads to the region. Much of the initial work on low-cost seals was carried out on rural road projects in Botswana. NRRL also conducted trials on the use low-grade materials in the pavement structure (Botswana Roads Department 1992), and much of the information on this was consulted during this programme of research.

7.2.5 CSIR

Other supporting evidence on the need for more relaxed standards is available from work carried out by the Council for Scientific and Industrial Research (CSIR) in South Africa. The results of this research has been incorporated into a design document for low volume roads in South Africa (Department of Transport, 1989).

7.3 Investment appraisal

Investment appraisal for many road projects is undertaken by the use of models, such as the RTIM3 (Cundill and Withnall 1995) and HDM3 (Watanatada et al 1987). In the models, the benefits accrue largely from a reduction in road user operating costs, although other benefits can be added exogenously. The models incorporated reflect the conventional approach to design, construction and maintenance. The development of these models has made a major contribution to achieving a more rational approach to the investment appraisal of road projects.

However, traffic volumes on rural roads are relatively low, and the proportion of benefits from reductions in road user costs are less. The use on these roads of models where benefits are derived predominantly from savings in motorised transport costs is questionable. There is also a need to assess the costs and benefits associated with non-motorised transport and social factors. These are now being addressed through the development of HDM4 (Kerali et al 1998). There is undoubtedly a need for models to enable a more rigorous evaluation of other benefits that are more appropriate for low-volume road projects.

A number of studies currently being carried out and recently completed, including this one, could provide valuable data for regional calibration of the deterioration models used for low volume sealed roads in HDM4.

7.4 Geometric Design

This study has shown that standard pavement design procedures over-estimate traffic-related deterioration on low volume roads and the recommended relaxation in designs could result in large cost savings.

Geometric design can also have a significant impact on construction costs. On major roads, with relatively high volumes of traffic travelling at high speeds, it may be important that current standards for horizontal and vertical alignment are maintained to provide safe and economical travel. On rural roads, where traffic volumes and speeds are generally less, it is possible to adopt a more relaxed approach without compromising road safety. During the construction of feeder roads in Zimbabwe, as part of the *Secondary and Feeder Road Development Programme*, a

more flexible approach was adopted for geometric design which allowed deviation from standard parameters.

This flexibility allowed variation of the design speeds and steeper gradients. On some of these roads no formal alignment calculations were made, and road layout was undertaken using a simple 'design-by-eye' approach. This involved engineering judgement and minimum surveying. Maximum use is made of the existing formation and the amount of additional earthworks, and hence cost, is reduced. The Zimbabwe Ministry of Transport has adopted this 'low cost' approach on many secondary and feeder roads. This approach has application in many other parts of the region.

This study has shown that standard design procedures over-estimate traffic-related deterioration on low volume roads. Although limits must still be placed on minimum thickness design because of the possibility of the road becoming saturated and carrying increased heavy or very over-loaded trucks, applying the results of this study can result in large savings.

7.5 Construction

7.5.1 Marginal materials

It is now clear that the previous strength requirement for the use of naturally occurring gravels of a minimum soaked CBR value of 80 per cent, at 98 per cent BS 4.5kg rammer or modified AASHTO compaction, for the roadbase is excessively conservative for most rural roads in the region. The characteristics of locally available materials, combined with relatively low traffic and a predominantly dry climate, allows materials with lower strength to be used with confidence. It is important that appropriate standards for pavement materials are used for these low volume roads, both from the point of view of economics and to preserve non-renewable natural resources. Good gravels are becoming increasingly scarce in the region, and future upgrading or rehabilitation of rural roads may become prohibitively expensive as haul distances increase or stabilisation procedures have to be adopted.

Materials are only marginal in terms of the specification applied to them. The use of inappropriate standards has probably cost more money than the few failures associated with the use of marginal roadbases. Most failures can be attributed to factors such as poor construction, poor drainage, poor axle load control, and the like, rather than as a direct effect of using a marginal base.

7.5.2 Compaction

The current investigation has underlined the importance of achieving the specified compaction standard during construction. Achieving even higher levels of compaction than those normally specified for subgrade, sub-base and roadbase, could be a relatively cheap method of increasing the stiffness of the pavement and increasing serviceability. A new approach could be developed that uses a method specification in conjunction with adequate knowledge of the local materials. Such an approach has attractions for low volume roads because it reduces the need and volume of on-site testing required with the current end product specifications. The approach would be assisted by the wider use of correctly-calibrated nuclear moisture-density gauges.

Proprietary additives to stabilise materials and increase the levels of compaction, such as sulphonated petroleum products (SPPs), are now marketed widely in the region. If successful, these products could be beneficial for low volume road works. A joint investigation by TRL and CSIR of a range of these products, funded by DFID, is now underway. Use of proprietary additives should be treated with caution until the results of this investigation are known.

In recent years the Botswana Roads Department has also embarked on a number of trials to demonstrate the benefits of extending the use of local materials such as Kalahari sand in the pavement layers, and by using innovative construction techniques such as impact roller compaction (Pinard and Ookeditse 1989).

7.5.3 Sealing shoulders

Early work done in the region by CSIR showed that there is a whole-life benefit from reduced maintenance costs from the sealing of shoulders. The present study has now identified the structural benefits which are obtained from maintaining a drier environment under the running surface, even in the wet season. This also allows weaker materials to be used in the pavement layers. There are also road safety benefits from sealed shoulders, although these have not been quantified.

7.5.4 Low cost surfacings

A wide range of thin bituminous seals are now available for consideration. Of those available, much interest has been given to the use of the graded (*Otta*) seal. Its ease of application, low cost and proven durability, make it an attractive option for sealing rural roads, particularly where conventional sources of surfacing aggregate might be unavailable. *Otta* seals have been used successfully in Botswana, Zimbabwe and Kenya.

7.5.5 Improved drainage

Drainage was identified as one of the major controlling variables on the performance of low volume roads constructed with marginal materials. The provision of sealed shoulders can reduce the susceptibility to weakening of pavements by moisture ingress. However, there is still a possibility that moisture can get into the pavement from below the load-bearing layers, particularly in areas of cut. Deeper or lined side drains in cut, or other areas where moisture ingress could occur, offer a low cost precaution. In one project in Malawi, materials for the road embankment were obtained from within the road reserve. This approach can enable the provision of higher embankments, better crown height, deeper and wider drains, and improved drainage, at little extra cost. The resulting wider road reserve also provides additional benefits to road safety for motorists and other non-motorised road users.

7.5.6 Alternative drainage structures

It is often uneconomic to provide all-season access even on sealed roads in rural areas. One method, which has been adopted on feeder roads in Zimbabwe, is to provide a combination design of a culvert and drift, known as a vented causeway. These structures are expected to be

over-topped for short periods in the wet season, and to be impassable for even shorter periods during peak floods. These low cost structures are providing large savings on provision of river crossings.

7.6 Maintenance

Over the past few decades or so, limited resources have been available for road maintenance in many countries in the region. During this period, there was a reluctance by donors (with a few exceptions) and politicians in many countries to fund maintenance despite the risk of losing the large capital investment in road projects. The performance of many roads was adversely affected by poor maintenance and these have deteriorated more rapidly than they would have done with better maintenance. This is evident from the overall better standard of the road network in countries (eg Botswana, South Africa, Zimbabwe) which allocated more funds to maintenance. However, many roads, some included in this study, continued to perform adequately beyond their design period (usually 20 years) despite poor maintenance (eg no resealing for periods in excess of 20 years), overloading (design loading exceeded well before the design period) and the use of materials below the recommended standards. If roads exceed their design life despite extreme adverse conditions, this can generally be taken to indicate excessive over design. Some engineers use examples of past attitudes to maintenance and overloading in support of overdesign. However, it is extremely difficult to anticipate or quantify the degree of overloading or to design for inadequate maintenance. Furthermore, overdesign is costly and can inhibit development by unnecessarily increasing the cost of road projects.

Current practice is aimed at a whole-life approach taking into account construction, maintenance and vehicle operating costs and whole-life benefits. Models such as HDM3 have been developed to aid this process. This approach results in an optimal design for road projects. Furthermore, donors and governments are now collaborating to ensure improved and sustainable provision for future maintenance through the Road Maintenance Initiative (RMI) and the establishment of fuel levies to provide funds dedicated to road maintenance.

In recent years, studies conducted by TRL and others have led to a greater understanding of the modes of deterioration of roads constructed with various non-standard materials, carrying relatively low levels of traffic and in the environmental conditions prevalent in southern Africa. The results of this work together with the results of the current study has led to the development of specifications and standards which are more appropriate for the materials, traffic and climatic conditions likely to be encountered in many future road projects in the region. The application of these relaxed standards in the improved maintenance environment can be expected to aid development through the cost-effective provision of secondary and feeder roads.

8. Conclusions and recommendations

Sections of the road network (some on trunk roads which had received very little maintenance) in three countries in the region (Botswana, Malawi and Zimbabwe) and constructed with sub-standard materials were investigated. The main conclusions and recommendations from this project are discussed in detail in the following sections but can be summarised as:

- 1) The minimum standard of 80 per cent soaked CBR for natural gravel roadbases is inappropriately high for many low volume sealed roads, which form the majority of new surfaced road projects in the region and in many developing countries. New limits are recommended depending on traffic, materials and climate and these are presented as a series of design charts.
- 2) Field/optimum moisture content ratios for roadbase in the outer wheel-track in the wettest condition ranged for between 0.5 to 0.7 in the driest areas to between 1.0 to 1.2 in the wettest areas. Roadbases constructed with materials of high PI (plasticity index) were less sensitive to moisture ingress than was envisaged at the outset of the project. Moisture ingress was greatest on bases constructed with non-plastic materials.
- 3) The grading envelopes for natural gravel roadbases are too narrow. Alternative (wider) envelopes are recommended for relatively light trafficked roads.
- 4) New pavement design tables have been produced which enable the strong subgrades prevalent in many areas in the region to be exploited.
- 5) Traffic below 300,000 to 500,000 esa was not as significant a factor on pavement deterioration as expected. Many sections, especially those on the trunk road network, had been subject to a high degree of overloading but deformation (rutting) was low even on roadbases with PI of 18. New limits for PI are recommended.
- 6) Drainage was a significant factor on performance even in dry areas. A minimum design crown height of 0.75 metre is recommended.
- 7) Sealed shoulders provide a structural and maintenance benefit and should be considered even on low volume roads if this enables local materials to be exploited and there is an overall whole-life benefit. A method is suggested for determining the optimum width for sealing shoulders but the evidence from this study suggests a minimum width of one metre.
- 8) Included in this study were sections of a road with base materials of gravel wearing course standard which was subsequently sealed. There are other such examples in the region and these roads have generally performed exceptionally well. (This is further evidence of the need for a relaxation in design standards). In recognition of this practice, a design class for sealing an existing gravel road is included in the design chart for traffic up to 10,000 esa.

8.1 Classification of engineering properties of natural gravels

8.1.1 Roadbase

- 9) The major groups of natural gravel roadbase materials used in Zimbabwe, Malawi and Botswana are quartzitic gravels, weathered rocks, lateritic gravels, and calcareous gravels and sands. The study has shown that all of these can be successfully used in the upper pavement layers of low volume roads.
- 10) A common feature of the specifications in the region are for natural gravel roadbase materials to meet strict compliance criteria on particle size distribution, to have plasticity index less than 6, and a soaked CBR of greater than 80 per cent at 98 per cent BS 4.5kg rammer or modified AASHTO compaction. In many parts of the region one of the biggest problems for the engineer is the location of materials which meet these specifications. Many natural gravels are often excluded from use because they fail to meet at least one of these criteria.
- 11) Where materials meeting the specification are not available locally, the alternatives are to:
 - Import suitable materials over long distances
 - Improve the materials by addition of stabilising agents such as lime or cement
 - Utilise sources of crushed stone if these are available
- 12) Grading was the parameter which most of the roadbases included in the study failed to meet. Plasticity requirements were less of a problem, except in the N<2 climatic areas.

8.1.2 Subgrades

- 13) The subgrades, encountered in the project were generally S5 and S6 soils, which were very good for road construction purposes.
- 14) Where poor subgrades are prevalent, judicious and selective stabilisation with lime may be warranted by the savings that can be made in pavement material thickness and quality.

8.2 Performance of natural gravel roadbases

8.2.1 Strength

- 15) In situ roadbase and sub-base strengths were generally above their design values. In the few cases where the in situ strengths were below the design value, the performance of the road was not adversely affected.
- 16) In situ structural numbers at the wettest time of the year, when the pavement is in its weakest condition, were in most cases higher than the design structural number.

- 17) The strengths measured in situ were very dependent on the compacted density. The soaked CBR was taken at the same compaction level as determined by the in situ density measurements to allow comparisons to be made to the design values.

8.2.2 Moisture in the roadbase

- 18) The seasonal effects of edge wetting of the roadbase, sub-base and subgrade were obvious in most situations. However, it was determined that the ingress of moisture was less serious where more plastic materials were used in the roadbase. Where low plasticity materials were used in the roadbase, ingress of moisture could extend over substantial widths of the carriageway.
- 19) Where materials were poorly compacted, these exhibited a higher risk of wetting. If the compaction standards could be improved on the 'low cost' roads in Zimbabwe, for example by compacting at the appropriate optimum moisture content, the structural standard and the durability of the roads could increase significantly.
- 20) The field/optimum moisture ratios in the outer wheel-track and at the wettest time of year for the roadbases were different in the different climatic zones. In $N < 2$ areas, the ratio ranged from 1.0 to 1.2; in $N = 2-4$ areas, the ratio was 0.6 to 0.8; and, in $N > 4$ zones, it ranged between 0.5 to 0.7.

8.2.3 Moisture in the subgrade

- 21) The field/optimum moisture ratios in the outer wheel-track at the wettest time of year for the subgrade were different in the different climatic zones. In $N < 2$ areas, the ratio ranged from 1 to 1.5; in $N = 2-4$ areas, the ratio was 0.75 to 1.25; and, in $N > 4$ zones, it ranged between 0.5 to 0.7. The drainage of the road had an equally strong influence on the moisture condition and the strength, with better drained sites exhibiting higher strengths.
- 22) It proved difficult to track and explain moisture changes on roads less than about four years old. This was probably an effect of the road settling down to an equilibrium after construction.

8.2.4 Road performance parameters

- 23) Performance data collected indicated that traffic-induced permanent deformation is relatively low until the traffic level gets to about 300,000 to 500,000 esa and, even at this level of traffic, rutting was generally less than 12mm. This was consistent with the magnitude of the structural numbers. There was also little evidence of a relationship between the level of roughness or the development of pavement cracking to the level of traffic.
- 24) The levels of cracking and roughness could be explained better by the drainage conditions at the site, as measured by the height of the crown of the road above the invert of the ditch, and the distance of the outer wheel-track from the edge of the sealed area. Longitudinal cracks were prevalent where the drainage was poor. It was also evident that cracking

increased with the age of the bituminous seal. In most cases, the severity of the cracking was less than 10 per cent, and crack sealing or re-sealing would arrest the problem. Where a higher degree of cracking was present, some improvement would also be needed to the drainage. High roughness was also associated with the 'low-cost' constructions in Zimbabwe.

- 25) The central deflection was much lower than expected given the nature of the pavement materials. High deflections were again explained by poor drainage conditions or lower strength of poorly compacted materials at depth in the structure.
- 26) The road environment can be considered to encompass both local climate (rainfall, temperature range and evaporation), drainage (effectiveness of drains, carriageway cross fall and the crown height) and topographic and sub-soil conditions. This was of major importance in determining the degree and severity of the cracking and roughness on these low volume roads. Applying design standards based solely on traffic may be inappropriate.
- 27) Another important variable controlling the performance of road with marginal bases is the durability of the bituminous seal. A number of thin sealing technologies have been highlighted which are appropriate to these types of roads. All thin seals are subject to the effect of ageing and embrittlement of the binder. They are also susceptible to the movements in the pavement caused by wetting and shrinkage of the soils in the lower parts of the structure. Crack sealing is usually adequate to protect the structure when double seals have been used. If single seals are used at construction, it is important to re-seal once the road environment has stabilised the structure.

8.2.5 Sealed shoulders

- 28) The study has shown that the use of sealed shoulders gives a structural benefit by maintaining a drier environment under the running surface. The provision of a sealed shoulder decreases the risk of using weaker materials in the upper pavement layers.
- 29) The addition of narrow (<750mm) sealed shoulders had only a marginal affect on the strength of the roadbase strengths in the outer wheel-track. Their use had no impact on subgrade strengths in the wheel-tracks.
- 30) The outer wheel-track moisture and strength conditions will remain fairly stable provided that the shoulders of the road are sealed to a sufficient width such that the outer wheel-track is more than 1.5 metres from the edge of the sealed area, and the drainage is ensured by maintaining the crown height greater than one metre above the ditch. A one metre sealed shoulder is therefore the minimum effective width. In some cases moisture can still ingress to this level, and wider shoulders are then required.
- 31) The need for sealed shoulders on existing roads can be determined by measuring the strength variation across the pavement in the wet and dry seasons, using a DCP, to assess the effective design width. In this way an estimate can be made of the likely ingress and the shoulder width can be designed accordingly. Shoulder sealing is now being carried out on many trunk roads in the region and the lessons should be learned for secondary road design.

8.3 Revisions to specifications and design criteria

8.3.1 Structural design charts

- 32) The results from this study have been used to develop a series of structural design charts for low volume roads. Guidelines for the selection of natural gravel roadbases have also been developed. The charts and recommendations given are specific to lightly trafficked roads in the Southern Africa and should be used as a guide. The design charts are not prescriptive and, if other local evidence and experience are available to the engineer, they should be modified or adjusted accordingly.
- 33) The design charts are applicable to two climatic zones: $N < 4$ and $N > 4$. The charts allow modifications to the materials selection which depend on the width of cross-section selected. The options provided in a climate where $N > 4$, and with wide cross-sections, makes good use of marginal quality materials, and therefore may be an attractive alternative when good quality materials are scarce.
- 34) Separate guidelines for the selection of lateritic and calcrete roadbases have been included, as these two groups of materials consistently exhibit better than expected performance.
- 35) Standards need not be relaxed over the whole road length, but there may be sections where such changes in approach are justified.

8.3.2 Sealing gravel roads

- 36) The design charts include an option for sealing a gravel road and using the existing wearing course as roadbase for traffic up to 10,000 esa. There are a number of examples of this approach in the region and analysis with HDM3 in Zimbabwe that the optimum traffic level for upgrading by this method is less than 40 vehicles per day.

8.3.3 Mitigating risks

- 37) The main risk factors to road pavements are:
 - Quality of the materials (strength and moisture susceptibility)
 - Construction control (primarily compaction standard)
 - Environment (particularly drainage)
 - Maintenance standards (drainage and surfacing)
 - Traffic and over-loading
- 38) Depending on the circumstances, some of these factors will be more important than others. Generally, the risk of failure can be expected to increase if a number of factors are relaxed together. However, in this study, many of the sections had been constructed with low-quality materials, received poor maintenance and had experienced considerable overloading but still performed well. The level of performance of these roads constructed

with sub-standard materials in adverse conditions is a further indication of unnecessary conservatism and the need for more appropriate designs for these roads. However, as with all road projects, control of construction quality, maintenance and overloading will ensure that the maximum benefits will be obtained from the recommended relaxation in roadbase standards.

