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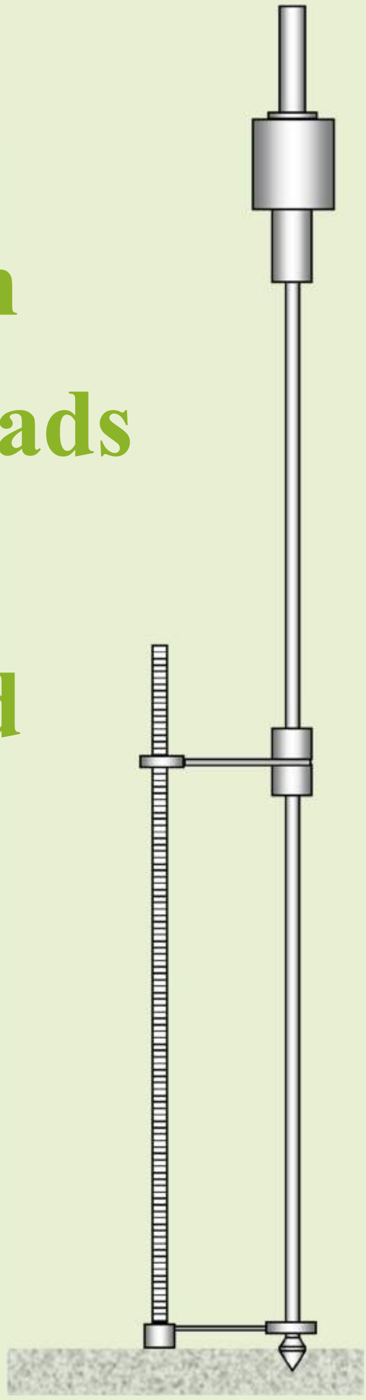

Pavement Design of Low Volume Roads using the DCP-DN Method

September 2020

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Layout: Infra Africa (Pty) Ltd
Gaborone
Botswana

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Foreword

UK Aid has been at the forefront of supporting research in the design of low volume roads (LVRs) in Africa and Asia to improve rural access to infrastructure and transport services. The research has included the development and enhancement of a method of pavement design for low volume roads (LVRs) based on DN values for materials strength derived with the Dynamic Cone Penetrometer (DCP). The DCP-DN method is an alternative to the more traditional methods of pavement design, which are based on the use of the California Bearing Ratio (CBR).

The purpose of this Manual is to provide guidance on the structural design of gravel and paved roads using the DCP-DN Design Method. The Manual takes account of the experience gained from a wide cross-section of experts over a seven-year period of application of the *Malawi Design Manual for Low Volume Sealed Roads Using the DCP Design Method,* which was published in 2013. This updated "generic" Manual incorporates the outcomes from the *Evaluation of Cost-Effectiveness and Value-for-Money of the DCP-DN Pavement Design Method in comparison with Conventional Designs* that was completed in 2019 under the Research for Community Access Partnership (ReCAP).

The Manual is intended for use principally in tropical and sub-tropical regions in Africa and Asia and other parts of the world with climate and geological conditions similar to the southern African geographical region where the method was originally developed. In dissimilar environments, further verification and performance monitoring of trial sections may be necessary. However, as the method is based on simple structural number principles, the method should be applicable in almost any environment.

The expected users of the Manual are practitioners from government agencies and private sector consultants who are responsible for the design of low volume roads (LVRs), as well as by academia and training institutions. The Manual includes all elements of the DCP-DN design procedure based on the accompanying ReCAP LVR DCP design software ver. 1.0, (2020). It is the key reference document for the DCP-DN pavement design method described in the Rural Road Note 01: *A Guide on the Application of Pavement Design Methods for Low Volume Rural Roads* developed under ReCAP.

It is my hope that this Manual will herald a new era in the more efficient and cost-effective design of LVRs in Africa, Asia, and elsewhere. It offers the potential to make a substantial contribution to the improved infrastructure of these countries and, in the process, enhance social and- economic growth and development for rural communities.

Dave Runganaikaloo
ReCAP Programme Director

Acknowledgements

The preparation of this Manual was financed by United Kingdom Department for International Development (DFID) (now the Foreign, Commonwealth & Development Office – FCDO) through the Research for the Community Access Partnership (ReCAP) – a DFID-funded research programme that promotes safe and sustainable access for rural communities in Africa and Asia.

The authors acknowledge the pioneering role played by Eduard Kleyn, dating back to the mid-1970s, as the Director of Roads of the then Transvaal Provincial Administration, South Africa, in developing the original concepts on which the DCP-DN design method was initially based. The significant additional work carried out in enhancing the method by Messrs. Phil-Paige-Green and Morris de Beer of the South African Council for Scientific and Industrial Research, and Gerrie van Zyl of Mycube Asset Management Systems, South Africa, is also acknowledged.

Project Management

The project was managed by Cardno Emerging Markets, UK, and was carried out under the general guidance of the ReCAP Deputy Team Leader - Infrastructure Research Manager, Eng. Nkululeko Leta.

Development Team

The Manual was developed by the following team of consultants led by Infra Africa (Pty) Ltd, Development Consultants, Botswana

Michael Pinard (Team Leader) - Lead author
Jon Hongve – Co-author
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Gerrie van Zyl – Contributing author
Dr. Estime Mukandila – Contributing author

Peer Review

The authors acknowledge the valuable comments made on the Draft Manual by Dr. Ian van Wijk, Adjunct Professor, University of Queensland, Brisbane Australia, and Technical Director, Aurecon International, Brisbane, Australia, and by the ReCAP Technical Panel.

Lastly, the authors acknowledge the contributions made by practitioners within the ReCAP community who have been involved in various aspects of the application of the DCP-DN method of pavement design.

Terminology

The terminology used to describe various components of a low volume road is illustrated below for ease of reference in this Manual.

Pavement[1]

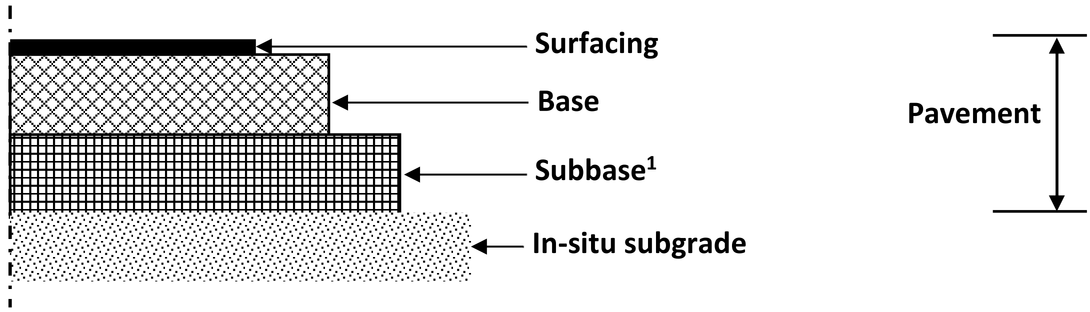

Figure 1: Main components of a LVSR pavement

Table 1: Purpose of main components of a LVSR pavement

Pavement component	Purpose
Surfacing	• Provides a smooth running surface. • Provides a safe, economical and durable all-weather surface. • Minimizes vehicle operating and maintenance costs. • Reduces moisture infiltration into the pavement. • Provides suitable properties for the local environment, e.g. dust suppression, skid resistance and surface texture. • Delineates traffic lanes and shoulders, bicycle paths, traffic calming devices. • Visually enhances the road environment for road users and adjacent residents.
Base (base course)	• Provides the bulk of the structural capacity in terms of load-spreading ability by means of shear strength and cohesion. • Minimizes changes in strength with time by having relatively low moisture susceptibility. • Minimizes the ingress of moisture into the pavement by having adequate shrinkage and fatigue properties. • Assists with the provision of a smooth riding surface by having volume stability with time and under load.
Subbase/improved subgrade (reformed & compacted existing road surface)	• Provides a stable platform for the construction of the base and surfacing. • Assists in providing adequate pavement thickness so that strains in the in-situ subgrade are kept within acceptable limits.
In situ subgrade	• Refers to the naturally occurring material on which the pavement and improved subgrade are constructed. The stiffness (related to the degree of compaction) of the subgrade influences the quality/thickness of the overlying pavement layers.

[1] Subbase can be from imported material or an improved in-situ subgrade layer.

Cross Section

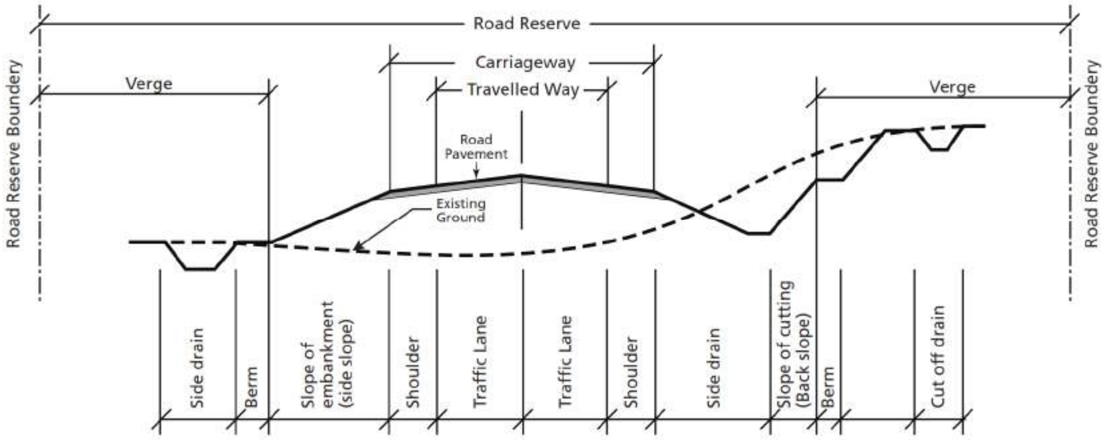

Figure 2: Cross-section elements

Drainage Elements

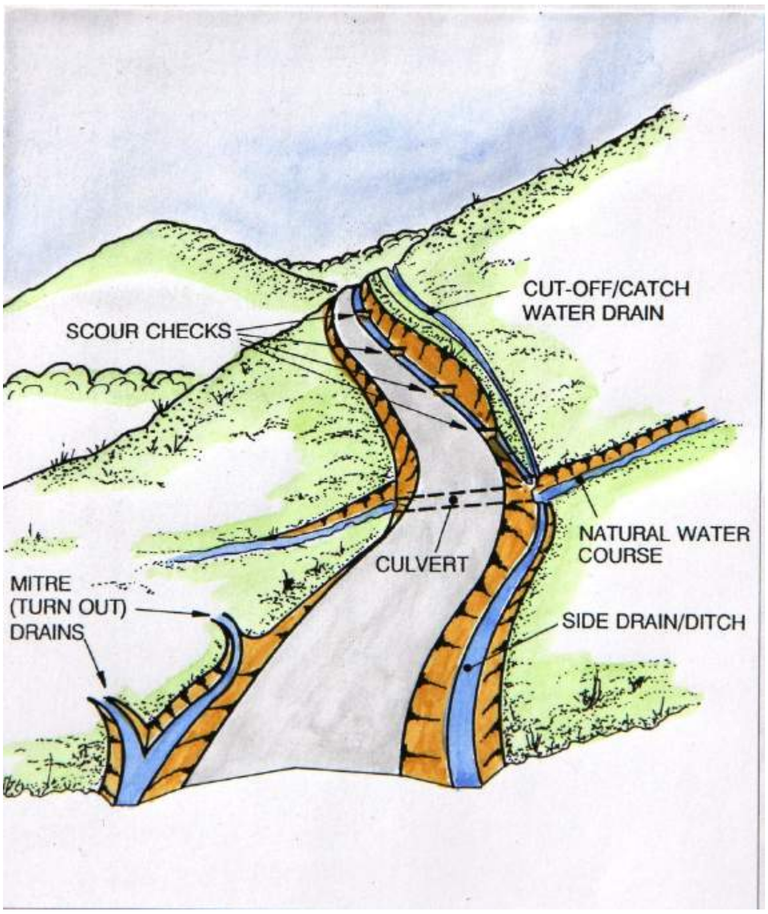

Figure 3: Main drainage elements

List of Abbreviations, Acronyms and Initialisms

A	The deviation between the SPBC and the measured balance curve (represents a "goodness of fit" parameter for the pavement).
AADT	Average Annual Daily Traffic
AASHTO	American Association of State Highway Officials
ACV	Aggregate Crushing Value
AIV	Aggregate Impact Value
ASTM	American Society for Testing and Materials
B	A parameter defining the Standard Pavement Balance Curve (SPBC)
BN_{100}	The number of DCP blows as a percentage of the DSN800
BS	British Standard
CBR	California Bearing Ratio
CESA	Cumulative Equivalent Standard Axles
CL	Centre Line
CSIR	Council for Scientific and Industrial Research
CUSUM	Cumulative Sum
DCP	Dynamic Cone Penetrometer
DES	Discrete Element Surfaces
DESA	Daily Equivalent Standard Axles
DF	Drainage Factor
DFID	Department for International Development
DN	The average penetration rate in mm/blow of the DCP through a pavement layer
DOS	Double Otta Seal
DSD	Double Surface Dressing
DSN_{450}	Number of DCP blows required to penetrate the top 450 mm of a pavement
DSN_{800}	Number of DCP blows required to penetrate the top 800 mm of a pavement
DSS	Double Sand Seal
EF	Equivalence Factor
EIP	Environmental Impact Plan
ENS	Engineered Natural Surface
EOD	Environmentally Optimized Design
ESA	Equivalent Standard Axle (80 kN)
FACT	Fines Aggregate Crushing Test
FCDO	Foreign, Commonwealth & Development Office
FHWA	Federal Highway Administration
FMC	Field Moisture Content
g	Grade
GB	Granular Base
Gc	Grading Coefficient
GM	Grading Modulus
GVM	Gross Vehicle Mass
HDM-4	Highway Development and Management Model - 4
HGV	Heavy Goods Vehicle
HVR	High Volume Road
HPS	Hand Packed Stone
LAA	Los Angeles Abrasion Value
LBM	Labour Based Methods
LCC	Life Cycle Cost
LGV	Light Goods Vehicle
LL	Liquid Limit
LVR	Low Volume Road

LVSR	Low Volume Sealed Road
MAASHTO	Modified AASHTO
MC	Moisture Content
MDD	Maximum Dry Density
MESA	Million Equivalent Standard Axles
MGV	Medium Goods Vehicle
NMT	Non-motorised Traffic
NPV	Net Present Value
O/D	Origin & Destination
OMC	Optimum Moisture Content
OWL	Outer Wheel Left
OWR	Outer Wheel Right
P2	Percentage of material passing the 2 mm sieve
P425	Percentage of material passing the 0.425 mm sieve
P075	Percentage of material passing the 0.075mm sieve
PL	Plastic Limit
PM	Plastic Modulus
PSD	Particle Size Distribution
PV	Present Value
QA	Quality Assurance
QC	Quality Control
R	Right
RED	Roads Economic Decision Model (a World Bank Model)
SADC	Southern African Development Community
SOS	Single Otta Seal
Sp	Shrinkage Product
SPBC	Standard Pavement Balance Curve
SS	Slow Setting
SSA	Sub-Saharan Africa
SSD	Single Surface Dressing
SSS	Single Sand Seal
TLC	Traffic Load Class
TRL	Transport Research Laboratory
UK	United Kingdom
UKAID	Development Assistance provided by the UK Department for International Development
URC	Unreinforced Concrete
VEF	Vehicle Equivalence Factor
VOC	Vehicle Operating Costs
vpd	Vehicles per day

1. Introduction

1.1 Background

> *"Low volume roads are a lower order of worldwide land transport. They begin where animal tracks and walking trails end. They are the beginning of the world economy and are the lifelines for rural communities. Everything that sustains us – grown, mined, or drilled – begins on a low-volume road"*[1].

The above words encapsulate the vital role played by low volume roads (LVRs) in the development of all countries. Such roads provide the essential links to the primary/trunk road system and facilitate economic opportunity; connect rural, developing, and underdeveloped areas; and provide access to education, medical facilities and markets. However, the attainment of the potentially catalytic role that LVRs can play in the economy of any country depends critically on the ability of a road agency to provide such roads in an economical manner. This requires the adoption of appropriate design standards and practices that maximise the use of the available, often limited, funding in an optimal manner.

The cost of providing LVR infrastructure based on traditional standards and design methods as used on higher-order roads can be prohibitive. This is because these approaches are generally conservative and too costly for application to a LVR environment for which more cost-effective solutions are required. In order to address these challenges, the UK Department for International Development (DFID), through the Research for Community Access Partnership (ReCAP), has supported research aimed at developing appropriate standards, specifications and design methods for LVRs. This has included the use of the Dynamic Cone Penetrometer (DCP) for pavement design purposes as an alternative to the more traditional design methods that are based on the use of the California Bearing Ratio (CBR), details of which may be found in the *Rural Road Note 01: A Guide on the Application of Pavement Design Methods for Low Volume Rural Roads* developed under ReCAP.

The Design Manual for Low Volume Sealed Roads Using the DCP Design Method was initially developed under the Africa Community Access Programme (AfCAP) in 2013 based on the outcome of a *Performance Review of Design Standards and Technical Specifications Used on Low Volume Sealed Roads in Malawi.* The Manual has subsequently been applied in the design of trial sections in a number of African countries, including Kenya, Tanzania, Malawi and Ghana, from which much experience has been gained and valuable feedback received from practitioners. This has led to a need to update the original design Manual by developing an updated, generic version that can be applied in appropriate environments in the African and Asian regions. Where there is doubt as to whether a given environment is appropriate, further guidance is given in Section 3.3.6.

The Manual draws on the outputs of several research and investigation projects that have been carried out in Africa and Asia, particularly since the 1990s. The corroborative findings of this research work provide a wealth of evidence-based information that has advanced previous knowledge on various aspects of LVR technology. This has allowed state-of-the-art guidance to be provided in this Manual, which is expected to serve as a regionally recognized document, the application of which will harmonize approaches to the provision of LVRs in Africa and Asia.

The Manual is intended to be the key reference document for the DCP-DN pavement design method described in the Rural Road Note 01 that was developed under ReCAP.

[1] Michael T. Long, Chair, US Transportation Research Board (TRB), Low-Volume Roads Committee

1.2 Purpose

The purpose of this Manual is to provide practitioners with the necessary guidance for using the DCP design method to undertake sound and economical pavement design of both gravel and paved LVRs. Such an approach is aimed at maximising the use of locally available materials and, in the process, minimising the life-cycle costs of road provision by also taking account of the many locally prevailing road environment factors that impact on the performance of LVRs. In so doing, the primary goal is to reduce the cost of providing LVR infrastructure leading to:

- increased public and commercial transport through lower road user costs;
- improved access to schools, clinics, jobs, urban centres, and neighbouring rural areas;
- improved environmental, health and social conditions; and
- enhanced socio-economic growth, development and poverty alleviation.

The Manual is intended for use by practitioners from government road agencies and private sector consultants who are responsible for the design of LVRs, as well as by academia and training institutions.

1.3 Scope

The Manual focuses on the structural design of both gravel and paved roads with thin, non-structural bituminous surfacings that are typically found in Africa and Asia. The environmentally optimised approach to the design of such roads is a key feature of the Manual. This approach can be applied to interventions that deal with individual critical sections or to the total length of a road link. In the latter case, this could comprise different design options along the total road length.

Because of the diverse physical features of the various countries in Africa and Asia, it would be impractical and inappropriate to provide recipe solutions for specific situations. Instead, the emphasis has been placed on guiding the practitioner towards evaluating alternative options and considering their pros and cons as a basis for adopting an appropriate solution.

1.4 Structure

The Manual is divided into ten chapters that collectively address various aspects of LVR provision, as presented below.

Chapter 1 - Introduction: Discusses the background, purpose and scope of the Manual as well as the manner of updating it.

Chapter 2 - Low Volume Roads in Perspective: Places in broad perspective the various factors that affect the provision of LVRs, including their definition and particular characteristics, the approach to their design, the environmentally optimised design philosophy, and the risk factors to be considered.

Chapter 3 – Approach to Design: Presents the approach to the design of both paved and unpaved LVRs based on the use of the Dynamic Cone Penetrometer (DCP) method of design, including the design philosophy, principles and concepts, as well as the strengths and limitations of the method and its applicability in practice.

Chapter 4 – Surveys and Investigations: Details the procedures to be followed in obtaining the basic inputs to the design of the road pavement, including traffic, route and materials surveys as well as moisture, drainage investigations and materials prospecting and sampling.

Chapter 5 – Materials: Provides the approach to selecting and using materials for the construction of LVRs, as well as the manner of testing and selecting them for use in the pavement structure.

Chapter 6 – Drainage and Climate Adaption: Deals with the sources of moisture into a pavement, the elements of both internal and external drainage, and measures for dealing with the impact of climate change.

Chapter 7 - Structural Design: Paved Roads: Presents a detailed procedure for undertaking pavement design based on the ReCAP LVR DCP software and covers the application of the method in various situations typically encountered in practice.

Chapter 8 – Structural Design: Gravel Roads: Provides details for the pavement design of gravel roads as well as the manner of selecting the gravel wearing course. It also discusses the use of chemicals for improved performance of gravel wearing course materials.

Chapter 9 - Surfacings: Provides an overview of the various types of bituminous and non-bituminous discrete-element surfacings that are potentially suitable for use on LVRs, their performance characteristics, and the factors that may govern their selection.

Chapter 10 – Life-Cycle Costing: Outlines the procedure to be followed in undertaking a life-cycle cost analysis to compare alternative pavement/surfacing/upgrading options, including the method of carrying out such analyses.

1.5 Sources of Information

In addition to providing detailed information and guidance, the Manual also serves as a valuable source document because of its comprehensive lists of references from which readers can obtain supplementary information to meet their particular needs. A bibliography is provided at the end of each chapter of the Manual. Where the sources of any tables or figures are not specifically indicated, they are attributed to the authors.

1.6 Use of the Manual

The Manual will require approval by the Ministry responsible for roads before its use. This will ensure that a consistent and harmonised approach in the design of LVRs is followed in the respective countries.

The Manual is based on current good practice in the design of LVRs based on the DCP-DN method, as described in this document. However, as LVR technology is continually being researched and improved, it may be necessary to periodically update the Manual to reflect improvements in practice. Such updating would typically take the form of officially released circulars to stakeholders.

There may also be situations where the designer may propose to deviate from the standards presented in this Manual. Proposals for such changes should reflect the standard approach to deviation from standards in engineering practice, including the provision of the following information:

- The aspect of design for which a departure from the stipulated standards is desired.
- A description of the standard, including the normal value, and the value of the departure from the stipulated standard.
- The reason for the departure from the standards.
- Any mitigation to be applied in the interests of reducing the risk of failure.

The designer must seek approval from the Ministry responsible for roads, via the respective road agency, for any proposed major or minor departures from the stipulated standards.

Bibliography

Behrens L C (1999). ***Overview of Low-Volume Roads:*** Keynote Address, 7th International Conference on Low Volume Roads. Transportation Research Record No. 1652, TRB, Washington, D.C., USA.

Faiz A (2012). ***The Promise of Rural Roads: Review of the Role of Low-Volume Roads in Rural Connectivity, Poverty Reduction, Crisis Management, and Liveability***. Transportation Research Circular No. E-C167. Transportation Research Board of the National Academies, Washington, D.C., USA.

Ministry of Transport and Public Works (2013). ***Design Manual for Low Volume Sealed Roads Using the DCP Design Method***, Lilongwe, Malawi.

Pinard M I (2011). ***Performance Review of Design Standards and Technical Specifications for Low Volume Sealed Roads in Malawi.*** AfCAP Report MAL/016.

Lebo J and D Schelling (2001). ***Design and Appraisal of Rural Infrastructure: Ensuring Basic Access for Rural Communities.*** World Bank Technical Paper 496. World Bank, Washington, D.C., USA.

Southern Africa Development Community (SADC) (2003): ***Guideline on Low-volume Sealed Roads***. SADC House, Gaborone, Botswana.

TRL Limited (2020). ***Rural Road Note 01: A Guide on the Application of Pavement Design Methods for Low Volume Rural Roads.*** Crowthorne, Berkshire, UK.

2. Low Volume Roads in Perspective

2.1 Introduction

2.1.1 Background

The traditional approaches to the provision of low volume roads (LVRs) in many tropical and sub-tropical countries are generally based on technology and research carried out in temperate climates. While these "standard" approaches might still be appropriate for much of the relatively heavily-trafficked primary road network, they remain conservative, inappropriate, and too costly for application on much of the relatively lightly-trafficked LVR networks in Africa and Asia. Thus, in facing the challenges of improving and expanding LVR networks, more appropriate approaches need to be considered.

The approach to the design of LVRs follows the general principles of any good road design. The focus should be on maximising the use of the existing pavement structure, as much as possible, without disturbing its inherent strength derived over many years from consolidation by traffic. New pavement layer(s) can then be added, as necessary, to cater to the design traffic.

2.1.2 Purpose and Scope

The primary purpose of this chapter is to place in broad perspective the various factors that affect the provision of LVRs. To this end, the chapter addresses the following topics:

- The particular characteristics of LVRs.
- The approach to the design of LVRs.
- Risk factors to be considered.

2.2 Characteristics of Low Volume Roads

2.2.1 Definition and Classification of Low Volume Rural Roads

A shared understanding of the definition of a LVR for pavement design purposes is crucially important as it will dictate the approach to undertaking the design of such roads in relation to their characteristics and the related criteria to be used in providing them at an appropriate level of service and minimum life-cycle cost.

There is no internationally accepted definition of a LVR. In developed countries such as the USA, roads carrying about 400 vehicles per day (vpd) are defined as "very low volume roads". Other countries, such as many in Africa and Asia, adopt a figure of about 300 motorised, 4-wheeled vehicles, including about 20-25% commercial vehicles, and a related traffic loading of up to about one million Equivalent Standard Axles (MESA) per lane over a design life of typically 10-15 years. For pavement design purposes, for which traffic loading, rather than traffic volume, is required, a design life traffic loading of up to about 1 MESA[1] has been adopted in this Manual to define a LVR.

For classification purposes, roads are usually categorised on the basis of the function that they serve within the overall road network. This functional classification signifies the purpose or role that these roads serve in terms of connecting different centres of population and economic activity. In general, there are three main functional distinctions, as described below and illustrated in Figure 2-1:

1) Primary or arterial/trunk/main roads linking major centres of population and production
2) Secondary or collector roads linking regional/local centres with the primary road network.
3) Tertiary or access roads linking village/district centres and local centres of population to higher-order roads of the road network.