- 39) More consideration needs to be given to determining the traffic growth rate. Projecting traffic growth and assigning accurate equivalence factors to the traffic is crucial if economic designs are to be achieved. Using unrealistically high growth rates or equivalence factors reduces the level of risk for the engineer but results in conservative pavement designs which can ultimately negate the feasibility of projects.
- 40) Where the road carries a larger proportion of heavy traffic, or where poor drainage conditions are unavoidable, the risk can be reduced by adjusting the design class upwards by one traffic class.

8.4 Concluding Remarks and outstanding issues

The main aim of this study was to derive specifications to enable greater use to be made of natural gravels in the roadbases of lightly trafficked sealed roads in order to reduce costs and encourage road projects as an aid to development. This objective has been achieved by the derivation of new specifications and design standards. Additional benefits will accrue from greater use of locally available gravels in other layers of the pavement, including exploitation of the strong sub-grades which occur extensively in the region. The actual savings in construction costs will depend on the design adopted but substitution of crushed gravel or crushed rock by natural gravel will, itself reduce the costs of the roadbase by a factor of between 3 and 8.

Changes currently taking place in Road Authorities in the region, which include increased participation by the private sector and communities, together with recognition by practitioners and politicians of the importance of maintenance and the need for dedicated funding has improved the climate for the application of more appropriate standards and innovative practices.

The need for standards derived from experience in the region was acknowledged and the results of the study endorsed by consultants, contractors and Roads Department engineers attending workshops held in the participating countries to discuss the recommendations.

A strong recommendation from the workshops, particularly by consultants and contractors, was that the revised specifications and standards should be incorporated into country documents which would confirm endorsement by Roads Departments and would facilitate implementation of the results of the research by practitioners.

It was also agreed that these and other recommendations and manuals should be used as guides only and that large cost savings can be made in the design and construction of low-volume roads by engineers being innovative.

There is also a need to increase awareness through workshops for other countries in the region and donors.

The materials testing in this study was generally carried out to British Standards(BS) test methods. Testing to BS methods is the current practice in some countries in southern Africa but there is considerable diversity in the test methods used throughout the SADC region. In some cases use of a different test method may not be important but in other circumstances the differences could have a significant impact on design and subsequent performance. This is a matter of concern in the region, which affects the recommendations from this study and other regional guidelines and manuals for materials testing, road design and construction.

The results of this study indicate the importance of drainage, especially in materials considered to be particularly moisture sensitive. Most of the sections investigated were on embankments. There is a need to extend the work of this study to sections in cut, where drainage problems are often most acute, to identify and quantify any additional risks of moisture ingress and recommend the most appropriate and cost-effective preventative measures.

The cost savings from this study will result from changes in the use of materials and pavement design standards. Similar and possibly much greater savings in some circumstances could be obtained from a relaxation in geometric standards for sealed roads carrying low volumes of traffic. The standards adopted on rural roads in the region are often far higher than those on rural roads in developed countries. Most engineers and many road safety experts *in the region* agree that the adoption of more appropriate standards would yield high benefits without compromising road safety and this approach has already been used on some projects in Zimbabwe. Little data is currently available and additional research is required to quantify the impact of adopting reduced geometric standards for rural roads in the SADC countries.

Whilst studies such as this are aimed at reducing costs, there is also a need to improve methods of identifying and quantifying the benefits from road projects. On low-volume roads, socio-economic benefits are often a large component of the total benefits and there is a need to derive methods to measure these benefits. Sealed roads also yield other benefits such as dust reduction which affect health, agriculture and road safety particularly on roads through villages, near schools and clinics, and near dust-sensitive crops in the dry season. In the wet season, the improved passability from sealed roads also ensures more regular public transport services, which are so essential to the well-being of rural populations.

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Appendix A: Characteristics of test sections

Table A-1 Details of test sections in Zimbabwe

Section	Road	Chainage	Roadbase material	Design traffic (esa million)	Within or outside grading	Plasticity Index	Soaked CBR (%)	Climatic N-value
WAMI 1	Wedza-Mutiweshiri	57+695-57+740	quartz gravel	0.3	marginal	slightly plastic	170	2-3.9
RENA 1	Rusape-Nyanga	9+220-9+300	lateritic gravel	0.3	out	4	65	2-3.9
HSMO 1	Headlands-Mayo	6+330-6+360	lateritic gravel	0.1	out	non plastic	100	2-3.9
HSMO 2	Headlands-Mayo	6+620-6+660	lateritic gravel	0.1	out	non plastic	50	2-3.9
NARE 1	Nyanga-Ruangwe	72+920-72+960	mixed gravel	0.1	out	3	130	2-3.9
NARE 2	Nyanga-Ruangwe	72+975-73+000	mixed gravel	0.1	out	slightly plastic	130	2-3.9
NARE 3	Nyanga-Ruangwe	73+000-73+040	quartz gravel	0.1	out	slightly plastic	65	2-3.9
NARE 4	Nyanga-Ruangwe	79+160-79+213	quartz gravel	0.1	out	4	80	2-3.9
NARE 5	Nyanga-Ruangwe	79+222-79+294	mixed gravel	0.1	out	4	150	2-3.9
NARE 6	Nyanga-Ruangwe	79+294-79+336	mixed gravel	0.1	out	slightly plastic	115	2-3.9
NARE 7	Nyanga-Ruangwe	90+163-90+203	quartz gravel	0.1	out	slightly plastic	145	2-3.9
NARE 8	Nyanga-Ruangwe	90+203-90+243	mixed gravel	0.1	out	slightly plastic	80	2-3.9
NARE 9	Nyanga-Ruangwe	90+243-90+283	mixed gravel	0.1	out	slightly plastic	85	2-3.9
MAMI 1	Marondera-Musami	43+390-43+430	lateritic gravel	0.05	out	5	100	2-3.9
MAMI2	Marondera-Musami	43+995-44+035	lateritic gravel	0.05	out	slightly plastic	110	2-3.9
MAMI 3	Marondera-Musami	50+075-50+115	lateritic gravel	0.05	out	non plastic	120	2-3.9
MAMI 4	Marondera-Musami	51+570-52+610	lateritic gravel	0.05	out	slightly plastic	135	2-3.9
MAMI 5	Marondera-Musami	52+860-52+900	lateritic gravel	0.05				2-3.9
MAME 1	Murewa-Madicheche	51+970-52+020	mixed gravel	0.1	out	5	160	2-3.9
GECY 1	Glendale-Centenary	80+560-80+630	quartz gravel	0.3	in	4	170	2-3.9
GECY 2	Glendale-Centenary	89+620-89+680	quartz gravel	0.1	out	slightly plastic	140	2-3.9
GECY 3	Glendale-Centenary	89+680-89+730	quartz gravel	0.1	out	4	170	2-3.9

continued

Section	Road	Chainage	Roadbase material	Design traffic (esa million)	Within or outside grading	Plasticity Index	Soaked CBR (%)	Climatic N-value
KIBA 1	Kamativi-Binga	30+140-30+170	weathered rock	0.05	out	4	70	2-3.9
KIBA 2	Kamativi-Binga	64+020-64+060	calcrete	0.05	out	5	30	2-3.9
KIBA 3	Kamativi-Binga	88+450-88+480	weathered rock	0.05	out	9	26	2-3.9
KIBA 4	Kamativi-Binga	93+500-93+540	calcrete	0.05	out	12	15	2-3.9
KIBA 5	Kamativi-Binga	134+200-134+240	Kalahari sand	0.05	out	TBA	35	2-3.9
MIAS 1	Mlibizi access	2+200-2+240	calcrete	0.05	out	7	110	2-3.9
MIAS 2	Mlibizi access	5+550-5+580	quartz gravel	0.05	marginal	slightly plastic	85	2-3.9
MIAS 3	Mlibizi access	5+710-5+740	quartz gravel	0.05	out	4	80	2-3.9
TOSA 1	Tsholotsho-Sipera	2+800-2+840	ferruginous gravel	0.05	in	6	80	4-5
BOKI 1	Bulawayo-Kezi	83+270-83+300	quartz gravel	TBA	in	slightly plastic	115	4-5
SSMA 1	St. Joseph's-Maphisa	79+170-79+200	quartz gravel	0.05	in	slightly plastic	220	5+

Table A-2 Details of test sections in Malawi

Section	Road	Chainage	Roadbase material	Design traffic (esa million)	Within or outside grading	Plasticity Index	Soaked CBR (%)	Climatic N-value
CABA 1	Chikwawa-Bangula	11+170-11+205	ferruginous gravel	-	out	14	24	1-2
LEZA 1	Liwonde-Zomba	4+570-4+600	weathered rock	-	out	17	35	1-2
LEZA 2	Liwonde-Zomba	15+780-15+810	lateritic gravel	-	out	non plastic	85	1-2
MIMY 1	Mangochi-Monkey Bay	15+400-15+430	weathered rock	-	out	16	45	1-2
MYGI 1	Monkey Bay-Golomoti	20+000-20+030	quartz gravel	-	out	16	45	1-2
NADA 1	Nkhotakota-Dwangwa	91+829-91+859	crushed stone	-	in	non plastic	130	1-2
CAJE 1	Chilumba-Jetty	0+624-0+654	quartz gravel	-	out	10	50	<1
CAKA 1	Chiweta-Karonga	13+355-13+385	quartz gravel	-	out	15	25	<1
KUMA 1	Kasungu-Mzimba	4+510-4+540	lateritic gravel	0.5	out	18	50	<1
KUMA 2	Kasungu-Mzimba	5+510-5+540	lateritic gravel	0.5	out	19	40	<1
KUMA 3	Kasungu-Mzimba	5+360-5+390	lateritic gravel	0.5	out	18	45	<1
KUMA 4	Kasungu-Mzimba	5+811-5+841	crushed stone	0.5	-	non plastic	140	<1
LEMI 1	Lilongwe-Mchinji	7+290-7+320	lateritic gravel	0.5	out	16	55	1-2
LEMI 2	Lilongwe-Mchinji	9+800-9+830	lateritic gravel	0.5	out	17	70	1-2
LEMI 3	Lilongwe-Mchinji	75+600-75+630	lateritic gravel	0.5	out	15	40	1-2
GINA 1	Golomoti-Mganja	1+300-1+330	weathered rock	0.5	out	slightly plastic	90	1-2

Table A-3 Details of test sections in Botswana

Section	Road	Chainage	Roadbase material	Design traffic (esa million)	Within or outside grading	Plasticity Index	Soaked CBR (%)	Climatic N-value
OASE 1	Orapa-Serowe	42+065-42+105	Kalahari sand	0.2	out	non plastic	35	5+
OASE 2	Orapa-Serowe	42+150-42+190	Kalahari sand	0.2	out	non plastic	35	5+
NAMN 1	Nata-Maun	21+440-21+470	calcrete	0.8	marginal	21	80	5+
NAKA 1	Nata-Kazungula	201+500-201+540	weathered basalt	0.5	in	10	60	4-5
NAKA 2	Nata-Kazungula	213+290-213+320	crushed stone	0.5	marginal	10	85	4-5
NAKA 3	Nata-Kazungula	259+500-259+540	weathered basalt	0.5	-	-	-	4-5
NAKA 4	Nata-Kazungula	289+500-289+540	crushed stone	0.5	out	non plastic	95	4-5
KAKE 1	Kazungula-Kasane	4+600-4+640	weathered basalt	0.5	in	11	55	2-3.9
TUNG 1	Tsau-Nokaneng	4+400-4+440	calcareous sand	0.5	out	non plastic	30	5+
SATU 1	Sehitwa-Tsau	10+400-10+440	calcareous sand	0.5	out	non plastic	35	5+

Table A-4 Distribution of sections with respect to the recommended limits for roadbases in Overseas Road Note 31

Material type	Number of sections	No of sections inside recommended limits	No of sections outside recommended limits			
			Grading	Plasticity	Soaked CBR (98% mod AASHTO)	All parameters
Quartz gravel	14	5	9	3	4	3
Mixed gravel	7	-	7	-	-	-
Lateritic gravel	15	-	14	7	8	7
Ferruginous gravel	2	1	1	1	1	1
Calcrete	4	-	3	3	2	1
Calcareous sand	2	-	2	-	2	-
Kalahari sand	3	-	3	-	3	-
Weathered rock	5	-	5	3	4	3
Weathered basalt	3	-	0	2	2	-
Crushed stone	4	2	1	1	N/A	N/A
Total	59	8	45	20	26	15
<i>Notes:</i>						
N/A Not applicable						

Table A-5 General characteristics of the Zimbabwe test sites

Site	No	Year of Construction	Month of Construction	Road width		Sealed Shoulder		Gradient per cent	Crossfall LHS	Crown Height LHS	Distance to Drain LHS	Crossfall RHS	Crown Height RHS	Distance to Drain RHS	Surfacing Type	Date Last Re-seal	General Surfacing Condition
				LHS	RHS	LHS	RHS										
BOKI	1	1963		3.30		0		-1.47	-0.86	0.7	10.0	-1.92	0.3	1.4	DSD	1992	F
GECY	1			3.40	3.35	0	0	-3.58	-2.46	1.3	10.0	2.30	0.9	11.0	DSD		G
GECY	2	1994	10	3.20	3.35	0.5	0.5	-0.1	-0.9	0.9	8.0	-1.36	0.8	8.5	DSD		VG
GECY	3	1994	10	3.75	3.45	1	1	-0.34	-2.07	1.0	8.5	-2.00	1.2	11.5	DSD		VG
HSMO	1	1990	8	2.95	2.95	0	0	1.34	2.73	0.3	5.0	1.60	0.3	5.4	DOS	1996	VG
HSMO	2	1990	8	3.00	3.00	0	0	1.93	2.32	0.4	4.6	4.56	0.5	5.6	SOS		P
KIBA	1	1985	12	3.00	3.00	0	0	2.6	1.42	0.5	7.5	-2.50	0.2	5.4	SSD	1993	P
KIBA	2	1984	5	3.10	3.20	0	0	-2.6	-1.92	F.D		-2.07	0.3	7.5	SSD	1991	G
KIBA	3	1988	6	3.10	3.10	0	0	1.47	-1.68	0.6	9.5	-0.46	0.4	7.5	SSD	1994	G
KIBA	4	1988	9	3.00	3.10	0	0	0.43	-2.82	F.D		2.18	0.4	8.5	SSD		G
KIBA	5	1990	2	3.00	3.00	0	0	-0.29	-2.5	0.4	6.5	-2.57	0.4	10.0	SSD		F
MAME	1	1992	9	3.50	3.60	0	0	-1.31	-1	0.7	7.5	-1.21	0.9	8.0	DSD		VG
MAMI	1	1993	10	3.64	3.72	0	0	1.88	-3.42	0.8	8.5	-1.18	0.5	6.5	SSD	1996	VG
MAMI	2	1994	3	3.70	3.60	0	0	1.22	-3.68	0.9	7.5	-0.93	0.7	6.5	SSD	1996	VG
MAMI	3	1994	3	3.60	3.60	0	0	1.09	0	0.7	8.5	-1.96	0.7	6.5	SSD	1996	VG
MAMI	4	1994	8	3.63	3.70	0	0	-1.13	-2.25	0.7	9.0	-1.39	0.8	6.5	SSD	1996	VG
MAMI	5	1993	11	3.58	3.62	0	0	-1.83	4.6	0.7	9.0	4.60	0.8	6.5	SSD	1996	VG
MIAS	1	1993	3	3.20	3.40	0	0	-1.49	-2.03	F.D		-1.14	0.4	7.0	SSD		G
MIAS	2	1993	3	3.10	3.60	0	0	0.73	-1.28	1.1	10.0	-1.14	0.6	8.5	SSD		G
MIAS	3	1993	2	3.10	3.20	0	0	0.67	-2.25	0.6	9.0	-2.46	0.4	7.5	SSD		G
NARE	1	1992	11	3.60	3.60	1	0.8	-2.03	-2.12	0.8	6.5	-1.64	F.D		DSD		VG
NARE	2	1992	11	3.55	3.50	0	0	-2.07	-1.36	0.7	6.5	-1.71	1.0	8.5	DSD		VG
NARE	3	1992	11	3.60	3.70	0.5	0.5	-2.07	-2.86	0.8	8.5	-3.57	F.D		DSD		VG
NARE	4	1993	11	3.55	3.55	0	0	1.11	-1.32	0.4	7.0	-2.03	F.D		DSD		VG
NARE	5	1993	11	4.00	4.10	0.5	0.5	0.08	-2.03	0.4	7.0	-2.50	0.5	7.0	DSD		VG
NARE	6	1993	11	4.00	4.00	0.6	0.6	-0.53	-2.57	0.6	7.5	-1.75	0.9	7.0	DSD		VG
NARE	7	1994	3	3.65	3.70	0	0	-2.41	-2.42	0.5	7.0	-3.28	F.D		DSD		VG
NARE	8	1994	3	3.60	3.70	0.35	0.45	-2.42	-2.28	0.5	7.5	-1.43	0.7	8.0	DSD		VG
NARE	9	1994	3	3.70	3.70	0.9	0.9	-2.49	-2	0.5	7.5	0.57	FD		DSD		VG
RENA	1	1992	3	3.80	3.60	1.4	1.4	-0.5	-2.36	0.8	10.5	-1.61	0.8	10.5	DSD		VG
SSMA	1	1990	12	3.03	2.96	0	0	2.21	-3.43	0.7	7.0	-1.71	0.5	7.5	SOS		G
TOSA	1	1995	3	3.60	3.60	0	0	0.9	-1.43	0.7	7.5	-0.14	0.4	7.0	DSD		F
WAMI	1	1991	10	3.60	3.60	0	0	1.39	-2.5	0.8	6.5	-2.21	0.7	7.5	DSD		VG