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[1] 1 MESA applies to the traffic loading per lane for roads ≥ 6.0 m wide. For narrower roads the traffic loading needs to be adjusted in accordance with the requirements of Table 4-4 in *Chapter 4 – Surveys and Investigations.*

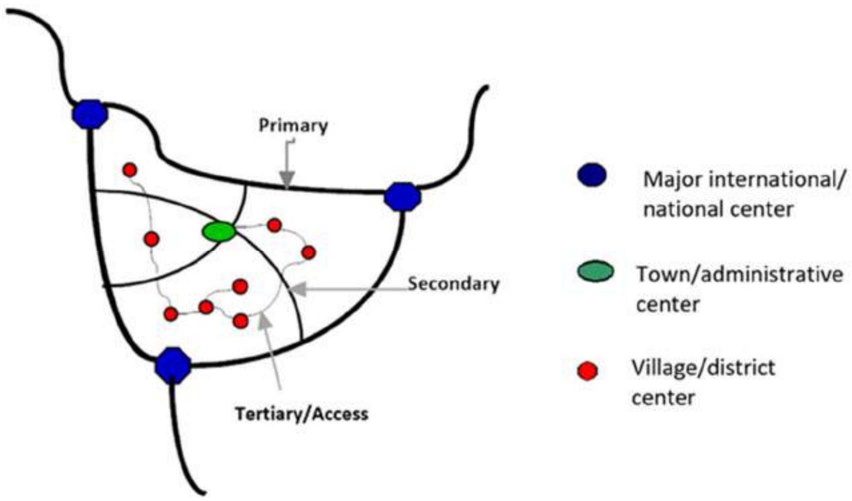


Figure 2-1: Typical road hierarchy and functions

Generally, but not necessarily, LVRs are associated with the lower-order tiers of the road network, i.e., tertiary access and sometimes secondary roads that provide a primarily *access* (relatively low speed, low geometric standard) function. However, in some countries, even primary roads may carry low levels of traffic, although they provide a primarily *mobility* (relatively high speed, high geometric standard) function. Thus, for pavement design purposes, LVRs within all tiers of the road network may be designed following the DCP-DN method presented in this Manual.

2.2.2 Special Features

The following special features of LVRs affect the manner of their provision and need to be fully appreciated by the designer:

- They are constructed mostly from naturally-occurring, often "non-standard", moisture-sensitive materials.
- Pavement deterioration is driven primarily by environmental factors, particularly moisture, with traffic loading being a relatively lesser influential factor, and drainage being of paramount importance.

It must also be appreciated that conventional economic analysis (focussing on consumer surplus or road user savings) often cannot fully justify the investment of public funds in the provision or improvement of LVRs. Thus, it can be relatively difficult to quantify the many other benefits that are of a broad socio-economic and environmental nature.

2.3 Approach to Design

2.3.1 General Approach

Whilst the approach to the design of LVRs follows the general principles of any good road design practice, the level of attention and engineering judgement required for optimal provision of such roads tends to be higher than that required for the provision of other roads. This is because optimising a design requires a multi-dimensional understanding of all of the project elements, and in this respect, all design elements become context-specific. This will require:

- A full understanding by the design engineer of the local environment (physical and social).
- Recognition and management of risk.
- Innovative and flexible thinking through the application of appropriate engineering solutions rather than following traditional thinking related to road design.

2.3.2 Influence of Road Environment

The term "road environment" is an all-encompassing one that includes both the natural or biophysical environment and the human environment. It includes the interaction between the different environmental factors and the road structure. Some of these factors are uncontrollable, such as those attributable to the natural environment, including the interacting influence of climate (e.g., wind, rainfall duration and intensity, temperatures, etc.), local hydrology and drainage, terrain and gradient. Collectively, these will influence the performance of the road, and the design approach needs to recognise such influence by providing options that minimise the adverse effects. Other factors, such as the construction and maintenance regime, safety and environmental demands, and the extent and type of traffic, are largely controllable and can be more readily built into the design approach.

Typical road environment factors that impact on the LVR design process are presented in Figure 2-2 and are covered in more detail in various parts of the Manual.

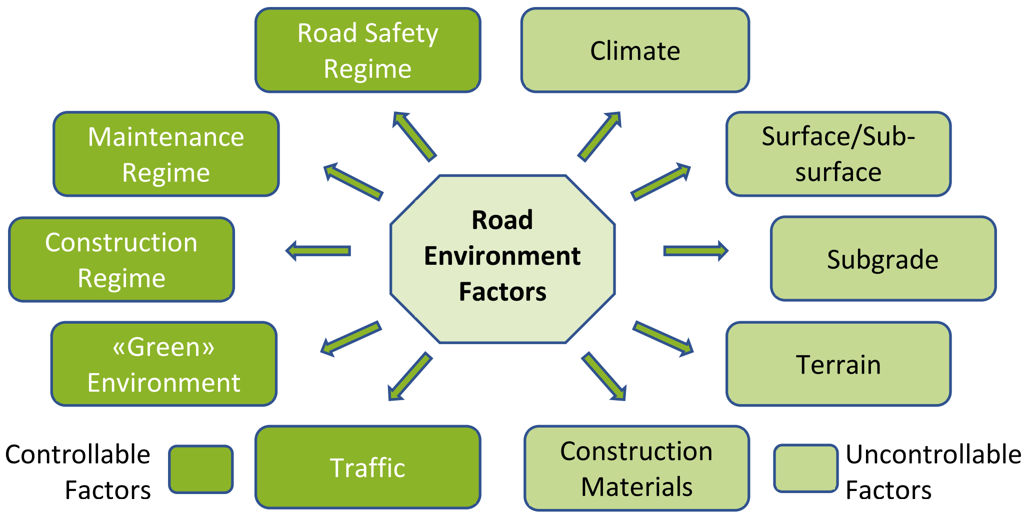


Figure 2-2: Various road environment factors affecting design

2.3.3 Road Deterioration Factors

The relative influence of environmental factors and traffic on pavement performance is shown schematically in Figure 2-3. Environmental factors - primarily in terms of moisture and temperature – tend to induce surface degradation and cracking, which, in turn, leads to moisture ingress and, ultimately, pavement failure. At the lower end of the traffic spectrum, this impact overshadows the traffic loading impact with the cross-over point tending to about 1 MESA, depending on climate, materials used, and pavement configuration. Thus, in the design of a LVR, particular attention needs to be paid to the influence of moisture on the performance of LVRs, and to the adoption of appropriate drainage measures, the use of a durable surfacing, as well as adequate maintenance, to mitigate against the adverse effects of moisture ingress into the pavement structure.

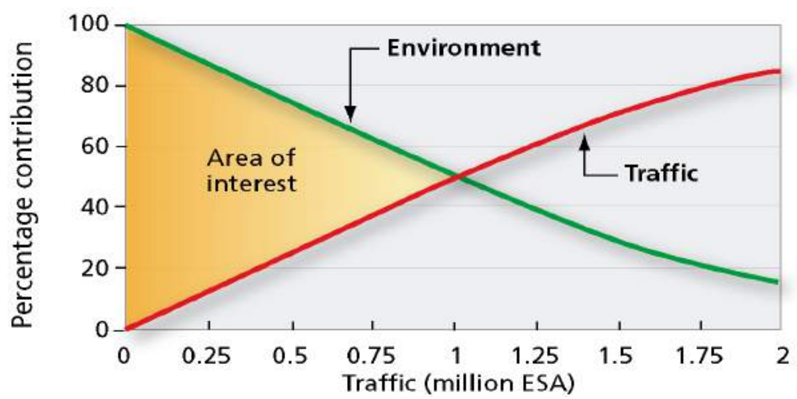


Figure 2-3: Traffic loading versus dominant mechanism of pavement distress (schematic)

2.3.4 Adoption of an Environmentally Optimised Design Approach

It is essential to adopt an approach to the design of LVRs that is guided by appropriate local standards and conditions. In this regard, there are significant benefits of applying the principles of "Environmentally Optimised Design" (EOD) to the provision of LVRs in a manner that is compatible with the local road environment, as illustrated in Figure 2-4.

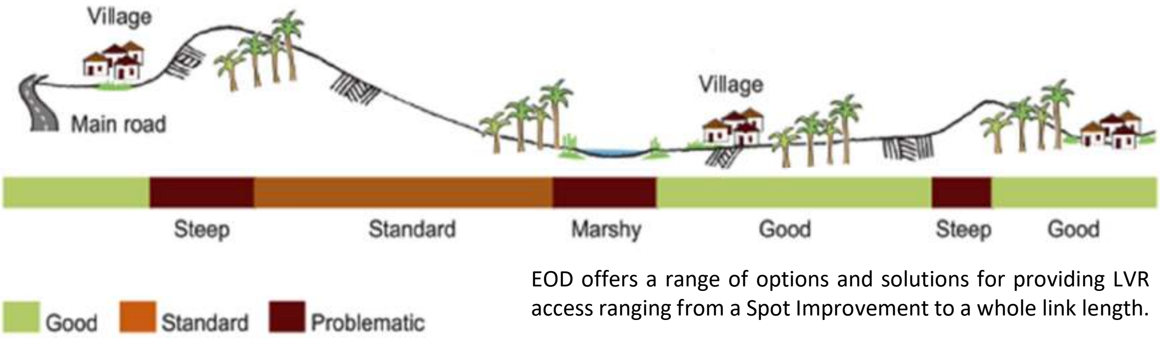


Figure 2-4: Environmentally optimised and spot improvement strategy (schematic)

The essence of the EOD approach is to satisfy several strategic objectives, including:

- The practicality of the recommended designs for implementation within the available resources and level of expertise in rural areas.
- The use of design standards and materials specifications that should aim at achieving an appropriate level of serviceability which should not fall below the minimum acceptable level for the prescribed standard of road during its design life.
- The availability of equipment/plant for construction and maintenance, as well as the level of quality control that can be effectively exercised in rural areas.
- The maximum use of local labour and skills.
- The maximum use of locally available materials or those that can be processed locally.
- In-built maintenance considerations in the design, such as the provision of adequate drainage, resistance to soil erosion along the side slopes, adequate lateral support from shoulders, etc. that would minimise subsequent maintenance requirements.

The EOD strategy should be applied with the overall aim of ensuring that each section of a road is provided with the most suitable pavement type for the specific circumstances prevailing along the road. This requires analysis of a broad spectrum of solutions to improve different road sections, depending on their requirements, ranging from engineered natural surfaces to paved roads. The chosen solution must be achievable with materials, plant and contractors available locally.

2.3.5 Use of Local Non-standard Materials

Construction costs of the upper pavement layers (base and subbase) are typically about 30 – 40% of the total construction cost of a LVR. Thus, a full understanding of the nature, engineering character, and properties of construction materials is essential aspect of the road environment assessment. The challenge is to adopt appropriate design and material standards that deliver acceptable solutions. In this regard, there is research-based evidence to support the lowering of the traditional standards and specifications typically applied to high volume roads, to more appropriate standards and specifications for application to LVRs. These aspects of the LVR design philosophy are addressed in *Chapter 7 – Structural Design: Paved Roads and Chapter 8 – Structural Design: Gravel Roads.*

2.3.6 Surface Improvement Technology

Earth and gravel roads are particularly vulnerable to the effects of the road environment. A range of more durable surfacing options is available for LVRs. These include thin bituminous surfacings and non-bituminous surfacings such as cobblestone, hand-packed stone, and concrete. The selection, outline materials requirements and use of various surfacing options in a context-sensitive manner are described in *Chapter 9 – Surfacing.*

Improved surfacings may be provided for the entire length of a road, or only on the most vulnerable sections. The approach may include spot improvements which deal only with individual critical sections on a road link (e.g., weak or vulnerable sections, roads through villages or settlements), or providing an overall total whole rural link design, which could comprise different design options along its length.

2.3.7 Upgrading Stages of a Low Volume Road

The decision as to when a LVR should be upgraded to a higher (more expensive) standard (service level) is often not a simple choice between a paved and an unpaved road. Over a period of time, a road will often undergo a number of improvements or upgrading iterations during its use. Figure 2-5 illustrates the various upgrading stages of LVRs. A life-cycle cost analysis, as discussed in *Chapter 10 – Lice-cycle Costing*, should be undertaken to determine when to upgrade from one standard to a higher one.

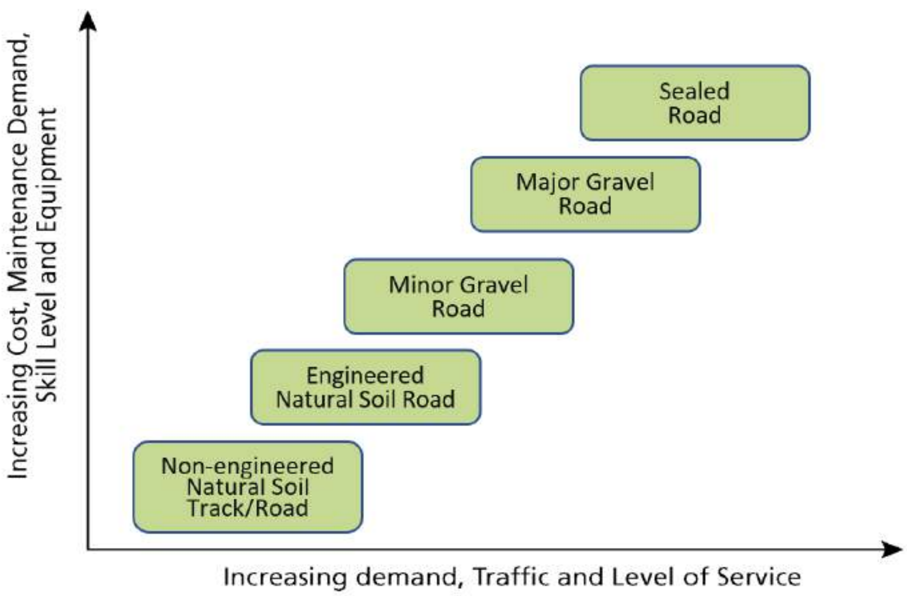


Figure 2-5: Upgrading stages of a LVR

2.4 Risk Factors

The departure from well-established, generally conservative, material quality specifications may carry some increased level of risk of failure for a LVR. However, such a risk should be a calculated one and not a gamble and must consider not just materials but the whole pavement and its environment. Thus, in any pavement design strategy, it is necessary to be aware of the main risk factors which could affect the performance of LVRs so that appropriate mitigating measures may be adopted to minimise them. These factors are summarised below:

1. **Quality and type of materials (strength, durability and moisture susceptibility).**
   Particular attention needs to be paid to the selection of subgrade and pavement materials by adherence to the specified testing standards during laboratory testing. Also, care needs to be exercised when contemplating the use of materials of basic igneous origin, which may decompose to various degrees as a result of the alteration of certain rock minerals to clay minerals.

2. **Construction control (primarily compaction standard and layer thickness).**
   Adherence to the specified thickness and compaction requirements is essential to ensuring that the minimum material strength assumed in the design is achieved in practice. Failure to do so will result in reduced strength and bearing capacity of the pavement and a lessening of its design life.

3. **Environment (particularly moisture ingress into the pavement and external and internal drainage).**
   It is essential that various mitigation measures against moisture ingress into the pavement, either through the surfacing or from the shoulders, are attained in practice (Ref. *Chapter 6 – Drainage and Climate Adaptation*). Failure to do so will result in a reduction in pavement material strength and related bearing capacity of the pavement and a lessening of its design life.

4. **Maintenance standards (drainage, surfacing and shoulders).**
   LVRs are particularly vulnerable to inadequate or deferred maintenance due to the extensive use of local, often moisture-sensitive materials for pavement construction. This vulnerability is further exacerbated by the projected climate changes in the coming decade. Thus, the highest priority should be given to timely, adequate maintenance of LVRs to avoid their premature deterioration.

5. **Vehicle loads (overloading).**
   LVRs are particularly prone to excessive deformation and failure due to overloading. Just a single, excessively loaded axle can cause the pavement to deform severely and fail. Thus, effective overload control measures are essential to the satisfactory performance of LVRs. If this is unlikely to be achieved in practice, then a more substantial pavement should be provided.

The risk of premature failure will depend on the extent to which the above factors are negative – the greater the number of factors that are unsatisfactory, the greater the risk of failure. However, this risk can be greatly reduced by adhering to the material specifications prescribed in this Manual, by ensuring that the construction quality is well controlled and that drainage measures are strictly implemented and, most importantly, that maintenance is carried out in a timely manner and vehicle overloading is reasonably well controlled.

Bibliography

Behrens L C (1999). ***Overview of Low-Volume Roads: keynote Address, 7th International Conference on Low Volume Roads.*** Transportation Research Record No. 1652, TRB, Washington, D.C., USA.

Cook J R and C S Gourley (2002). ***A Framework for the Appropriate Use of Marginal Materials.*** World Road Association (PIARC) – Technical Committee C12 Seminar in Mongolia.

Gourley C S and P A K Greening (1999). ***Performance of Low-Volume Sealed Roads: Results and recommendations from studies in southern Africa.*** TRL Project Report PR/OSC/167/99. Transport Research Laboratory, Crowthorne, Berkshire, UK.

Faiz A (2012). ***The Promise of Rural Roads: Review of the Role of Low-Volume Roads in Rural Connectivity, Poverty Reduction, Crisis Management, and Liveability***. Transportation Research Circular No. E-C167. Transportation Research Board of the National Academies, Washington, D.C., USA.

Greenstein J (1993). ***Issues Related to Administration of Low-volume Roads in Developing Countries.*** Transportation Research Record 1426. Transportation Research Board, Washington, D.C., USA.

Kelly K and S Juma (2015). ***Environmentally Optimised Design for Low-Volume District Roads in Tanzania***. Transportation Research Record: Journal of the Transportation Research Board Volume 2772.

Naidoo K, Purchase R and T Distin (2004): ***Blacktop Roads to Developing Communities Using Appropriate Technologies.*** Proc. 8th Conference on Asphalt Pavements for Southern Africa, South Africa.

Paige-Green P (1991). ***Recommendations on the use of marginal base course materials in low volume roads in South Africa.*** Research Report RR 91/201, Department of Transport, Pretoria, RSA.

Paige-Green P (1999). ***Materials for sealed low volume roads.*** Transportation Research Record 1652, TRB, National Research Council, Washington, D.C., USA.

Pinard M I, Paige-Green P and J Hongve (2015): ***Developments in Low Volume Roads Technology: Challenging Traditional Paradigms.*** Conference on Asphalt Pavement for Southern Africa, Sun City, South Africa.

Rolt J K, Mukura K, Dangare F and A Otto. (2013). ***Back analysis of previously constructed rural roads in Mozambique.*** African Community Access Programme Project MOZ/001/G. CPR 1612. DFID, UK.

Southern Africa Development Community (SADC) (2003). ***Guideline on Low-volume Sealed Roads.*** SADC House, Gaborone, Botswana.

South East Asia Community Access Programme (SEACAP) (2009). ***Low Volume Rural Road Environmentally Optimised Design Guideline.*** Department of Roads, Ministry of Public Works and Transportation, Lao PDR.

Transport Research Board/National Research Council (1995). ***Assessing Worldwide Low-volume Roads: Problems, Needs and Impacts***. Transport Research Circular No. 446, TRB, Washington, D.C., USA.

Toole T, Rice Z, Latter L and K Sharp (2018): ***Appropriate Use of Marginal and Non-Standard Materials in Road Construction and Maintenance.*** Austroads Ltd, Sydney NSW 2000 Australia.

Visser A T and T van Niekerk (1987). ***The implementation of appropriate technology in the design of light pavement structures,*** Transp. Res. Record, 1106, 222–31

3. Approach to Design

3.1 Introduction

3.1.1 Background

The approach to the design of LVRs follows the general principles of any good road design. However, there are several important differences from the traditional approaches that need to be appreciated by the designer in order to provide appropriate, cost-effective designs. For example, a recognition that pavement distress is generally attributable more to the effects of the natural environment than to the traffic loading. Also, materials that may be unsuitable for high volume roads can be "fit-for-purpose" and suitable for use in LVRs.

Ultimately, the approach adopted for the design of LVRs should result in the provision of a pavement structure that is appropriate to the road environment in which it operates and fulfills its function at minimum life-cycle cost at an optimal level of service. However, positive action in the form of timely and appropriate maintenance, as well as adequate control of vehicle overloading, will be necessary to ensure that the assumptions of the design phase hold true over the design life of the road.

Fortunately, considerable research has been carried out in the past few decades in Sub-Saharan Africa and Southeast Asia that has led to the development of more appropriate pavement design methods. This has enabled unpaved roads to be upgraded economically to a paved standard by making optimal use of local materials that do not necessarily have to meet the standard specifications that are applied to HVRs.

3.1.2 Purpose and Scope

The main purpose of this chapter is to present the approach to the design of both paved and unpaved LVRs based on the use of the Dynamic Cone Penetrometer (DCP) method of design (hereafter referred to as the DCP-DN design method). The chapter addresses the following topics:

- General LVR design principles.
- Development of the DCP-DN design method.
- DCP-DN design philosophy and concepts.
- Strengths and limitations of the DCP-DN design method.
- Applicability of the DCP-DN design method.

An outline, generic Table of Contents for a design report is presented In Appendix 3.2.

3.2 General Design Principles

3.2.1 General

The principle behind the environmentally optimised design of a LVR is to produce a sound pavement structure at minimum life-cycle costs that achieve the functionality of the road. This typically entails:

- Making optimum use of the in-situ and other locally available materials.
- Utilising the existing pavement structure, which often has been moulded under traffic for a prolonged period to a relatively high density and stiffness.
- Adding only a new pavement layer(s) of minimum thickness and appropriate material quality to satisfy the requirements of the design catalogue or reworking, modifying the upper 100 – 150 mm.

3.2.2 Pavement Structure and Function

When the natural subgrade of a road is not strong enough to support the repeated application of axle loads without deforming, it will be necessary to protect it from overstressing by traffic loads. This can be achieved by introducing stronger materials above the subgrade (the pavement layers) to provide a chosen design life as cost-effectively as possible. The pavement layers must be in balance with each other, i.e., they must decrease gradually in strength from the upper to the lower layers, and must possess the following attributes if the pavement structure is to perform satisfactorily.

- Sufficient strength and stiffness to withstand repeated cycles of vertical stress without excessive deformation.
- Sufficient bearing capacity which is achieved through:
  - Adequate thickness of material required to protect the subgrade for given traffic levels
  - Inter-particle friction and shear strength, which depend on the presence of lateral confining stresses.

Figure 3-1 illustrates conceptually the way in which a pavement functions under loading.

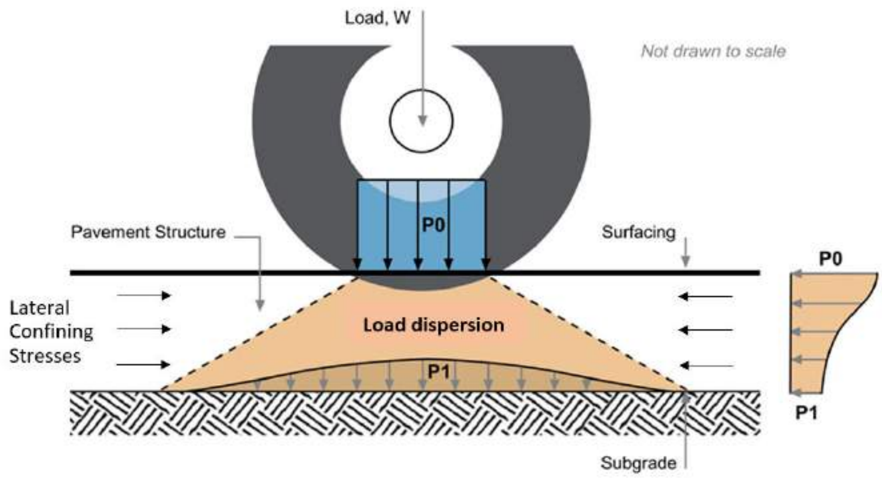


Figure 3-1: Dispersion of surface load through a granular pavement structure

In essence, the wheel load, W, is transmitted to the pavement surface through the tyre. The pavement structure (one or more layers) then spreads the wheel load to the subgrade so that the maximum pressure on it is reduced sufficiently to avoid overstressing it. This can be achieved by the proper selection of pavement materials of appropriate thickness and quality to withstand the design traffic loading.

3.2.3 General design procedure

As with all methods of pavement design, the main requirements of the DPC-DN design procedure are as follows:

- Assessment of design traffic loading.
- Consideration of climatic factors (especially rainfall) and drainage conditions.
- Assessment of the strength of the subgrade or the layers of an existing road prior to improvement or upgrading.
- Determination of pavement layer requirements (thickness and strength).
- Selection of pavement materials.

The above procedure for designing a LVR is presented in more detail in *Chapter 7 - Structural Design: Paved Roads.*

3.3 The DCP-DN Design Method

3.3.1 General

Most pavement design methods for LVRs have been developed empirically by comparing the performance of existing roads with their properties and layer strengths in relation to subgrade support, the environmental and drainage conditions, and the volume and type of traffic. This information usually results in a pavement catalogue or a structural number based on the combined contributions of each pavement layer to the overall bearing capacity of the pavement. The DCP-DN method is no different and, in fact, the Structural Numbers of the DCP-DN catalogue are very similar to other LVR design catalogues. The manner of development of the DCP-DN method is summarised below.

3.3.2 Background to the DCP-DN Method (Paved roads)

Early Development

The original development of the DCP equipment dates back to the mid-1950s in Australia based on an older Swiss model and was used initially as a non-destructive testing device to evaluate the in-situ shear strength of subgrade materials. The use of the DCP for pavement design purposes was developed in the mid-1960s and 1970s in South Africa where results from the back analysis of numerous (more than 1100) road sections with different traffic loadings (0.04 – 20 MESA), materials (calcrete, ferricrete, weathered granite, quartzite, dolerite, diabase, sandstone, etc.) and climatic (annual rainfall 300 mm to > 1600 mm) environments. This led to the development of a structural catalogue based on DN values (penetration in mm/blow, using the conventional 60 degree cone) for a range of design traffic loadings. Following a more detailed investigation into the performance of 57 LVRs throughout South Africa, the findings were compared with the earlier DCP design catalogues and confirmed their applicability and reliability.

During the mid-1980s, new concepts and methodologies were developed, including the concept of pavement strength balance and the development of a unique DCP classification system and a related Standard Balance Classification System (SBCS) and quantification of imbalances.

The DCP method of design has a long history of application in South Africa. Following the development of the original design method, it was used on numerous roads in the country and played a major part in the International Labour Organization (ILO)-managed Gundo Lashu labour-based programme in the Limpopo province in South Africa where many roads were upgraded from gravel to low volume paved standard based on the DCP design method. At that time, although the layer strength requirements were expressed in DN terms, the materials were selected on the basis of their equivalent CBR values because most engineers were not sufficiently familiar with the DN concept.

It should also be noted that the DCP design method (based on the same original criteria) is widely used in southern Africa for the rehabilitation of existing roads (from low volume to national highways) and is an integral part of the SATCC Rehabilitation Guideline.

Software Development

Following the early development and use of the DCP for pavement evaluation in the late 1970s, a simple programme in Fortran was developed, which was subsequently migrated to a Disc Operating System (DOS) for use on laptop computers. This development underwent various iterations until a Windows-based version (WINDCP5.1) was released in 1998, which was essentially for converting field DCP data into layer strength diagrams and comparing it with those required for various traffic classes under various moisture regimes. This was the basis for the development of the AfCAP LVR DCP analysis and design software for low volume roads.

The first version of the AfCAP LVR DCP v 1.03 was produced in mid-2016 as an enhancement of the original WinDCP5.1.

Recent Developments

The DCP design method, as described above, was refined and re-named the DCP-DN design method, which included the following enhancements:

- Minor refinement of the design catalogue to optimise pavement balance.
- Moving away from converting DN to CBR values due to poor correlation and material dependence to avoid the introduction of errors in the conversion.
- Laboratory DCP evaluation of borrow pit materials as structural layers.

Based on extensive use of v1.03 of the AfCAP LVR DCP software, further improvements have become necessary to reflect the latest developments in the DCP-DN design method. This has led to the development of an upgraded version of the software - renamed the ReCAP LVR DCP v 1.0. The examples presented in this Manual are based on this latest version of the software.

LVR Performance History

The outcome of several investigations into the performance of a large number of LVRs in the Sub-Saharan Africa region has produced many significant findings that are relevant to the design approach adopted in the DCP-DN design method. These findings may be summarised, as follows:

- The Structural Numbers (SNs) of the good-performing roads are below the design SNs obtained using most current LVR manuals.
- Most of the pavement failures were caused by failures of the surfacings (high pothole and crack intensities) but are non-structural.
- No structural failures (no terminal conditions reached in the case of rutting) have been observed in the bases or sub-bases or subgrade except in-situations where water ingress into the base occurred after the surfacing had been breached.
- There is almost no correlation between the measured CBR and the traditional soil indicator properties such as Grading Modulus and Plasticity Index (PI).
- There was no significant difference between the Plasticity Modulus (PM) of the bases of sections that are performing well and those that are performing poorly.
- Road bases and sub-bases with PIs and PMs, which were significantly higher than the specification limits on roads with significantly higher traffic loading, also performed well.
- The ranges of particle size distributions of the base and sub-base materials differed widely (both much finer and much coarser) than the traditional specification envelopes; these can be widened to allow the use of very coarse materials.
- In general, sections with low crown heights tended to perform worse than sections with higher crowns (except where coarse or sandy materials were used in the pavement layers).
- Drainage is a significant factor in performance, with the latter being influenced by the crown height, type of shoulders (sealed/unsealed), and whether or not a permeability inversion in the structure exists.
- Given adequate drainage, and based on the micro-climate in the project area, the moisture content in the subgrade tends to equilibrate at or just above OMC, and in the pavement layers to below OMC.

The above findings provide valuable insight into the key factors that affect the performance of LVRs, particularly as regards the manner of selecting materials for use in the pavement structure.

3.3.3 Background to the DCP-DN Method (Unpaved roads)

The method was initially developed in the mid-1980s based on the investigation and monitoring of 110 sections of unpaved roads that were monitored every three weeks for more than three years. The roads were located in various provinces in South Africa as well as in Namibia and covered a wide range of climate (rainfall from 360 to 1500 mm), materials (acid crystalline, basic crystalline, high silica, arenaceous, argillaceous, pedogenic), and traffic (20 – 375 ADT and 2 – 100 heavy vehicles per day).

The pavement design requirements were based on the performance and structural capacities of the 110 sections of unpaved roads, using DCP testing. The resulting design catalogue was incorporated in the *Draft TRH20: Structural Design, Construction and Maintenance of Unpaved Roads* (1990) and implemented for the following 20 years. The design catalogues were subsequently refined in 2010 to provide a better pavement balance, specifically for potential later use of the unpaved road in a structure upgraded to a paved standard, with the minimal replacement of the lower layers. Initially, CBR strengths derived from DCP tests were employed, but these were subsequently revised back to the original DCP DN values and developed into fully balanced layer strength diagrams for various traffic categories of unpaved roads, as described in *Chapter 8 – Structural Design: Gravel Roads.*

3.3.4 DCP-DN Design Philosophy and Concepts

General

This method is based entirely on the use of the DCP device that provides a close approximation of the strength of the soil. The DCP is used for assessing the strength of the subgrade for new roads, existing pavement structures on unpaved gravel and earth roads as well as borrow pit materials. Many readings can be taken at relatively low cost, thus enabling the design engineer to subdivide the road into uniform sections to derive appropriate, environmentally optimized pavement design (EOD) solutions. The DCP can also be used on site during construction to verify that the design requirements have been achieved.

Design philosophy

The philosophy behind the DCP-DN method is to achieve a balanced pavement design whilst also optimizing the utilization of the in-situ material strength as much as possible. This is achieved by:

1) Determining the design strength profile needed for the expected traffic, and
2) Integrating the in-situ strength profile with the required strength profile.

To utilise the strength of the existing gravel or earth road, the materials in the pavement structure need to be tested for their actual in-situ strength, using a DCP. This device has been designed to provide a rapid, relatively low-cost, non-destructive method of estimating the in-situ strength of fine-grained and granular subgrades, base and subbase materials, and weakly cemented materials.

Design concepts

DCP Number (DN): The DCP measures the penetration per blow into a pavement through each of the different pavement layers. This rate of penetration in mm/blow (the DN value) is a function of the in-situ shear strength of the material at the in-situ moisture content and density of the pavement layers at the time of testing. However, the pavement design requires an estimate of the values of strength (DN) that would be obtained under the anticipated long-term moisture conditions. Through the Laboratory DN test, the designer can determine the strength of the materials at the anticipated field density and long-term moisture conditions. The test also provides a measure of the sensitivity of the materials to moisture and density variations and gives the designer a good basis for a realistic estimate of the material strength for design purposes.

DCP Structure Number (DSN): This is the number of DCP blows required to penetrate a pavement structure or layer to a specified depth. This DSN value allows the bearing capacity of different pavements to be compared. Accordingly, the DSN_{800} is the number of blows required to penetrate the pavement to a depth of 800 mm. However, for LVRs, the DSN_{450} is also determined as it represents the bearing capacity of the pavement to a depth of 450 mm, below which the traffic stresses are negligible.

Layer-strength diagram: Each DCP test provides a profile through the depth of the pavement, which gives an indication of the in-situ strength properties (related to other proxies for strength such as grading and plasticity) of the materials in all the pavement layers down to the depth of penetration of 800 mm.

A schematic of the DCP is shown in Figure 3-2, while a typical in-situ strength profile is shown in Figure 3-3.

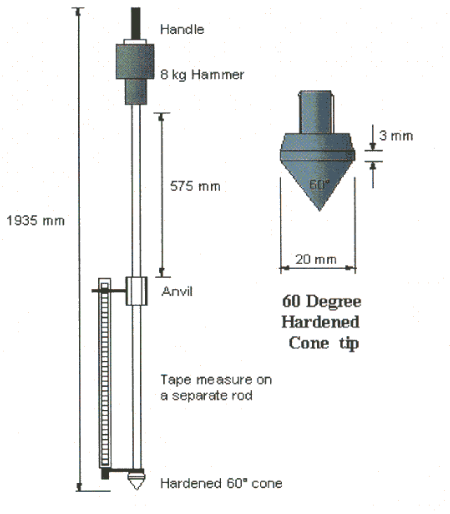


Figure 3-2: The DCP device

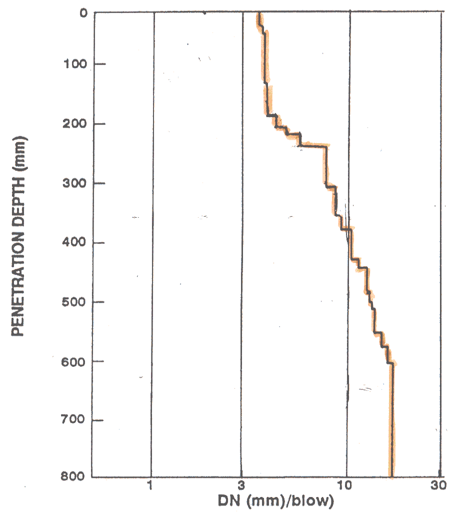


Figure 3-3: Layer strength profile

Pavement strength/balance: This is a fundamental feature of the DCP-DN method in which the strength balance of a pavement structure is defined as the change in the strength of the pavement layer with depth. A well-balanced pavement structure is one in which the strength of the pavement layers decreases progressively and smoothly with depth from the surface without any discontinuities. Although the layer strength diagram indicates steps between the different layers, research into pavement has shown that re-moulding of the materials occurs under traffic to "iron out" these steps.

From a knowledge of the DN values of various pavement layers, those of relatively high and relatively low strength can be distinguished from each other, and the balance of the pavement at any depth can be evaluated. This has led to the development of a pavement classification system in which shallow, deep and inverted pavements can be distinguished from each other and further differentiated in terms of whether they are well-balanced, averagely balanced, or poorly balanced.

The more the final bearing capacity is derived from the upper pavement layers (base and subbase) relative to the lower layers, the "shallower" the pavement structure. In contrast, the more the lower layers (subgrade) contribute to the final bearing capacity relative to the upper layers, the "deeper" the pavement structure. The manner of determining the strength-balance of a pavement structure is presented in Appendix 3.1.

Assessment of subgrade/pavement layer strength: Understanding what influences the performance of a LVR pavement, and how the performance can be predicted and controlled is the key to the use of fit-for-purpose materials. The strength of a material is determined by its basic properties (grading, plasticity, aggregate hardness, etc.). However, the strength of the material is also influenced by the operating conditions in the pavement and will vary with moisture content and compacted density. Therefore, to fully understand how a material is expected to perform under a specific design scenario, and ultimately how fit for a particular purpose it will be, an assessment process is required to determine the risk associated with the design assumptions.

The material's fundamental properties will remain unchanged unless they are modified, for example, by some form of stabilisation, thereby creating a changed or modified material. Given the basic properties, it is possible, however, to examine how the strength varies with different combinations of moisture content and density. A good understanding of how these interacting variables affect material strength is essential for assessing the adequacy of a material for a specific design scenario. Figure 3-4 shows a typical output of the materials assessment process.

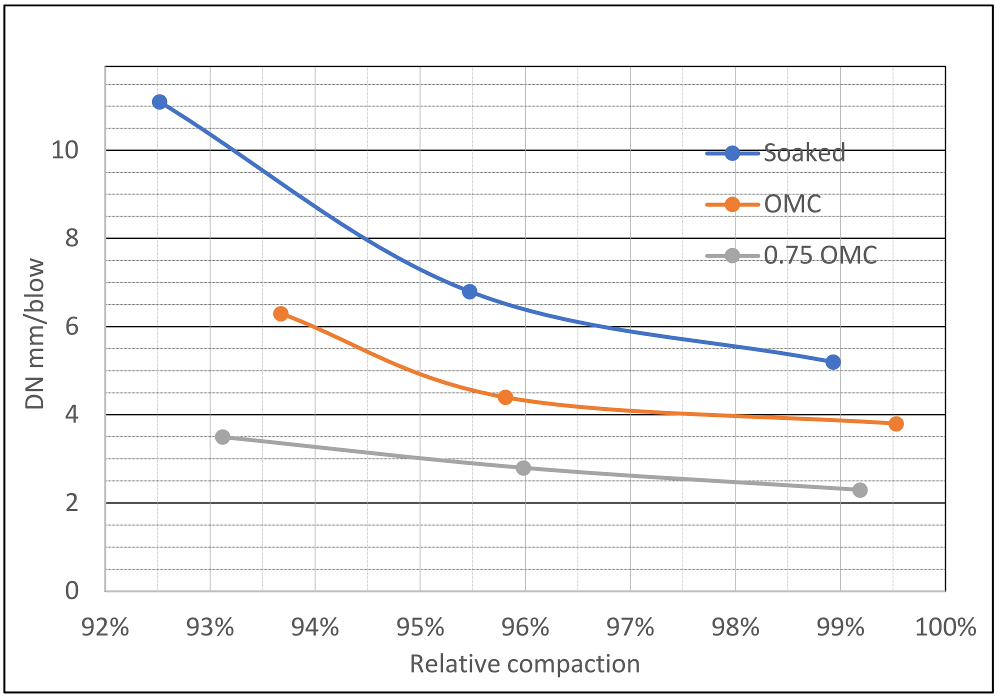


Figure 3-4: DN/density/moisture relationship

Figure 3-4 illustrates how a material's strength, as measured by the laboratory DN value, varies with changes in moisture and density. The gradient of the curves and the separation between them indicate the following about the particular material:

- The steeper the slope of the lines, the greater the sensitivity of the material's strength to changes in density (a function of particle size distribution); and
- The greater the separation of the lines, the greater the sensitivity of the material's strength to changes in moisture (a function of plasticity).

It follows, therefore, that:

- An acceptable DN value (based on the design assumptions) represents a composite measure of the key interacting variables that affect material strength, and
- Acceptable grading and plasticity requirements are implicitly controlled by an acceptable DN value and need not be separately specified.

From the output of the materials assessment process illustrated in Figure 3-4, the following essential design considerations will ensure optimum use of the material:

- Achieving the highest practicable level of density, i.e., without degrading the material (so-called "compaction to refusal"), by employing the heaviest rollers available (type, mass, vibration amplitude and frequency). This will result in a stiffer material with a lower voids content and a reduced permeability, thereby enhancing the overall properties and performance of the material.
- Adopting appropriate measures to keep the subgrade and pavement layer materials as dry as possible in service. This can be achieved by the provision of adequate drainage, both external and internal, as discussed in *Chapter 7 – Structural Design: Paved Roads.*

The information presented in Figure 3-4 provides the designer with the required evidence to ascertain under which moisture and density conditions the material will satisfy the design DN requirement. Details of the various tests required to select the subgrade and pavement layer materials are described in *Chapter 4 – Surveys and Investigations.*

3.3.5 Strengths and Limitations of the DCP Device

Strengths

The main strengths of the DCP device are as follows:

- Relatively low cost, robust apparatus that is quick and simple to use allowing comprehensive characterization of the in-situ road conditions.
- Provides improved precision limits compared to the CBR test.
- Very little damage is done to the pavement being tested (effectively non-destructive), and very useful information is obtained.
- The pavement is tested in the condition at which it performs, and the test can be carried out in an identical manner both in the field and in the laboratory.
- The simplicity of the test allows repeated testing to minimise errors and also to account for temporal effects.
- The laboratory DN value is determined over the full depth of the sample and not just the top 25 – 75 mm as with the CBR test.
- The DCP is as good or better than any other device in taking into account variations in moisture content and provides data quickly for analysis.

Limitations

The main limitations that are likely to affect the results and their interpretation and that need to be considered when using the DCP design method include:

- Use in very coarse granular materials.
- Very hard cemented layers in the pavement structure
- The possibility of not recording very weak or thin layers when taking depth measurements every 5 blows unless variable blows are used.
- Poorly executed tests (hammer not falling the full distance, non-vertical DCP, excessive movement of the depth measuring rod, etc.).
- Use in areas of significant cut and/or fill (> 1.5 m), including widening in similar situations.

Many of the above limitations are controllable if taken into account when using the DCP. Ultimately, the onus remains on the designer to understand the environment and implications of each test in relation to the in-situ state of the material. This includes aspects such as material composition, presence of large stones or hard layers, moisture content, density, etc. Sound engineering judgment and understanding, as well as knowledge of the specific site, are necessary to maximize the information that can be obtained from a DCP profile. Thus, unless the field conditions are fully comprehended, the design engineer may draw erroneous conclusions or wrongly extrapolate data provided by site teams.

3.3.6 Applicability of the DCP-DN Method

General

A clear distinction should be made between the application of the DCP-DN design method and the use of the DCP device, as described above. The DCP-DN pavement design method is essentially about applying the DCP-DN design catalogue for pavement design and is not linked intrinsically to the use of the DCP device to undertake measurements as part of a survey along a road alignment. Situations may be encountered in practice where it is not possible to use the DCP to obtain the in situ strength of the existing pavement structure. In such cases, as described in *Section 7.3 – Application in Practice,* the material properties and design strength can be obtained from bulk samples taken at the level of the finished roadbed.

Environment of development

As with any empirically designed method, the DCP-DN design method and related design catalogue were developed within a broad range of environmental conditions as summarised below:

- **Climate:** Annual rainfall 300 mm to > 1600 mm
- **Materials types**: calcrete, ferricrete, weathered granite, quartzite, dolerite, diabase, sandstone, etc.
- **Cumulative traffic loading**: 0.04 – 20 MESA (the majority of roads carried relatively light traffic (< 500 vpd).

The failure criterion adopted in the development of the design catalogue was a 20 mm rut depth measured under a 2 m straight edge. This figure is now generally considered to be unduly stringent, and, based on the outcome of recent back-analysis projects, a less conservative figure of 30 mm is now suggested, which implies that the catalogues are on the conservative side.

Applicability to other environments

In principle, from a scientific perspective, the DCP-DN catalogue, like any other empirically developed LVR catalogue, is valid for application only to environments that fall within the range of data of its development. However, the boundaries of this range are seldom precise, and a designer may be required to consider designs for conditions that are not fully within the range of all of the data presented above. In such a situation, the number of data variables, and the extent of their departure from the original data range, need to be considered carefully, and sound engineering judgement will be required to make a sensible design decision. Based on these principles, four scenarios may be considered, as follows:

Scenario 1 – Full application: This applies to environments similar to the environment of development of the original DCP-DN design method.

Scenario 2 – Qualified application (low risk): This applies to environments where one variable, e.g., rainfall, may be outside the range of the original data. In such a situation, there would be a very low risk of using the design method if the design moisture contents were properly assessed and other supplementary drainage measures addressed in the road design.

Scenario 3 – Qualified application (higher risk): This applies to environments where a number of variables, e.g., climate, material and traffic loading, may all fall outside the range of the original data. In such a situation, there would be a higher risk than Scenario 2 of using the design method. Thus, it would be prudent to proceed with caution by, for example, first undertaking further empirical studies through closely monitored and evaluated trial sections. It is also useful to compare the results of a DCP design in such cases with the pavement structure developed by some other method for a reasonable fit.

Scenario 4 – No application: This applies to environments where very different data variables occur, for example, in temperate climates with very different climatic, material and traffic loading environments.

3.3.7 Further Research Work

There is always scope for the continuing refinement of any design method, the DCP-DN method being no exception. In this regard, the following topics would warrant further research work:

- The precision limits of the DCP test.
- The effect of confinement on the laboratory DN test.
- A review of the DN relationship with fundamental design principles and materials properties.
- A more appropriate definition of failure as it applies to LVRs.

Bibliography

AfCAP (2013). ***Design Manual for Low Volume Sealed Roads Using the DCP Design Method***. Ministry of Transport and Public Works, Malawi.

Ayers M E, Thompson M R and D R Uzarski (1989). ***Rapid Shear Strength Evaluation of In-situ Granular Materials***. Journal of the Transportation Research Board, 1227, 134-146.

Beavis H M (1984). ***Observations on sections of rural road at three sites in Western Australia and South Australia, 1967 to 1982***, Australian Road Research Board, Vol. 14, 1984.

De Beer, M (1988). ***Towards a Classification System for the Strength-Balance of Thin-Surfaced Flexible pavements.*** Proc. Annual Transport Convention, Vol. 3D, CSIR, Pretoria, South Africa

Emery S J (1985). ***Prediction of Moisture Content for Use in Pavement Design***. Ph.D. Thesis, University of Witwatersrand, Johannesburg, South Africa.

George K P and W Uddin (2000). ***Subgrade Characterisation for Highway Pavement Design,*** University of Mississippi, Mississippi, USA.

Gourley C S and PAK Greening (1999). ***Performance of low volume sealed roads: results and recommendations from studies in Southern Africa (Vol. 1).*** TRL Project Report PR/OSC/167/99. Transport Research Laboratory, Crowthorne RG45 6AU. UK.

Gourley C S and PAK Greening (1999). ***Proceedings of Four Country Workshops in Southern Africa: Outcomes and Indicative Actions (Vol. 5).*** TRL Project Report PR/OSC/171/99. Transport Research Laboratory, Crowthorne RG45 6AU. UK.

Harison J A (1989). ***Correlation between California Bearing Ratio and Dynamic Cone Penetrometer Strength Measurements of Soils***. Australian Road Research, **19**, pp 130-136.

Hassan A B (1996). ***The Effects of Materials Testing Parameters on Dynamic Cone Penetrometer Results for Fine-Grained Soils and Granular Materials***, Ph.D. Thesis, Oklahoma State University, Oklahoma, USA.

Jacobsz S W and P Day (2008). ***Are we getting what we paid for from commercial testing laboratories?*** Civil Engineering Magazine, South African Civil Engineering Society.

Livneh M (2007). ***Uncertainty Associated with Pre-Defined Correlative Expressions of Various In-situ Test Outputs.*** FAA Worldwide Airport Technology Transfer Conference, Atlantic City, USA.

Otto A, Rolt J, Musenero L and K Mukura (2019). ***Development of Guidelines and Specifications for Low Volume Sealed Roads through Back Analysis***, Phase 3 Final Report, RAF2069A. London: ReCAP for DFID.

Paige-Green, P (1994). ***Recommendations on the use of marginal base course materials in low volume roads in South Africa.*** Pretoria. Department of Transport, Research Report RR 91/201.

Paige-Green P and G Van Zyl (2019). ***A Review of the DCP-DN Pavement design Method for Low Volume Sealed Roads:* Development and Applications**. Journal of Transportation Technologies, 9, pp 397-422.

Pinard M I, van Zyl G D and J Hongve (2019). ***Evaluation of Cost-Effectiveness and Value-for-Money of the DCP-DN Pavement Design method for Low Volume Roads in Comparison with Conventional Designs.*** ReCAP Project RAF2128A.

Rallings R (2014). ***The CBR Test – A Case for Change?*** Australian Geomechanics Vol 49 No 1.

Rolt J, Mukura K, Dangare F and A Otto (2013). ***Back Analysis of Previous Low Volume Rural Roads in Mozambique.*** AFCAP/MOZ/001/G. TRL Final report RRN 2588.

Rolt J and M I Pinard. ***Designing low-volume roads using the dynamic cone penetrometer.*** Proceedings of the UK Institution of Civil Engineers, Transport 169, June 2016, Issue TR3.

Smith R B and D N Pratt (1983). ***A field study of in-situ California Bearing Ratio and dynamic cone penetrometer testing for road subgrade investigations.*** Australian Road Research, 13, No. 4, 1983, 285-94. ARRB, Australia.

Appendix 3.1 – Pavement Strength Balance

General

The strength balance of a pavement structure is defined as the change in the strength of the pavement layers with depth. If the strength decrease is smooth and without discontinuities, the pavement can be regarded as being in a state of balance.

The pavement balance at any depth can be determined from the following formula:

$$DSN(\%) = D[400B + (100 - B)^2]/[4BD + (100 - B)^2]$$

Where: DSN(%) = pavement structure number (%)
B = parameter defining the standard pavement balance curve (SPBC)
D = pavement depth (%)

The above formula allows a series of curves to be developed for different pavement structure numbers and depths. These can be plotted as Standard Pavement Balance Curves (SPBC) as shown in Figure 3-5.

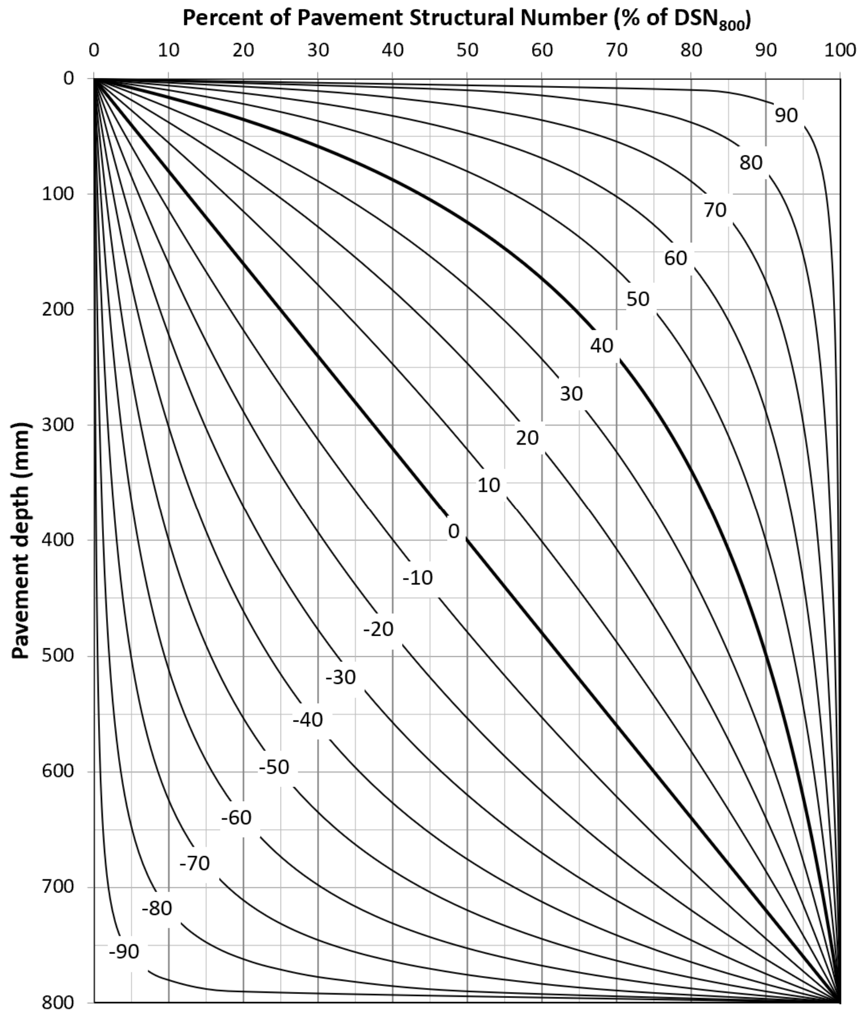


Figure 3-5: Standard Pavement Balance Curves (SPBC)

The value of B allows characterisation of the pavement structure, as follows:

-90 < B < 0: Inverted pavement (strength increasing with depth)

0 ≤ B < 40: Deep pavement (gradually decreasing strength with depth)

40 ≤ B ≤ 90: Shallow pavement (very strong in the upper layer(s) and rapidly decreasing strength with depth)

Balance number

The number of blows of the DCP required to reach a certain depth for a balanced pavement expressed as a percentage of the DCP Structural Number (DSN_{800}) is defined as the Balance Number (BN) at that depth, as illustrated in Figure 3-6. For example, the BN_{100} in the figure is the number of blows as a percentage of the DSN_{800} required to penetrate to a depth of 100 mm.