Table A-6 General characteristics of the Malawi test sites

Site	No	Year of Construction	Month of Construction	Road width		Sealed Shoulder		Gradient per cent	Crossfall LHS	Crown Height LHS	Distance to Drain LHS	Crossfall RHS	Crown Height RHS	Distance to Drain RHS	Surfacing Type	Date Last Re-seal	General Surfacing Condition
				LHS	RHS	LHS	RHS										
CABA	1	1974		3.41	3.48	0.69	1.02	-2.26	-3.64	0.7	9.5	-2.10	0.6	10.0	50 Asphalt	1982	P
CAKA	1	1975		2.90	2.67	1.2	1.03	-0.64	-5	1.0	6.5	-3.57	FD	FD	DSD	RS	P
CAJE	1	1972		2.56	2.56	0.9	1.04	0.29	-1.36	0.4	10.5	-3.53	1.8	13.0	DSD		P
GINA	1	1972		3.08	3.08	1.1	0.8	0.69	-0.96	FD	FD	-2.46	1.2	11.0	DSD		VG
KUMA	1	1985	1	3.26	3.24	1.39	1.25	-4.95	-3.28	1.1	7.0	-3.00	1.1	6.5	DSD		VG
KUMA	2	1985	7	3.23	3.27	1.17	1.23	-1.07	-2.61	FD	FD	-2.57	0.8	6.5	DSD		VG
KUMA	3	1985	1	3.28	3.26	1.21	1.34	-3.44	-3.36	1.3	10.0	2.89	0.9	6.5	DSD		VG
KUMA	4	1985	1	3.22	3.25	1.48	1.35	-0.2	-3.11	1.0	7.0	-3.18	1.1	7.0	DSD		VG
LEMI	1	1976		3.34	3.32	1.96	1.42	0.76	-2.43	2.3	12.0	-2.50	FD	FD	SSD	1985	VG
LEMI	2	1976		3.38	3.38	1.44	1.59	-0.5	-3.75	FD	FD	-3.68	1.0	10.0	SSD	1985	VG
LEMI	3	1979		3.32	3.35	1.38	1.45	-0.44	-2.43	1.2	13.5	-3.06	1.4	18.5	SSD	1985	G
LEZA	1	1970		3.28	3.25	1.32	1.4	0.77	-2.57	1.0	11.0	-2.96	1.4	10.0	DSD		P
LEZA	2	1970		3.27	3.29	1.93	1.31	-1.76	-2.86	1.1	9.5	-2.53	1.3	9.0	DSD		G
MIMY	1	1972		3.43	3.40	1.17	0.7	0.28	-4.39	0.6	8.0	2.27	1.0	9.5	DSD		F
MYGI	1	1971		2.63	2.77	1.5	1.1	0.48	-3.64	0.7	9.0	4.03	1.0	9.0	DSD		F
NADA	1	1978		2.80	2.80	0	0	1.3	-4.32	FD	FD	-4.32	0.8	8.0	DSD	RS	VG

Table A-7 General characteristics of the Botswana test sites

Site	No	Year of Construction	Month of Construction	Road width		Sealed Shoulder		Gradient per cent	Crossfall LHS	Crown Height LHS	Distance to Drain LHS	Crossfall RHS	Crown Height RHS	Distance to Drain RHS	Surfacing Type	Date Last Re-seal	General Surfacing Condition
				LHS	RHS	LHS	RHS										
KAKE	1	1986		3.08	3.12	0	0	0.04	-2.68	0.5	6.5	-2.18	0.6	7.5	DSD		F
NAKA	1	1984		3.30	3.31	0.4	0.6	2.5	-3.21	0.4	6.0	-1.32	0.4	6.0	DSD		F
NAKA	2	1984			3.85	0.7		-1.62	-3.61	0.9	9.5	N/A			DSD		F
NAKA	3	1984		3.40	3.40	4.1	3.85	-1.34	-3.42	0.9	13.5	-1.14	FD		DSD		VG
NAKA	4	1984		3.40	3.40	0.64	0.66	3.41	-1.18	0.7	11.0	-7.14	1.5	10.5	DSD		VG
NAMN	1	1991		3.50	3.45	1	1		-3.64	FD		-0.60	FD		DSD		VG
ORSE	1	1989		3.41	3.10	1.1	1.2		2.61	0.5	13.0	-2.61	0.5	7.0	DSD	1996	VG
ORSE	2	1989		3.45	3.28	1.3	1.6		3.27	0.5	13.0	-3.27	0.6	9.5	DSD	1996	VG
SATU	1	1994		3.45	3.50	0	0	-1.86	-3.64	0.4	8.5	1.07	FD	FD	DSD	1996	VG
TUNG	1	1988		2.60	2.60	0	0	-0.32	-1.68	FD		-2.00	0.4	8.5	DSD		P

Notes:
 FD free draining
 DSD double surface dressing
 SSD single surface dressing
 DOS double Otta seal
 SOS single Otta seal

Table A-8 Summary of traffic data

Zimbabwe					Malawi					Botswana				
Section	AADT	Esa (x 10 ³)	Per cent heavy vehicles	Per cent buses	Section	AADT	Esa (x 10 ³)	Per cent heavy vehicles	Per cent buses	Section	AADT	Esa (x 10 ³)	Per cent heavy vehicles	Per cent buses
BOKI 1	189		30	12	CABA 1	177	342	29	5	KAKE 1	235	240	10	4
GECY1-3	29	13	28	7	CAJE 1	95	764	32	14	NAKA 1-3	242	340	24	1
HSMO 1-2	126	19	26	12	CAKA 1	95	501	33	9	NAKA 4	242	340	24	1
KIBA 1	88	23	31	5	GINA 1	44	32	2	5	NAMN 1				
KIBA 2-3	94	23	31	5	KUMA 1-4	440	504	23	9	ORSE 1-2				
KIBA 4	75	23	31	5	LEMI 1-2	1 166	839	13	7	SATU 1	268	330	6	4
KIBA 5	120	24	31	5	LEMI 3	390	637	16	8	TUNG 1	268	330	6	4
MAME 1	122	36	63	23	LEZA 1	1 005	1 015	13	11					
MAMI 1-5	121	31	36	13	MIMY 1	773	525	21	9					
MIAS 1-3	128	3	69	2	MYGI 1	76	117	8	7					
RENA 1	410	26	12	3	NADA 1	239	303	29	10					
SSMA 1	28	15	43	14										
TOSA 1	22	7	55	23										
WAMI	247	29	48	16										
<i>Notes:</i>														
Esa Cumulative equivalent standard axles														
AADT Annual average daily traffic														

Table A-9 Roadbase characteristics for Zimbabwe test sections

Site	No	% Passing British Standard Sieve (mm)																	Compactive effort	Design						
		53	37.5	26.5	19	9.5	4.75	2.36	2	1.18	0.6	0.425	0.300	0.150	0.075	IR	GM	WL		Ip	PM	PP	MDD	OMC	CBR	
BOKI	1	100	100	97	92	80	60	49	48	40	32	29	23	16	12	0	2.11	SP	SP		29	12	BSH	2210	6.8	115
GECY	1	100	100	96	81	61	44	38	37	33	29	26	23	16	12	0	2.25	SP	SP	4	104	48	BSH	2225	5.6	340
GECY	2	100	100	94	87	72	58	51	50	46	40	36	34	24	18	0	1.96	SP	SP		36	18	BSH	2180	6.5	140
GECY	3	100	100	98	92	77	63	54	53	49	43	40	36	26	19	0	1.88		21	4	160	76	BSH	2190	6.5	170
HSMO	1		100	98	97	90	74	64		55	46	40	33	24	19	1		SP	SP			20	BSH	2110	9	100
HSMO	2		100	98	96	86	67	58		52	44	38	32	23	16	0		SP	SP			15	BSH	2100	9	50
KIBA	1	100	100	100	98	92	82	67	65	57	50	44	38	27	20	0	1.71		22	4	176	80	BSH	2170	7.1	70
KIBA	2	100	100	100	98	93	88	83	82	77	66	56	46	31	24	0	1.38		21	5	280	120	BSH	2120	8.5	30
KIBA	3	100	100	100	98	93	88	82	80	77	71	67	64	58	54	0	0.99		26	9	603	486	BSH	2115	8.3	25
KIBA	4	100	100	100	99	96	92	87	85	81	76	74	71	64	57	0	0.84		27	12	888	684	BSH	2040	10	15
KIBA	5																						BSH	2100	6	34
MAME	1	100	100	99	98	92	84	73	70	55	38	32	27	20	16	0	1.82		19	5	160	80	BSH	2140	6.5	170
MAMI	1	100	100	94	89	77	67	62	60	52	37	35	28	22	19	0	1.86		20	5	175	95	BSH	2215	6.5	100
MAMI	2	100	100	98	94	84	77	72	70	58	42	36	29	20	14	0	1.8	SP	SP		36	14	BSH	2155	7	110
MAMI	3	100	100	96	88	71	62	58	55	52	40	37	29	19	13	0	1.95	NP	NP		37	13	BSH	2155	7	120
MAMI	4	100	100	100	99	85	68	60	58	53	42	38	29	19	14	0	1.9	SP	SP		38	14	BSH	2090	8.6	135
MIAS	1	100	100	99	90	80	63	53	51	48	41	35	28	18	14	0	2		19	7	245	98	BSH	2110	6.6	110
MIAS	2	100	100	97	84	65	51	44	43	42	37	33	26	16	13	0	2.11	SP	SP		33	13	BSH	2185	7.2	85
MIAS	3	100	100	100	96	87	80	76	75	71	65	55	47	31	24	0	1.46		16	4	220	96	BSH	2190	7.5	80
NARE	1	100	100	97	95	87	77	66	64	56	44	42	32	22	17	0	1.77	SP	SP	3	126	51	BSH	2180	7	130
NARE	2	100	100	97	94	87	74	63	62	54	43	42	32	22	15	0	1.81	SP	SP		42	15	BSH	2180	7	130
NARE	3	100	100	98	94	88	77	69	67	60	50	48	40	28	22	0	1.63	SP	SP		48	22	BSH	2110	7.5	65
NARE	4	100	100	98	96	89	79	64	60	52	43	38	33	23	18	0	1.84		21	4	152	72	BSH	2190	9.3	80
NARE	5	100	100	98	94	88	78	68	65	57	45	40	34	24	19	0	1.76	SP	SP	4	160	76	BSH	2120	7.5	150
NARE	6	100	100	97	96	88	78	69	65	57	45	40	35	23	16	0	1.79	SP	SP		40	16	BSH	2185	7.7	115
NARE	7	100	100	95	92	85	75	65	63	53	42	36	31	19	13	0	1.88	SP	SP		36	13	BSH	2130	7.2	145
NARE	8	100	100	95	90	79	69	61	60	51	42	37	33	22	16	0	1.87	SP	SP	10	370	160	BSH	2160	7.8	80
NARE	9	100	100	98	97	91	82	75	74	65	54	48	43	28	20	0	1.58	SP	SP		48	20	BSH	2160	7	85
RENA	1	100	100	100	98	91	83	74	72	62	44	38	32	24	20	0	1.7		19	4	152	80	BSH	2125	7.4	65
SSMA	1		100	95	90	78	62	44		33	27	24	21	16	12	5		NP	NP			10	BSH	2120	7.5	220
TOSA	1	100	100	96	92	78	60	46	44	38	34	28	24	15	11	0	2.17		21	6	168	66	BSH	2250	7	80
WAMI	1	100	100	98	93	82	66	54	52	44	36	32	28	22	17	0	1.99	SP	SP		32	17	BSH	2210	6.2	170

Table A-10 Roadbase characteristics for Malawi test sections

Site	No	% Passing British Standard Sieve (mm)																				Compactive effort	Design		
		53	37.5	26.5	19	9.5	4.75	2.36	2	1.18	0.6	0.425	0.300	0.150	0.075	IR	GM	WL	Ip	PM	PP		MDD	OMC	CBR
CABA	1	100	100	100	100	100	99	98	97		93	92	86	72	61	0	0.5	28	14	1288	854	BSH	1970	7.5	25
CAKA	1	100	100	100	95	90	86	79	73		46	41	32	23	19	0	1.67	25	15	615	285	BSH	2165	6.9	25
CAJE	1	100	100	96	95	87	74	61	53	48	36	31	28	22	19	0	1.97	32	10	310	190	BSH	2170	5.7	50
GINA	1	100	100	99	98	93	85	75	72	64	52	45	39	26	20	0	1.63	SP	SP	45	20	BSH	2300	7	90
KUMA	1	100	100	100	98	89	76	70	69		57	53	44	33	28	0	1.5	35	18	954	504	BSH	2075	10.7	50
KUMA	2	100	100	100	100	90	78	72	65	69	63	56	49	37	33	0	1.46	33	19	1064	627	BSH	2045	10.5	40
KUMA	3	100	100	100	100	96	87	81	80		71	67	56	44	37	0	1.16	34	18	1206	666	BSH	2045	12	45
KUMA	4	100	100	100	100	98	87	81	80		71	67	56	44	37	0	1.16	NP	NP			BSH	2278	4.2	140
LEMI	1	100	100	100	98	87	65	51	50		36	33	27	20	17	0	2	31	16	528	272	BSH	2200	11.4	53
LEMI	2	100	100	98	98	86	63	52	50	45	39	34	30	22	20	0	1.96	29	17	578	340	BSH	2290	8.9	70
LEMI	3	100	100	100	99	91	71	58	55		41	38	33	24	19	0	1.88	30	15	570	285	BSH	2190	10.5	40
LEZA	1	100	100	100	98	95	82	67	65	54	46	42	39	33	28	0	1.65	39	17	714	476	BSH	2110	7.8	35
LEZA	2	100	100	100	100	93	72	60	58		42	38	32	25	21	0	1.83	SP	SP	38	21	BSH	2130	8.9	85
MIMY	1	100	100	96	93	89	78	67	64		42	37	29	20	16	0	1.83	34	16	592	256	BSH	2120	9.5	45
MYGI	1	100	100	98	97	92	84	73	70	62	49	43	38	28	23	0	1.64	30	16	688	368	BSH	2165	7.5	45
NADA	1	100	100	80	62	44	33	28	26		19	18	14	8	6	0	2.5	NP	NP	18	6	BSH	2239	5.6	130

Table A-11 Roadbase characteristics for Botswana test sections

Site	No	% Passing British Standard Sieve (mm)																				Compactive effort	Design			
		53	37.5	26.5	19	13.2	9.5	4.75	2.36	2	1.18	0.6	0.425	0.300	0.150	0.075	IR	GM	WL	Ip	PM		PP	MDD	OMC	CBR
KAKE	1	100	100	98	95	89	82	57		32	24		16		10	8	0	2.44	34	11	176	88	BSH	1970	13	55
NAKA	1																						BSH	2280	6.6	60
NAKA	2	99	90	75	66	56	51	42		33	29		22		13	10	10	2.35	28	10	220	100	BSH	2380	6	85
NAKA	4	100	100	99	94	86	75	55		37	31		24		16	12	0	2.27	NP	NP	24	12	BSH	2320	4.8	95
NAMN	1	100	100	100	88	81	76	61		51	47		38		20	15	0	1.96	52	21	798	315	BSH	1860	12.5	80
OASE	1-2	100	100	100	100	100	100	99		97	97		57		39	22	0	1.24	NP	NP	57	22	BSH	1940	6.9	35
SATU	1	100	100	98	97	97	95	90		84	82		74		25	12	0	1.3	NP	NP	74	12	BSH	1890	9.8	35
TUNG	1	100	100	100	100	99	99	93		86	84		77		26	12	0	1.25	NP	NP	77	12	BSH	1895	8.7	30

Table A-12 Sub-base characteristics for Zimbabwe and Malawi test sections

Site	No	% Passing British Standard Sieve (mm)															Compactive effort					Design					
		53	37.5	26.5	19	13.2	9.5	4.75	2.36	2	1.18	0.6	0.425	0.300	0.150	0.075	IR	GM	WL	Ip	PM	PP	BSH	MDD	OMC	CBR	
BOKI	1	100	100	97	95		82	65	52	49	41	31	26	22	16	12	0	2.13	SP	SP	26	12	BSH	2220	7.5	45	
GECY	1																						BSH	2180	6.8	55	
KIBA	5																						BSH	1980	7	17	
RENA	1	100	100	100	98		89	79	71	67	62	51	45	37	27	24	0	1.64	SP	SP	45	24	BSH	2190	8.2	70	
WAMI	1	100	100	94	87		71	54	46	43	39	32	27	24	18	14	0	2.16	SP	SP	27	14	BSH	2195	5.7	150	
CABA	1	100	100	100	100		100	100	100	100	96	81	71	54	36	30	0	0.99	27	15	1065	450	BSH	2080	7.5	18	
CAKA	1	100	100	100	95		90	86	79	70		46	41	32	23	19	0	1.7	28	12	492	228	BSH	2170	5.9	30	
CAJE	1	100	100	92	88		79	69	56	50		34	31	26	20	16	0	2.03	32	16	496	256	BSH	2010	10.9	5	
GINA	1	100	100	100	100		97	92	82	75		54	49	42	29	21	0	1.55	NP	NP	49	21	BSH	2240	8		
KUMA	1	100	100	100	100		97	87	81	77		70	65	52	38	33	0	1.25	25	12	780	396	BSH	2040	11.4	30	
KUMA	2	100	100	100	99		91	80	75	70		64	59	48	37	32	0	1.39	35	16	944	512	BSH	2100	10.4	35	
KUMA	3	100	100	100	100		97	86	82	81		80	76	64	51	45	0	0.98	34	18	1368	810	BSH	2050	11.7	25	
KUMA	4																					BSH	2100	11.2	18		
LEMI	1	100	100	100	100		98	82	70	65		53	49	41	30	26	0	1.6	33	16	784	416	BSH	2100	11.4	10	
LEMI	2	100	100	100	100		90	74	63	60		46	42	36	27	22	0	1.76	25	15	630	330	BSH	2080	12.2	14	
LEMI	3	100	100	95	93		86	76	69	65		54	50	42	32	27	0	1.58	33	14	700	378	BSH	2185	10.2	35	
LEZA	1	100	100	100	98		93	86	77	74		60	57	51	40	35	0	1.34	44	17	969	595	BSH	2060	10	30	
LEZA	2	100	100	100	100		98	87	71	68		47	44	40	33	29	0	1.59	33	15	660	435	BSH	2190	9.6	70	
MIMY	1	100	100	100	98		96	94	92	86		66	59	47	35	29	0	1.26	SP	SP	59	29	BSH	2160	8.2	17	
MYGI	1	100	100	100	100		91	82	70	66		45	42	36	26	20	0	1.72	26	13	546	260	BSH	2135	6.6	30	
NADA	1	100	100	100	100		98	87	79	66		62	56	43	26	20	0	1.58	NP	NP	56	20	BSH	2220	8	50	
KAKE	1	89	83	83	81	73	67	57		34	26		18		11	9	17	2.39	29	9	162	81	BSH	2040	11.6	45	
NAKA	1	100	100	98	96	92	85	60		42	36		30		22	19	0	2.09	50	20	600	380	BSH	2010	11.4	20	
NAKA	2	99	86	68	59	49	43	31		21	18		14		11	9	14	2.56	38	17	238	153	BSH	2210	7.8	70	
NAKA	4	100	100	100	99	94	87	70		55	48		31		19	90	0	1.24	21	5	155	450	BSH	2310	5.8	60	
NAMN	1	100	100	98	84	72	65	54		48	46		37		18	12	0	2.03	35	14	518	168	BSH	1915	11.7	30	
ORSE	1/2	100	100	100	100	99	97	95		94	93		55		28	9	0	1.42	NP	NP	55	9	BSH	2040	7.2	40	
SATU	1	100	100	100	100	100	99	97		95	94		78		26	12	0	1.15	NP	NP	78	12	BSH	1885	8.5	25	
TUNG	1	100	100	100	100	100	99	96		91	89		82		29	14	0	1.13	NP	NP	82	14	BSH	1930	10	30	