Figure 3-6 illustrates the BN_{100} for typical natural gravel and lightly cemented pavements in the Southern African region:

A: BN_{100} = 12.5% for a SPBC with B = 0

B: BN_{100} = 80% for a SPBC with B ≈ 68

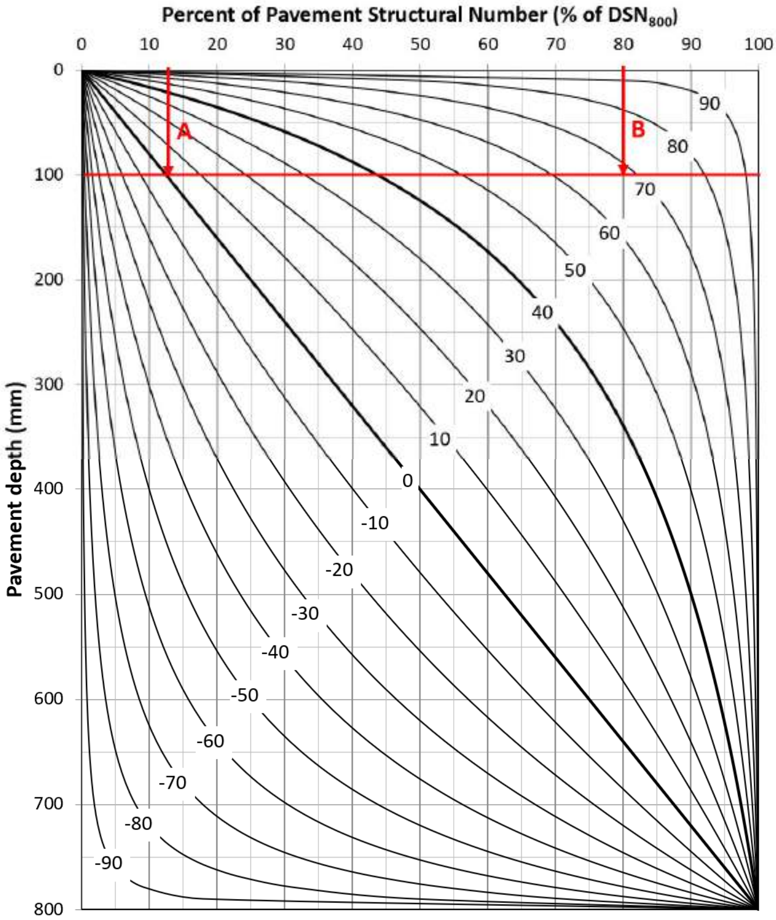


Figure 3-6: Graphic illustration of the pavement Balance Number (BN)

The higher the BN_{100} value, the greater is the contribution to overall strength from the top 100 mm of the pavement. Such pavements are considered to be "shallow" (analogous to "glass on feathers") and tend to be composed of one strong, relatively stiff, upper layer with rapidly diminishing support at depth from the underlying material. In contrast, the lower the BN_{100} value, the greater the contribution to overall strength from the lower pavement layers. Such pavements are considered to be "deep" and tend to be composed of a number of relatively less rigid layers of relatively equal strength, affording generous support at depth.

In the development of the DCP-DN design method, it was established that optimum balance would be achieved for a BN_{100} = 38%, which corresponds to the Standard Pavement Balance Curve (SPBC) for B=35. On this basis, the standard DCP-DN design catalogue presented in Chapter 6 was developed.

Alternative design catalogues

Using the pavement balance concept, as explained above, it is possible to develop alternative design catalogues for use in particular design situations. For example, in areas with strong subgrades, a lower

BN100, say BN100 = 24%, corresponding to the SPBC for B = 20, facilitates a wider use of naturally occurring materials without having to resort to modification or stabilisation, while providing the required total bearing capacity (DSN800), as in the standard catalogue.

It is also possible to develop catalogues for different pavement layer configurations, e.g., for labour-based projects where thinner lifts for subbase and base may be warranted.

An alternative DCP-DN catalogue for BN100 = 24%, as well as a detailed explanation of how to develop new catalogues for different pavement layer configurations, is provided in *Chapter 6: Appendix 6.1.*

Pavement Classification

The pavement strength balance curves have been used to develop a pavement classification system in which any pavement is classified in terms of the Balance Curve (B), which is the balance curve of the pavement and the deviation (A) between the SPBC and the measured balance curve which represents a "goodness of fit" parameter for the pavement. This deviation from an SPBC represents the state of imbalance in the structure.

By way of example, Figure 3-7 illustrates a well-balanced pavement structure, while Figure 3-8 illustrates a poorly-balanced structure in which the imbalance is indicated by the deviation of the pavement balance curve from the SPBC.

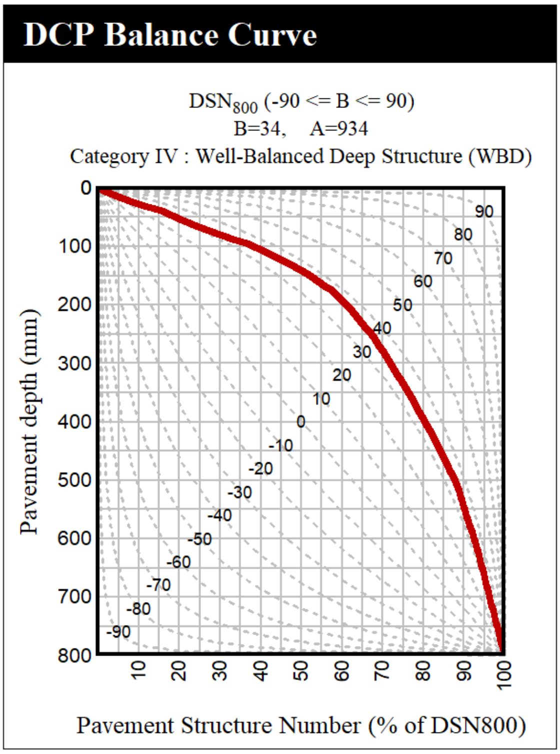


Figure 3-7: Well-balanced structure

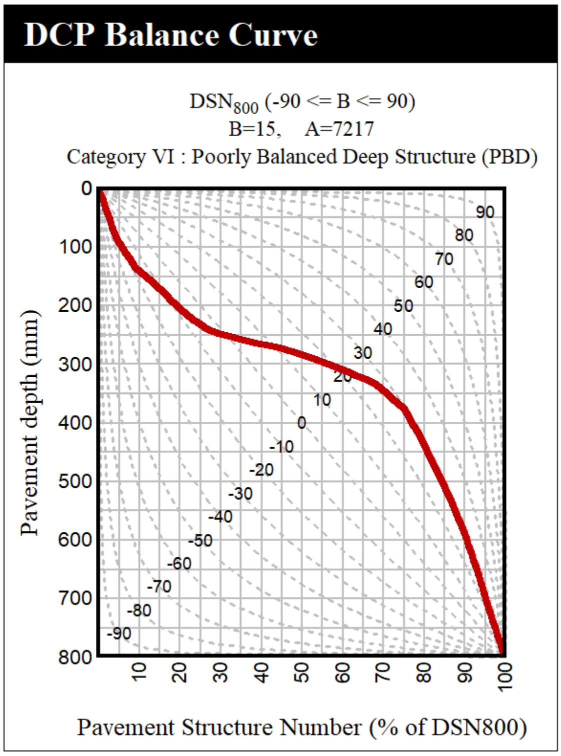


Figure 3-8: Poorly-balanced structure

The A and B parameter limits for defining the different categories in the pavement classification system are summarised below:

Shallow pavements	40 ≤ B < 90 (BN ≥ 42%)
Deep pavements	0 ≤ B < 40 (12.5% ≤ BN < 42%)
Inverted pavements	-90 ≤ B < 0 (BN < 12.5%)
Well balanced	0 ≤ A ≤ 1200
Averagely balanced	1200 < A ≤ 3000
Poorly balanced	A > 3000

Each cell in the classification system is defined by an A and a B descriptor, resulting in a possible 9 classification categories, as shown in Table 3-1 .

Table 3-1: Pavement balance classification matrix

B < 0 A > 3000 Poorly Balanced Inverted (PBI)	0 ≤ B < 40 A > 3000 Poorly Balanced Deep (PBD)	B ≥ 40 A > 3000 Poorly Balanced Shallow (PBS)
B < 0 1200 < A ≤ 3000 Averagely Balance Inverted (ABI)	0 ≤ B < 40 1200 < A ≤ 3000 Averagely Balanced Deep (ABD)	B ≥ 40 1200 < A ≤ 3000 Averagely Balanced Shallow (ABS)
B < 0 0 ≤ A ≤ 1200 Well Balanced Inverted (WBI)	0 ≤ B < 40 0 ≤ A ≤ 1200 Well Balanced Deep (WBD)	B ≥ 40 0 ≤ A ≤ 1200 Well Balanced Shallow (WBS)

Power exponent 'n'

In the DCP design method, the value of "n", which is used for estimation of Vehicle Equivalence Factors (VEF), has been found to be affected by the composition of the pavement structure in terms of its "balance" which influences the load sensitivity of the pavement.

Moreover, the exponent "n" is related to the Pavement Balance Number, BN_{100}, (the number of blows to penetrate the top 12.5% of the pavement as a percentage of the number required to penetrate 800 mm, as explained above), through the derived model which is illustrated in Figure 3-9.

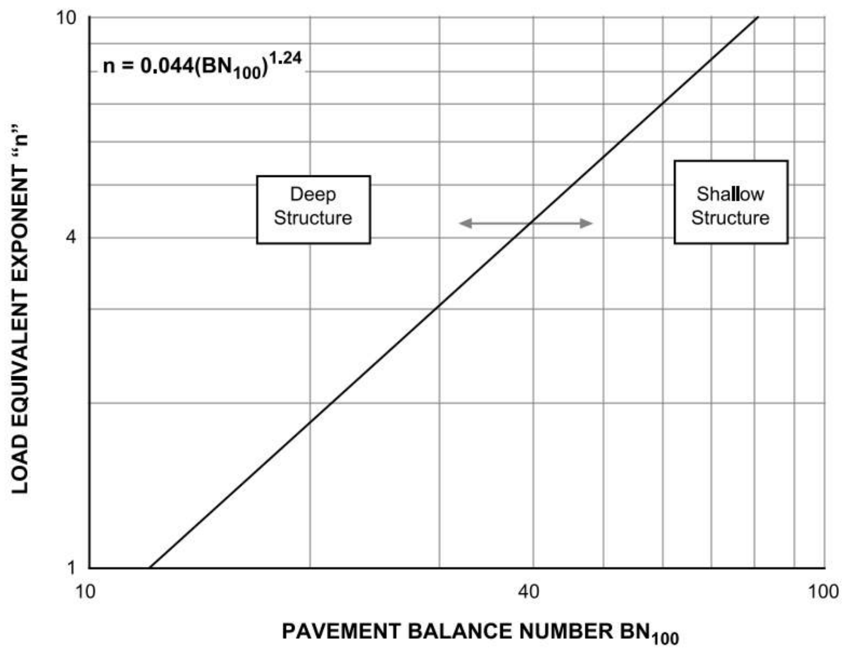


Figure 3-9: Relationship between BN100 and power exponent 'n'

This relationship may be used to re-calculate the design traffic loading for pavements with BN_{100} < 40. However, it is not recommended to use 'n' values < 3. It was also found that shallower pavements are more susceptible to overloading and that higher 'n' values than the commonly applied values of 4.0 to 4.5 may be warranted.

Appendix 3.2 – Generic Table of Contents for a Design Report

The design report that should be compiled after completing a pavement design for the upgrading of an existing unpaved road to a paved standard – a common situation in most countries - should typically contain the following topics, some of which are not addressed in this Manual:

1. **Title page**
2. **Table of Contents**
3. **Introduction**
    a. Background
    b. Project description
    c. Location map
    d. Purpose and scope of report
4. **Physical Environment**
    a. Topography/geology/hydrology
    b. Soils/vegetation
    c. Climate
5. **Site Investigations**
    a. Topographic surveys
    b. Visual assessment
        i. Road condition
        ii. Drainage condition
    c. Road inventory and strip map
6. **Soils and Materials Investigations**
    a. Centreline surveys
        i. Determination of uniform sections
    b. Subgrade evaluation
        i. In-situ materials sampling and testing
    c. Borrow pit investigations
        i. Pavement materials sampling and testing
    d. Construction water
7. **Traffic Surveys and Analysis**
    a. Traffic counts and axle load surveys
    b. Design traffic loading
8. **Road Geometry**
    a. Horizontal alignment
    b. Vertical alignment
    c. Cross section design
9. **Road Safety**
    a. Road safety audit
    b. Traffic calming measures
    c. Traffic signs and markings
    d. Other (roadside environment, traffic segregation, road furniture, etc.)
10. **Pavement Design**
    a. Structural design
    b. Surfacing options and selection
    c. Engineering adaptions to climate change
    d. Life-cycle cost analysis
        i. Comparison of designs
        ii. Selection of preferred option
11. **Hydrology and Drainage**
    a. Hydrological analysis
    b. Drainage structure requirements and design
    c. External and Internal road drainage
    d. Erosion control measures
12. **Cost Estimate**
    a. Bill of Quantities
    b. Engineer's cost estimate

4. Surveys and Investigations

4.1 Introduction

4.1.1 Background

The physical environment exerts a significant influence on the design and performance of a LVR. For example, the subgrade soils along the alignment of the road are a primary determinant of the requirements of the pavement structure. In addition, drainage design is dependent on climatic factors such as rainfall intensity and duration, while binder selection for bituminous surfacings is influenced by the prevailing ambient temperatures. For these reasons, it is essential for the designer to have a comprehensive understanding of the various factors that make up the physical environment.

The designer also requires information on the characteristics of the existing road as well as the traffic volume and composition before its upgrading requirements can be established. This information can be obtained from appropriately staged site investigations aimed at collecting and compiling information on the road's characteristics and surrounding environment as inputs for subsequent decision making and design.

4.1.2 Purpose and Scope

The purpose of this chapter is to:

- Highlight the various features of the physical environment that could affect the design of a LVR.
- Outline the scope of the required site investigations.
- Describe the various surveys that are required to collect and process the information required for the actual pavement design.

4.2 Characteristics of Physical Environment

4.2.1 General

Information on the following characteristics of the physical environment is required for design purposes:

- Climate (current and projected future)
- Topography
- Geology
- Hydrology
- Vegetation
- Soils

The significance of the above characteristics of the physical environment and their impact on design are discussed briefly below.

4.2.2 Climate

The climate of a country, primarily in terms of its rainfall and temperature, can exert a considerable influence on both the design and performance of a LVR. For example, rainfall influences the supply and movement of water and impacts upon the road in terms of direct erosion through runoff.

Climatic indices have a significant influence on the selection of pavement options and their design for "wet" or "dry" conditions. Unpaved surface performance is particularly influenced by the quantity and intensity of rainfall, and the manner of dealing with runoff.

Table 4-1 provides general guidance for selecting the appropriate climatic zone for pavement design purposes. If information about the moisture indices is lacking, the mean annual rainfall in the project area can be used as a proxy.

Table 4-1: Guide for selection of climatic zone

Description		Weinert N value	Thornthwaite Moisture Index[1] (I_m)	Typical Mean Annual Rainfall (mm)
Arid	Dry	5+	< - 40	< 250
Semi-arid		4-5	- 20 to - 40	250 – 500
Semi-arid to Sub-tropical	Moderate	2-4	- 20 to + 20	500 – 1000
Humid tropical	Wet	< 2	+ 20 to + 100	> 1000

Weinert N value = (12 x Evaporation in warmest month)/annual precipitation (Weinert, 1980)
Thornthwaite I_m = (100 x water surplus – 60 x water deficiency)/water need (http://glossary.ametsoc.org/wiki/Moisture_index)

It is also important that an indication of projected future climate changes, particularly precipitation, is obtained as this would certainly influence the runoff volumes and velocities for different return periods used for the design of water crossing structures.

4.2.3 Topography

The topography reflects the geological and geomorphological history of a region. Apart from its influence on the geometry (grade and alignment) of the road and earthwork requirements, the characteristics of the terrain also reflect and influence the availability of materials and resources and potential slope instability problems.

4.2.4 Geology

The rock types (lithologies) beneath the surficial soil cover can be used to get a preliminary indication of the type of residual material that would form from the underlying rock. For example, residual materials derived from granites and quartzites are usually gravelly with low plasticity compared with those derived from basic volcanic rocks, which normally have higher clay contents with high plasticity.

4.2.5 Hydrology and drainage

The movement of water within and adjacent to the road structure has a major impact on the performance of the road pavement and surfacing. Seasonal moisture variations influence pavement behaviour adjacent to unsealed shoulders. Changes in subgrade moisture conditions are the trigger for significant subgrade and earthwork volume changes in pavements, particularly those underlain by “expansive” clay materials, collapsible sands and dispersive soils.

4.2.6 Vegetation

Knowledge of the type of vegetation might provide a clue as to the type of soil and how the parent geology of an area interacts with rainfall to produce certain types of soils.

4.2.7 Soils

The nature and engineering properties of the subgrade and pavement materials are key aspects of the physical environment. For LVRs, where the use of local materials is a priority, the fundamental issue should be to ensure that the materials are “fit for purpose” in the environment in which they are being used. Thus, the design options should be compatible with the available materials rather than attempting to find materials that meet pre-defined standard specifications, as is the case with higher-level roads. Specifications need to be appropriate to the local environment and locally available materials.

[1] A moisture index which relates water demand by plants to available precipitation, by means of potential evapotranspiration, calculated from measurements of air temperature and day length. In humid regions the index is positive and in arid regions it is negative.

4.3 Site Investigations

4.3.1 General

The choice of methods for site investigation is determined by the type of road project, practical problems arising from site conditions, terrain, and climate. Only techniques appropriate for LVRs are described in this chapter.

4.3.2 Types and Scope of Site Investigations

In general, site investigations will be undertaken in two main phases:

- Desk Study and Initial Investigations that will identify the main issues; and
- Detailed Investigations that will provide all the information needed for design and identification of areas requiring specialised input.

For LVRs, investigations should employ standard and simple engineering methods. These include visual inspection and description of test pits along the proposed alignment, use of Dynamic Cone Penetrometer (DCP) testing to identify uniform sections, and use of appropriate material tests to assess the in-situ soils and borrow materials for incorporation in the pavement. Complex and expensive procedures should only be employed when a significant geotechnical problem is encountered or suspected. Under such circumstances, it is advisable to seek specialist assistance.

It is the responsibility of the design engineer to determine the frequency and type of test necessary for the specific road project and to assess when and how samples should be taken for laboratory testing in accordance with the appropriate standard. Guidance on this is provided in *Chapter 5 – Materials.*

New roads

Totally new roads are uncommon but may be encountered periodically. Short sections of existing roads, however, may need to be re-aligned for various reasons. For such "greenfield" sections, more emphasis should be placed on the investigation of the subgrade conditions as there will be little visual data to collect in terms of inventory and condition. The techniques that will be used will, however, be the same as for existing roads.

Existing roads

Where a road already exists and needs to be upgraded to a higher class, for example, from a gravel road to a paved road, some minor realignments may be warranted. For such a situation, standard inventory and condition data collection, in addition to subgrade materials testing, as described in Section 4.4, are essential to inform the design as described in *Chapter 7 – Structural Design: Paved Roads* and *Chapter 8 – Structural Design: Gravel Roads.* Existing drainage deficiencies must be noted for particular attention.

4.3.3 Desk Study and Initial Investigations

The requirements of the desk study and initial investigations are summarised in Table 4-2. The outcome of these activities will provide the basis for planning the more detailed site investigations discussed in Section 4.3.4.

Table 4-2: Summary of requirements for Desk Study and Initial Investigations

Stage of design	Study	Comments
Desk study	Engineering	Normally a road or track is in existence already. Major engineering problems identified.
	Social	The need for the road improvement would have been based on the current planning process at the regional or local level. Social assessment is based on desk study information and concentrated on major issues such as land take and resettlement and potential economic development, if any.
	Environment	Assessment based on a desk study and scoping of any EIA required to comply with national regulations.information including issues such as land take, reinstatement, the existence of any conservation areas, climate, rainfall, vegetation, geology, etc.
	Cost estimation	Historic data only. Based principally on terrain and number of structures. Accurate to only ±30%.
Site Visit (General)	Consultation with local people	Social and economic issues. Flooding (high water levels) and stream flows, adequacy of culverts, accident locations, weak sections of road that are impassable for part of the year, the nature of impassability, availability of materials, etc.
	Engineering	The following aspects should be addressed: • Confirm information obtained from consultation with local people. • Assessment of defects visible on the road surface. • Assessment of geometric characteristics. • Assessment of road drainage, stream and river crossings and extent of flooding of water crossings and low-lying areas. • Location of all possible bridge sites, water crossings and all culvert locations. • Slope stability and potential landslide problems. • Other possible major hazard areas such as poorly drained soils, problem soils, springs, and erosion in river courses. • Extent of erosion problems with road drainage requirements. • Possible sources of water for construction. • Possible sources of construction materials. • Assessment of land acquisition/site clearance problems. • Traffic and its potential growth.
	Environment	All environmental aspects are important. However, the extent to which they can or must be addressed on LVR projects varies. Particular attention must be paid to borrow and spoil areas and likely changes in drainage patterns plus possible effects of the road on biodiversity and ecology.
	Cost estimation	Largely based on historical records but now supplemented with more detail about the scale (and therefore likely cost) of the pavement and structures.

4.3.4 Detailed Site Investigations

General

Depending on the scope of the project, detailed site investigations could be broken down into a feasibility stage, which, upon approval, would move forward to a detailed design stage. For an LVR project, these activities may be combined in one site investigation. The sections below describe the activities to be carried out.

Feasibility and Detailed Design Study

A feasibility and detailed design study typically comprise the following activities:

a) Structural assessment

The site investigation should establish the condition of the existing pavement structure or the subgrade on new alignments in order to maximise its use for the new pavement structure so that it can carry the expected future traffic. An efficient and inexpensive way to examine the structural properties of each layer is with the Dynamic Cone Penetrometer (DCP), as described in Section 4.4.3 below.

b) Drainage and erosion

A thorough assessment of the existing road drainage system is necessary, including the following:

- Culverts:
    - Adequacy of opening (size, flooding, length of culvert)
    - Inlet and outlet conditions (ponding, silting, erosion, headwalls)
    - Structural strength (condition of concrete or other materials)
- Low-level structures (causeways, drifts, etc.) and bridges:
    - Flood levels and time of closure
    - Adequacy of the existing structure to cope with floods
    - Structural condition
    - Width
    - Erosion
- Bridges (if any)
- Surface drainage:
    - Standing water next to the road and in the side drains
- Subsurface drainage:
    - Seepage from higher ground, springs, high water tables, etc.
- Drainage channels:
    - Adequacy of side drains (shape of the drain, ponding, silting, scour, erosion)
    - Catchwater drains and cut-off drains (shape of drain, ponding, silting, erosion)
    - Mitre drains (frequency, the shape of the drain, ponding, silting, erosion, outlet efficiency and water dispersion)
- Down chutes (condition, erosion)

Erosion is closely related to drainage and depends on soil type, grade, climate and site conditions. A general assessment of erosion potential is needed for embankments, cuttings, road reserve and borrow areas, leading to the design of erosion control measures where necessary.

c) Materials assessment and laboratory testing

Samples of the base material and, if necessary, the support layers in each uniform section must be tested in the laboratory to provide information to aid construction and to ensure that the materials meet the relevant physical requirements.

4.4 Site Surveys

4.4.1 General

Various surveys are required to collect and process the information required for pavement design and include the following:

- Traffic
- Route survey (DCP survey, test pit and sampling, moisture and drainage, problem soils)
- Construction materials survey (in-situ materials, borrow pit materials, aggregate sources)
- Materials testing

Each of the above is critical to the final pavement design and must be carried out accurately and comprehensively to ensure the most cost-effective pavement structure is designed and to minimize the potential for construction delays due to unforeseen circumstances.

4.4.2 Traffic Surveys

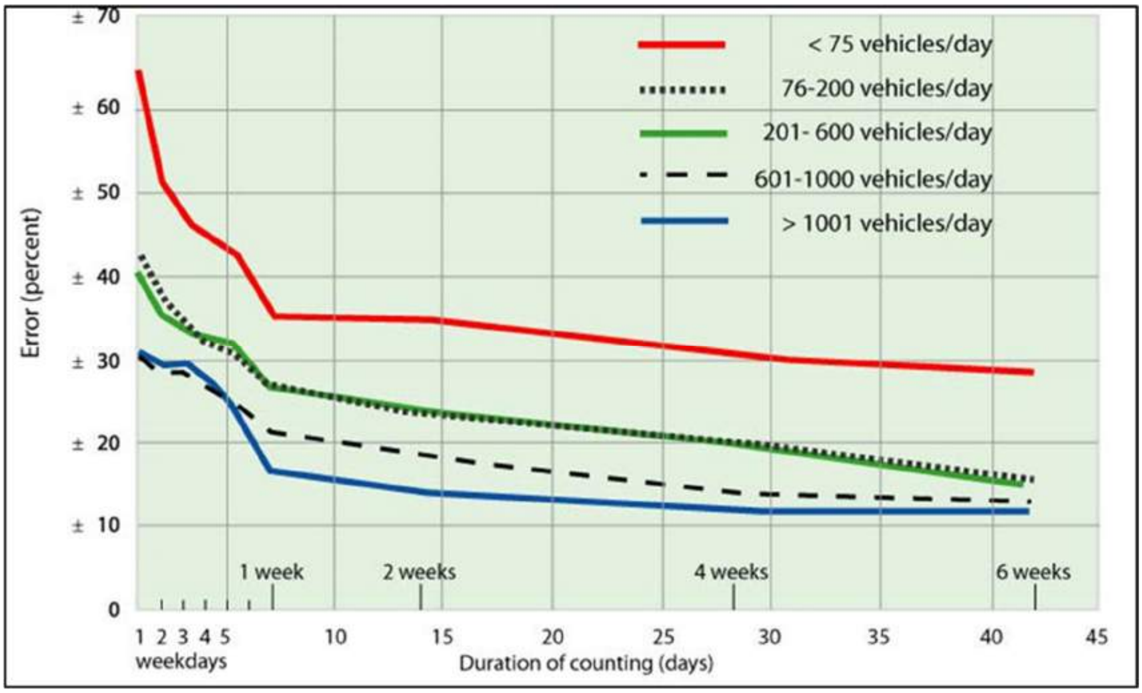
Reliable data on traffic volumes and load characteristics are an essential input for the pavement design process. Thus, a reliable estimate of existing (baseline) and future traffic statistics is required to undertake the design of the road in an appropriate manner.

The following types of traffic surveys are typically carried out in the project area where the road is located:

- Classified Traffic Counts
- Origin-Destination Surveys
- Axle Load surveys

Classified Traffic Counts

Different traffic survey requirements are necessary for the upgrading of an existing road and for a new road. For an existing road, the timing, frequency and duration of traffic surveys should be given very careful consideration in terms of striking a balance between cost and accuracy. As indicated in Figure 4-1, short duration traffic counts in low traffic situations can lead to large errors in traffic estimation.



Source: Howe, 1972

Figure 4-1: Possible errors in ADT estimates from random counts of varying duration

In the case of a new road, an approximate estimate should be made of traffic that would use the road considering the number of villages, and their population, along the road, and other socio-economic parameters. This can be achieved by carrying out traffic counts on an existing road, as described above, in the vicinity with similar conditions and knowing the population served as well as agricultural/industrial products to be transported. Likely traffic on the new road can also be estimated from Origin-Destination (O-D) surveys along the nearby existing roads which presently serve the villages proposed to be connected.

For either a new road or an existing road, due consideration also needs to be given to the anticipated “Diverted” and “Generated” traffic because of the development of the proposed road, land use of the area served, the probable growth of traffic, and the design life.

Reducing errors in estimating traffic for LVRs: Errors in estimating traffic can be reduced by:

- Counting for at least five or preferably seven consecutive days.
- On some days, counting for a full 24 hours, preferably with one 24-hour count on a weekday and one during a weekend; on other days, 16-hour counts (typically 06:00 – 22:00 hours) should be made and expanded to 24-hour counts using a previously established 16:24 hour conversion ratio.
- Avoiding counting at times when road travel activity increases abnormally; for example, just after the payment of wages and salaries, or at harvest time, public holidays, or any other occasion when traffic is abnormally high or low. However, if the harvest season is during the wet season (often the case, for instance, in the timber industry), it is important to obtain an estimate of the additional traffic typically carried by the road during these periods.
- Repeating the five or seven-day counts several times throughout the year.

Care should be exercised in selecting appropriate locations for conducting the traffic counts to ensure a true reflection of the traffic using the road and to avoid under- or over-counting. Thus, locations within or close to villages or marketplaces should be avoided. If any junctions occur along the road length, counts should be conducted before and after them.

The accuracy of traffic counts can be improved by increasing the count duration or by counting in more than one period of the year. Improved accuracy can also be achieved by using local knowledge to determine whether there are days within the week or periods during the year when the flow of traffic is particularly high or low.

Adjustments for the season: A weighted average adjustment is made to the traffic count data according to the season in which the count was undertaken, and the length of the wet and dry seasons, as illustrated in Figure 4-2.

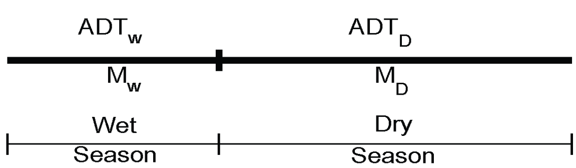


Figure 4-2: Basis for traffic count adjustment in relation to seasonal characteristics

The weighted average of the traffic count in relation to the seasonal characteristics of the region in which the counts were undertaken is obtained as follows:

$$Weighted\ Average\ ADT = \frac{ADT_w \times M_w}{12} + \frac{ADT_D \times M_D}{12}$$

Where: ADT_W = Average daily traffic count in wet season
ADT_D = Average daily traffic count in dry season
M_W = Number of months comprising the wet season
M_D = Number of months comprising the dry season

Example: For a wet season ADT of 240 vpd over 4 months and a dry season ADT of 360 vpd over 8 months:

Weighted Average ADT= (240 * 4)/12 + (360 * 8)/12 = 80 + 240 = 320

Origin-Destination Surveys

Origin-Destination (OD) surveys can be undertaken using a variety of survey techniques. They are carried out for a variety of reasons, including the provision of data on traffic diversion likely to occur after a particular link in the road network has been improved. Such diversion may occur due to drivers wishing to travel on a quicker or cheaper route, although this may not be the shortest. When combined with other estimates of traffic growth following a road improvement, it allows the total traffic flow to be estimated, as illustrated in Figure 4-3.

Axle Load Surveys

Axle load surveys provide critical and essential information that is required for both cost-effective pavement design as well as the preservation of existing roads. The importance of this parameter is highlighted by the well-known "fourth power law" which exponentially relates increases in axle load over 80 kN to pavement damage (e.g., an increase in axle load of 20 % produces an increase in damage of about 120 %). Information about the loading of vehicles is essential for pavement design and also for overload control. Methods of acquiring vehicle load data are described below.

Full axle load surveys: The type of equipment which may be used for axle load surveys varies widely and includes:

- Static or dynamic weighing equipment.
- Manual or automatic recording of loads.
- Portable or fixed installation.

The quality of the data obtained depends on the type of equipment used, the duration of the survey, and the degree of quality control performed. In general, the higher the quality of the data, the greater are the resources required to collect it. However, axle load surveys can be expensive and are not always undertaken for an individual LVR project for which simplified methods are required.

Simplified axle load surveys: If a full axle load survey is not being carried out, information about the vehicle loading can be obtained by observation during the traffic counting survey. The enumerator merely records, for every heavy vehicle in the heavy vehicle classes, the state of loading (full, partial or empty), and the type of load (heavy, medium, or light). The number of VEFs (ESAs per vehicle) can then be estimated based on a knowledge of the VEFs obtained from full axle load surveys carried out elsewhere in the vicinity of the project.

Determination of Design Traffic

The procedure for determining the traffic loading for pavement design purposes is summarized in Figure 4-3, and each step is explained thereafter. The traffic analysis for pavement design cannot be separated from the analysis for geometric design since the geometric design requirements, and ultimately the selection of road class and cross-section width, will influence the traffic lane distribution. The analysis for geometric and pavement design purposes should, therefore, always be carried out together. However, the geometric design aspects of LVRs are not discussed in this Manual.

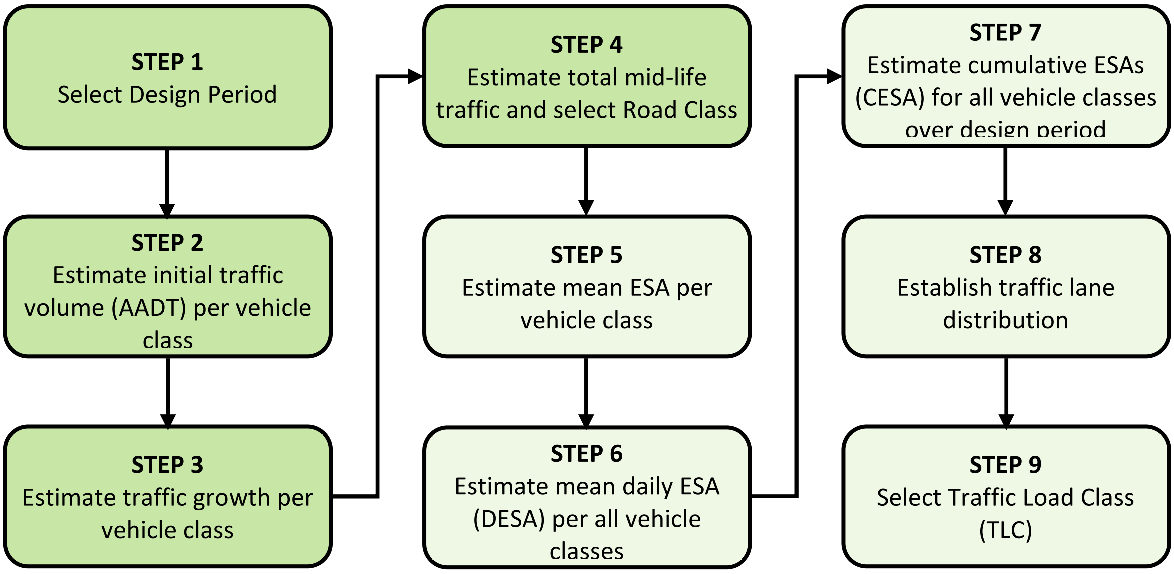


Figure 4-3: Procedure for establishing design traffic

Step 1: Select Design Period

A structural design period must be selected over which the cumulative axle loading is determined as the basis of designing the road pavement. The design period is defined as the time span in years considered appropriate for the road pavement to function before reaching a terminal value of serviceability, after which major rehabilitation or reconstruction would be required. The terminal serviceability is usually expressed in terms of rut depth.

Various factors that influence the choice of design period include:

- Functional classification and selected life-cycle strategies.
- Strategic importance of the road.
- Funding considerations.
- Maintenance strategies (highly trafficked facilities will demand long periods of low maintenance activity.
- Anticipated time for further upgrading of the road.
- The likelihood that factors other than traffic, e.g., a highly reactive subgrade, will cause distress necessitating major rehabilitation in advance of any load-related distress.

Based on the above factors, Table 4-3 provides guidance on the selection of the structural design period. Choosing a relatively short design life reduces the problem of long-term traffic forecasting whilst choosing a relatively long design life requires greater care in estimating the design traffic loading if over-/under-design of the pavement, and the related cost implications, are to be avoided.

Table 4-3: Structural design period

Importance/level of service	
Low	High
10 - 15 years	15 – 20 years

Step 2: Estimate initial traffic volume per vehicle class

Based on the traffic surveys described in Section 4.4.2 above, the initial traffic volume for each vehicle class can be determined. For structural design purposes, it is only the commercial vehicles that make any significant contribution to the total number of equivalent standard axles.

Step 3: Estimate traffic growth per vehicle class

Following the establishment of the baseline traffic, further analysis is required to establish the total design traffic based on a forecast of traffic growth in each vehicle class. To forecast such growth, it is first necessary to sort traffic in terms of the following categories, as shown in Figure 4-4.

Normal traffic: Traffic that would pass along the existing road in the absence of any upgrading to a higher standard.

Diverted traffic: Traffic that changes from another route to the project road, but still travels between the same origin and destination points. Unless origin-destination surveys have been carried out, this can only be estimated based on a judgment of the traffic on nearby roads that could benefit from a shorter or more comfortable route.

Generated traffic: Additional traffic that occurs in response to the new or improved road. This traffic is essentially 'suppressed' traffic that does not currently exist because of the poor state of the existing road. Local historical precedent can sometimes assist in estimating this; otherwise, generated traffic can be assumed to be about 20% of the existing traffic.

Diverted traffic occurs quickly after the completion of the road, whereas generated traffic increases with time as a result of developments within the road corridor.

Estimating traffic growth over the design period is sensitive to economic conditions and prone to error. It is, therefore, prudent to assume low, medium and high traffic growth rates as an input to a traffic sensitivity analysis for pavement design purposes.

The growth rate of each vehicle class may differ considerably. Traffic by Light Goods Vehicles, for example, is usually growing at a faster rate than that of Heavy Goods Vehicles, and this should be taken into account when estimating the traffic loading.

There are several methods for estimating traffic growth, including the following:

Local historical precedent: Evidence of traffic growth on roads recently upgraded in the area is a good guide as to what to expect.

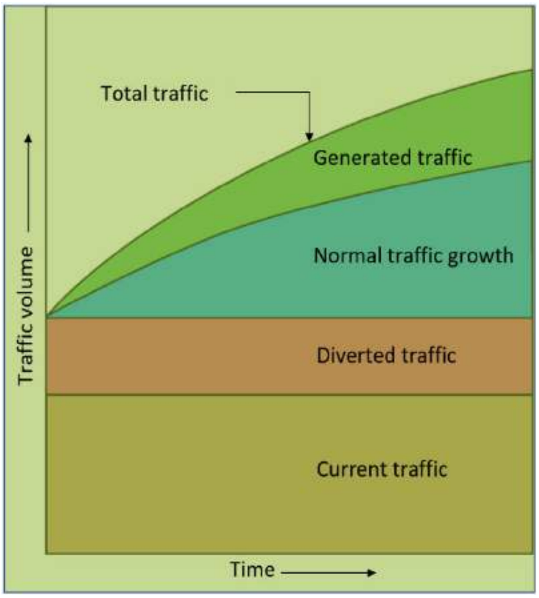


Figure 4-4: Traffic development on an improved road

Government predictions of economic growth: Traffic growth is closely related to the economic growth of a country. Economic growth rates can be obtained from government plans and government-estimated growth figures. However, the traffic growth rate should preferably be based on regional growth estimates, where available.

It should be borne in mind that both geometric design classes and structural design classes are quite wide in terms of traffic range, typically a range of 100% or more; hence the precision required of traffic estimation is not high. A common method of choosing the design traffic is simply to estimate the initial traffic, including diverted and generated traffic, and to accommodate traffic growth by choosing the next higher road class for both geometric and structural design.

The AADT in both directions in the first year of analysis consists of the current traffic plus an estimate of the generated and diverted traffic. Thus, if the total traffic is denoted by AADT and the general growth rate is **r** per cent per annum, then the traffic in any subsequent year, **x**, is given by the following equation:

$$AADT_X = AADT_0 \times \left(1 + \frac{r}{100}\right)^X$$

Step 4: Estimate the total mid-life traffic and select Road Class

This is the last step for geometric design purposes (not addressed in this Manual). The Road Class will determine the cross-section width and influence the traffic lane distribution, as shown in Table 4-4.

Step 5: Estimate mean ESA per vehicle class

Static axle load data on the vehicles expected to use the road is required to determine the mean axle load Equivalence Factor (EF) and, subsequently, the mean Vehicle Equivalence Factor (VEF), i.e., the sum of the axle load EFs for each vehicle. Ideally, such data should be obtained from surveys of commercial vehicles using the existing road or, in the case of new roads on new alignments, from existing roads carrying similar traffic. However, such surveys may not be justified for LVRs, in which case reliance will need to be placed on existing information and visual surveys.

The VEF is determined from converting the surveyed individual axle loads to axle load EF (ESAs/axle), adding up the EFs for each vehicle, and then deriving a representative weighted average value for each vehicle class. In some cases, there will be distinct differences in each direction, and separate EFs should be derived for each direction.

The EF (ESAs/axle) is derived as follows:

EF = $[P/8160]^n$ (for loads in kg) or = $[P/8.16]^n$ (for loads in tonnes) = $[P/80]^n$ (for loads in KN)

The formula for calculating the VEF for each individual vehicle can then be expressed as follows:

$$VEF = \sum_{1}^{i} \left(\frac{P}{8.16}\right)^{n}$$

where: i = the number of axles on the vehicle class (e.g., for a 3-axle truck i = 3). Note that Heavy Goods Vehicles (HGV) may not all have the same number of axles. The VEF is determined per vehicle class, not distinguishing between vehicles with a different number of axles.

P = axle load in tonnes. The standard axle load is taken as 8160 kg, 8.16 tonnes or 80 kN.

n = power exponent (typically 4.0 – 4.5 applied by road agencies. Recent research evidence suggests that n = 3.0 – 4.0 may be more appropriate for LVRs with flexible pavements. For the time being, a value n = 4.0 is recommended. See also discussion in *Chapter 3 – Approach to Design, Appendix 3.1.*

If the information on average VEF for different vehicle classes is not available, data from any recent axle load survey on the road in question or a similar road in the vicinity is better than using countrywide averages.

Step 6: Estimate Mean Daily ESA for all Vehicle Classes

The estimated mean daily ESAs for each vehicle class (DESA) is obtained from the traffic data derived in Step 3, and the VEFs derived in Step 5 as follows:

$$DESA = AADT \times VEF$$

Step 7: Cumulative ESA (CESA) for all Vehicle Classes over the Design Period

For pavement design: The cumulative equivalent standard axles (CESA) in each direction for each traffic category expected over the design life may be obtained from the following formula:

$$CESA = 365 \times DESA \times [(1 + r)^{n} - 1]/r$$

where: DESA = mean daily ESAs for each vehicle class in the first year in each direction (from Step 6).

r = assumed annual growth rate expressed as a decimal fraction. (Different traffic categories may have different growth rates).

n = design period in years (from Step 1).

Step 8: Establish Traffic Lane Distribution

The actual design traffic loading (ESAs) needs to be corrected for the distribution of heavy vehicles between the lanes in accordance with Table 4-4.

Table 4-4: Pavement width adjustment factors for design traffic loading

Cross section	Paved width	Corrected design traffic loading (ESA)	Explanatory notes
Single carriageway.	< 3.5m.	Double the sum of ESAs in both directions.	The driving pattern on this cross-section is very channelized.
	Min. 3.5m but less than 4.5m.	The sum of ESAs in both directions.	Traffic in both directions uses the same lane, but not all in the same wheel tracks as for the narrower road.
	Min. 4.5m but less than 6m.	80% of the ESAs in both directions.	To allow for overlap in the centre section of the road.
	6 m or wider.	Total ESAs in the heaviest loaded direction.	Minimal traffic overlap in the centre section of the road.
More than one lane in each direction.		90% of the total ESAs in the studied direction.	The majority of vehicles use one lane in each direction.

Note that for a carriageway width of 6.0 m and above, the pavement design is based on the total ESAs in the heaviest loaded direction. In such cases, the best approach is to carry out the traffic count and estimation of the TLC for each direction separately rather than summing up the traffic in both directions and then estimating a directional split.

Step 9: Select Traffic Load Class

The traffic classes for structural design are shown in Table 4-5.

Table 4-5: Traffic Load Classes for structural design

DCP-DN Design Method	
Traffic Load Class	**Cumulative traffic load during design life (MESAS)**
TLC 1.0	0.7 – 1.0
TLC 0.7	0.3 – 0.7
TLC 0.3	0.1 – 0.3
TLC 0.1	0.01 – 0.1
TLC 0.01	< 0.01

Sensitivity Analysis: For the final selection of the Traffic Load Class, it is prudent to carry out a sensitivity analysis taking account of:

- Different traffic growth rate scenarios.
- The likelihood of future developments in the area, e.g., new industry, mining operations, agricultural development, new road projects, etc., which have not already been accounted for in the traffic growth estimates.

If the estimated design traffic loading is close to the upper boundary of the TLC, and the sensitivity analysis indicates that the upper boundary may be exceeded, the next higher TLC is adopted unless this has a significant impact on the construction cost. The impact may be negligible if the required material quality is readily available, or significant if the higher TLC would require longer haulage distances or modification of the materials available in the vicinity of the road.

If the project budget cannot accommodate the design for the higher TLC, the effect will not necessarily be that the pavement will fail, only that it would reach the end of its design life earlier than planned, e.g., after 12 years rather than 15 years.

An example of the procedure for calculating the Traffic Load Class (TLC) of a LVR is presented in Appendix 4.1.

4.4.3 Route Surveys

The successful design of a road depends on ensuring that the pavement is appropriate for the characteristics of the subgrade or the embankment on which it is placed. To this end, a good understanding of the following is essential:

- General soil types along the route.
- Required test pits and trenches.
- Sampling and testing requirements.
- Moisture and drainage conditions.
- Problem subgrade soils.

When upgrading an existing track or road, it is also important to determine the subsurface characteristics of the material in the structure because this material should be utilised in the new pavement to the maximum extent possible. The DCP is used to obtain such sub-surface information along the entire route at appropriate intervals to a depth of 800 mm

DCP Surveys

General: The DCP is well suited for characterising the strength of the subgrade or existing layers at the in-situ moisture and density. It is also useful for quality control testing during construction. The strength of using this device is that it "sees" the variation in the ground conditions down to a depth of 800 mm. It is thus important that the DCP survey is carried out correctly (ASTM D6951) and in a consistent manner by trained personnel to obtain useful and reliable data. It is equally important that the designer participates in the DCP survey to develop a "feel" for the ground conditions. This will enable him/her to interpret the data correctly once back in the office.

Frequency of DCP measurements: The frequency of the DCP measurements depends on the variability in road conditions and the level of confidence required. Where obvious changes in surface conditions occur, the frequency of the testing should be increased in the vicinity of the locations where the changes occur. Similarly, where surface conditions are uniform, the frequency of testing may be reduced. A guideline for the minimum frequency of testing for upgrading an existing track or unpaved road to a paved standard is shown in Table 4-6.

Table 4-6: Frequency of DCP testing

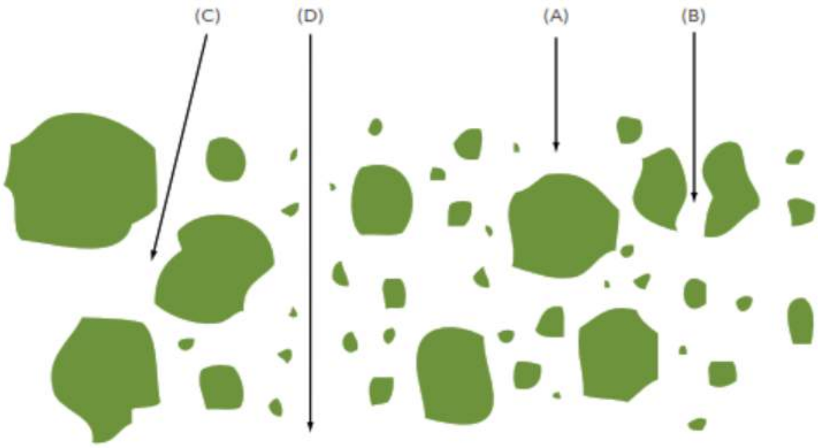
Road condition	Frequency of testing (number/km) (minimum)
Uniform, fairly flat, reasonable drainage – low risk	5
Non-uniform, rolling uneven terrain, variable drainage – medium risk	10
Distressed, uneven terrain, poor drainage – high risk	20

Pattern of DCP measurements: The tests should be staggered across the width of the road at outer wheel-tracks (left and right) and the centerline. However, the variability of the road subgrade strength will only become fully apparent when the tests have been carried out. In order to ensure statistical reliability, at least ten tests must be taken in each uniform section. Hence additional tests may be required after analyzing the first set of results.

The general procedure for undertaking the DCP survey as part of the overall design process differs between that for new roads and for existing roads, as discussed below:

Care must be exercised in carrying out the DCP survey to discard any measurements that could produce anomalous results. Such results could arise, for example, where large stones occur in the pavement layers or subgrade, as illustrated in Figure 4-5.

Where hard aggregate layers (e.g., water-bound macadam) already comprise a part of the pavement, this layer should be removed, and the DCP test carried out from the base of the layer, after measuring the thickness of this layer. This layer is then included in the pavement design as an existing strong layer.



(a) Cone cannot penetrate	(b) Cone breaks the stone. DCP profile shows a plateau and subsequent readings may be low.	(c) Rod pushed aside and tilts at an angle. Excessive friction on rod gives a low reading	(d) Normal result

Figure 4-5: DCP effects where large stones are present

The data should be captured in the form of the number of blows of the DCP hammer and the corresponding depth of penetration and can be entered directly into a spreadsheet for use in available software. Destructive testing (sampling) is not required at each DCP test site.

DCP survey for existing roads: On existing roads and tracks, the new road is expected to be built directly on the material currently forming the road or track, and the DCP test will indicate the properties of the materials that will be a part of the new pavement structure. Even if the formation is raised slightly to facilitate drainage or pipe cover, the materials tested will usually influence the structural capacity of the new pavement structure.

The DCP survey must thus be carried out along the full length of the road, with each measurement being taken to a depth of at least 800 mm, in order to assess the balance of the final pavement.

DCP Survey for new roads: A DCP survey for new roads can result in two processes, as follows:

1) **The new road will be at or slightly above the natural ground level:** In this case, the DCP survey can be carried out along the alignment as described above.
2) **The new road is to be constructed on a new fill or embankment higher than about 500 mm:** In this case, the DCP method cannot be used directly as the formation level of the new road would be outside the influence zone of an existing alignment DCP survey. In such a situation, samples of the proposed embankment material can be obtained at appropriate frequencies for laboratory classification and strength testing. Fills can then be designed in accordance with the DCP-DN design catalogue to ensure that all the layers comply with the specifications.
3) **The new road is to be constructed in a soil cut deeper than about 500 mm:** In this case, the material properties and design strength can be obtained from test pit samples taken at the level of the finished roadbed, at appropriate frequencies along the length of the cut

section, for laboratory classification and strength testing. The pavement layers can be tested in the usual manner. Cuts can then be designed in accordance with the DCP-DN design catalogue to ensure that all the layers comply with the specifications.

4) **In areas of significant widening**: The approach would be as described above, depending on whether the widening would involve a cut or fill situation.

In some cases, the DCP can be used at natural ground level to identify potential drainage and moisture problems, or potential subgrade problems, e.g., expansive or collapsible soils, beneath embankments that will be up to about 1.0 m or more in height, which may influence the later stability or settlement of the embankment. However, DCP testing of the in-situ material at the prevailing moisture and density is generally meaningless for such embankments. Instead, testing of samples of the material in the laboratory, after compaction to the expected density and at the expected moisture content, can be used to determine whether the local material can be used for the construction of the embankment. It is also possible, if the proposed embankment borrow source has been identified, to test this material in the laboratory to determine its characteristics.

Interpretation and analysis of DCP data: The analysis of the DCP test results is done automatically in the ReCAP LVR DCP program. However, the interpretation of the data and variations caused by, for instance, weak or hard interlayers, may require further investigations and sound engineering judgment on how to deal with the localized situation to provide sound and cost-effective design solutions. The most common problem found in practice is the occurrence of subsurface moisture due to seepage from higher ground or simply because of inadequate drainage of the existing road or track. The former may require installation of subsurface drainage (expensive) while the latter can often be remedied simply by ensuring that the level of the pavement is raised (i.e., by importing selected fill material or constructing a thicker pavement), and that proper side-drains are constructed.

For the analysis of the DCP test results, it is good practice to exclude "outliers," i.e., very high or very low DN values, in order not to distort the determination of representative values within the uniform sections. Weak points should, however, then be investigated separately to identify the cause of the problem. Additional DCP tests at close intervals may be required to determine the extent of the problem.

Determination of uniform sections: Following the exclusion of "outliers," the DCP data is used to determine uniform sections from a CUSUM analysis that is carried out automatically in the ReCAP LVR DCP software or using spreadsheets as described in *Chapter 7 – Structural Design: Paved Roads*.

Test pits and trenches

Test pits and trenches are used to take samples for testing to provide information on the in-situ subgrade soil conditions and potential fill material.

The location, number and depth of pits and trenches required for characterizing the subgrade depend on the type and condition of the road and the general characteristics of the project area (soil type and variability). The DCP tests provide a good indication of the subsurface conditions and variability with the depth of the subgrade and the depths at which samples should be taken.

The determination of uniform sections, as described in *Chapter 7 - Structural Design: Paved Roads,* statistically limits the variability of the subgrade. For gravel or earth roads in a reasonable condition, where the DCP tests indicate fairly uniform subgrades with depth, three test pits per uniform section for the purpose of material sampling and description in uniform materials to a depth of 450 mm below the surface are sufficient for the determination of the design subgrade strength. Engineering judgment must be used to locate the uniform sections at points deemed to be representative of the section.