Appendix B: Performance of test sections

Table B-1 In situ CBR and structural numbers for test sections in Zimbabwe

Site	No	Lane	Wheel-track	Wet season CBR			Wet season structural No's				Dry season CBR			Dry season structural No's			
				Base	Sub-base	Subgrade	BSN	SN	SNC	SGC	Base	Sub-base	Subgrade	BSN	SN	SNC	SGC
BOKI	1	LHS	OWT	150	150	125	0.14	1.277	3.47	2.19	150	150	125	0.14	1.277	3.47	2.19
BOKI	1	CL	CL	150	150	150	0.14	1.277	3.46	2.18	150	150	150	0.14	1.277	3.46	2.18
BOKI	1	RHS	OWT	150	150	150	0.14	1.277	3.46	2.18	150	150	150	0.14	1.277	3.46	2.18
GECY	1	LHS	OWT	150	150	80	0.14	2.611	4.78	2.17	150	150	150	0.14	2.611	4.79	2.18
GECY	1	LHS	IWT	150	150	80	0.14	2.611	4.78	2.17	150	150	150	0.14	2.388	4.57	2.18
GECY	1	CL	CL	150	150	80	0.14	2.611	4.78	2.17	150	150	100	0.14	2.611	4.80	2.19
GECY	1	RHS	IWT	150	150	80	0.14	2.611	4.78	2.17	150	150	150	0.14	2.611	4.79	2.18
GECY	1	RHS	OWT	150	150	80	0.14	2.611	4.78	2.17	150		150	0.14	2.611	4.79	2.18
GECY	2	LHS	OWT	140		50	0.14	0.85	2.93	2.08	150		30	0.14	0.851	2.75	1.90
GECY	2	LHS	IWT	150		90	0.14	0.851	3.03	2.18							
GECY	2	CL	CL	150		55	0.14	0.851	2.96	2.10	150		65	0.14	0.851	2.99	2.14
GECY	2	RHS	IWT	150		65	0.14	0.851	2.99	2.14							
GECY	2	RHS	OWT	135		55	0.14	0.849	2.95	2.10	145		50	0.14	0.851	2.93	2.08
GECY	3	LHS	OWT	150		100	0.14	1.561	3.75	2.19	150		150	0.14	1.561	3.74	2.18
GECY	3	LHS	IWT	150		80	0.14	1.561	3.73	2.17							
GECY	3	CL	CL	150		100	0.14	1.561	3.75	2.19	150		90	0.14	1.561	3.74	2.18
GECY	3	RHS	IWT	150		150	0.14	1.561	3.74	2.18							
GECY	3	RHS	OWT	140		85	0.14	1.558	3.74	2.18	140		130	0.14	1.558	3.75	2.19
HSMO	1	LHS	OWT	55		50	0.11	0.51	2.59	2.08	105		45	0.14	0.662	2.71	2.05
HSMO	1	LHS	IWT	120		55	0.14	0.674	2.78	2.10	140		70	0.14	0.68	2.83	2.15
HSMO	1	CL	CL	150		90	0.14	0.681	2.86	2.18	115		60	0.14	0.671	2.80	2.12
HSMO	1	RHS	IWT	100		50	0.14	0.655	2.74	2.08	120		65	0.14	0.674	2.81	2.14
HSMO	1	RHS	OWT	55		55	0.11	0.51	2.61	2.10	90		55	0.13	0.637	2.74	2.10
HSMO	2	LHS	OWT	40		16	0.09	0.38	1.94	1.56	75		30	0.13	0.547	2.45	1.90
HSMO	2	LHS	IWT	65		28	0.12	0.512	2.38	1.87	95		40	0.14	0.593	2.60	2.01
HSMO	2	CL	CL	95		70	0.14	0.593	2.75	2.15	120		12	0.14	0.618	1.99	1.37
HSMO	2	RHS	IWT	90		60	0.13	0.584	2.71	2.12	115		60	0.14	0.615	2.74	2.12
HSMO	2	RHS	OWT	35		35	0.08	0.345	2.31	1.96	75		50	0.13	0.547	2.63	2.08
KIBA	1	LHS	OWT	40		45	0.09	0.242	2.29	2.05	150		150	0.14	0.397	2.58	2.18
KIBA	1	LHS	IWT	85		70	0.13	0.365	2.52	2.15	150		75	0.14	0.397	2.56	2.16
KIBA	1	CL	CL	70		75	0.12	0.338	2.50	2.16	150		115	0.14	0.397	2.59	2.19
KIBA	1	RHS	IWT	90		80	0.13	0.372	2.54	2.17	150		80	0.14	0.397	2.57	2.17
KIBA	1	RHS	OWT	45		120	0.10	0.262	2.46	2.19	140		80	0.14	0.397	2.57	2.17
KIBA	2	LHS	OWT	65		65	0.12	0.931	3.07	2.14	120		45	0.05	0.399	2.59	2.19
KIBA	2	LHS	IWT	150		140	0.14	1.135	3.32	2.19	150		20	0.06	0.482	2.66	2.18
KIBA	2	CL	CL	150		150	0.14	1.135	3.32	2.18	150		25				

continued

Site	No	Lane	Wheel-track	Wet season CBR			Wet season structural No's				Dry season CBR			Dry season structural No's			
				Base	Sub-base	Subgrade	BSN	SN	SNC	SGC	Base	Sub-base	Subgrade	BSN	SN	SNC	SGC
KIBA	2	RHS	IWT	150		130	0.14	1.135	3.33	2.19	150		50	0.14	1.135	3.21	2.08
KIBA	2	RHS	OWT	35		150	0.08	0.628	2.81	2.18	150		80	0.14	1.135	3.31	2.17
KIBA	3	LHS	OWT	55		40	0.11	0.85	2.86	2.01	50		45	0.10	0.802	2.85	2.05
KIBA	3	LHS	IWT	60		27	0.11	0.893	2.75	1.85	25		55	0.06	0.482	2.59	2.10
KIBA	3	CL	CL	70		40	0.12	0.965	2.98	2.01	25		50	0.06	0.482	2.56	2.08
KIBA	3	RHS	IWT	145		60	0.14	1.134	3.26	2.12	25		95	0.06	0.482	2.67	2.19
KIBA	3	RHS	OWT	85		70	0.13	1.043	3.20	2.15	25		115	0.06	0.482	2.68	2.19
KIBA	4	LHS	OWT	90		130	0.13	1.062	3.25	2.19	150		150	0.14	1.135	3.32	2.18
KIBA	4	LHS	IWT	35		20	0.08	0.628	2.33	1.70	150		100	0.14	1.135	3.33	2.19
KIBA	4	CL	CL	20		28	0.05	0.399	2.27	1.87	80		150	0.13	1.021	3.20	2.18
KIBA	4	RHS	IWT	23		8	0.06	0.45	1.50	1.05	100		150	0.14	1.092	3.28	2.18
KIBA	4	RHS	OWT	150		150	0.14	1.135	3.32	2.18	150		150	0.14	1.135	3.32	2.18
KIBA	5	LHS	OWT	90		150	0.13	0.691	2.81	2.12	150		145	0.14	0.851	3.04	2.19
KIBA	5	LHS	OWT	80		150	0.13	0.663	2.71	2.05	105		150	0.14	0.827	3.01	2.18
KIBA	5	CL	CL	75		150	0.13	0.647	2.83	2.19	65		70	0.12	0.605	2.76	2.15
KIBA	5	RHS	IWT	120		150	0.14	0.731	2.92	2.19	60		55	0.11	0.67	2.77	2.10
KIBA	5	RHS	OWT	60		150	0.11	0.58	2.63	2.05	60		50	0.11	0.67	2.75	2.08
MAME	1	LHS	OWT	80		75	0.13	0.868	3.03	2.16	85		85	0.14	0.965	3.14	2.18
MAME	1	LHS	IWT	90		100	0.13	0.903	3.09	2.19	150		150	0.14	0.965	3.15	2.18
MAME	1	CL	CL	135		120	0.14	0.962	3.16	2.19	150		150	0.14	0.965		
MAME	1	RHS	IWT	100		135	0.14	0.928	3.12	2.19	100		85	0.14	0.928	3.11	2.18
MAME	1	RHS	OWT	110		125	0.14	0.945	3.14	2.19	115		150	0.14	0.951	3.13	2.18
MAMI	1	LHS	OWT	60		60	0.11	0.67	2.79	2.12	70		85	0.12	0.724	2.90	2.18
MAMI	1	LHS	IWT	80		80	0.13	0.766	2.94	2.17	75		100	0.13	0.746	2.94	2.19
MAMI	1	CL	CL	60		60	0.11	0.67	2.79	2.12	60		70	0.11	0.67	2.82	2.15
MAMI	1	RHS	IWT	40		50	0.09	0.519	2.60	2.08	55		65	0.11	0.638	2.78	2.14
MAMI	1	RHS	OWT	60		50	0.11	0.67	2.75	2.08	65		65	0.12	0.698	2.84	2.14
MAMI	2	LHS	OWT	55		50	0.11	0.68	2.76	2.08	70		60	0.14	0.874	3.03	2.15
MAMI	2	LHS	IWT	90		90	0.13	0.85	3.03	2.18	105		100	0.14	0.883	3.07	2.19
MAMI	2	CL	CL	105		115	0.14	0.883	3.08	2.19	100		105	0.14	0.874		
MAMI	2	RHS	IWT	90		100	0.13	0.85	3.04	2.19	85		85	0.13	0.835	3.01	2.18
MAMI	2	RHS	OWT	50		45	0.10	0.642	2.69	2.05	70		65	0.12	0.772	2.91	2.14
MAMI	3	LHS	OWT	65		50	0.12	0.745	2.82	2.08	65		50	0.13	0.796	2.94	2.14
MAMI	3	LHS	IWT	70		65	0.12	0.772	2.91	2.14	50		75	0.12	0.745	2.82	2.08
MAMI	3	CL	CL	80		65	0.13	0.817	2.96	2.14	70		65	0.13	0.796	2.95	2.15
MAMI	3	RHS	IWT	85		60	0.13	0.835	2.96	2.12	85		75	0.13	0.835		
MAMI	3	RHS	OWT	60		45	0.11	0.714	2.76	2.05	65		50	0.12	0.745	2.82	2.08
MAMI	4	LHS	OWT	40		55	0.09	0.484	2.59	2.10	55		65	0.11	0.595	2.73	2.14
MAMI	4	LHS	IWT	50		85	0.10	0.562	2.74	2.18	60		80	0.11	0.625	2.80	2.17
MAMI	4	CL	CL	65		100	0.12	0.652	2.84	2.19	65		105	0.00	0	2.19	2.19
MAMI	4	RHS	IWT	60		70	0.11	0.625	2.78	2.15	60		95	0.00	0.755	2.94	2.19
MAMI	4	RHS	OWT	45		55	0.10	0.525	2.63	2.10	55		65	0.00	0.652	2.79	2.14
MAMI	5	LHS	OWT	70		65	0.12	0.868	3.01	2.14	70		95	0.12	0.868	3.06	2.19

continued

Table B-2 In situ CBR and structural numbers for test sections in Malawi

Site	No	Lane	Wheel-track	Wet season CBR			Wet season structural No's				Dry season CBR			Dry season structural No's			
				Base	Sub-base	Subgrade	BSN	SN	SNC	SGC	Base	Sub-base	Subgrade	BSN	SN	SNC	SGC
CABA	1	LHS	OWT	75	20	35	0.13	1.429	3.39	1.96	90	60	75	0.13	1.684	3.85	2.16
CABA	1	LHS	IWT								65	50	60	0.12	1.558	3.68	2.12
CABA	1	CL	CL	150	110	150	0.14	1.85	4.03	2.18	90	75	50	0.13	1.726	3.81	2.08
CABA	1	RHS	IWT								80	80	45	0.13	1.709	3.76	2.05
CABA	1	RHS	OWT	75	30	28	0.13	1.506	3.38	1.87	80	70	60	0.13	1.684	3.81	2.12
CAKA	1	LHS	OWT	115	70	70	0.14	0.827	2.88	2.05	95	80	30	0.07	0.463	2.63	2.17
CAKA	1	LHS	IWT	100	75	75	0.14	0.81	2.77	1.96	135	65	30	0.06	0.392	2.53	2.14
CAKA	1	CL	CL	120	65	65	0.14	0.831	2.63	1.80	120	65	23				
CAKA	1	RHS	IWT	90	60	60	0.13	0.791	2.69	1.90	110	60	35	0.14	0.823	2.79	1.96
CAKA	1	RHS	OWT	85	65		0.13	0.778	2.74	1.96	80	50	24	0.13	0.763	2.56	1.80
CEJE	1	LHS	OWT	90	65	65	0.13	1.202	3.39	2.19	95	150	150	0.14	1.262	3.45	2.18
CEJE	1	LHS	IWT	150	150	150	0.14	1.306	3.50	2.19	90	100	110	0.13	1.226		
CEJE	1	CL	CL	150	135	135	0.14	1.3	3.48	2.18	150	150	150	0.14	1.306	3.49	2.18
CEJE	1	RHS	IWT	100	125	125	0.14	1.262	3.46	2.19	55	130	115	0.11	1.07	3.26	2.19
CEJE	1	RHS	OWT	60	65		0.11	1.066	3.25	2.18	150	130	150	0.14	1.298	3.48	2.18
GINA	1	LHS	OWT	35	50	27	0.08	0.825	2.68	1.85	50	30	45				
GINA	1	LHS	IWT														
GINA	1	CL	CL	45	25	40	0.10	0.797	2.81	2.01	35	30	35	0.08	0.768	2.73	1.96
GINA	1	RHS	IWT														
GINA	1	RHS	OWT	40	40	50	0.09	0.826	2.91	2.08	30	40	40	0.07	0.772	2.78	2.01
KUMA	1	LHS	OWT	50	55	45	0.10	1.163	3.21	2.05	50	55	45	0.10	1.163	3.21	2.05
KUMA	1	LHS	IWT														
KUMA	1	CL	CL	45	50	35	0.10	1.119	3.08	1.96	55	50	45	0.11	1.179	3.23	2.05
KUMA	1	RHS	IWT														
KUMA	1	RHS	OWT	50	50	29	0.10	1.15	3.04	1.89	55	50	30	0.11	1.179	3.08	1.90
KUMA	2	LHS	OWT	45	55	27	0.10	1.475	3.33	1.85	40	50	30	0.09	1.412	3.31	1.90
KUMA	2	LHS	IWT														
KUMA	2	CL	CL	50	60	35	0.10	1.533	3.50	1.96	45	75	40	0.10	1.53	3.54	2.01
KUMA	2	RHS	IWT														
KUMA	2	RHS	OWT	30	35	16	0.07	1.241	2.81	1.56	60	50	23	0.11	1.573	3.35	1.77
KUMA	3	LHS	OWT	40	30	30	0.09	1.119	3.02	1.90	45	35	50	0.10	1.184	3.26	2.08
KUMA	3	LHS	IWT								23			0.06			
KUMA	3	CL	CL	50	30	30	0.10	1.203	3.10	1.90		55	35	0.00	0.682	2.64	1.96
KUMA	3	RHS	IWT											0.10			
KUMA	3	RHS	OWT	60	50	40	0.11	1.339	3.35	2.01	60	55	50				
KUMA	4	LHS	OWT	150	40	18	0.14	1.751	3.39	1.64	150	50	24	0.14	1.785	3.58	1.80
KUMA	4	LHS	IWT														
KUMA	4	CL	CL	150	55	20	0.14	1.8	3.50	1.70	150	45	30	0.14	1.769	3.67	1.90
KUMA	4	RHS	IWT				0.05										
KUMA	4	RHS	OWT	150	30	20					150	55	40	0.14	1.8	3.81	2.01

continued

Site	No	Lane	Wheel-track	Wet season CBR			Wet season structural No's				Dry season CBR			Dry season structural No's			
				Base	Sub-base	Subgrade	BSN	SN	SNC	SGC	Base	Sub-base	Subgrade	BSN	SN	SNC	SGC
LEMI	1	LHS	OWT	115	65	55	0.14	1.414	3.52	2.10	90	65	75				
LEMI	1	LHS	IWT	95	75	45	0.14	1.418	3.47	2.05	65	50	60	0.13	1.386	3.55	2.16
LEMI	1	CL	CL	125	90	40	0.14	1.473	3.48	2.01	90	75	50	0.12	1.277	3.40	2.12
LEMI	1	RHS	IWT	90	80	45	0.13	1.421	3.47	2.05	80	80	45	0.13	1.41	3.49	2.08
LEMI	1	RHS	OWT	105	70	55	0.14	1.419	3.52	2.10	80	70	60	0.13	1.4	3.45	2.05
LEMI	2	LHS	OWT	85	55	45	0.13	1.557	3.61	2.05	90	80	80				
LEMI	2	LHS	IWT	115	65	100	0.14	1.638	3.83	2.19	120	70	95	0.13	1.633	3.80	2.17
LEMI	2	CL	CL	120	85	100	0.14	1.687	3.88	2.19	140	85	100	0.14	1.654	3.84	2.19
LEMI	2	RHS	IWT	110	65	105	0.14	1.633	3.83	2.19	125	85	120	0.14	1.693	3.88	2.19
LEMI	2	RHS	OWT	90	60	70	0.13	1.585	3.74	2.15	130		90	0.14	1.689	3.88	2.19
LEMI	3	LHS	OWT	70	55	28	0.12	1.501	3.37	1.87	70	60	35	0.12	1.511	3.47	1.96
LEMI	3	LHS	IWT	65	55	22	0.12	1.469	3.22	1.75	65	50	19	0.12	1.459	3.13	1.67
LEMI	3	CL	CL	80	105	55	0.13	1.626	3.73	2.10	80	95	60	0.13	1.615	3.74	2.12
LEMI	3	RHS	IWT	55	70	55	0.11	1.419	3.52	2.10	55	60	50	0.11	1.402	3.48	2.08
LEMI	3	RHS	OWT	70	55	45	0.12	1.501	3.55	2.05	70	60	45	0.12	1.511	3.56	2.05
LEZA	1	LHS	OWT	65	18	21	0.12	1.318	3.04	1.72	60	23	25				
LEZA	1	LHS	IWT														
LEZA	1	CL	CL	140	45	25	0.14	1.623	3.44	1.82	150	150	150	0.14	1.851	4.03	2.18
LEZA	1	RHS	IWT														
LEZA	1	RHS	OWT	55	16	24	0.11	1.243	3.04	1.80	55	26	40	0.11	1.335	3.35	2.01
LEZA	2	LHS	OWT	150	60	35	0.14	1.459	3.42	1.96	150	120	55	0.14	1.582	3.69	2.10
LEZA	2	LHS	IWT	150	150	100	0.14	1.622	3.81	2.19	150	150	95	0.14	1.622	3.81	2.19
LEZA	2	CL	CL	150	150	80	0.14	1.622	3.79	2.17	150	150	65	0.14	1.622	3.76	2.14
LEZA	2	RHS	IWT	150	150	95	0.14	1.622	3.81	2.19	150	120	70	0.14	1.582	3.73	2.15
LEZA	2	RHS	OWT	150	95	35	0.14	1.54	3.50	1.96	150	95	65	0.14	1.54	3.68	2.14
MIMY	1	LHS	OWT	150	30	18	0.14	1.476	3.11	1.64	150	40	35	0.14	1.533	3.50	1.96
MIMY	1	LHS	IWT														
MIMY	1	CL	CL	150	35	24	0.14	1.506	3.30	1.80	150	80	45	0.14	1.672	3.72	2.05
MIMY	1	RHS	IWT											0.10			
MIMY	1	RHS	OWT	150	140	35	0.14	1.784	3.75	1.96	150	45	50				
MYGI	1	LHS	OWT	65	35	65	0.12	1.444	3.58	2.14	150	140	45	0.14	1.838	3.89	2.05
MYGI	1	LHS	IWT	55	30	21	0.11	1.362	3.09	1.72	150	55	29	0.14	1.662	3.55	1.89
MYGI	1	CL	CL	150	35	40	0.14	1.576	3.59	2.01	75	60	24	0.13	1.587	3.38	1.80
MYGI	1	RHS	IWT	45	50	27	0.10	1.393	3.25	1.85	75	70	41	0.13	1.616	3.64	2.02
MYGI	1	RHS	OWT	70	90	50	0.12	1.644	3.72	2.08	150	150	54	0.14	1.851	3.95	2.10
NADA	1	LHS	OWT	55	35	50	0.11	1.257	3.34	2.08	85	70	85				
NADA	1	LHS	IWT	85	70	90	0.13	1.498	3.68	2.18	95	75	100	0.14	1.536	3.73	2.19
NADA	1	CL	CL	135	100	105	0.14	1.614	3.81	2.19	130	80	120	0.14	1.587	3.78	2.19
NADA	1	RHS	IWT	95	90	90	0.14	1.556	3.74	2.18	95	85	100	0.14	1.55	3.74	2.19
NADA	1	RHS	OWT	75	60	55	0.13	1.44	3.54	2.10	100	100	75	0.14	1.579	3.74	2.16

Table B-4 Roadbase moisture condition for climatic zone N<2

Site	No	Left hand side		Centre-line	Right hand side	
		OWT	IWT		IWT	OWT
KAKE	1	0.58	0.55	0.47	0.40	0.60
KIBA	3	0.96	0.69	0.83	0.97	1.11
KIBA	4	0.47	0.42	0.60	0.44	0.74
KIBA	5	0.55	0.43	0.51	0.41	0.64
MIAS	1	0.55	0.72	0.66	0.70	0.72
MIAS	2	0.53	0.53	0.57	0.54	0.64
MIAS	3	0.67	0.86	0.94	0.87	0.62
MAME	1	0.64	0.65	0.69	0.63	0.65
MAMI	1	0.89	0.81	0.84	0.86	0.91
MAMI	2	0.60	0.48	0.60	0.54	0.68
MAMI	4	0.74	0.62	0.58	0.54	0.72
MAMI	3	0.66	0.52	0.57	0.49	0.61
MAMI	5	1.11	0.94	0.92	1.09	0.95
WAMI	1	0.63	0.58	0.53	0.58	0.66
GECY	1	0.56	0.59	0.63	0.64	0.66
GECY	2	0.65	0.67	0.55	0.61	0.61
GECY	3	0.56	0.55	0.56	0.57	0.56
HSMO	1	0.96	0.53	0.44	0.61	0.89
HSMO	2	0.77	0.70	0.70	0.95	0.85
KIBA	1	0.35	0.33	0.43	0.41	0.44
KIBA	2	0.84	0.52	0.56	0.56	0.95
NARE	1	0.61	0.61	0.53	0.66	0.78
NARE	2	0.71	0.64	0.67	0.69	0.76
NARE	3	0.58	0.51	0.37	0.42	0.64
NARE	4	0.42	0.29	0.36	0.31	0.37
NARE	5	0.63	0.51	0.46	0.43	0.52
NARE	6	0.47	0.50	0.50	0.42	0.52
NARE	7	0.41	0.43	0.45	0.41	0.40
NARE	7	0.42	0.56	0.50	0.44	0.59
NARE	7	0.51	0.53	0.54	0.44	0.56
NARE	8	0.51	0.45	0.52	0.56	0.61
NARE	9	0.68	0.57	0.66	0.65	0.79
RENA	1	0.76	0.00	0.80	0.00	0.83

Table B-5 Roadbase moisture condition for climatic zone N=2-4

Site	No	Left hand side		Centre-line	Right hand side	
		OWT	IWT		IWT	OWT
CABA	1	0.9133		0.5093		0.624
CAJE	1	0.6404		0.7509		0.6333
CAKA	1	0.8145		0.671		0.7029
GINA	1	0.8386		0.9057		1.0343
KUMA	1	1.0542		0.9112		1.1355
KUMA	2	0.8333		0.8105		0.9495
KUMA	3	1.0842		1.2458		1.1658
KUMA	4	1.0214		0.9857		0.7333
LEMI	1	0.6281	0.7377	0.6947	0.6386	0.7746
LEMI	2	1.0022	0.8865	0.8719	0.9753	0.9337
LEMI	3		0.8438	0.8124	0.7162	0
LEZA	1	1.2833		1.1231		1.0987
LEZA	2	1.0315	0.8899	0.8596	0.9236	0.9708
MIMY	1	0.7021		0.6979		0.8126
MYGI	1	0.892		0.8507		0.852
NADA	1	0.3607	0.3518	0.2786	0.3161	0.3661

Table B-6 Roadbase moisture condition for climatic zone N>4

Site	No	Left hand side		Centre-line	Right hand side	
		OWT	IWT		IWT	OWT
NAKA	1	0.55	0.54	0.61	0.56	0.71
NAKA	2			0.51	0.61	0.55
NAKA	3					
NAKA	4	0.40	0.34	0.34	0.35	0.42
NAMN	1	0.63	0.64	0.58	0.68	0.42
SATU	1	0.56	0.46	0.58	0.61	0.80
ORSE	1	0.45	0.50	0.51	0.53	0.50
ORSE	2	0.56	0.85	0.53	0.64	0.57
TUNG	1	0.40	0.58	0.63	0.53	0.52
TOSA	1	0.92	0.79	0.74	0.68	0.96
SSMA	1	0.63	0.43	0.37	0.41	0.43

Table B-7 In situ roadbase densities for test sections in Zimbabwe

Site	No	CBR LHS OWT	Relative Density	CBR LHS IWT	Relative Density	CBR centreline	Relative Density	CBR RHS IWT	Relative Density	CBR RHS OWT	Relative Density
BOKI	1	150				150	93.4			150	
GECY	1	150	93.9	150	95.4	150	94.7	150	97.3	150	95.1
GECY	2	133	111	150	110	150	112.8	150	109.4	142	110.5
GECY	3	150	112.9	150	109.9	80	107.7	150	107	131	111.9
HSMO	1	53	92.3	102	88	150	103.4	120	94.7	55	92.7
HSMO	2	33	89.4	89	99.3	94	99.8	67	101.2	38	95.4
KIBA	1	43	98.5	90	99.8	68	96.8	83	97	42	95.4
KIBA	2	34	99.3	150	94	150	92.9	150	96	65	93.4
KIBA	3	84	97.4	144	102.3	71	98.8	61	101.9	55	103.5
KIBA	4		94.3		97.4		98		96.5	87	104.2
KIBA	5	57	97.6	119	96.6	73	96	80	90.8	89	90.2
MAME	1	110	106.5	97	104.1	136	104.4	91	102	80	105.5
MAMI	1	58	93.9	40	92.1	62	92.3	62	90.6	63	93.1
MAMI	2	51	92.3	89	91.5	106	93.3	92	94.7	54	94
MAMI	3	59	94.4	84	92.1	77	92.7	70	89.9	63	94.5
MAMI	4	45	93.4	61	94	64	95.2	51	95	41	93.1
MAMI	5	57		104		108		69		68	
MIAS	1	112	99.7	129	99.4	192	94.9	103	94.7	91	95.1
MIAS	2	44	89.2	96	91.1	104	91.2	60	91.3	69	87.2
MIAS	3	47	96.8	113	96.8	119	97.4	150	100.1	126	96.6
NARE	1	68	97.9	150	104.8	150	103	150	98	59	93.4
NARE	2	43	97.1	49	97.7	67	99.6	55	91.1	48	93.5
NARE	3	37	98.8	50	110.9	65	107.3	40	107.1	38	104.5
NARE	4	53	99.9	84	97.9	82	98	64	95.5	62	98.3
NARE	5	80	105.6	104	107.3	137	106.11	150	102.9	150	105.3
NARE	6	52	96.9	150	99.9	110	101.6	84	102.5	81	99.6
NARE	7	75	107.2	150	102.1	150	102.5	137	108.4	86	104.1
NARE	8	84	104.1	77	103.5	88	104.4	77	105.1	50	100.3
NARE	9	62	108.7	150	102.6	107	97.4	84	101.9	64	100.5
RENA	1	75	94.5			150	93.5			82	96.5
TOSA	1	85	102.1	129	89	113	94.4	92	94.7	69	96.3
WAMI	1	131	106.9	118	107.4	150	109.9	123	107.4	134	107.4

NB Relative density as a percentage of British Standard 4.5kg rammer (heavy) compaction

Table B-8 In situ roadbase densities for test sections in Malawi

Site	No	CBR		Relative Density	CBR		Relative Density	CBR		Relative Density	CBR		Relative Density
		LHS	OWT		LHS	IWT		centreline	RHS		IWT	RHS	
KAKE	1	74		100.2	94		102.3	106		103.6	106		103.7
NAKA	1	86		100.2	95		107.8	74		101.8	82		104.1
NAKA	2							150		103.1	106		99.3
NAKA	4	150		111.8	150		102.5	150		112	150		111.5
NAMN	1	52		94.5	94		98.9	48		101.5	61		101.6
ORSE	1	51		99	80		97.9	89		94.5	71		98.1
ORSE	2	73			92			71			71		96
SATU	1	104		85.5	150		86.9	150		86.8	150		85.7
SSMA	1	60		98.4	81		99.5	139		96.1	86		102.3
TUNG	1	64		90.5	59		91.2	65		93.7	41		96.4

NB Relative density as a percentage of British Standard 4.5kg rammer (heavy) compaction

Table B-9 In situ roadbase densities for test sections in Botswana

Site	No	CBR		Relative Density	CBR		Relative Density	CBR		Relative Density	CBR		Relative Density
		LHS	OWT		LHS	IWT		centreline	RHS		IWT	RHS	
CABA	1	74		106				150		93.6			77
CAJE	1	60		97	99		93.5	166		94.7	152		95.5
CAKA	1	84		108.2	92		111.1	122		109.6	99		112.4
GINA	1	42		99.2				47		98.9			35
KUMA	1	47		93.2				44		91.2			50
KUMA	2	32		96.4				52		96.3			44
KUMA	3	63		93.8				51		93.2			38
KUMA	4	150		98.3				150		102.4			150
LEMI	1	106		91.3	88		89	124		96.4	93		89.8
LEMI	2	93		96.8	110		93.7	119		93.8	116		90.8
LEMI	3	53		97.5	54		89.4	67		93	65		89.8
LEZA	1	51		89.7				138		94			63
LEZA	2	150		97.9	150		95.6	150		98.2	150		97.4
MIGI	1	72		84.8	45		98.1	150		93.8	54		98.8
MIMY	1	141		101.8				150		99.6			150
NADA	1	75		110.4	94		107.3	135		108	86		106.1

NB Relative density as a percentage of British Standard 4.5kg rammer (heavy) compaction

Table B-10 Deflection results

Site	Dry season 1995								Wet season 1996							
	OWT LHS		IWT LHS		IWT RHS		OWT RHS		OWT LHS		IWT LHS		IWT RHS		OWT RHS	
	Mean	SD	Mean	SD	Mean	SD	Mean	SD	Mean	SD	Mean	SD	Mean	SD	Mean	SD
Zimbabwe																
BOKI1	399	83					465	145	273	1					295	3
GECY1	444	102	323	117	235	101	305	154	457	43	306	102	214	51	194	14
GECY2	444	102	323	117	235	101	305	154	585	2	509	26	550	9	611	38
GECY3									481	26	425	22	424	14	519	9
KIBA1	271	70	306	114	386	128	281	120	267	55	292	67	448	174	374	54
KIBA2	185	5	190	19	262	84	173	81	538	71	186	19	260	41	358	31
KIBA3	2618	3	2052	671	1630	746	1826	974	1983	132	1932	715	1593	760	1482	427
KIBA4	808	59	813	47	712	278	615	270	816	57	822	117	702	49	752	116
KIBA5	205	54	373	127	339	123	290	186	294	139	424	211	390	180	447	49
MAME1	617	25	657	49	629	224	511	218	712	13	635	0	642	1	629	20
MAMI1	485	8	486	11	568	195	435	196	574	3	549	7	588	5	588	5
MAMI2	504	15	465	86	555	167	432	188	708	35	473	9	506	14	747	20
MAMI3	627	9	587	8	596	218	507	224	736	6	546	21	529	19	654	29
MAMI4	681	39	488	11	488	178	502	230	681	2	546	24	567	2	852	18
MAMI5									814	12	701	27	679	22	817	22
MIAS1	388	35	336	31	400	130	297	128	328	14	280	18	336	3	367	12
MIAS2	318	127	157	32	342	118	287	103	456	52	352	113	316	46	484	48
MIAS3	808	59	813	47	712	278	615	270	377	29	255	1	373	20	355	23
RENA1	432	9	376	20	383	135	356	156	411	3	363	8	361	15	399	25
SSMA1	573	81	384	85	363	120	563	250	713	24	432	99	382	44	943	91
TOSA1	489	59	303	2	325	116	339	168	610	77	268	2	284	38	403	8
WAMI1	513	41	333	2	508	169	448	185	641	10	481	24	589	82	713	37
Malawi																
CABA1	607	36	366	72	310	116	521	240	838	56	316	2	295	13	930	130
CAJE1	336	72	467	72	290	160	279	112	325	46	353	6	350	5	324	22
CAKA1	937	26	1035	18	1066	384	773	335	1071	34	923	10	1064	17	1064	9
GINA1	1274	40	1439	64	1334	500	1071	462	1376	17	1507	79	1432	49	1399	22
KUMA1	737	10	781	2	887	312	642	285	743	14	737	19	862	68	744	27
KUMA2	754	25	626	2	658	239	584	274	600	3	600	3	634	64	634	64
KUMA3	939	42	804	32	751	283	715	319	901	1	811	45	726	37	696	49
KUMA4	310	16	317	31	322	111	268	114	393	49	269	10	274	5	302	4
LEMI1	604	12	565	30	598	205	495	217	600	6	578	33	618	12	605	7
LEMI2	289	0	472	39	432	158	236	106	380	13	443	13	454	16	476	82
LEMI3	566	6	635	34	573	218	462	205	595	34	608	32	559	53	570	17
LEZA1	623	79	902	44	936	328	592	249	1464	255	1055	77	1147	7	1828	111
LEZA2	574	75	603	46	628	215	495	196	725	119	415	102	566	98	614	105

continued

Site	Dry season 1995								Wet season 1996							
	OWT LHS		IWT LHS		IWT RHS		OWT RHS		OWT LHS		IWT LHS		IWT RHS		OWT RHS	
	Mean	SD	Mean	SD	Mean	SD	Mean	SD	Mean	SD	Mean	SD	Mean	SD	Mean	SD
MIMY1	907	110	1098	92	1000	376	704	289	1622	296	1404	73	1342	67	1534	7
MYGI1	451	26	1071	36	901	372	350	159	887	33	997	35	922	9	923	11
NADA1	791	52	513	92	681	204	547	291	415	167	998	565	632	22	744	51
Botswana																
KAKE1	600	61	610	21	548	214	542	226	737	21	580	15	529	42	887	13
NAKA1	149	16	107	5	182	64	122	49	450	36	379	16	414	1	562	14
NAKA2	349	19	426	3	353	151	285	120	438	8	460	16	468	1	537	39
NAKA3	404	44	496	6	438	176	377	158	377	30	401	3	377	25	328	2
NAKA4	346	42	377	12	403	141	296	118	218	61	171	7	148	3	166	22
NAMN1	534	9	716	99	774	257	508	241	1297	6	1322	31	1418	35	1448	107
ORSE1	302	10	292	12	362	119	303	148	347	21	321	4	377	7	509	12
ORSE2	265	7	263	6	288	100	253	118	275	9	258	5	290	2	290	2
SATU1	343	48	300	12	334	115	299	117	369	20	330	11	341	17	362	20
TUNG1	364	122	381	138	476	139	341	154	593	67	624	123	683	24	736	95

NB Deflections in microns

Table B-11 Cracking on test sections in Zimbabwe

Site	No	% Cracking Left hand side						% Cracking Right hand side						Other
		Outer	Me	Inner	Lane	Width	Type	Outer	Me	Inner	Lane	Width	Type	
BOKI	1	0	8	0	8	<1-3	T/L	0	0	0	0			
GECY	1	0	0	0	0			0	0	1	1	1-3	L	
GECY	2	0	0	0	0			0	0	0	0			
GECY	3	0	0	0	0			0	0	0	0			
HSMO	1	0	0	0	0			0	0	0	0			
HSMO	2	13	13	12	38	<1-3	L	10	2	0	12	<1-3	T/L	Edge drop
KIBA	1	17	17	17	51	1	C	10	20	20	50	1-3	C	
KIBA	1	7	10	17	34	1-3	L/T	0	13	17	30	1-3	L/T	Cracks sealed
KIBA	2	1	2	2	5	<1-3	L/C	2	2	2	6	<1-3	L/C	Cracks Sealed
KIBA	3	9	0	0	9	1-3	L	3	0	1	4	1-3	T/L	
KIBA	3	2	2	2	6	1-3	T	0	0	0	0			
KIBA	4	6	0	0	6	1-3	T/L	0	0	0	0			
KIBA	5	4	2	0	6	1-3	T/L	16	0	4	20	<1-3	B/T/L	
KIBA	5	9	0	0	9	1-3	T/L	8	0	0	8	<1	T/L	
MAME	1	0	0	0	0			0	0	0	0			
MAMI	1	0	0	0	0			0	0	0	0			Ravelling
MAMI	2	0	0	0	0			0	0	0	0			
MAMI	3	0	0	0	0			0	0	0	0			
MAMI	4	0	0	0	0			0	0	0	0			
MAMI	5	0	0	0	0			0	0	0	0			
MIAS	1	0	0	0	0			6	1	0	7	1-3	T/L	
MIAS	3	0	0	0	0			0	0	0	0			
MIAS	2	0	0	0	0			0	0	0	0			
NARE	1	0	0	0	0			0	0	0	0			
NARE	2	0	0	0	0			0	0	0	0			
NARE	3	0	0	0	0			0	0	0	0			
NARE	4	0	0	0	0			0	0	0	0			
NARE	5	0	0	0	0			0	0	0	0			
NARE	6	0	0	0	0			0	0	0	0			
NARE	7	0	0	0	0			0	0	0	0			
NARE	8	0	0	0	0			0	0	0	0			
NARE	9	0	0	0	0			0	0	0	0			
RENA	1	0	0	0	0			0	0	0	0			
SSMA	1	0	0	1	1	<1	T/L	0	0	0	0			
TOSA	1	0	0	0	0			0	0	0	0			
WAMI	1	0	0	0	0			0	0	0	0			