If there is reason to suspect the occurrence of problematic subgrade soils, in particular for "greenfield" projects, i.e., on sections where the horizontal alignment is new or changed, deeper test pits may be required. These are also useful for investigation of the water table level.

The location of each test pit within a uniform section should be carefully chosen, and all layers, including topsoil, should be accurately described, and their thicknesses measured. All horizons thicker than 75 mm below the topsoil should be sampled. This will also provide a proper assessment of the materials excavated in soil or suitable weathered cuts that are to be used in embankments. The samples should be taken over the full depth of the layer.

It is sometimes impossible to dig trial pits to the depth of all layers of soil or weathered rocks that need to be assessed for the foundation design of structures or the treatment of weak or problem soils. In this case, it is recommended that hand or power augers are used for identification (AASHTO T203). Borings could also be necessary to investigate the materials that lie below the pavement layers. This is especially true in areas where a thick layer of problem soils or soft deposits exist and where the road alignment passes through landslide zones, solution cavities and unconsolidated soils.

Sampling and Testing

The contribution of a thin layer of, say, less than 75 mm, of remaining gravel wearing course on the road, to the strength of the in-situ pavement, can be disregarded. This thin layer should not be mixed with the subgrade samples; it is most likely not of base or subbase quality due to loss of fines and will normally be blended with the top of the subgrade in the construction process. This will provide a factor of safety in the design by increasing the strength of the subgrade above the subgrade strength determined in the laboratory. However, a thicker gravel wearing course layer may be augmented by the addition of more gravel to form a full base- or subbase layer, in which case the properties and strength of the gravel wearing course must be determined in the laboratory.

The subgrade design strength is determined through the laboratory DN test. If there are two (or more) distinctly different subgrade material types, as indicated in Figure 4-6, both layers must be tested separately.

Samples collected from the test pits are used to provide the basic information on the properties of the in-situ materials and subgrade along the alignment. The standard laboratory tests to be carried out and test methods to be used are shown in Table 4-7.

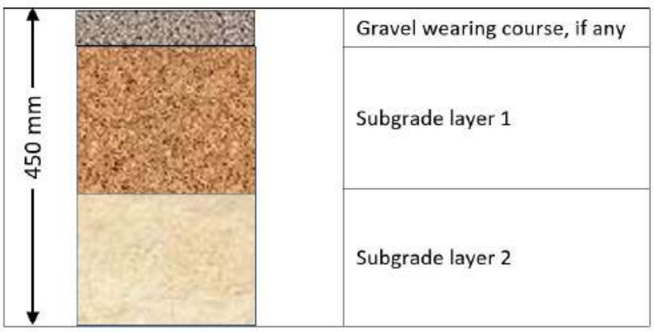


Figure 4-6: Typical soil profile in test pit

Table 4-7: Soil profile description and standard laboratory testing

Parameter	Test Methods
Soil Profile: • Overburden • Layer/horizon thickness • Visual description • In-situ moisture content • In-situ density	 ASTM D2216 ASTM D1566 or D6031
Index tests	ASTM D6913 & D4318
Compaction (Density/Moisture)	ASTM D588
Strength	Laboratory DN (Chapter 5, Appendix 5.1); Field (ASTM 6951)
Mineralogical and Durability	Max. Secondary Mineral Content – ASTM C856 Magnesium or Sodium Sulphate Soundness – ASTMC88-13 Methylene Blue Index – ASTM C837-09)

The soil profile is described, and index and compaction tests are carried out for the material from each test pit to ascertain the representative nature of the materials. Then, the bulk samples of each separate layer are mixed for the determination of the subgrade design strength, as described in *Chapter 7 – Structural Design: Paved Roads.*

Figure 4-7: Logging of subgrade profile

Moisture and drainage

Two different design approaches are provided for in the Manual and in the ReCAP LVR DCP software:

Option 1: This option is recommended for design situations where the moisture conditions along the alignment are not likely to be uniform. The determination of the representative layer strength profile within each uniform section is carried out through laboratory testing of the in-situ pavement layers and subgrade at the anticipated, in-service equilibrium moisture content (EMC) and density. The determination of the in-situ moisture content in the pavement at the time of the DCP survey is therefore not required.

Option 2: This option may be considered in situations where the moisture conditions along the alignment are likely to be fairly uniform, as would be the case with an improved gravel road having a functional drainage system.

Based on representative moisture samples from each uniform section, the DN values can be adjusted to represent the anticipated in-service EMC within each layer by the use of percentiles. Applying, say, the 80th percentile of the weighted average of the in-situ DN values, would indicate that it is expected that the long-term EMC will be higher than at the time of the DCP survey. Conversely, applying the 20th percentile would indicate that it is expected that the long-term EMC will be lower than at the time of the DCP survey. Lastly, if no significant changes in the moisture regime are anticipated, the 50th percentile (equals the weighted average DN values) may be used for the design.

Depending on the TLC for the road under design, other combinations of percentiles may be used to provide for an increased factor of safety.

For either option, guidance on the selection of in-service EMC is provided in *Chapter 7 – Structural Design: Paved Roads.*

Assessment of moisture conditions along the alignment: For Option 2, it is essential that an estimate of the in-situ moisture condition is made at the time of the DCP survey for comparison with the expected moisture regime in service.

At least two samples should be collected per kilometre of the in-situ pavement layers and subgrade materials for moisture content and Optimum Moisture Content (OMC) determination from the outer wheel tracks of the road at depths of 0-150, 150-300 and 300-450 mm. This is best done during the test pitting. Synchronisation of the identification of uniform sections and test pitting needs to be done as soon as possible after the DCP survey, within about seven days, to ensure that moisture content samples are representative of all the uniform sections.

As far as possible, it is recommended that the DCP survey is carried out towards the end of the wet season, at which point the pavement layers and subgrade will normally be in their weakest (and wettest) condition. Upgrading of the pavement, by addition of new layers and improvement of the drainage, will then ensure that the in-service EMC in the pavement and subgrade is lower than, or, at worst equal to, the moisture regime at the time of the DCP survey.

Local drainage problems and requirements: During the survey, any drainage problems or constraints that would affect drainage locally (streams, marshy areas, flat poorly drained areas, etc.) need to be identified so that areas requiring specific side or cross drainage can be pin-pointed for the design.

Cognisance should be taken of the potential climate change effects in the long term, particularly in terms of drainage structures. Projections indicating that the annual rainfall and intensity of storms will increase, with more frequent extreme events, will lead to potentially more flooding situations in certain areas. It is recommended that localised information is obtained from the relevant authorities regarding expected climate changes in areas being investigated. Areas that are visibly prone to possible flooding and water accumulation under extreme precipitation or flooding conditions must be noted, as soaked designs would be necessary for these areas.

Problem Subgrade Soils

Many subgrades may be classified as problem soils. These include a wide range of possible materials such as:

- Expansive/heaving clays ("cotton soils")
- Low-strength soils (often found in wet/waterlogged areas)
- Collapsible soils
- Dispersive soil
- Erodible soils
- Saline soils

Each of the potentially problematic soils listed above requires unique investigation and test protocols. During the site investigation, it is important that such problem areas are identified, and useful advice is obtained regarding the implications and treatment from geotechnical specialists where it is felt necessary. The fact that most of these are affected by moisture fluctuations is also relevant to long-term climate changes. The manner of identifying and treating the types of problematic soils listed above is of a specialist nature and is beyond the scope of this Manual. Such information may be obtained from various ReCAP manuals on LVRs and climate change, as listed in the bibliography.

4.4.4 Construction Materials Surveys

General

Part of the initial survey programme is to identify potential sources of construction materials. Although the DCP design method attempts to optimise the use of in-situ materials and minimise additional material usage, in many cases, the local materials may not be of suitable quality, and other materials may be required for fill and the upper pavement layers. Thus, sources of road-building materials must be identified within an economic haulage distance, and they must be available in sufficient quantity and of the required quality for the purposes intended. Previous experience in the area plus local knowledge may assist with locating such materials, but additional surveys are usually required.

Types of construction material

The types of construction materials required for a typical LVR, are as follows:

- Common embankment fill
- Capping layer / imported subgrade
- Pavement and wearing course material
- Road surfacing aggregate
- Paving stone (e.g., for cobblestone pavements)
- Aggregates for structural concrete
- Filter/drainage material
- Rockfill, e.g., for gabion baskets
- Construction water

If the project is in an area where good quality construction materials are scarce or unavailable, alternate solutions that make use of the local materials should be considered to avoid long and expensive haulage. For example, consideration could be given to either modifying the material (e.g., mechanical or chemical stabilisation) or to material processing (e.g., crushing, screening, blending).

Fill: In general, the location and selection of fill material for LVRs pose few problems. Exceptions include organic soils and clays with high liquid limit and plasticity. Problems may also exist in lacustrine (stratified deposits at the bottom of a lake) and flood plain deposits where very fine materials are abundant. Where possible, fill should be taken from within the road alignment (balanced cut-fill operations) or by the excavation of the side drains (where materials meet the requirements). Borrow pits producing fills should be avoided as far as possible due to cost implications, and special consideration should be given to avoid winning fill in agriculturally productive areas where land expropriation impacts can be high. Silty materials should be avoided in fills as their high capillary suction can increase the moisture content in the pavement structure.

Improved subgrade: The improved subgrade can be made of the same material as any fill. Where in-situ and alignment soils are weak or problematic, the importation of improved subgrade (capping layer) may be necessary. As far as possible, the requirement to import material from borrow areas should be avoided due to the additional haulage costs. However, the importation of strong subgrade materials can provide economies because pavement thickness design can be reduced (refer to *Chapters 6 and 7 – Structural Design*). Where improvement is necessary or unavoidable, mechanical and chemical stabilisation methods should be considered.

Base and subbase: Where possible, naturally occurring unprocessed materials should be selected for base and subbase layers in paved low volume roads. However, under certain circumstances, mechanical treatment may be required to improve the quality to the required standard. This often requires the use of special equipment and processing plants that are relatively immobile or static. In such cases, the borrow pits for base and subbase materials are usually spaced widely.

The main sources of base and subbase materials are rocky hillsides and cliffs, high steep hills, river banks, and naturally-occurring residual soil deposits and pedocretes, e.g., calcretes and laterites.

The minimum thickness of a deposit that is normally considered workable for excavation for materials for subgrade, sub-base and base is of the order of one metre. However, thinner horizons can be exploited if there are no alternatives. The absolute minimum depends on material availability, the thickness of the overburden, and the methods of excavation. If there is no overburden, as may be the case in arid areas, horizons as thin as 300 mm may be excavated.

Hard stone and aggregate: A variety of rocks can be used as material sources for concrete aggregate, bituminous road surfacing aggregate, masonry and cobblestone. In most areas, a relatively fresh rock can be encountered at some depth as there is a gradual transition from one weathering state to the other. Fresh rock is usually close to the surface in steep slopes and cliff faces. The recovery of a suitable material is, therefore, a matter of understanding the geological history and weathering profile at the quarry site. It is then necessary to make sure that only un-weathered rock of the specified quality is excavated for future use.

Construction water: Road construction requires significant sources of water to ensure proper compaction, stabilisation, and finishing of the structural layers. During the initial investigations and surveys, it is important to identify potential sources of construction water. Normal river water is usually adequate for construction, but groundwater resources may be slow and expensive to extract and may also be excessively saline for road construction. Typically, water that is potable to humans is suitable for construction and should be used.

4.4.5 Materials prospecting

General

Prospecting for construction materials is aimed at ensuring that such materials are located as efficiently as possible, instead of the "haphazard or random methods" often used. The art of prospecting involves looking for clues as to the occurrence of useful materials and then digging test pits to see what may be there. Learning to identify features that indicate the presence of gravel from the interpretation of maps and other information is a key aspect of the process.

Stages in Prospecting

The various stages in the materials prospecting process are shown in the flow chart in Figure 4-8.

At the desk study stage, records of roads already built can provide a valuable source of data, not only on the location of construction materials but also on their excavation, processing, placement and subsequent performance. Potential problems with materials can also be identified, but care must be taken to distinguish between genuine material problems and poor performance caused by inadequate drainage, as is often the case. Construction records may be available with the Roads Authority, local authorities, or by road design consultants and contractors. These, and any other materials-related reports, should be consulted to assist with material location.

Mapped data on topography, geology, soils, hydrology, vegetation, land-use and climate in the area should be used to plan the field survey and laboratory testing programmes. The field investigation programme may be undertaken by specialised firms or in-house. Due regard must be given to environmental considerations that impact the viability of potential material sources.

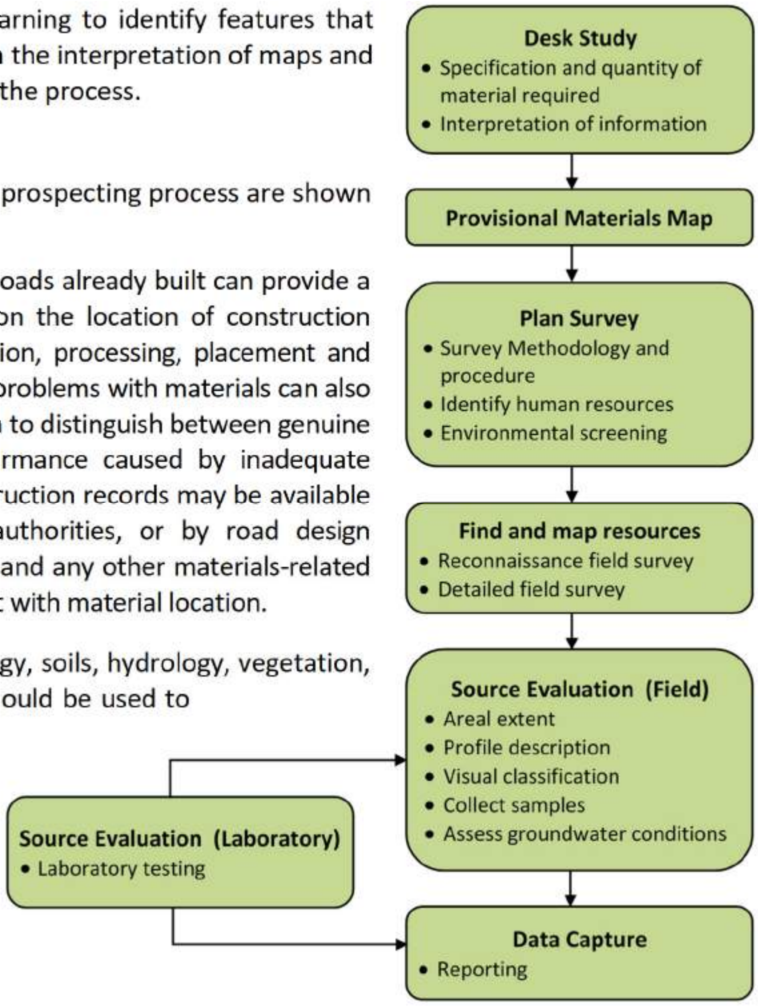


Figure 4-8: Flowchart of prospecting procedure

To assist with material location in the field, a number of techniques can be utilised. Many plants preferentially grow on materials with specific mineralogical/chemical or physical properties. Certain plant species grow particularly well on calcium-rich or iron-rich materials, and by identifying these plants, the presence of calcrete or laterite, for instance, in the underlying material can be identified. Other plants may have a preference for sandy/gravelly (free-draining) materials compared with those that prefer more water-logged conditions (clayey materials).

The geomorphology is also a strong indicator of potential materials. Specific features such as pans, depressions, ridges or trenches can indicate material differences. Flat lying areas tend to have deeper weathering profiles (or transported soils) than more steeply inclined areas. Termite hills and animal burrows may also give clues to the subsoil conditions.

Exploration

The completion of the initial prospecting stage is followed by the exploration stage, which has the following objectives:

- Determination of the nature of the deposit, including its geology, history of previous excavation, and possible mineral rights.
- Determination of the depth, thickness, extent and composition of the strata of soil and rock that are to be excavated.

- Analysis of the condition of groundwater, including the position of the water table, its seasonal variations, and possible flow of surface water into the excavation ground.
- Assessment of the properties of soils and rocks for the purposes intended.

The outcome of the exploration stage would be a comprehensive list of the location of potential borrow pits and quarries, along with an assessment of their proposed use and the volumes of material available.

Apart from the quality and quantity of material, the borrow pits and quarries should ideally be:

- Accessible and suitable for efficient and economic excavation.
- Close to the site to minimize haulage costs.
- Of suitable quality to enable cost-effective construction with little or no treatment.
- Located such that their exploitation will be environmentally acceptable and legally possible.

Two of the most common reasons for the escalation of construction costs, once construction has started, and material sources have been fully explored, are that the materials are found to be deficient in quality or quantity. This leads to expensive delays whilst new sources are investigated, or the road is redesigned to take account of the quality of the materials available.

4.4.6 Materials Sampling

General

A high proportion of the LVR construction costs is materials related. Construction using natural gravels with inherently variable properties can also lead to poor performance, premature failures, and increased maintenance costs unless the use of materials of the specified quality is strictly upheld at all times. A key requirement for the successful construction of LVRs is, therefore, that a materials sampling and testing program is designed and implemented as appropriate to the circumstances for each project.

Sampling

A variety of sub-surface sampling and investigation procedures appropriate for different materials is used to recover the samples needed for laboratory testing. These include disturbed sampling from test pits, trenches and auguring as well as undisturbed block sampling from exposed faces of excavations. It is important that adequate representative samples of each material are obtained for testing. Table 4-8 gives a guide to the required sample sizes for the most common soil tests applicable to LVRs. The relevant test protocol should be consulted before the collection of samples from the field.

Table 4-8: General guide for sample size requirements for common soil tests

Test		Minimum mass required (kg)		
		Fine grained	Medium grained	Coarse grained
Classification	Water / Moisture Content	0.05	0.35	4.00
	Liquid limit (Cone / Casagrande)	0.50	1.00	2.00
	Liquid limit (One Point Cone)	0.10	0.20	0.40
	Plastic Limit	0.05	0.10	0.20
	Shrinkage Limit	0.50	1.00	2.00
	Linear Shrinkage	0.50	0.80	1.50
	Particle Size Distribution (PSD)	0.15	2.50	17.00
Compaction	CBR / DN (per mould)	6.00	6.00	12.00
	Compaction (Heavy 4.5 kg / Light 2.5 kg, CBR mould)		80.00	
	Compaction (Heavy 4.5 kg / Light 2.5 kg, 1 ltr mould)		25.00	
	Vibrating Hammer (also used for determining "refusal density")		80.00	
Aggregate strength	Aggregate Crushing Value (ACV)		2.00	
	Aggregate Impact Value (AIV)		2.00	
	Los Angeles Abrasion (LAA)		5.00-10.00	

Notes: The laboratory definitions of fine and coarse soils differ from those used for engineering soil descriptions.
Fine-grained = not more than 10% > 2 mm (incl. clay, silt and sand)
Medium grained = some > 2 mm, not more than 10% > 20 mm (incl. fine and medium gravel)
Coarse-grained = some > 20 mm, not more than 10% > 37.5 mm (incl. coarse gravel)

Materials must be sampled at a regular frequency, or whenever the material source changes, and correct sampling procedures must be followed to ensure that the samples are representative.

Potential borrow pits shall be surveyed by trial pit excavation and sampling at the detailed design stage. The survey shall prove sufficient quantities for all pavement layers, and the sampling frequency shall be *at least* as indicated in Table 4-9 per DN test.

Table 4-9: Guideline for materials sampling in borrow pits

Intended use	Maximum volume per laboratory DN test (m^3)
Base	5,000
Subbase	10,000

Bibliography

ASIST (1998). ***Technical Brief Number 9: Material Selection and Quality Assurance for Labour-based Unsealed Road Projects.*** International Labour Organisation Advisory Support, Information Services, and Training (ASIST) Nairobi, Kenya.

ASTM D6591/D6591M-18 (2018). ***Standard Test Method for Use of the Dynamic Cone Penetrometer in Shallow Pavement Applications.*** ASTM International, West Conshohocken, PA., USA.

Bennet C R, Soliminiha H and A Chamaro (2006). **Data collection technologies for road management.** Transport Note 30. World Bank, Washington, D.C., USA.

Cook J R, Bishop E C, Gourley C S and N E Elsworth (2001). ***Promoting the Use of Marginal Materials.*** TRL Report PR/INT/205/2001. TRL, Crowthorne, Berkshire, UK.

Ministry of Works and Transport, Botswana (2000). ***Guideline No. 3: Methods and Procedure for Prospecting for Road Construction Materials***. Roads Department, Gaborone, Botswana.

Ministry of Works, Tanzania (1999). ***Pavement and Materials Design Manual***. TANROADS, Dar es Salaam, Tanzania.

Paige-Green P, Coetzer K, Lea J and C Semmelink (1993). ***Appropriate use of locally available materials in concrete, bituminous surfacings and layer works for roads in rural areas.*** CSIR Transportek, Research Report RR93/263, Pretoria, South Africa.

Robinson P, Oppy T and G Giumarra G (1999). ***Pavement Materials in Road Building, Guidelines for Making Better Use of Local Materials.*** Australian Road Research Board, Transport Research Ltd. Victoria, Australia.

SANRAL (2013). ***South African Pavement Engineering Manual, Chapter 7: Geotechnical Investigations and Design Considerations***. South African National Road Agency. Pretoria, South Africa.

Appendix 4.1: Traffic analysis

Example of estimation of Traffic Load Class

This design example is for illustrative purposes only for which typical input parameters are used.

Step 1: Select Design period

Design period = 15 years

Step 2: Estimate Initial Traffic Volume per Vehicle Class

A 7-day traffic count summary (AADT of commercial vehicles in both directions) is as follows:

Table 4-10: Example of traffic count figures

Day	Large bus	Small bus	LGV	MGV	HGV
Mon	1	4	9	1	0
Tue	2	4	11	2	0
Wed	2	5	7	1	0
Thu	3	8	9	3	0
Fri	2	8	6	2	0
Sat	3	10	25	4	0
Sun	1	3	10	1	0
ADT	2	6	11	2	0

Step 3: Estimate Traffic Growth per Vehicle Class

Vehicle growth rate r = 4.5% (average for all vehicle classes).

Step 4: Estimate total mid-life traffic and select road class

Step 5: Estimate Mean VEF (ESA per Vehicle Class)

The Vehicle Equivalence Factors have been determined as follows using n = 4:

Table 4-11: Example of VEFs

Vehicle Type	VEF (ESA/vehicle)	
	Direction 1	Direction 2
Large bus	2.4	1.2
Small bus	0.3	0.15
LGV	1.5	0.75
MGV	4	2
HGV	7	3.5

Step 6: Estimate Mean Daily ESA for all Vehicle Classes

Estimation of mean daily ESA (DESA) for all vehicle classes in Direction 1.

- Large bus 1 x 2.4 = 2.4
- Small bus 3 x 0.3 = 0.9
- LGV 5.5 x 1.5 = 8.3
- MGV 1 x 4.0 = 4.0
- HGV = 0

Total ESA/day = 15.6 (direction 1)

Estimation of mean daily ESA (DESA) for all vehicle classes in Direction 2.

- Large bus 1 x 2.4 = 2.4
- Small bus 3 x 0.15 = 0.5
- LGV 5.5 x 0.75 = 4.1
- MGV 1 x 2.0 = 2.0
- HGV = 0

Total ESA/day = 9.0 (direction 2)

Step 7: Cumulative ESA (CESA) for all Vehicle Classes over the Design Period

The design CESA can be computed from the following equation:

$$\begin{aligned} CESA &= 365 * DESA* [(1 + r)^n – 1]/r \\ &= 365 \times (15.6+9.0) \times [(1 + 0.045)^{15} – 1]/0.045 \\ &= 365 \times 24.6 \times [(1.045)^{15} – 1]/0.045 \\ &= 365 \times 24.6 \times [1.935- 1]/0.045 \\ &= 365 \times 24.6 \times 20.78 \\ &= 186,583 \text{ ESA} \end{aligned}$$

Step 8: Determine traffic load distribution

From Table 4-4, the traffic loading for a design for a 5 m carriageway is 80% of the ESAs in both directions.

Traffic loading = 0.8 x 186,583 = 149,266
= 0.15 MESA

Step 9: Select traffic load class

From Table 3-5 : Traffic Load Class = TLC 0.3

Appendix 4.2: Use of the DCP

The DCP test shall be carried out in accordance with *ASTM D6951 / D6951M – 18: Standard Test Method for the use of the Dynamic Cone Penetrometer in Shallow Pavement Applications*.

When carrying out a DCP test, whether in the field or in the laboratory, attention should be paid to the following issues :

- The equipment must be held vertically at all times. Any deviation from the vertical will result in difficulties in getting repeatable readings from the measuring staff. In addition, the friction effects between the falling mass and the upper rod reduce the energy imparted to the cone.
- The hammer must touch the base of the handle before being released, without jolting the equipment vertically. The hammer should be released to fall under its own mass and not “thrown” down.
- When testing “hard” materials, the hammer will often bounce a number of times on the anvil before coming to rest. It should not be lifted for the next drop before coming to total rest.
- The test should start with the upper portion of the shoulder of the cone flush with the surface of the layer being tested.
- During testing, it is common to note that uplift or mounding of the layer around the DCP hole occurs. This may result in a gradual rise of the measuring staff relative to the equipment and hence a reduction in the reading being obtained. Care should be taken that the base of the measuring staff is not affected by this “mounding”.
- The cones will suffer from wear and deformation, particularly when testing hard materials, and need to be replaced periodically. Prior to any test, the condition of the cone should be checked to ensure that the point is sharp, the whole cone is screwed into the shaft, and the lower surface is not excessively rough (see Photograph 2A-1). High tensile or tempered cones are not recommended as they tend to shear off when striking a hard stone.

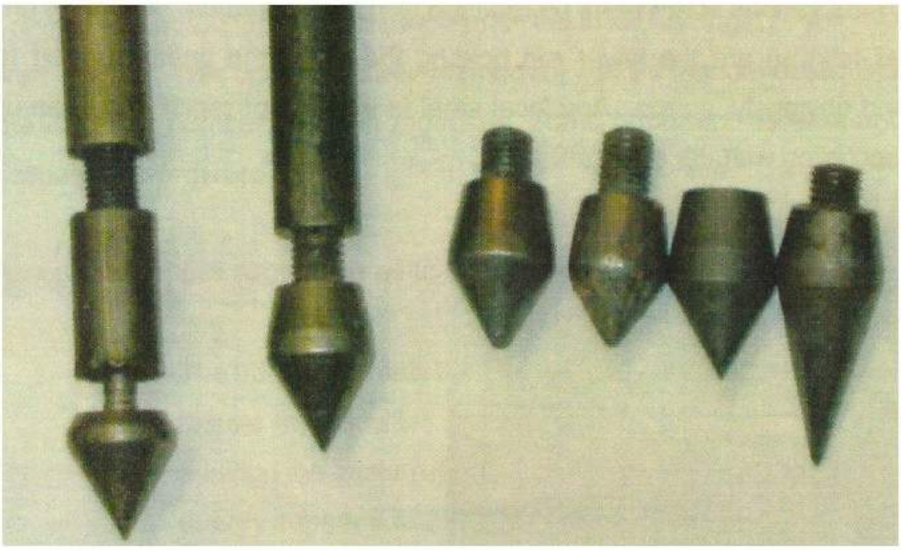


Figure 4-9: Typical cones and problems

(From left to right: disposable cone, conventional 60° cone, worn cone tip, worn cone, incorrect cone with shoulder too wide, 30° cone)

Appendix 4.3: DCP field form

Table 4-12: Typical DCP form and example measurements

DCP Test Measurements							
Project Name: xxxxxxxxx xxxxxx							
Chainage (km):	0.060			Surface Type:	Unpaved		
Direction:	West			Thickness (mm):	-		
Location/offset::	Lane 1/ 2.00 m			Base type:	Natural gravel (laterite)		
Zero Error (mm):	44 mm			Thickness:	120 mm		
Test date:	06.08.2010			Surface Moisture:	Dry		
No.	**Blows**	**Cumulative**	**Penetration**	**No.**	**Blows**	**Cumulative**	**Penetration**
1	0	0	44	26	5	125	379
2	5	5	62	27	5	130	388
3	5	10	72	28	5	135	411
4	5	15	86	29	5	140	426
5	5	20	93	30	5	145	441
6	5	25	101	31	5	150	460
7	5	30	115	32	5	155	478
8	5	35	131	33	5	160	497
9	5	40	148	34	5	165	510
10	5	45	166	35	5	170	519
11	5	50	183	36	5	175	546
12	5	55	202	37	5	180	563
13	5	60	222	38	5	185	578
14	5	65	238	39	5	190	597
15	5	70	259	40	5	195	610
16	5	75	271	41	5	200	629
17	5	80	288	42	5	205	643
18	5	85	300	43	5	210	684
19	5	90	310	44	5	215	710
20	5	95	321	45	5	220	736
21	5	100	332	46	5	225	758
22	5	105	341	47	5	230	790
23	5	110	358	48	5	235	830
24	5	115	365	49	5		
25	5	120	372	50	5		

5. Materials

5.1 Introduction

5.1.1 Background

Naturally occurring soils, gravel soil mixtures and gravels occur extensively in many tropical and sub-tropical countries of the world. These unprocessed materials are a valuable resource as they are relatively cheap to exploit compared, for example, with processed materials such as crushed rock, and are often the only source of material within a reasonable haul distance of the road alignment. Thus, in order to minimise construction costs, maximum use must be made of locally available materials. However, their use requires not only a sound knowledge of their properties and behaviour but also of the traffic loading, the physical environment, and their interactions.

Although many naturally occurring materials do not meet conventional specifications that are generally more appropriate for high volume roads, they can, nonetheless, still provide satisfactory performance on LVRs. Their use must, therefore, be based on appropriately developed selection criteria and laboratory testing, coupled with attention to construction techniques.

5.1.2 Purpose and Scope

The purpose of this chapter is to provide the background for the general understanding of the approach to selecting and using materials for the construction of LVRs in an economical and sustainable manner and to ensure that satisfactory levels of quality are attained.

The chapter covers the range of construction materials typically found in tropical and sub-tropical countries. The benefits of using such materials are highlighted, as well as the approach to testing and selecting them for use in the pavement structure.

5.2 Material Types

5.2.1 General

Materials for the structural layers in LVRs will usually consist of local gravels derived from weathering of in-situ rock or materials that have been transported by some natural force (e.g., water, wind, ice, gravity). The use of expensive aggregate derived from the crushing of hard rock should be minimized, such materials typically being used solely for bituminous surfacings or concrete structures.

The main types of materials that may be considered for use in LVRs include:

- Weathered and residual materials
- Transported materials
- Pedogenic materials

The characteristics of the above material types are discussed briefly below.

5.2.2 Weathered and residual materials

Chemical weathering of rocks causes the alteration of the minerals in the rocks (except quartz, which is relatively resistant) to form different minerals, mostly clays, and changes the hard rock to a residual material that could be used as natural gravel for road construction. These materials are of particular interest for use in LVRs as they can be easily worked without requiring expensive blasting or heavy equipment for ripping. However, the quality and durability of borrow materials and crushed stones can be greatly affected by the weathering or alteration processes.

The type and rate of weathering vary from one region to another. In the tropics, high temperatures associated with high humidity often produce physical and chemical changes to a considerable depth in surface rocks. In drier areas, weathering is predominantly physical, and rock masses disintegrate by alternate heating and cooling and wetting and drying, but still keep their general appearance. In more humid areas, chemical weathering proceeds quite rapidly, and rock masses may be partially or completely weathered.

Figure 5-1: Variable weathering in borrow source

Weathering effects generally decrease with depth, although zones of differential weathering can occur in many outcrops. Of equal importance, however, is the presence of thick deposits of transported materials in the foothills of mountains, which can make good construction gravels, although they often require some single-stage crushing. The properties of the final residual material will, however, depend on the mineralogy of the parent rock and the climatic regime causing the weathering.

Table 5-1 summarizes the typical properties of the residual gravels obtained from the chemical weathering of various rock types. This should only be seen as a guide, as many local conditions (e.g., perched water tables, good drainage conditions) could affect the actual individual properties.

Table 5-1: Typical properties of residual materials derived from various rock types

Rock type	Typical rock types	Dominant particle sizes	Plasticity	Material strength
Acid crystalline	Granite, gneiss, felsite, syenite	Sands and gravels	Low to medium	Medium to high
Basic crystalline	Basalt, lava, schist, dolerite, andesite	Silts and clays	Medium to high	Low to medium
High silica	Quartzite, chert, hornfels	Gravels	Low	Medium to high
Arenaceous	Sandstone, arkose	Sands	Low to medium	Medium
Argillaceous	Shale, schist, slate	Clays	Medium to high	Low
Carbonate	Limestone, marble, dolomite,	Mixed gravels	Low	Medium to high
Diamictite	Tillite, greywacke	Mixed gravels	Low to high	Low to high
Pedogenic	Calcrete, laterite, silcrete, gypcrete	Mixed gravels	Low to high	Low to high

5.2.3 Transported Materials

Surficial soils can be moved to different locations by wind, rain, rivers, ice or gravity. During this process, the properties of the materials change as large particles are broken down, finer materials are removed, and the sorting of different size fractions may occur. Such transported materials are found in the foothills of the mountains, in river valleys, in desert and inland deposition areas, and can occur almost everywhere except on steep slopes. Many of these can make suitable construction gravels, although they often require some single-stage crushing.

Figure 5-2: Typical deposit of transported sandy material

5.2.4 Pedogenic Materials

Pedogenic materials, collectively known as pedocretes or pedogenic soils, are unique types of soil in which the existing material is "fully or partially cemented" by certain minerals. Typical cementing materials include iron and aluminium oxides, calcium carbonate, and, to a lesser extent, silica. These materials can be formed by:

- an accumulation of the cementing material resulting from the leaching or washing out of soluble bases leaving material rich in the cementing materials; or
- an accumulation of the cementing material where the cementing material is carried in solution and deposited/precipitated in an existing soil somewhere else to cement the existing material particles together.

Depending on the quantity of cementing material, pedogenic soils can be exceptionally good construction materials, although they frequently do not comply with existing material specifications. Experience with their testing and use will provide a good understanding of their properties and a knowledge of how best to use these materials. It should be noted, however, that their unique properties usually require special sample preparation methods.

Some pedogenic materials have the property of self-stabilisation (or self-hardening), in that, with time, the properties improve apparently as a result of alternating dissolution and precipitation or changes in the chemistry of the cementing materials. This can certainly have benefits in the long term, but the use of these materials should ensure that the pavement layers have sufficient strength immediately after construction and opening to traffic and prior to the development of any self-stabilisation. The major benefits are that in the long-term, the materials become less susceptible to moisture-related damage.

Two of the most common types of pedogenic materials that are found in tropical and sub-tropical countries are laterite and calcrete. Their characteristics and properties are discussed briefly below.

Laterite

Laterite, in particular, is generally a suitable construction material for all layers up to the base course, if the material properties are tested correctly and understood. A large number of factors control how a particular type of laterite is developed, and the material tends to exhibit both vertical and lateral variability within an often deep and irregular weathering profile. Two typical types of laterite are illustrated in Figures 5-3 and 5-4 below.

Figure 5-3: Nodular laterite stockpile

Figure 5-4: Hardpan laterite in quarry

The behaviour of lateritic materials in pavement structures depends mainly on their iron and aluminium oxide (sesquioxide) contents, particle size characteristics, the nature and strength of the gravel-sized particles, the degree of compaction as well as traffic and environmental conditions. The most important requirements for a laterite pavement to perform well are that the material is well graded with a high content of hard particles and adequate fines content. However, when judging the gradation of lateritic gravel, it is important to assess its composition to decide if separate specific gravity determinations of the fines and coarse fractions should be made. For example, for nodular laterites, the coarse fraction is iron-rich whilst the fine fraction is often mostly quartz and kaolinite.

The requirements for selection and use of lateritic gravels for pavement layers are different from those typically specified for other natural gravels, and this needs to be taken into account during their testing. Conventional testing using oven drying, for instance, can have a major effect on the test results. Other aspects, such as mixing times for the Atterberg limits, can also affect the results. For these reasons, assessing the qualities of laterites as road building materials in terms of conventional specifications must be undertaken carefully.

Calcrete

Calcrete is formed in place either by cementation or replacement–sometimes both – of pre-existing soils, usually by calcium carbonate and, to a lesser extent, by magnesium carbonate precipitated from the soil or groundwater. This results in the original material being transformed into a new one - calcrete - comprising varying quantities of $CaCO_3$ whose properties vary from an almost pure, very hard, massive limestone, in which there is hardly any trace of the host material, to a very loose material consisting largely of the host material (Figure 5-5).

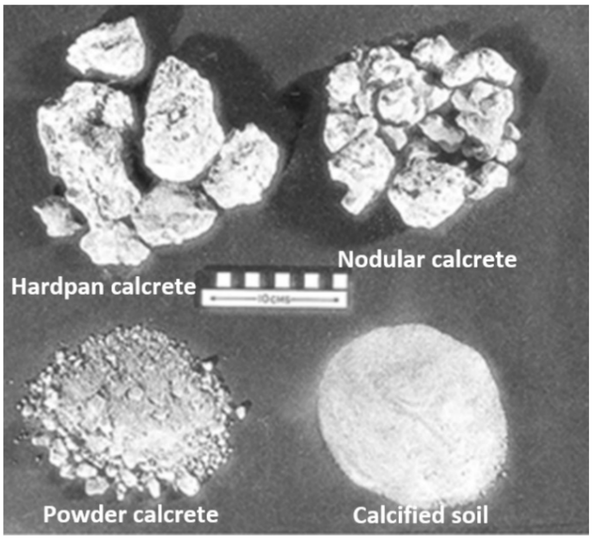


Figure 5-5: Typical types of calcrete

The most commonly encountered clay minerals in calcretes are palygorskite, montmorillonite and sepiolite. These minerals possess a number of unusual properties, most of which are likely to be beneficial to a road material. For example, palygorskite clay (and probably sepiolite) possesses a far greater shear strength at the same moisture content than other clays. Calcretes, therefore, possess a composition that is unusual among road materials. Because of the high carbonate content, the unusual clay minerals present, and the presence of compound porous particles and amorphous silica micro-fossil remains, they can be expected to exhibit some unusual properties, including a form of self-cementation that can result in an increase in strength. Moreover, although some of the properties of calcrete, such as their plasticity and grading, do not conform with traditional requirements, this material can still provide satisfactory performance in a LVR pavement.

5.3 Use of Locally Available Materials

5.3.1 General

Making maximum use of naturally occurring, unprocessed materials is a central pillar of the LVR design philosophy. In so doing, a key objective is to match the available construction materials to the road function and environment. Conventional specifications, when applied to LVRs, tend to limit the use of many naturally occurring, unprocessed materials in upper pavement layers in favour of more expensive crushed rock or other processed materials. However, recent research has shown quite clearly that so-called “non-standard” materials can often be used successfully and cost-effectively in LVR pavements provided appropriate precautions are observed. These precautions include effective drainage of the pavement structure, good construction practices and quality conrol, and regular maintenance, as discussed in other chapters.

5.3.2 Optimum Utilisation of Local Materials

To minimise the cost of LVR projects, the materials used in their structural layers should be sourced as close to the project as possible (haulage costs are frequently the highest component of material provision) and should be of the most appropriate standard for the respective layer. This means that the material should:

- Provide the necessary strength and stiffness for the proposed layer in the road.
- Be able to retain those properties over the design life, and preferably longer, under the impacts of traffic and climate.

It is essential that appropriate specifications are adopted, considering the main properties required, and that appropriate test methods are used (Section 5.5.3). Thus, the design philosophy should be changed from “finding materials to suit the proposed pavement design” to “designing the pavement to suit the available materials”.

The main requirements of materials in the structural layers of roads are to provide adequate strength and stiffness to avoid shear failure or traffic-induced compaction in the subgrade and to retain these properties over the expected life of the road, i.e., to be durable. It should be noted that traffic-induced compaction (or rutting) can be almost eliminated by ensuring that all materials in the pavement are compacted to as high a density as possible (exclusion of as many voids as possible).

Recent developments and research have indicated that plasticity is, in fact, inherent in the strength of a material, together with other properties. Thus, provided the strength requirements are met, the plasticity is of secondary importance. Nonetheless, it does provide an indication of the potential moisture susceptibility of the material. One of the problems of prescribing various "interrelated" properties, in terms of specifications, is that if a material fails to satisfy one of these specifications, it will be rejected for use. In this way, materials that easily satisfy the "most important" strength criterion, but are marginally deficient on the plasticity or grading, are often rejected for use.

Despite the innumerable differences that exist among local materials, there are some dominant characteristics that affect pavement performance, which should be appreciated to design and construct LVRs using such materials with confidence. These characteristics depend on whether the materials are used in an unbound or bound state, which affects the way they derive their strength in terms of the following intrinsic properties:

- Inter-particle friction.
- Cohesive effects from fine particles.
- Soil suction forces.
- Physico-chemical (stabilization) forces.

The relative dependence of a material on each of the above components of shear strength, and the influence of moisture and density, significantly influences the way the material can be incorporated within a pavement. In this regard, Table 5-2 summarises the typical relative characteristics of unbound and bound materials that critically affect the way in which they can be incorporated into a pavement in relation to their properties and the prevailing conditions of traffic, climate, economics and risk.

Table 5-2: Pavement material types and characteristics

Parameter	Pavement Type			
	Unbound			Bound
	Unprocessed	Moderately processed	Highly processed	Very highly processed
Material types	Category 1 As-dug gravel	Category 2 Screened gravel	Category 3 Crushed rock	Category 4 Stabilised gravel
Variability	High	Decreases		Low
Plasticity Modulus	High	Decreases		Low
Development of shear strength	Cohesion and suction.	Cohesion, suction & some particle interlock.	Particle interlock	Particle interlock & chemical bonding
Susceptibility to moisture	High	Decreases		Low
Design philosophy	Material strength maintained only in a dry state	Selection criteria reduce the volume of moisture sensitive, soft and poorly graded gravels		Material strength maintained even in a wetter state
Appropriate use	Low traffic loading in a very dry environment	Traffic loading increases, environment becomes wetter		High traffic loading in wetter environments
Cost	Low	Increases	High	High
Maintenance requirement	High	Decreases		Low
(shaded)	Of particular significance to LVSRs			

Unprocessed materials (Category 1), such as laterite, are highly dependent on soil suction and cohesion forces. Gravel components will also add some interlock and friction, provided the particles are strong enough to avoid excessive breakdown, for the development of shear resistance, which will only be generated at relatively low moisture contents. Consequently, special measures must be taken to ensure that moisture ingress into the pavement is prevented; otherwise, suction forces and shear strength will be reduced, which could result in failures (Figure 5-6).

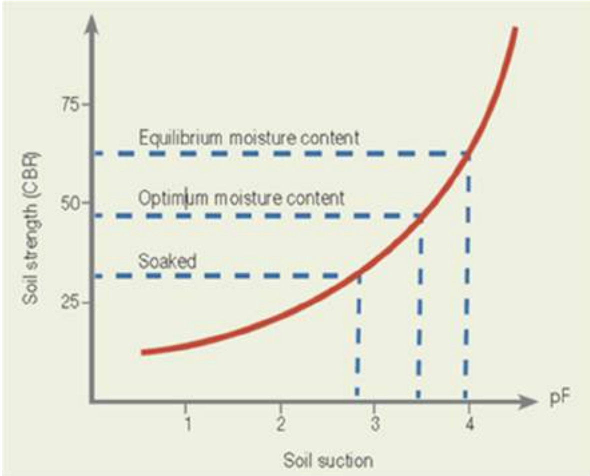


Figure 5-6: Illustrative soil strength-soil suction relationship

Since most LVRs are constructed from unbound materials, a good knowledge of the performance characteristics of such materials is necessary for their successful use as discussed below:

- **Category 1 materials:** are highly dependent on soil suction and cohesive forces for the development of shear resistance. The typical deficiency in hard, durable particles prevents reliance on inter-particle friction. Thus, even modest levels of moisture, typically approaching 60% saturation, may be enough to reduce confining forces sufficiently to cause distress and failure.

- **Category 2 materials:** have a moderate dependency on all forms of shear resistance – friction, suction forces and cohesion. Because these materials have rather limited strength potential, concentrations of moisture, typically 60-80% saturation, may be enough to reduce the strength contribution from suction or cohesion sufficiently to cause distress and failure. This would occur at moisture contents lower than those necessary to generate pore pressures.

- **Category 3 materials:** have only minor dependency on suction and cohesion forces but have a much greater reliance on internal friction, which is maximized when the aggregate is hard, durable and well graded. Very high levels of saturation, typically 80-100%, are necessary to cause distress, and this will usually result from pore pressure effects.

- **Category 4 materials:** rely principally on physio-chemical forces which are not directly affected by water. However, the presence of water can lead to distress under repetitive load conditions through layer separation, erosion, pumping and breakdown.

The management of moisture during the construction and operational phases of a pavement affects its performance, especially when unbound, unprocessed, relatively plastic materials are used. It is therefore very clear that emphasis should be placed on minimising the entry of moisture into a LVR pavement to ensure that it operates as much as possible at an unsaturated moisture content.

5.4 Materials Testing

5.4.1 General

The samples collected during the route survey must be tested in the laboratory to assess their properties and suitability for possible use in the pavement. All testing should comply with local standards and should not be mixed (e.g., AASHTO with BS). The test methods must be meticulously followed, using the correct and recently calibrated equipment.

5.4.2 Approach to Evaluation of Materials

The approach to the evaluation of subgrade/earthworks and pavement layer materials is based on consideration of the following:

(a) Knowledge of the key engineering properties of the subgrade/earthworks and pavement materials in order to detect those materials with deleterious properties associated with "problem soils", such as excessive swell, erodibility, or collapse potential. This is obtained from traditional classification, grading, and other appropriate tests carried out on at least two bulk samples obtained from each uniform section along the road.

(b) The selection of materials in terms of acceptability for specific use in the subgrade or pavement layers is then based on engineering judgment related to the outcome of the above tests, bearing in mind the preference for local material use on LVRs.

(c) Knowledge of the key parameter required in a pavement layer - the in-situ shear strength of the material - which is a function of the material properties, including grading and plasticity. This parameter is strongly correlated to the laboratory DN value of the material, which is determined at the highest practicable density anticipated in the field ("compaction to refusal") and at the anticipated EMC in the pavement of the upgraded road. Thus, as discussed in *Chapter 3 – Approach to Design, Section 3.3.4*, the finally specified material selection parameter is a DN value which represents a composite measure of the key interacting variables that affect material strength, i.e., compacted density, moisture content, grading and plasticity (i.e., the PM of the soil). This approach avoids potentially suitable materials being rejected on the basis of one or other of the traditionally specified parameters not being complied with, even though the strength, represented by the DN value, may be adequate.

5.4.3 Laboratory Testing

Samples of the local in situ materials, as well as the possible construction materials collected during the soil survey, should be tested in an approved laboratory for routine properties that allow the material to be classified. Classification testing will include particle size distribution, plasticity, durability (where appropriate) and compaction characteristics (maximum dry density and optimum moisture content), and material strength.

The laboratory testing program provides all of the information needed to determine the characteristics, potential use, and available volume of construction materials.

The program will comprise the following tests:

- Grading and Atterberg Limits
- Mineralogical and Durability
- Compaction
- Determination of material strength

See also *Chapter 4 – Surveys and Investigations, Tables 4-7 and 4-8.*

Grading and Atterberg limits: Grading envelopes and Atterberg limits (PI and PM) are not specified for the DCP-DN method. Nonetheless, the standard tests to determine these parameters must be carried out for all material samples to enable the design engineer to consider their influence on the strength of a material in service, as discussed in *Chapter 3 – Approach to Design*, *Section 3.3.4 - Assessment of subgrade/pavement layer strength.*

Limits on the material grading are specified as a prerequisite for subsequent testing to exclude overly fine or coarse materials from being considered for use in the pavement layers. The Grading Modulus (GM) is calculated by the following formula:

$$GM = [300 - (P2 + P425 + P075)]/100$$

where P2, P425 and P075 denote the percentages passing through the 2.0 mm, 0.425 mm and 0.075 mm sieve sizes, respectively.

Mineralogical and Durability tests: The use of residual basic igneous rock (including ophiolites, basalt and dolerite/diabase) gravels could result in significant savings provided the characteristics of the material are good enough to serve as a pavement material. The following are indicative limits:

- Maximum secondary mineral content of 20 % (determined from petrographic analysis).
- Maximum loss of 12 or 20 per cent after 5 cycles in the sodium or magnesium sulphate soundness tests, respectively (ASTM C88-13).
- Clay index of less than 3 in the methylene blue absorption test (ASTM C837-09).

The use of residual soils derived from a basic igneous rock is potentially problematic if the limits stated above are exceeded.

Compaction: Standard compaction tests shall be carried out to determine the Maximum Dry Density (MDD) and the Optimum Moisture Content (OMC) of the material. At least three tests must be carried out on material from the same sample to determine the average values, which are then taken as the representative value for the particular material.

It is recommended that for the DCP-DN design method, each specimen shall be penetrated with the DCP to get a measure of the DN value at the different moisture contents and densities, as illustrated in Figure 5-7.

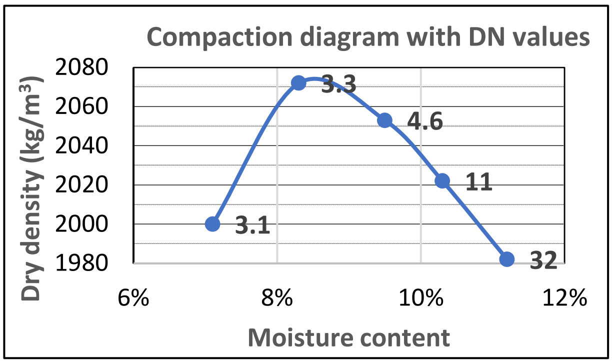


Figure 5-7: Dry density curve with DN values (mm/blow) determined on each mould

Determination of material strength: The method for determination of material strength constitutes the greatest difference between the DCP-DN method and other CBR-based design methods. The procedure for undertaking the laboratory DN test is described in Appendix 5.1, whilst a typical output of the testing programme is illustrated in Figure 5-9.

5.5 Material specifications

Provided all materials are assessed as suitable, as discussed above, then the three parameters that need to be specified for the imported pavement layers are as follows:

- **Grading modulus:** 1.0 ≤ GM ≤ 2.25
- **Maximum aggregate size:**
  - Base/Wearing course: ≤ 37.5 mm
  - Subbase: ≤ 63 mm or 2/3 of layer thickness
- **DN value:** The DN value of the materials to be used at the anticipated design moisture content and minimum density and, as per the DCP-DN structural design catalogue. The pavement layers must be compacted to the highest practicable density, i.e., “compaction to refusal”.

Bibliography

AfCAP (2013). ***Guideline on the Use of Sand in Road Construction in the SADC Region.*** http://r4d.dfid.gov.uk/pdf/outputs/AfCAP/AfCAP-GEN028-C-Sand-in-Road-Construction-Final-Guideline.pdf

ASTM C88-13 (2013). ***Standard Test Method for Soundness of Aggregates by Use of Sodium Sulphate or Magnesium Sulphate***, ASTM International, West Conshohocken, PA., USA.

ASTM C837-09 (2014), ***Standard Test Method for Methylene Blue Index of Clay***, ASTM International, West Conshohocken, PA., USA

Ayers M E, Thompson M R and Uzarski D (1989). ***Rapid Shear Strength Evaluation of In-situ Granular Materials.*** Transportation Research Record 1227.

Botswana Roads Department (2010). ***The Use of Kgalagadi Sands in Road Construction.***

Brink A B A (1985). ***Engineering Geology of Southern Africa.*** Building Publications, Pretoria, South Africa.

Construction Industry Research and Information Association (CIRIA) (1995). ***Laterite in road pavements***, Special Publication 47, CIRIA, London, UK.

Geological Society (1990). ***Tropical Residual Soils***. Special Publication. Geological Society, UK.

Grace H and D G Toll (1987). ***Recent Investigations into the Use of Plastic Laterites as bases for Bituminous-Surfaced Low-Volume Roads***. Proc. Fourth Int. Conf. on Low-Volume Roads, TRB, Washington, D.C., USA.

Cook J R, Bishop E C, Gourley C and N E Elsworth (2001). ***Promoting the use of marginal materials***. TRL Report PR/INT/205/2001. TRL, Crowthorne, Berkshire, UK.

Grace H (1991). Investigations in Kenya and Malawi using as-dug laterite as bases for bituminous surfaced roads. Geotechnical and Geological Engineering 9, 183–95.

Krill J L, Cratchley C R, Early K RGallois RW, Humphreys J D, Newbery J, Price D G and R G Thurrell (1970). ***The logging of rock cores for engineering purposes***. Geological Society Engineering Group Working Party. December 1970 Quarterly Journal of Engineering Geology and Hydrogeology 3(ii):1-24

McLennan A K (1986). ***Towards a strategy for the use of marginal and naturally occurring materials in pavements.*** 24th ARRB Regional Symposium, Bundaberg, Queensland, Australia.

Metcalf J B (1991). ***Use of naturally occurring but non-standard materials in low-cost road construction,*** Geotechnical and Geological Engineering, 9.

Mitchell M F, Petzer E C P and N Van der Walt (1979). ***The optimum use of natural materials for lightly trafficked roads in developing regions***. Transp. Res. Record 702, Washington, DC., USA.

Netterberg F and Paige-Green P (1988). ***Pavement materials for low volume roads in Southern Africa: A review.*** Proceedings ATC Conference, Vol. 2B – Appropriate Materials and Methods. Pretoria, South Africa.