Type: L = Longitudinal; T = Transverse; C = Crocodile; B = Block

Width: in mm

Table B-12 Cracking on test sections in Malawi

Site	No	% Cracking Left hand side						% Cracking Right hand side						Other
		Outer	Me	Inner	Lane	Width	Type	Outer	Me	Inner	Lane	Width	Type	
CABA	1	3	20	8	31	>3	T/L	3	17	10	30	>3	C/T/L	
CAKA	1	3	20	20	43	1-3	C/T/L	3	10	20	33	<1-3	C/T/L	
CAJE	1	13	10	3	26	1-3	L/T	8	8	5	21	<1-3	L/T	
GINA	1	0	0	0	0			0	0	0	0			Minor Edge break
KUMA	1	0	0	0	0			0	0	0	0			
KUMA	2	0	0	0	0			0	0	0	0			
KUMA	3	0	0	0	0			0	0	0	0			
KUMA	4	0	0	0	0			0	0	0	0			
LEZA	1	27	27	17	71	1-3	L/C/T	30	17	20	67	1-3	C/L	Edge break
LEZA	2	0	0	0	0			0	0	0	0			Edge break
LEMI	1	0	0	0	0			0	0	0	0			
LEMI	3	0	0	0	0			7	0	0	7	1-3	T/L	Edge break
MYGI	1	20	3	0	23	>3	L/T	0	0	0	0	1-3	L/T	Loss of surfacing
MIMY	1	10	0	0	10	>3	L	0	0	0	0			
NADA	1	0	0	0	0			0	0	0	0			Minor edge break
LEMI	2	0	0	0	0			0	0	0	0			

Type: L = Longitudinal; T = Transverse; C = Crocodile; B = Block. Width: in mm

Table B-13 Cracking on test sections in Botswana

Site	No	% Cracking Left hand side						% Cracking Right hand side						Other
		Outer	Me	Inner	Lane	Width	Type	Outer	Me	Inner	Lane	Width	Type	
KAKE	1	0	0	0	0			0	0	20	20	<1-3	L	
NAKA	1	3	1	0	4	<1	L	10	3	0	13	<1-3	T	
NAKA	2	11	8	0	19	<1-3	T/L	0	0	0	0			
NAKA	3	0	0	0	0			0	0	0	0			
NAKA	4	0	0	0	0			0	0	0	0			
NAMN	1	0	0	0	0			0	0	0	0			
OASE	1	14	6	6	26	<1	T	15	5	5	25	<1	T/L	*Closed Cracks
OASE	2	0	0	0	0			0	0	0	0			
SATU	1	0	0	0	0			0	0	0	0			
TUNG	1	24	0	10	34	>3	B/T/L	18	0	10	28	<1-3	B	Shoulders poor

Type: L = Longitudinal; T = Transverse; C = Crocodile; B = Block. Width: in mm

Table B-14 Roughness on test sections in Zimbabwe

	No	Lane	Wheel track	Roughness IRI	Wheel track	Roughness IRI
BOKI	1	LHS	OWT	3.02	-	-
BOKI	1	RHS	OWT	3.95	-	-
GECY	1	LHS	OWT	4.19	IWT	3.25
GECY	1	RHS	OWT	4.03	IWT	3.64
GECY	2	LHS	OWT	4.58	IWT	3.48
GECY	2	RHS	OWT	4.03	IWT	4.11
GECY	3	LHS	OWT	4.19	IWT	3.25
GECY	3	RHS	OWT	3.80	IWT	3.64
HSMO	1	LHS	OWT	3.25	IWT	3.64
HSMO	1	RHS	OWT	4.03	IWT	3.48
HSMO	2	LHS	OWT	3.56	IWT	3.80
HSMO	2	RHS	OWT	3.72	IWT	4.19
KIBA	1	LHS	OWT	3.64	IWT	3.33
KIBA	1	RHS	OWT	3.33	IWT	4.03
KIBA	2	LHS	OWT	3.33	IWT	3.02
KIBA	2	RHS	OWT	3.25	IWT	3.25
KIBA	3	LHS	OWT	5.91	IWT	5.52
KIBA	3	RHS	OWT	7.39	IWT	5.52
KIBA	4	LHS	OWT	7.24	IWT	6.06
KIBA	4	RHS	OWT	5.83	IWT	5.67
KIBA	5	LHS	OWT	5.75	IWT	5.75
KIBA	5	RHS	OWT	7.55	IWT	7.86
MAME	1	LHS	OWT	3.72	IWT	3.41
MAME	1	RHS	OWT	2.78	IWT	3.56
MAMI	1	LHS	OWT	3.64	IWT	3.56
MAMI	1	RHS	OWT	3.95	IWT	3.25
MAMI	2	LHS	OWT	3.25	IWT	3.02
MAMI	2	RHS	OWT	3.56	IWT	2.94
MAMI	3	LHS	OWT	3.09	IWT	3.02
MAMI	3	RHS	OWT	3.56	IWT	3.02
MAMI	4	LHS	OWT	3.48	IWT	3.41
MAMI	4	RHS	OWT	3.17	IWT	2.78
MAMI	5	LHS	OWT	3.72	IWT	3.48
MAMI	5	RHS	OWT	3.02	IWT	2.04
MIAS	1	LHS	OWT	3.33	IWT	4.03
MIAS	1	RHS	OWT	3.80	IWT	4.11
MIAS	2	LHS	OWT	5.28	IWT	4.58
MIAS	2	RHS	OWT	3.72	IWT	4.42
MIAS	3	LHS	OWT	5.20	IWT	4.89
MIAS	3	RHS	OWT	6.22	IWT	4.74
NARE	1	LHS	OWT	3.02	IWT	3.25
NARE	1	RHS	OWT	3.17	IWT	3.09
NARE	2	LHS	OWT	2.86	IWT	2.86
NARE	2	RHS	OWT	3.02	IWT	2.86
NARE	3	LHS	OWT	2.94	IWT	3.02
NARE	3	RHS	OWT	2.94	IWT	3.02
NARE	4	LHS	OWT	3.48	IWT	3.25
NARE	4	RHS	OWT	3.41	IWT	3.09
NARE	5	LHS	OWT	3.41	IWT	3.25
NARE	5	RHS	OWT	3.09	IWT	3.09
NARE	6	LHS	OWT	3.09	IWT	2.29
NARE	6	RHS	OWT	3.02	IWT	3.02
NARE	7	LHS	OWT	3.02	IWT	3.88
NARE	7	RHS	OWT	3.25	IWT	3.33
NARE	8	LHS	OWT	3.02	IWT	3.64
NARE	8	RHS	OWT	3.41	IWT	3.25
NARE	9	LHS	OWT	3.41	IWT	4.03
NARE	9	RHS	OWT	3.09	IWT	3.25
RENA	1	LHS	OWT	3.25	IWT	3.17
RENA	1	RHS	OWT	2.94	IWT	2.94
SSMA	1	LHS	OWT	4.19	IWT	3.56
SSMA	1	RHS	OWT	4.03	IWT	3.48
TOSA	1	LHS	OWT	7.24	IWT	5.67
TOSA	1	RHS	OWT	6.14	IWT	5.13
WAMI	1	LHS	OWT	3.41	IWT	2.94
WAMI	1	RHS	OWT	3.41	IWT	3.25

Table B-15 Roughness on test sections in Malawi

Site	No	Lane	Wheel track	Roughness IRI	Wheel track	Roughness IRI
CABA	1	LHS	OWT	5.61	IWT	5.47
CABA	1	RHS	OWT	5.14	IWT	4.56
CAJE	1	LHS	OWT	2.57	IWT	2.54
CAJE	1	RHS	OWT	2.87	IWT	3.04
CAKA	1	LHS	OWT	2.24	IWT	2.22
CAKA	1	RHS	OWT	2.40	IWT	2.22
GINA	1	LHS	OWT	2.10	IWT	2.38
GINA	1	RHS	OWT	2.26	IWT	1.68
KUMA	1	LHS	OWT	3.04	IWT	2.87
KUMA	1	RHS	OWT	3.25	IWT	3.06
KUMA	2	LHS	OWT	2.57	IWT	2.68
KUMA	2	RHS	OWT	3.15	IWT	3.01
KUMA	3	LHS	OWT	3.13	IWT	2.85
KUMA	3	RHS	OWT	2.71	IWT	2.68
KUMA	4	LHS	OWT	3.15	IWT	3.43
KUMA	4	RHS	OWT	3.25	IWT	3.60
LEMI	1	LHS	OWT	2.08	IWT	2.24
LEMI	1	RHS	OWT	2.12	IWT	2.12
LEMI	2	LHS	OWT	2.31	IWT	2.54
LEMI	2	RHS	OWT	2.52	IWT	2.29
LEMI	3	LHS	OWT	2.33	IWT	2.33
LEMI	3	RHS	OWT	2.29	IWT	2.45
LEZA	1	LHS	OWT	2.57	IWT	2.19
LEZA	1	RHS	OWT	3.13	IWT	1.91
LEZA	2	LHS	OWT	2.10	IWT	2.03
LEZA	2	RHS	OWT	2.12	IWT	2.05
MIMY	1	LHS	OWT	3.15	IWT	3.57
MIMY	1	RHS	OWT	3.08	IWT	3.43
MYGI	1	LHS	OWT	3.06	IWT	2.61
MYGI	1	RHS	OWT	2.85	IWT	2.64
NADA	1	LHS	OWT	3.67	IWT	3.25
NADA	1	RHS	OWT	3.27	IWT	3.25

Table B-16 Rutting on test sections in Zimbabwe

Site	No	Lane	Wheel-track	Percentiles			Mean	SD	Wheel-track	Percentiles			Mean	SD	n
				50	80	90				50	80	90			
BOKI	1	LHS	OWT	5.5	7	8	8.4	1.48							31
BOKI	1	RHS	OWT	6.5	8.7	9.8	7.4	2.14							31
GECY	1	LHS	OWT	5.5	7	9.5	6.8	2.05	IWT	5.7	7.2	8.2	6.3	1.56	41
GECY	1	RHS	OWT	7	8.8	9.8	7.6	1.88	IWT	5	6.7	7.9	6.1	1.79	41
GECY	2	LHS	OWT	4	5	5.5	4.7	0.81	IWT	5.4	6.6	8.1	6.1	1.81	41
GECY	2	RHS	OWT	4.5	5.4	5.8	5.05	0.86	IWT	6.2	8.4	10.5	7.2	2.25	41
GECY	3	LHS	OWT	4.7	6.3	6.9	5.3	1.8	IWT	5.3	6.2	6.7	5.8	1.04	41
GECY	3	RHS	OWT	4.8	6	6.6	5.3	1.36	IWT	5.4	7	7.5	5.9	1.62	41
HSMO	1	LHS	OWT	4.6	5.6	6	5.2	1.02	IWT	5.4	6.5	8	6.1	1.68	41
HSMO	1	RHS	IWT	4.5	5.7	6.1	5.02	1.31	IWT	5.2	6.2	6.6	5.6	1.32	41
HSMO	2	LHS	OWT	4.8	6.1	6.6	5.4	1.34	IWT	4.7	5.6	6	5.2	1.06	41
HSMO	2	RHS	OWT	5	5.8	6.4	5.4	1.02	IWT	5	5.8	6.3	5.3	1.08	41
KIBA	1	LHS	OWT	4.3	5	5.5	4.8	1.02	IWT	4.7	5.8	7	5.39	1.32	41
KIBA	1	RHS	OWT	4.7	5.8	6.5	5.3	1.1	IWT	4.7	5.8	6.5	5.4	1.18	41
KIBA	2	LHS	OWT	4.8	6	6.6	5.4	1.16	IWT	4.8	6.1	6.8	5.4	1.22	41
KIBA	2	RHS	OWT	4.8	5.6	5.9	5.2	1.04	IWT	5	5.9	6.3	5.5	0.95	41
KIBA	3	LHS	OWT	7.4	9.3	10.2	7.8	2.54	IWT	10	13	14.7	11	3.04	41
KIBA	3	RHS	OWT	4.7	5.9	7	5.4	1.32	IWT	8.4	10.4	16	9.8	4.48	41
KIBA	4	LHS	OWT	5.5	7.5	8.9	6.4	2.05	IWT	6.4	8.2	9.5	7.3	2.03	41
KIBA	4	RHS	OWT	6.4	8.3	10	7.5	3.09	IWT	6.8	8.2	8.8	7.1	2.06	41
KIBA	5	LHS	OWT	7.4	10.1	11.2	8.2	2.56	IWT	8	11.3	12.7	9.3	3.24	41
KIBA	5	RHS	OWT	9.1	10.2	11.5	9.1	2.67	IWT	8	10.8	11.5	8.5	2.52	41
MAME	1	LHS	OWT	4.6	5.6	6.3	5.2	1.02	IWT	4	5.1	5.6	4.6	1.07	41
MAME	1	RHS	OWT	4.7	6	6.5	5.3	1.07	IWT	3.8	4.9	5.4	4.3	1.01	41
MAMI	1	LHS	OWT	6.5	7.5	8	7	1.13	IWT	5.6	6.8	7.5	5.9	1.49	41
MAMI	1	RHS	OWT	13.5	15	15.6	13.4	2.07	IWT	5	5.9	6.6	5.5	1.21	41
MAMI	2	LHS	OWT	4.4	5.5	6	4.8	1.45	IWT	5.1	6.6	7.2	5.5	1.6	41
MAMI	2	RHS	OWT	6.5	10.6	12	7.6	2.93	IWT	4.1	5.3	11.4	4.6	1.56	41
MAMI	3	LHS	OWT	5	5.9	6.5	5.5	1	IWT	4	5	5.4	4.6	0.92	41
MAMI	3	RHS	OWT	4.6	5.6	6.2	5.2	1.13	IWT	4.4	5.2	5.8	4.8	1.02	41
MAMI	4	LHS	OWT	4	5	5.6	4.61	0.94	IWT	4.5	5.4	5.9	5	0.95	41
MAMI	4	RHS	OWT	4	4.7	5	4.4	0.94	IWT	4.1	5.2	5.6	4.6	1.14	41
MAMI	5	LHS	OWT	3.8	4.5	4.7	4.2	0.83	IWT	4.5	5.8	6.4	5.1	1.17	41
MAMI	5	RHS	OWT	4.1	5.1	5.7	4.7	1.07	IWT	4.8	5.8	6.3	5.4	0.99	41
MIAS	1	LHS	OWT	5.5	7.1	7.8	6.1	1.67	IWT	4.9	6	7	5.5	1.82	41
MIAS	1	RHS	OWT	5.5	7.2	8.5	6.4	1.66	IWT	5	6.5	7.2	5.6	1.82	41
MIAS	2	LHS	OWT	7.1	9.1	9.7	7.8	2.07	IWT	6.5	8	9.4	7.2	1.62	41
MIAS	2	RHS	OWT	6.2	7.5	8.6	6.7	1.78	IWT	6.2	7.7	8.2	6.8	1.51	41
MIAS	3	LHS	OWT	5.6	7	8	6.3	1.4	IWT	5.3	6.8	8	6.2	1.61	41
MIAS	3	RHS	OWT	5	6.3	7.1	5.6	1.36	IWT	5.2	7.4	8.5	6.3	2.01	41

continued

Site	No	Lane	Wheel-track	Percentiles			Mean	SD	Wheel-track	Percentiles			Mean	SD	n
				50	80	90				50	80	90			
NARE	1	LHS	OWT	5	6.2	6.8	5.7	1.14	IWT	4.3	5.1	5.7	4.9	0.95	41
NARE	1	RHS	OWT	5.8	6.5	6.8	6.2	1.01	IWT	4.2	5	5.4	4.7	0.82	41
NARE	2	LHS	OWT	4.7	5.2	5.8	4.8	1.04	IWT	4.2	5.3	5.6	4.7	1.03	41
NARE	2	RHS	OWT	4	5	5.5	4.5	1.1	IWT	5.6	6.7	7.2	6.05	1.22	41
NARE	3	LHS	OWT	3.9	5	5.5	4.5	1.12	IWT	5.3	6.2	6.6	5.8	0.94	41
NARE	3	RHS	OWT	4.1	5.1	5.8	4.7	1.29	IWT	4.7	5.8	6.4	5.2	1.16	41
NARE	4	LHS	OWT	4.2	5	5.5	4.8	0.82	IWT	4.9	5.7	6.1	5.4	0.94	41
NARE	4	RHS	OWT	4	4.8	5.3	4.6	0.86	IWT	5.5	6.5	6.9	6.2	1.22	41
NARE	5	LHS	OWT	5.3	6.3	6.8	5.8	1.1	IWT	5.3	6.7	7.8	6.1	1.91	41
NARE	5	RHS	OWT						IWT	4.5	5.5	6	5.2	1.04	41
NARE	6	LHS	OWT	5	5.9	6.5	5.7	1.38	IWT	3.8	4.6	5	4.4	0.92	41
NARE	6	RHS	OWT	5	6.5	6.8	5.6	1.14	IWT	4.1	5.2	5.6	4.7	0.98	41
NARE	7	LHS	OWT	5	6	6.5	5.4	1.14	IWT	4.5	5.5	5.8	5	1	41
NARE	7	RHS	OWT	5.7	6.8	7.3	6.3	1.52	IWT	4.4	5.5	5.8	5	1.05	41
NARE	8	LHS	OWT	5	6	6.5	5.3	1.13	IWT	5.8	7	8.2	6.6	1.55	41
NARE	8	RHS	OWT	4.5	5.5	6	5.1	0.97	IWT	5.2	6.5	7	5.9	1.41	41
NARE	9	LHS	OWT	4	5.1	5.5	4.7	1.08	IWT	5.5	7	9	6.4	2.01	41
NARE	9	RHS	OWT	4	5	5.5	4.7	0.88	IWT	5	6.5	6.8	5.7	1.27	41
RENA	1	LHS	OWT	5.1	5.8	6.3	5.6	0.81	IWT	4.9	6.4	6.6	5.41	1.02	41
RENA	1	RHS	OWT	4.9	7.4	6	5.39	0.86	IWT	5.8	6.8	7.4	6.36	1.09	41
SSMA	1	LHS	OWT	6.5	8.1	8.8	7.3	1.47	IWT	5.8	6.8	8	6.5	1.27	41
SSMA	1	RHS	OWT	6.4	7.8	8.5	7	1.37	IWT	6	7	7.7	6.5	1.23	41
TOSA	1	LHS	OWT	4.8	5.9	6.5	5.4	1.12	IWT	4.7	5.6	6	5.3	1.15	41
TOSA	1	RHS	OWT	4.7	5.9	7	5.3	1.25	IWT	4.5	5.5	5.8	5.1	1.01	41
WAMI	1	LHS	OWT	5.7	7	7.5	6.1	1.31	IWT	4.4	6	6.5	5.05	1.2	41
WAMI	1	RHS	OWT	5.7	7.6	9	6.1	1.3	IWT	4.5	5.7	6.3	5.05	1.2	41

NB Rutting in mm
 SD = Standard deviation

Table B-17 Rutting on test sections in Malawi

Site	No	Lane	Wheel-track	Percentiles			Mean	SD	Wheel-track	Percentiles			Mean	SD	n
				50	80	90				50	80	90			
CABA	1	LHS	OWT	9.4	11.5	12	9.4	3.63	IWT	0	0	3	0.6	1.5	30
CABA	1	RHS	OWT	3.6	5.7	8.8	4.2	3.68	IWT	0	0	0	0.3	1.15	30
CAJE	1	LHS	OWT	6.5	7.7	8.9	6.7	2.01	IWT	1.3	3.5	3.9	2.3	1.95	31
CAJE	1	RHS	OWT	3.7	5.2	6	4.2	2.57	IWT	2.3	3.8	4.4	2.8	1.86	31
CAKA	1	LHS	OWT	2	3.5	3.8	2.4	1.96	IWT	0	3.5	4	1.8	2.24	31
CAKA	1	RHS	OWT	0	0	1.3	0.6	1.31	IWT	0.7	3.2	3.7	2.03	1.82	31
GINA	1	LHS	OWT	6.3	8	8.5	6.6	2	IWT	0	3.1	3.8	1.4	1.98	31
GINA	1	RHS	OWT	4.5	7	7.7	5	2.4	IWT	3	4	5.4	3.2	1.91	31
KUMA	1	LHS	OWT	7.5	9.2	9.7	7.94	1.73	IWT	6.4	7.9	8.6	7.2	1.33	31
KUMA	1	RHS	OWT	5.5	8.5	11.5	7.3	3.62	IWT						31
KUMA	2	LHS	OWT	5.1	6.8	8.1	6.1	1.79	IWT	4.1	5.1	5.6	3.8	2.27	31
KUMA	2	RHS	OWT	8.9	11.2	12	9.55	2.43	IWT	5.4	6.9	8	6	2.01	31
KUMA	3	LHS	OWT	3.5	4.4	5	3.8	1.62	IWT	4	4.7	5	4.3	1.42	31
KUMA	3	RHS	OWT	7.1	9.3	10.2	7.2	2.52	IWT	5.6	6.9	7.5	6.2	1.26	31
KUMA	4	LHS	OWT	4.8	5.9	6.4	5.2	1.36	IWT	4.6	5.5	5.8	5	1.25	31
KUMA	4	RHS	OWT	4.3	4.9	5.7	5.1	1.18	IWT	5.2	6.6	7.4	6	1.66	31
LEMI	1	LHS	OWT	6.9	8	8.7	7.1	1.71	IWT	5.4	7.3	8.2	6.2	1.64	30
LEMI	1	RHS	OWT	10	11.7	13.4	10.7	2.2	IWT	5.3	6.8	7.8	5.7	1.95	30
LEMI	2	LHS	OWT	9.3	10.7	11.4	9.6	1.86	IWT	4.1	7.4	8	5.3	2.15	32
LEMI	2	RHS	OWT	3.2	5.7	6.5	3.8	2.47	IWT	5.4	6.5	7.4	5.8	1.39	33
LEMI	3	LHS	OWT						IWT						
LEMI	3	RHS	OWT						IWT						
LEZA	1	LHS	OWT	9.9	12.6	14.5	10.6	3.28	IWT	3.3	4	4.5	3.03	1.91	31
LEZA	1	RHS	OWT	7.9	11.3	13.9	9.7	3	IWT	0	5.3	12	1.5	2.41	31
LEZA	2	LHS	OWT	7	8.2	9.2	7.6	1.77	IWT	0	2.8	4.2	1.3	1.97	31
LEZA	2	RHS	OWT	0	1.8	3	1.2	1.68	IWT	0	0	0	0.3	1.01	31
MIMY	1	LHS	OWT	10.7	15.2	16	11.6	4.34	IWT						31
MIMY	1	RHS	OWT	9.4	11.9	13.5	9.8	3.63	IWT	3.3	5.4	6	3.4	2.75	31
MYGI	1	LHS	OWT	6	7.5	8.2	6.2	2.34	IWT	3.5	4.7	6.1	3.3	2.48	30
MYGI	1	RHS	OWT	4.4	6.4	7.3	4.3	2.82	IWT	5	6.9	8.2	5.7	2.14	30
NADA	1	LHS	OWT	5.7	7.6	9.3	6.4	2.32	IWT	4	5.2	5.6	4	2.01	31
NADA	1	RHS	OWT	6.2	7.7	10	7	2.34	IWT	0	4.4	4.8	2.3	2.51	31

NB Rutting in mm
 SD = Standard deviation

Table B-18 Rutting on test sections in Botswana

Site	No	Lane	Wheel-track	Percentiles			Mean	SD	Wheel-track	Percentiles			Mean	SD	n
				50	80	90				50	80	90			
KAKE	1	LHS	OWT	7.6	10.9	11.5	8.1	3.02	IWT	2.2	3.9	4.4	2.6	2.06	31
KAKE	1	RHS	OWT	4.6	6.7	7.9	5	2.88	IWT	3.5	4.9	5.5	3.7	1.96	31
NAKA	1	LHS	OWT	4.5	5.5	5.9	4.8	1.49	IWT	3.7	4.8	5.6	4	1.97	41
NAKA	1	RHS	OWT	3.9	4.7	5	4.5	0.81	IWT	3.9	4.8	5.4	4.5	1.34	41
NAKA	2	LHS	OWT	3.3	4.7	5.4	3.3	2.15	IWT	3.7	4.7	5	4	1.43	41
NAKA	2	RHS	OWT	3.9	5	5.5	4.5	1.05	IWT	3	3.8	9.2	3.02	1.74	41
NAKA	3	LHS	OWT	6.6	7.7	8.2	6.8	1.81	IWT	3.3	4.1	4.7	3.2	2.07	31
NAKA	3	RHS	OWT	5.6	7.4	8	6.3	1.54	IWT	3	4.3	5	3.2	3.32	31
NAKA	4	LHS	OWT	4	4.8	5.2	4.5	1.15	IWT	4	4.8	5.2	4.6	0.99	31
NAKA	4	RHS	OWT	3.7	4.4	5	4.3	0.83	IWT	4.5	5.5	5.9	5.2	1.11	31
NAMN	1	LHS	OWT	8.2	10.5	11.3	8.7	2.48	IWT	8.9	10.8	12.1	9.6	2.17	41
NAMN	1	RHS	OWT	8.8	11.4	12.6	9	2.6	IWT	8.8	10.7	11.4	9.3	2.11	41
OASE	1	LHS	OWT	6.4	8	9.5	7.2	2.04	IWT	8.2	10	10.6	8.4	2.15	41
OASE	1	RHS	OWT	8.5	10	10.8	9.2	2.14	IWT	4.6	5.6	6	5.2	1.2	41
OASE	2	LHS	OWT	6	10	10.8	7.4	3.08	IWT	7.1	9.5	11.2	7.8	2.46	41
OASE	2	RHS	OWT	7.4	11.2	12.1	8.5	2.91	IWT	6.5	8.4	9	7.2	1.92	41
SATU	1	LHS	OWT	5.7	7.6	8.7	6.5	1.9	IWT	4	5.4	6.7	4.1	2.54	41
SATU	1	RHS	OWT	6.5	7.7	8.3	6.9	1.46	IWT	4.3	5.5	7	4.9	1.74	41
TUNG	1	LHS	OWT	5.6	7.6	7.3	6.2	1.23	IWT	4.5	5.9	6.5	5	1.81	43
TUNG	1	RHS	OWT	4.3	5.4	6	4.7	2.05	IWT	3.6	4.7	5.5	4.2	1.72	41

NB Rutting in mm

SD = Standard deviation

Appendix C: Calcrete roadbases

Background

Good quality road-building materials in the Kalahari region of Southern Africa are scarce. Apart from sand, the principal material available is calcrete, which is historically regarded by engineers as a poor quality gravel. Some use has been made of the better calcretes for low volume roads in South Africa, but this has not led to the general acceptance of calcrete as a road building material. Nevertheless, since the haulage of good quality material from elsewhere is prohibitively expensive, it was important to establish reliable guidelines for its use. A joint research project was therefore set up between TRL and the Roads Department of the Ministry of Works and Communications of Botswana to investigate the use of calcretes as road building materials. The results of this research are relevant to the current project. The approach adopted was to construct experimental sections of road using different types of calcrete as roadbases. The road between Kanye and Jwaneng was selected for the trials, principally because of the relatively high levels of traffic that were expected. It was intended that the trials would be monitored until pavement sections were close to the end of their useful life.

Location of calcretes

The scarcity of deposits of road-building materials in the Kalahari makes their location notoriously difficult by surface surveying. However, remote sensing techniques using aerial photography and satellite imagery proved particularly useful in the location of the calcrete (see Lawrance et al 1993).

Four calcrete groups covering the range of materials found in Botswana were investigated in the trials: hardpan calcrete (HC); nodular calcrete (NC); powder calcrete (PC) and calcified sand (CS). The principal engineering properties of the samples are summarised in Table C-1 to Table C-3. All of the calcretes failed to meet at least one of the normal requirements specified in the Botswana Road Design Manual and Overseas Road Note 31 for strength, plasticity and grading. However HC-1 and NC-2 both met the minimum 4-day soaked CBR requirement of 80 per cent for roadbases. They were potentially much better road-building materials than PC-3 and CS-4.

Table C-1 Calcrete properties compared with Botswana Road Design Manual

Material property	Original BRDM	Project calcretes (mean values)			
		HC-1	NC-2	PC-3	CS-4
Grading modulus (minimum)	2.0	2.2	2.1	1.9	1.2
Maximum size (mm)	53	75	75	75	37.5
Passing 425µm sieve (maximum)	10 - 30	39	37	62	81
Passing 63µm sieve (maximum)	5 - 15 ^(a)	12	14	33	28
Liquid limit (maximum)	25	25	44	39	36
Plasticity index (maximum)	6	7	20	9	15
Linear shrinkage (LS) (maximum)	3	4	9	4	6
LS x % passing 425µm sieve (maximum)	170	156	629	248	486
Plasticity modulus (PM)	60-180	273	740	558	1215
Minimum CBR (4 days soaked)	80 ^(b)	150 ^(c)	120 ^(c)	50 ^(c)	40 ^(c)

Notes:
 (a) 75µm sieve in Botswana/South Africa
 (b) At 98% mod AASHTO compaction (AASHTO T180, 1986)
 (c) At BS heavy compaction (BS 1377 Part 4, 1990)
 BRDM Botswana Road Design Manual
 Samples taken from borrow pits

Table C-2 Laboratory CBRs

Material	Average CBR ^(a)			Ratio	
	4-day soaked	At OMC	Dried back	Optimum/ soaked	Dried/ soaked
HC-1	80 (24)	90 (7)	>150 (67)	1.1	2.4
NC-2	85 (41)	140 (27)	>150 (24)	1.6	2.9
PC-3	60 (9)	75 (9)	120 (12)	1.3	1.6
CS-4	23 (12)	90 (11)	>150 (40)	3.9	10.0

Notes:
 (a) Figures in brackets are the standard deviations
 HC Hardpan calcrete
 NC Nodular calcrete
 PC Powder calcrete
 CS Calcified sand
 Samples taken from the road

Table C-3 Typical aggregate tests on calcrete roadbase materials

Sample	10% Fines crushing value (kN)		Modified aggregate impact value (per cent)	
	Dry	Soaked	Dry	Soaked
HC-1	18-20	9-11	96-130	120-150
NC-2	37-42	43	42-84	40-87

Notes:
 The powder calcrete and the calcified sand were too weak for these tests
 Samples taken from borrow pits

As well as basic strength, sensitivity to moisture is an important aspect influencing the potential performance of calcrites. The ratio of CBR at optimum moisture content to the CBR in the soaked state is a measure of this sensitivity, as shown in Table C-2. The sensitivity of the calcified sand is noteworthy. The test results illustrate the high variability in properties that are found in most calccrete deposits. To use calcrites satisfactorily, great care is needed in quarrying, stockpiling, sampling and testing. Compaction trials are also recommended to determine the as-built properties of the materials when they are incorporated within a road.

Road trials

The design traffic for the Jwaneng-Kanye road was 500,000 cumulative equivalent standard axles (esa). The traffic in the direction towards Jwaneng was much heavier than that towards Kanye, thereby providing an opportunity to examine the applicability of the 4th power axle load/pavement damage law to these materials and this type of construction in Botswana. Eight test sections with calccrete roadbases were constructed with four control sections as part of a larger series of experiments. Kalahari sands were used as the sub-base material throughout the trial.

Traffic

Pavement performance was defined in terms of the traffic that the pavement carried before reaching a defined ‘failure’ or ‘terminal’ condition at which major repairs were required. In fact, many of the test sections did not reach a terminal condition. For these, potential traffic carrying capacity was based on extrapolations of their performance history to date, although this was not ideal from a statistical and scientific point of view. Simple performance models were developed relating rut depth to traffic and/or age for each section and, where possible, for each direction of travel. The cumulative traffic in the Jwaneng direction by the end of the study had reached 450,000 esa, so traffic levels above this were extrapolated. The models predict the rut depths to better than 10 per cent of the measured value. The actual, or predicted, traffic carrying capacities of the four roadbase materials to different 80th percentile rut depths are shown in Table C-4.

Table C-4 Traffic required to reach terminal rut depth

Material	Traffic (esa x 10 ⁶) to reach given 80th percentile terminal rut depth (mm)			
	15 mm	20 mm	25 mm	30 mm
Hardpan, HC-1	0.5	0.65	0.8	1.0
Nodular, NC-2	0.4	0.5	0.6	0.75
Powder, PC-3	0.6	0.8	1.0	1.15
Calc. sand, CS-4	0.2	0.3	0.35	0.45

Performance of the roadbase

Moisture content

The main mode of deterioration determined during the study was deformation due to shear failures within the roadbase, primarily in the outer wheel-track of the Jwaneng lane. The evidence from numerous moisture/strength profiles has led to the conclusion that the deformation was caused by a moisture-induced loss in strength.

The moisture content of the middle two thirds of the pavements constructed with HC-1 and NC-2 was typically 50-70 per cent of the optimum moisture content. The figures were higher for PC-3 and CS-4. From the moisture/strength profiles across the sections, it can be assumed that, if the shoulders on the trials had been sealed, then the moisture regime in the roadbase in the outer wheel-track would have been similar to that of the inside wheel-track. Under these conditions, the outer wheel-track should perform better than in the trials, although not as well as the inside wheel-track. This is confirmed by evidence from this and other studies which indicate that, in whole-life cost terms, an overall maintenance benefit results from sealing the shoulders of pavements constructed with natural gravel roadbases.

Calcified sand

From the results of laboratory tests, the calcified sand would normally be considered suitable only for sub-base material. Its potential for use as roadbase in arid conditions stems from the relatively high strengths obtained in the laboratory. CBR values of 90 per cent were obtained when the material was tested at optimum moisture content, and values greater than 150 per cent were obtained when in the dried-back state typical of the middle two thirds of the pavement. Conversely, the material was the most sensitive to moisture, with a large reduction in strength when wet, as shown in Table C-2. Over time, large variations in deflection, in the range of 0.6 to 1.0mm, have occurred in the outer wheel-track of this section. Deflections in the inside wheel-track position were generally 0.3 to 0.4mm. These results are also indicative of the difference in moisture condition of the pavement at these positions. The outer wheel-track of the Jwaneng lane developed an average rut of 20mm after the passage of about 350,000 esa.

Nodular calcrete

In terms of strength, the NC-2 material satisfied the general requirements for the higher traffic categories for unbound gravel roadbases given in *Overseas Road Note 31* (TRL 1993). However, the high plasticity index and the high percentage of material passing the 425 μ m sieve (Table C-1) showed that the material failed to meet the Botswana specification. Despite this, high values of in situ CBR were recorded consistently across the whole paved area of the section. The mean rut depth in the outer wheel-track reached 12mm after the passage of 400,000 esa. The models of average rut depth and traffic indicate that the failure rut depth of 20mm would be reached at a traffic level of about 600,000 esa. Thus, its use in conditions similar to those of the trial could be acceptable for traffic levels up to around 500,000 esa (Table C-4). No significant rutting developed in the inside wheel-track. Moisture contents were typically less than 60 per cent of optimum, and in situ CBRs were in excess of 100 per cent. Deflections were consistently less than 0.4mm in both wheel-tracks.

Hardpan calcrete

HC-1 was the best of the materials used in the trials in terms of suitability for roadbase. It easily met the GB3 category in *Overseas Road Note 31* for roadbase in terms of strength, although it did not satisfy the plasticity requirements. Deflections in the outer wheel-track of the Jwaneng lane averaged 0.4mm and were typically 0.1mm less in the other wheel-tracks. The deformation in the outer wheel-track of the Jwaneng lane increased, but at a relatively slow rate. The last measurements showed an average rut of 12mm.

Powder calcrete

The performance of this section was the most difficult to explain. The average in situ CBR values measured with a DCP were low, averaging 24 per cent in 1991. This is reflected in a low radius of curvature. The moisture content has increased since construction to values above optimum across the whole of the paved area. In 1994, the rut depths averaged 9mm in the outer wheel-track of the Jwaneng lane, but this is an increase of less than 4mm since the road was opened to general traffic.

Performance of the sub-base

The moisture content of the sand sub-base was always less than the optimum for compaction. The ratio of field to optimum moisture content varied from 0.3 to 0.8, reflecting the variations occurring in the roadbases, described above. The average *overall* in situ strengths for each section also mirrored the strengths of the roadbases: the strongest sub-bases being associated with the strongest roadbases. The sub-base for PC-3 was the weakest, with a CBR of 40 per cent, while that of HC-1 was the strongest with a CBR of over 100 per cent. The values in the outer wheel-track were very similar for PC-3, CS-4 and NC-2, namely 45, 55 and 50 per cent respectively, whereas the value for the HC-1 section was 95 per cent.

Performance of the subgrade

Strength tests carried out on the subgrade shortly after pavement construction gave in situ CBRs ranging from 11 to 22 per cent, with a mean value of 16 per cent. The predominantly dry and relatively free-draining conditions yielded relatively high strengths in the sand subgrade. The results of subsequent measurements were consistently higher than at construction because the subgrade had dried back. The mean CBR at 550mm depth for all sections, except for the NC-2 section, was above 30 per cent, indicating a subgrade category of S6. The value for the NC-2 section was 14 per cent, indicating a subgrade category of S4. The centre of the pavement was always stronger.

Shoulders and embankments

The trials demonstrated that calcified sand is unsuitable as shoulder material because of its susceptibility to erosion. The coarse-grained calcretes performed well as shoulder material, but the unsealed shoulders, when wet, acted as a potential source of moisture. This infiltrated the

roadbase layer and influenced the performance of some of the calcrete roadbases. When used as embankment material, Kalahari sand was susceptible to erosion. This could be reduced greatly by flattening the embankment slopes from 1:4 to 1:6.

Appendix D: Case studies

Malawi demonstration section

Background

Malawi has 35 trunk routes with an estimated total length of 1,859 kilometres. Twenty six of these, with a total length of 1 384 kilometres, have been constructed using crushed stone, which is nearly 75 per cent of the total. However, on many of these routes the design traffic was below three million esa, and natural gravels could have been utilised for roadbase construction at far lower cost.

The Limbe-Thyolo-Mulange-Muloza road project in south-east Malawi had a design which specified the use of crushed stone for the roadbase. Abundant deposits of lateritic gravel occur in the area, and these were to be used for the project sub-base. The results of this study were used as the basis for constructing a demonstration section using a lateritic roadbase, and this was undertaken in collaboration with the Roads Department and the European Union. The main purpose of the demonstration was to increase confidence that lateritic gravels can be used successfully as roadbase materials for design traffic levels less than three million esa, and to promote the construction of more economic pavements in the future.

It was also intended that data collected from the demonstration could be used to extend the design guidelines on lateritic materials.

The project road

The project road extends from Limbe to the Mozambique border, just beyond the village of Muloza. The existing road consists of a 3.3 to 4.0 metre wide single bituminous strip with gravel shoulders. It was in poor condition and was to be upgraded to a 6.7 metre carriageway with 1.5 metre shoulders. The design traffic varies along the project, but was 1.5 million esa in the area where the demonstration section was proposed. The pavement structure of the main project consisted of a 200mm thick natural gravel sub-base and a 200mm crushed stone roadbase. The design subgrade CBR was 8 per cent throughout the project. The sections between Limbe and Thyolo Boma have a semi-structural asphalt concrete surfacing, whilst the remaining sections were surfaced with a double surface dressing. The shoulders were sealed with a single surface dressing.

A design for the road based on *ORN 31* would recommend 200mm of granular roadbase (GB1-GB3), 200mm of granular sub-base (GS), and a double surface dressing. This is similar to that being used for the project. However, *ORN 31* would also have allowed use of a GB3 natural gravels with a CBR of at least 80 per cent, if available, rather than a crushed stone roadbase.

The demonstration section

The demonstration section was located on a 300 metre stretch between Luchenza and the Midima road junction. Although strong lateritic gravels with soaked CBRs in excess of 80 per cent were readily available in the area, material with a soaked CBR between 55 and 70 per cent were utilised for the roadbase. This was the only difference between the specification of the project road and that of the demonstration section. Details for the demonstration section are set out in Box D-1.

The borrow area for supply of the base material at Kululira was located within half a kilometre of the section. Pre-selection testing of the materials was carried out to designate an area within the pit for supply of material for the section. Approximately 1 500m³ of material was stockpiled for use.

A compaction trial was carried out to determine plant requirements and the number of passes to be used for compaction of the roadbase. A 7.5 tonne pneumatic roller and a 11.6 tonne vibrating steel roller were available. The trial showed that four passes of the pneumatic roller, followed by two high frequency and four low frequency passes of the vibrating roller, were adequate to give the desired compaction to the full depth of construction.

A control section comprising 300 metres of crushed stone base was established adjacent to the demonstration section.

Other relevant features

The roadbase layer was constructed in November 1996. The section remained un-surfaced until August 1997, when the prime and surfacing were applied, because of the early on-set of the rains and other logistical problems on the site. In the interim period, cracks were recorded on the base in late November, but had self-sealed by the time an inspection could be organised. Due to the severity of the rains, which caused some gulying on the side slopes, traffic often moved off the diversion road and onto the section. No damage was reported and, prior to surfacing, it was only necessary to proof roll with a pneumatic roller, check the levels, and brush the surface prior to priming.

Box D-1 Design features of the demonstration section on the Limbe-Thyolo-Mulange-Muloza road	
Location	km56+150 to km56+450.
Cross section	Constructed on embankment with design features as for project road. Carriageway width of 6.7 metres.
Subgrade design CBR	Subgrade class S4 (soaked CBR of 8-14%). Soils are plastic (liquid limit = 50, plasticity index = 25) comprising lake sediments, silty sands (60% passing 75µm) of the Phalombe plain. Soaked CBR of 9% and CBR swell of 1%. Laterite encountered at depths of 0.5-1.2 metres.
Geometrics	Straight section with gentle gradient (1.0%) and cross-fall as for the project road.
Length	300 metres with a 30 metre transition to project road.
Subgrade construction	Subgrade treatment as for project road. Subgrade compacted to greater than 93% BS 4.5kg rammer compaction and OMC.
Sub-base construction	(a) 200mm natural gravel, as for project road (b) Sub-base soaked CBR in the range of 30-45 per cent compacted at 95% BS 4.5kg rammer compaction and OMC (c) Compacted field density average 97.1% of BS 4.5kg rammer compaction, with a range of 95.5 to 99.3%
Roadbase material	(a) Lateritic gravel; source Kululira pit (b) MDD 2 150kg/m ³ (1 930 to 2 195kg/m ³) and OMC 8.9% (7.6 to 13.2%) (c) Soaked CBR of 60% at 98% (75 % at 100%) BS 4.5kg rammer compaction and OMC (d) Liquid limit = 32; plasticity index = 14 (e) Maximum particle size 20mm (outside the finer <i>ORN 31</i> limits for 20mm materials) (f) Expansion (CBR swell) <0.5%
Roadbase construction	(a) Average thickness 201mm roadbase (range 180-215mm) cf 200mm specified (b) Compacted at average field moisture 0.99% OMC (range 0.84-1.15%) cf specification of -2% to +1% of OMC (c) Compacted field density average 98.0% of BS 4.5kg rammer (range 94.0 to 102.4%) cf 98% specified (d) In situ CBR of 40% at construction and 135% prior to priming
Surfacing	Prime at a spread rate of 0.85litres/m ² and double surface dressing
Shoulders	1.5m wide, primed and single surface dressed
Finishing works	As for project road, with sign boards demarcating demonstration section

Monitoring performance

The section caused great interest during its construction, but this has not been sustained by carrying out subsequent regular monitoring. Measurements should now be taken, once or twice a year, of both the demonstration and control sections. These should include:

- In situ strength using the dynamic cone penetrometer
- Moisture content and density using nuclear methods
- Deformation using levelling
- Rut depth
- Visual assessment
- Deflections using FWD or Benkelman beam
- Roughness with a MERLIN or abay beam
- Automatic traffic counting, classified traffic counting and axle load surveys
- Collection of climatic data

Some ad hoc measurements of roughness and rutting have been taken since construction and, at this early stage, are indicating that the section is performing as well, if not better, than the parts of the road built with crushed stone.

If the maximum benefit is to be obtained from the demonstration section, it is essential that costing information for the demonstration section and the crushed stone is collected from the consultant and contractor before completion of the road project.

Worked examples

A number of case studies, using the design procedure in the study, were developed by SweRoad for the *Secondary and Feeder Road Development Programme* in Zimbabwe. These examples have been reproduced to illustrate the application of the procedure.

The following basic cost data are used in the case studies. The example is based on a design traffic class of 100,000 esa and on an S3 subgrade. Assumed unit rates are given in Table D-1, and determination of pavement designs using Charts 1 and 2 are given in Table D-2 and Table D-3 respectively. A summary of pavement construction costs is given in Table D-4. Note that costs used were derived in 1997.

Table D-1 Assumed unit costs

	Average haul distance	All costs excluding haulage	Haul cost	Total (US\$/m ³)
Fill	free	1.98	0	1.98
Selected fill CBR 15 %	free	2.34	0	2.34
Sub-base CBR 30%(compact 95%)	2.0km	3.36	0.37	3.73
Base CBR 45% (compact 98%)	2.0km	3.41	0.37	3.78
Base CBR 55%	3.5km	3.71	0.65	4.36
Base CBR 65%	5.0km	3.71	0.92	4.63
Base CBR 80%	6.5km	4.01	1.20	5.21

Table D-2 Determination of pavement design based on Chart 1

Description	Unit	Quantity per lin-m	Unit price (US\$)	Cost (US\$)	
				per /lin-m	per km
<i>Cross-section: total sealed width 6m</i>					
Fill 200mm	m ³	1.68	1.98	3.32	
Sub-base 150 mm (CBR 30%)	m ³	1.12	3.73	3.33	
Roadbase 150mm (CBR 65%)	m ³	0.97	4.63	4.49	
Single <i>Otta</i> seal (incl prime)	m ²	6.00	0.80	4.80	
Total				15.94	15,940
<i>Cross-section: total sealed width 7m</i>					
Fill 200mm	m ³	1.88	1.98	3.72	
Sub-base 150mm (CBR 30%)	m ³	1.27	3.73	4.74	
Roadbase 150mm (CBR 65%)	m ³	1.12	4.63	5.18	
Single <i>Otta</i> seal (incl prime)	m ²	7.00	0.80	5.60	
Total				19.24	19,240
<i>Cross-section: total sealed width 8m</i>					
Fill 200mm	m ³	2.08	1.98	4.11	
Sub-base 150mm (CBR 30%)	m ³	1.42	3.73	5.29	
Roadbase 150mm (CBR 55%)	m ³	1.27	4.36	5.25	
Single <i>Otta</i> seal (incl prime)	m ²	8.00	0.80	6.40	
Total				21.34	21,340
<i>Note:</i> Only pavement layers are included in the comparison, <i>i.e.</i> costs are not total construction cost per kilometre					

Table D-3 Determination of pavement design based on Chart 2

Description	Unit	Quantity per lin-m	Unit price (Z\$)	Cost (US\$)	
				per /lin-m	per km
<i>Cross-section: total sealed width 6m</i>					
Fill 200mm	m ³	1.64	1.98	3.24	
Sub-base 120mm (CBR 30%)	m ³	0.87	3.73	3.24	
Roadbase 150mm (CBR 55%)	m ³	0.97	4.36	4.23	
Single Otta seal (incl prime)	m ²	6.00	0.80	4.80	
Total				15.52	15,525
<i>Cross-section: total sealed width 7m</i>					
Fill 200mm	m ³	1.84	1.98	3.64	
Sub-base 120mm (CBR 30%)	m ³	0.99	3.73	3.69	
Roadbase 150mm (CBR 55%)	m ³	1.12	4.36	4.88	
Single Otta seal (incl prime)	m ²	7.00	0.80	5.60	
Total				17.82	17,823
<i>Cross-section: total sealed width 8m</i>					
Fill 200mm	m ³	2.04	1.98	4.04	
Sub-base 120mm (CBR 30%)	m ³	1.14	3.73	4.25	
Roadbase 150mm (CBR 55%)	m ³	1.27	4.36	5.53	
Single Otta seal (incl prime)	m ²	8.00	0.80	6.40	
Total				20.22	20,228
<i>Note:</i> Only pavement layers are included in the comparison, <i>i.e.</i> costs are not total construction cost per kilometre					

Table D-4 Summary of pavement construction costs

Cross section	Cost (US\$/km)	
	Chart 1	Chart 2
6 m	15,940	15,525
7 m	21,340	17,823
8 m	21,340	20,228

The following case studies are based on these costs, adjusted to fit the assumptions made in each case study.

Case study 1**Assumptions**

- Climate area N < 4
- Design traffic class 0.1M
- Subgrade S3
- All sections along the road were well drained, with the crown height of the road over 750mm above side drain or ground level
- Pavement materials were available at haul distances and unit costs as assumed in the *basic cost data* above; the option was not available of using base materials with relaxed specifications (increased limit on PM by 20%) on an 8m wide road using design chart 2, since material of this specification could not be found closer to the road; therefore, no cost reduction was possible for this option

Options

The use of a 6m, 7m or 8m sealed road was considered, the choice being mainly dependent on costs. The 8m cross-section could be designed with thinner pavement layer and lower base quality material.

The costs of 6m or 7m cross-sections designed using chart 1, and an 8m cross-section designed using chart 2, were estimated using the costs provided in the *basic cost data* above. The following costs were obtained:

6m cross-section	US\$15,940/km
7m cross-section	US\$21,340/km
7m cross-section on fills > 1.2m	US\$17,823/km
8m cross-section	US\$20,228/km

Comments

Cross-sections of 6m and 7m width gave the lowest costs. The cross-section of 7m width was selected, although the difference in cost between those of 7m and 8m was relatively small. The cross-section of width 6m was excluded, since frequent bus traffic was expected on the road, and this width was considered too narrow. In addition, more than 15 per cent of the road was on embankments of more than 1.2m. Therefore, cost savings could be made by using design chart 2 for a 7m road in these areas.

Case study 2

- Assumptions**
- As in Case study 1 above, plus
 - Roadbase quality material of CBR 55 was now available at the same cost as material of quality CBR 45 (US\$3.78/m³)

Options The new assumptions gave the following costs:

6m cross-section	US\$15,940/km
7m cross-section	US\$21,340/km
7m cross-section on fills > 1.2m	US\$17,180/km
8m cross-section	US\$19,499/km

Comments Due to the shorter haul distance of CBR 55 materials, the costs of the 7m (design chart 1) cross-sections and the 8m (design chart 2) cross-sections were now almost the same. Thus, on the basis of cost, either cross-section could be selected. In addition to the large number of buses expected (as in Case study 1), it was realised that there were also many villages along the road and, consequently, many carts and pedestrians. It was therefore decided to use the 8m cross-section in areas close to the villages, in combination with the 7m cross-section in other areas. The pavement design of the 7m cross-section varied depending on whether or not the road was on an embankment > 1.2m.

Case study 3

Assumptions

- Climate area N>4
- Design traffic class 0.1M
- Subgrade class S4

Tentative designs

- Design chart 2 to be used
- Cross-section: 6m (total sealed width)

Assessment of materials

- Roadbase material (fine graded plastic calcrete) was available within 2km haul distance and had the following quality
 $I_p = 15$
 $CBR = 45$
 $PM = 400$
- Roadbase specification for the geo-climatic area and the intended cross-section was
 $I_p = 12$
 $CBR = 45$
 $PM = (320 \times 1.4) = 448$
- The available roadbase material was out of specification on I_p ; roadbase materials that were within specifications could be obtained at a haul distance of 30km

Options

- Option 1) hauled materials that were within specifications 30km, or
 Option 2) use the wider cross-section (8m)

The wider cross-section allowed use of the material available within two kilometre's haul. The required quality of roadbase materials for the 8m wide road was:

$$I_p = 15$$

$$CBR = 45$$

$$PM = 490$$

The costs of options 1) and 2) were calculated, and the most economical selected.

Using the unit costs from the *basic costs data*, above, adjusted for actual haul distances, the following costs were obtained for the two options:

Option 1) US\$16,813 per km
 Option 2) US\$16,727 per km

The cost of the 7m cross-section on fills > 1.2m, for which the same materials specifications apply as for the 8m cross-section, was US\$14,796 per km.

Comments

The above case study showed that the selection of cross-section, based on cost, was dependent on the availability, quality and haul distance of materials. Different haul distances can easily change the costs of the options. It is therefore of utmost importance that the materials survey clearly identifies sources and quality of materials. The design charts provide various options that enable optimum use of available materials to be made. The optimal design may require the use of different cross-sections along the length of the road.

In this case, since the costs for the two cross-sections were almost the same, either could have been selected. It might, however, have been advantageous to select the 8m width which gave better protection to the pavement in the vicinity of the outer wheel-track from the ingress of water, and was better from a traffic safety point of view.

Case study 4**Assumptions**

- Climate area: N < 2
- Topography: mountainous
- Subgrade S5

Tentative designs

- Cross-section: 7m (total sealed width)
- Traffic: many heavy and wide vehicles carrying timber expected on the road, although the number and axle loads were uncertain
- Design class 0.1M was selected because of the uncertainty of number of heavy vehicles that will be using the road, even though the 0.05M class had originally been intended

Assessment of materials

- Roadbase material was available within a free haul, with the following quality
Ip = 15
CBR = 45
PM = 460
- Roadbase specification for the climate area and intended cross section was
Ip = 15
CBR = 45
PM = 400
- Roadbase material available within free haul was out of specification on PM, so material that was within specifications had to be obtained using an 8m haul

Options

- Option 1) use design chart 1 and 7m cross-section, haul material to specifications 8km, or
- Option 2) use design chart 2 and the wider cross-section of 8m (7m cross-section designed to chart 2 can also be used on high fills)

The wider cross-section allowed the use of the materials available within free haul.

Specification for roadbase materials for the 8 m wide road:

Ip = 15
CBR = 45
PM = 460

The costs were calculated for options 1) and 2), and most economical selected.

Using the unit costs from the *Basic cost data*, above, adjusted for actual haul distances, the following costs were obtained for the two options:

Option 1) US\$17,120 per km
Option 2) US\$14,492 per km

Comments

Option 2, with the wider cross-section of 8m, was the cheapest. This was because a thinner pavement, with a roadbase of 150mm, was required by design chart 2. The wider cross section was also preferred for reasons relating to climate, topography, and type of traffic. The wider seal would protect the vulnerable area of the outer wheel-track, which in this case might carry heavy axle loads.

Other examples from the region

Over the 52km length of the Kasungu-Mzimba (KUMA), the cost of the crushed stone base was about 15 per cent of the total project cost. The differential between the cost of one kilometre of crushed stone roadbase and the one kilometre lateritic gravel roadbase test section on this road, where both materials were locally available, was about 4:1. Substantial savings on the cost of construction could have been achieved if these locally abundant lateritic gravels had been utilised for the roadbase construction.

On the Lilongwe-Mchinji road, there are no sources of crushed rock readily available. The differential cost between this and the lateritic gravel roadbase, that was used, would have been higher than the 4:1, and probably closer to 6:1.

A survey of seven contracts in Botswana indicated that the cost of construction of a natural gravel roadbase was between Pula10 and Pula16 per cubic metre (US\$1 = Pula3.5) compared with Pula 80-100 per cubic metre for crushed stone. This is equivalent to savings of about US\$30,000 per kilometre on the cost of the roadbase. If there is a need to seal the shoulders to guarantee a drier environment in the roadbase, then the extra costs will be about US\$3,000 per kilometre, although savings on shoulder re-gravelling costs will reduce this figure.

The use of sand as sub-base material can also result in considerable savings. Most of the calcretes are suitable for use in the sub-base but the use of the abundant sources of Kalahari sand has resulted in savings in haulage costs on recent projects. Savings of approximately US\$3,000 per kilometre can be expected where sub-base sand material is available adjacent to construction sites in the Kalahari.