Netterberg F (1994). ***Low-cost local road materials in southern Africa***. Geotechnical and Geological Engineering, **12**, 35-42.

Paige-Green P (1994). ***Recommendations for the use of marginal base course materials in low volume roads in South Africa***, CSIR Transportek, Pretoria, South Africa.

Southern African Transport and Communications Commission (SATCC). (1994). ***Standard specifications for road and bridge works.*** Maputo: SATCC-TU.

TRL (Undated). ***Guidelines for the Selection and Use of Construction Materials***. Project Report R6898. Crowthorne, Berkshire, UK

Weinert H H (1980). ***The natural road construction materials of Southern Africa***. Pretoria, South Africa: Academica, Cape Town, South Africa.

Appendix 5.1: Laboratory DN Test

The laboratory DN test is central to the DCP-DN design method, and it is crucially important that it is carried out strictly in accordance with the procedure described below and to the highest standards. It is used both for evaluating imported materials for new pavement layers as well as for the determination of the in-situ subgrade strength. The following explains in detail how to carry out the laboratory DN test as well as a full testing program to characterize construction materials. However, in practice, the full testing program may not be necessary, following the sample preparation procedure described below.

Preparation of test samples

The samples must be prepared in accordance with the procedure described below:

Procedure 1 – Scalping Method: This applies to materials that have 30% or more (by weight) retained on the 20 mm sieve, and may be summarized as follows:

- Remove material passing the 37.5 mm sieve and retained on the 20 mm sieve and lightly crush by means of a steel tamper so that all the material passes the 20 mm sieve.
- Recombine a portion of the crushed material, representing 30% by mass of the original sample, with the rest of the original sample and mix thoroughly before testing.

Procedure 2 - Crushing Method: This applies to materials that have 30% or less (by weight) retained on the 20 mm sieve, and may be summarized as follows:

- Screen field sample on 20 mm sieve.
- Remove material retained on the 20 mm sieve and lightly crush by means of a steel tamper so that all material passes the 20 mm sieve.
- Recombine the crushed material with the rest of the original sample and mix thoroughly before testing.

Note: Care should be taken that the aggregate is not crushed unnecessarily small. If the material contains soil aggregations, these should be disintegrated as finely as possible with a mortar and pestle without reducing the natural size of the individual particles.

Some natural, particularly pedogenic gravels (e.g., laterite, calcrete) can exhibit a self-cementing property in service, i.e., they gain strength with time after compaction. This effect must be evaluated as part of the test procedure by allowing the samples to cure/equilibrate prior to testing in the manner prescribed below:

Thoroughly mix and split each borrow pit sample into nine sub-samples for DN testing in a CBR mould at three moisture contents and three compactive efforts, as shown in Table 5-3.

Table 5-3: Matrix for a full laboratory DN test programme

Compactive effort*	Moisture regime		
	Soaked	OMC	0.75 OMC
Light (2.5 kg rammer, 3 layers, 55 blows/layer)	3 samples	3 samples	3 samples
Intermediate (4.5 kg rammer, 5 layers, 25 blows/layer)	3 samples	3 samples	3 samples
Heavy (4.5 kg rammer, 5 layers, 55 blows/layer)	3 samples	3 samples	3 samples

*The terms "Light", "Intermediate" and "Heavy" refer to different compactive efforts and are not linked to any international standard.

The compacted samples should be allowed to equilibrate for the periods shown below before DN testing is carried out to dissipate pore-water pressures and compaction stresses and to allow the moisture content to equilibrate within the sample.

- **4-days soaked:** After compaction, soak for 4 days, allow to drain for at least 15 minutes, then undertake a DCP test as described below in the CBR mould to determine the soaked DN value.
- **At OMC:** After compaction, seal in a plastic bag and allow to "equilibrate" for 7 days (relatively plastic, especially pedogenic, materials (PI > 6)), or for 4 days (relatively non-plastic materials (PI < 6)), then undertake a DCP test in the CBR mould to determine the DN value at OMC.
- **At 0.75 OMC:** Air dry the compacted samples in the sun (pedogenic materials) or place the sample in the oven to a maximum of 50°C (non-pedogenic materials) to remove moisture. Check from time to time to determine when sufficient moisture has been dried out to produce a sample moisture content of about 0.75 OMC (it doesn't have to be exactly 0.75 OMC, but as close as possible). Once this moisture content is reached, seal the sample in a plastic bag and allow to cure for 7 days (pedogenic materials) or for 4 days (non-pedogenic materials) to allow moisture equilibration before undertaking the DCP test at approximately 0.75 OMC. Weigh again before DCP testing to determine the exact moisture content at which the DN value was determined.

Test procedure

The procedure to be followed for determining the DN value of a material is similar to that for the more traditional CBR test except that a DCP is used to penetrate the CBR mould instead of the CBR plunger.
Each of the specimens should be subjected to DCP testing in the CBR mould as summarized below.

(a) Secure the CBR mould to the base plate, place the mould on a level (preferably concrete) floor, and place the annular weight on top of the mould.

(b) Measure the height of the compacted specimen inside the mould. This is to enable the operator to stop the test just before the tip of the cone hits the base plate.

(c) Place an empty CBR mould upside down or another device (e.g., bricks or cement blocks) next to the full mould, as shown in Figure 5-8 to support the base of the DCP ruler level with or slightly higher than the top of the full mould.

(d) Position the tip of the DCP cone in the middle of the CBR mould, hold the DCP in a vertical position, knock it down carefully until the top of the 3 mm shoulder of the cone is level with the top of the sample and record the zero reading.

(e) Knock the cone into the sample with "n" number of blows and record the reading on the ruler after every "n" blows. At OMC and 0.75, OMC "n" may be any number between 1 and 10 depending on the hardness of the sample. At 4-days soak "n" may be 1 or 2. "n" does not have to be the same number for all measurements.

    Stop just before the tip of the cone touches the base plate, and in order not to blunt the cone (the last reading minus the "zero blows" reading must be less than the height of the sample inside the mould).

(f) Enter the test data (sample description, number of blows and corresponding readings, etc.) into the Laboratory Module of the ReCAP LVR DCP Software. With a laptop at hand, the data can be entered directly as the test is carried out.

(g) Take a representative sample from the middle of the specimen for determination of the actual moisture content at which the DN value was determined.

Analysis of the test data

A typical output from the Laboratory Module from the test of one sample is shown in Figure 5-8. The representative DN value for the specimen is taken as the slope of the "best fit" line from the middle of the mould. The DN value in the top and bottom 15 mm of the specimen often diverges from this "best fit" DN due to lack of vertical confinement at the top and possibly a higher density at the bottom of the mould.

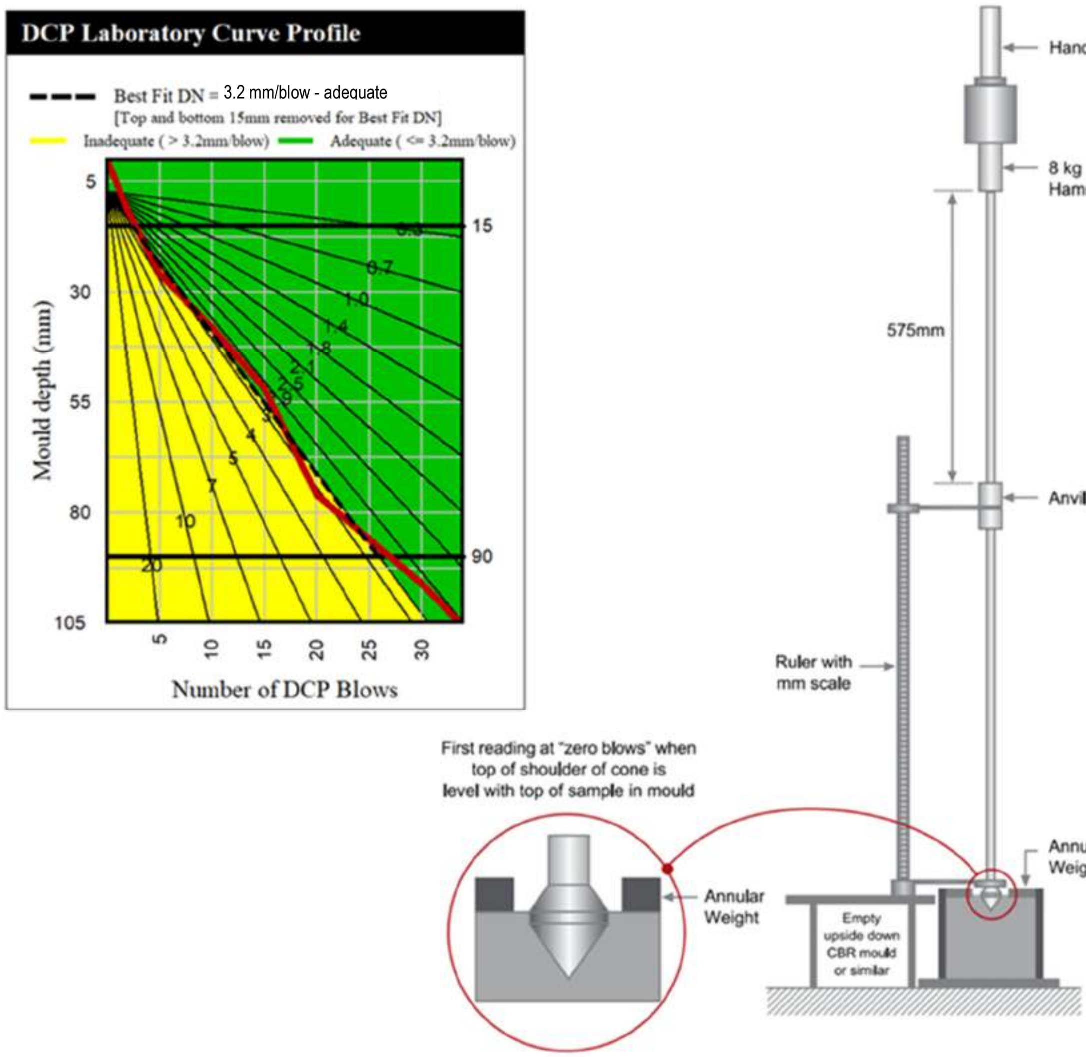


Figure 5-8: Set-up and typical output from the laboratory DN test

Note that there will always be a slight variation in the dry densities of specimens compacted with the same compactive effort and at the same moisture content. It is therefore imperative that the volume of each mould is pre-determined and that the laboratory equipment (particularly the scales) is properly calibrated to ensure that the actual dry densities of each specimen can be calculated with the required level of accuracy. The average results of a minimum for three specimens with the same compactive effort and moisture content are taken as representative values for the material.

Table 5-4 shows a summary of a typical laboratory DN test, as described above. Plot the average of the "best fit" DN values against the actual dry densities (average values of three specimens) in a diagram, as shown in Figure 5-9.

Table 5-4: Summary of typical laboratory DN test results

Compactive effort	DN mm/blow		
	Soaked	OMC	0.75 OMC
Light	11	6.4	3.6
Intermediate	6.9	4.5	2.9
Heavy	5.3	3.9	2.4

Compactive effort	Density kg/m^3		
	Soaked	OMC	0.75 OMC
Light	2165	2192	2179
Intermediate	2234	2242	2246
Heavy	2315	2329	2321

MDD 2340 kg/m^3				
Soaked	Relative compaction	92.5 %	95.5 %	98.9 %
	DN mm/blow	11.2	6.9	5.3
OMC	Relative compaction	93.7 %	95.8 %	99.5 %
	DN mm/blow	6.4	4.5	3.9
0.75 OMC	Relative compaction	93.1 %	96.0 %	99.2 %
	DN mm/blow	3.6	2.9	2.4

Figure 5-9 illustrates the relationships between DN, density and moisture content for a naturally occurring material. This will enable the designer to determine whether the material is suitable for use in the pavement, and where in the pavement it can be used based on an assessment of the anticipated long-term moisture condition in the pavement and the field density of the layer(s) after compaction, by comparison with the requirements specified in the DCP-DN design catalogue for each pavement layer.

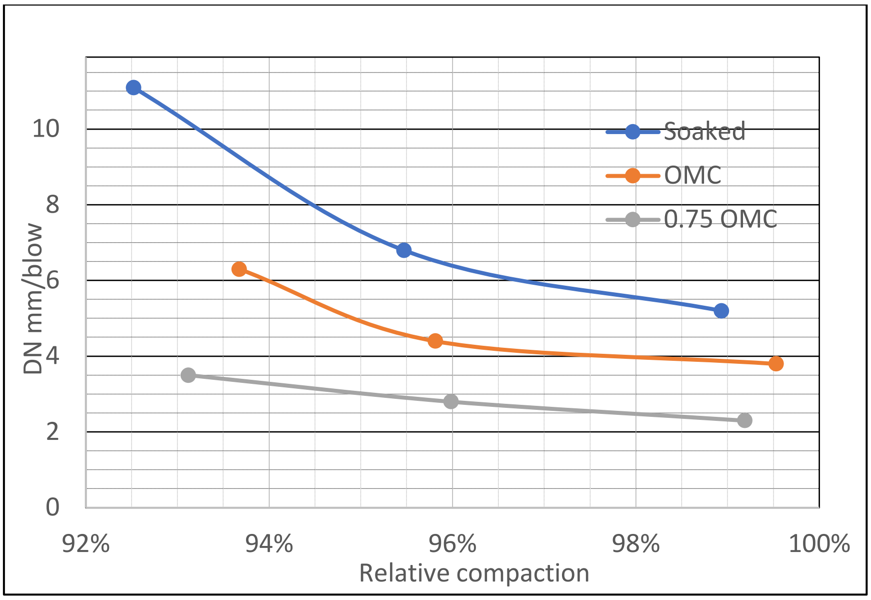


Figure 5-9: DN/density/moisture relationship

Figure 5-9 illustrates two critical factors that crucially affect the long-term performance of the road:

- The need to specify the highest level of density practicable (so-called “compaction to refusal” without incurring material breakdown) by employing the heaviest rollers available. This will result in a stronger material with lower voids and a reduced permeability, enhancing the overall properties of the material. Compaction to refusal (without degrading the material) is indicated by the number of roller-passes, established through compaction trials, at which no additional density is achieved for any specific compaction effort. Additional compaction thereafter is a waste of time and money and may result in the breakdown of individual particles of the material.
- The need to ensure that the moisture content in the outer wheel track of the road does not rise above OMC. This will require careful attention to drainage, as discussed in *Chapter 6 – Drainage and Climate Adaption.*

6. Drainage and Climate Adaptation

6.1 Introduction

6.1.1 Background

Moisture is the single most important environmental factor affecting pavement performance and long-term maintenance costs of LVRs. Thus, one of the significant challenges faced by the designer is to provide a pavement structure in which the weakening and erosive effects of moisture are constrained to acceptable limits and degree of acceptable risk. Most LVRs will be constructed from natural, often unprocessed materials, which tend to be moisture sensitive. This places extra emphasis on the provision of adequate external and internal drainage and moisture control for achieving satisfactory pavement life.

6.1.2 Purpose and Scope

The chapter deals with the sources of moisture in a pavement, the elements of both internal and external drainage, and measures for dealing with the impact of climate change.

The scope of the chapter is limited to the point at which water has entered some suitable collection system, e.g., pipes or side drains, and deals mainly with various aspects of the road surface and subsurface drainage.

6.2 Sources of Moisture in a Pavement

The various causes of water movement into and out of a pavement are listed in Table 6-1, highlighting those aspects that should be addressed when designing an effective drainage system.

Table 6-1: Typical causes of water movement into and out of a road pavement

Means of Water Ingress	Causes
Through the pavement surface	Through cracks and potholes caused by pavement failure.
	Penetration through permeable, intact layers.
From the subgrade	Artesian head in the subgrade.
	Pumping action at formation level.
	Capillary action in the subbase.
From the road margins	Seepage from higher ground, particularly in cuttings.
	Reverse falls at formation level.
	Lateral/median drain surcharging.
	Capillary action in the subbase.
	Through an unsealed shoulder collecting pavement and ground run-off.
Through hydrogenesis (aerial well effect)	Condensation and accumulation of water from vapour phase onto the underside of an impermeable surface.
Into the subgrade	Soakaway action.
	Subgrade suction.
To the road margins	Into lateral/median drains under gravitational flow in the subbase.
	Into positive drains through cross-drains acting as collectors.

6.3 External Drainage

External drainage is concerned with the control of water that is outside the road structure. There are three important components of external drainage:

- Preventing water from entering the road structure; for example, aspects of geometric design (e.g., camber) and waterproofing (e.g., surfacings).
- Collecting the water and channelling it safely away from the road.
- Allowing water to cross the road effectively from one side to the other.

6.3.1 Road Surfacing

The most effective means of preventing water from entering the road pavement from above is using a durable, waterproof surfacing that is adequately maintained over the design life of the road. There are many types of bituminous surfacings that can be used for this purpose, some being more impermeable than others.

6.3.2 Shoulders

Shoulders provide several beneficial functions, including:

- Providing structural support to the carriageway.
- Allowing wide vehicles to pass one another without causing edge damage to the carriageway.
- Providing extra room for temporarily stopped or broken-down vehicles.
- Allowing pedestrians, cyclists, and other vulnerable road users to travel in safety.

On paved roads, the whole roadway width should normally be sealed, whether shoulders are provided or not. Such sealing offers many benefits, including:

- Confinement of the zone of seasonal moisture variation to within the shoulder.
- Reduced road maintenance.
- Increased road safety.

The design of shoulders needs to be undertaken carefully if typical drainage problems are to be avoided, as illustrated in Figure 6-1.

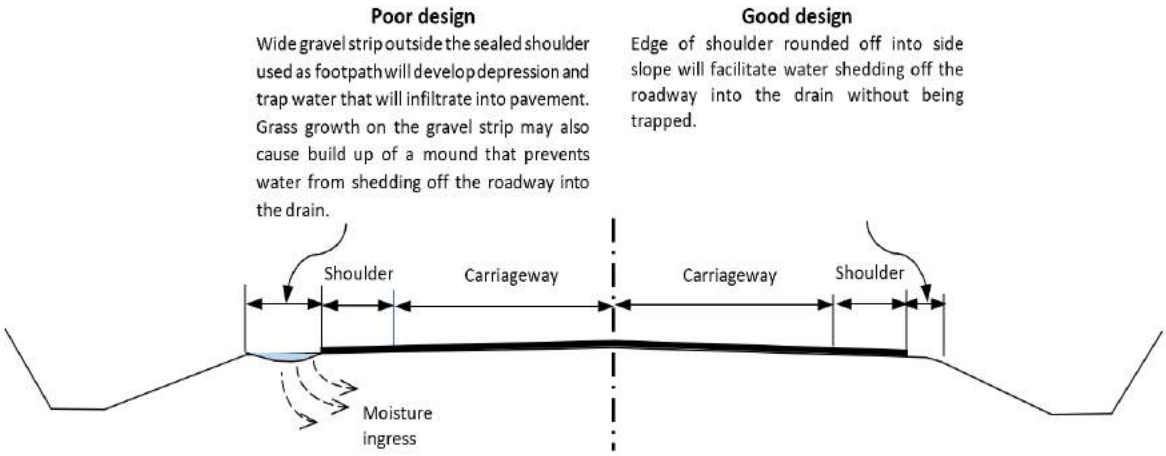


Figure 6-1: Examples of poor and good design of shoulders

6.3.3 Camber/Crossfall

Effective surface drainage is facilitated by ensuring that the road is designed with an appropriate crossfall to drain water from the road surface. Three types of crossfall may be used for this purpose, as illustrated in Figure 6-2.

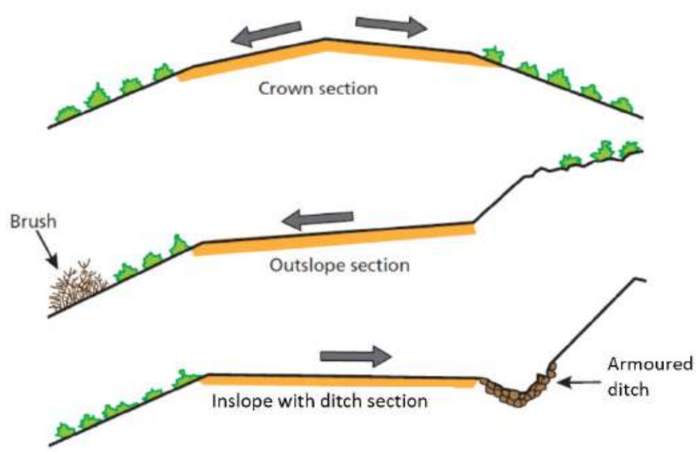


Figure 6-2: Three types of crossfall

Carriageway camber/crossfall slope: The design of the crossfall is often a compromise between the need for a reasonably steep crossfall for drainage and a relatively flat crossfall for driver comfort and safety. The ideal crossfall depends on the pavement surfacing.

On paved roads, a camber of 3.0-3.5% is recommended. Although steeper than many traditional specifications, it does not cause problems for drivers in a low-speed environment and improves climate resilience. It also accommodates reasonable construction tolerance of +/- 0.5%, thereby taking into account the skills and experience of small-scale contractors and labour-based methods (LBM) of construction. In addition, it provides an additional factor of safety against water ingress into the pavement should slight rutting occur after trafficking.

Failure to achieve the minimum values of crossfall/camber will, in combination with rutting or other minor depressions, result in possible ponding of water on the road surface, leading to potholing and eventual ingress of water into the road pavement.

Shoulder crossfall: When permeable base materials are used, particular attention must be given to the drainage of this layer. Ideally, the base and sub-base should extend right across the shoulders to the drainage ditches. In addition, proper crossfall is needed to assist the shedding of water into the side drains. A slope of about 4-6% is suitable for unpaved shoulders. However, it is not usually possible to increase the crossfall from the value used for the running surface to a greater value for paved shoulders; hence, every effort should be made during construction to ensure that the crossfall of the road running surface is correct, preferably at the upper limit of the specification range.

6.3.4 Crown Height

To achieve adequate external drainage, the road must also be raised above the level of existing ground such that the crown height of the road (i.e., the vertical distance from the bottom of the side drain to the finished road level at the centreline) is maintained at a minimum height, h_{min}. This height must be sufficient to prevent moisture ingress into the potentially vulnerable outer wheel track of the carriageway (Figure 6-3).

The recommended minimum crown height of 0.75 m applies to unlined drains located in relatively flat ground (longitudinal gradient, g, less than 1%). The recommended values for sloping ground (g > 1%) or where lined drains are used, for example, in urban or peri-urban areas, are shown in Table 6-2. The capacity of the drain should meet the requirements for the design storm return period, and sufficient mitre or cross drains should be incorporated to ensure the effective and rapid removal of water from the side drains.

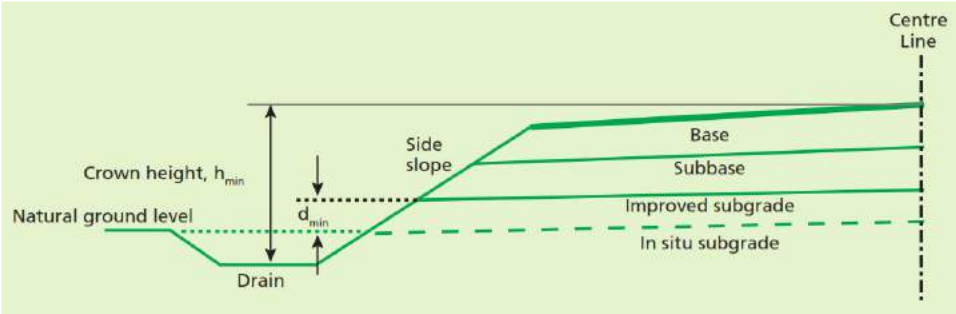


Figure 6-3: Crown height for paved road in relation to depth of drainage ditch

Table 6-2: Recommended crown height, h_{min} , above drainage ditch invert

Unlined drains		Lined drains	
Gradient < 1%	Gradient > 1%	Gradient < 1%	Gradient > 1%
0.75	0.65	0.65	0.50

In addition to observing the crown height requirements, in flat terrain, it is equally important to ensure that, where practicable, the bottom of the sub-base is maintained at a height of at least 150 mm above the existing ground level (d_{min} as indicated in Figure 6-3) to minimize the likelihood of wetting up of this pavement layer from moisture infiltration from the drain.

Irrespective of climatic region, if the site has effective side drains and adequate crown height, then the in-situ subgrade moisture will probably remain at or below OMC. If the drainage is poor, the in-situ moisture will increase above OMC with a corresponding loss of strength.

6.4 Internal Drainage

6.4.1 General

Internal drainage is concerned with the control of water that enters the road structure, either directly from above the road pavement or from below, and the measures that can be adopted to avoid trapping water within the pavement. This is an essential element of road design because the strength of the pavement layers, especially the subgrade, depends critically on the moisture content during the most likely adverse conditions. Such drainage depends primarily on the properties of the materials, including their permeability. Shoulders are also an important aspect of the internal drainage system in that they contribute to the effective drainage of water out of the structure.

6.4.2 Avoiding Permeability Inversion

A permeability inversion exists when the permeability of the pavement and subgrade layers decreases with depth. Under the infiltration of rainwater, there is potential for moisture accumulation at the interface of the layers. The creation of such a perched water table often leads to rapid lateral wetting under the seal. This may lead to base or subbase saturation in the outer wheel track and result in catastrophic failure of the base layer when trafficked.

A permeability inversion often occurs at the interface between sub-base and subgrade since many subgrades are of cohesive and relatively impermeable fine-grained materials. Under these circumstances, a more conservative design approach is required that specifically caters for these conditions, for example, designing for wetter subgrade conditions.

Preventing a permeability inversion can be achieved by ensuring that the permeability of the pavement and subgrade layers are at least equal or are increasing with depth. For example, the permeability of the base must be less than or equal to the permeability of the sub-base in a three-layered system. For a paved road, if a permeability inversion is unavoidable, the road shoulder should be sealed to an appropriate width to ensure that the lateral wetting front does not extend under the outer wheel track of the pavement.

6.4.3 Lateral drainage

Lateral drainage can also be encouraged by constructing the lower pavement layers with an exaggerated crossfall, especially where a permeability inversion (decreasing permeability as you move down the pavement layers) occurs. This can be achieved by constructing the top of the fill or lower subgrade with a crossfall of 4%-5%. Although this may cause difficulties in setting out for construction, it is still worth considering, particularly as full under-pavement drainage is rarely likely to be economically justified for LVRs. In addition, it provides some increase in pavement strength due to the slightly greater thickness of subgrade material at the outer wheel path where the structure is more vulnerable to damage. However, this ideal drainage arrangement, as illustrated in Figure 6-4, may be more difficult to achieve in practice than constructing all layers with the same crossfall.

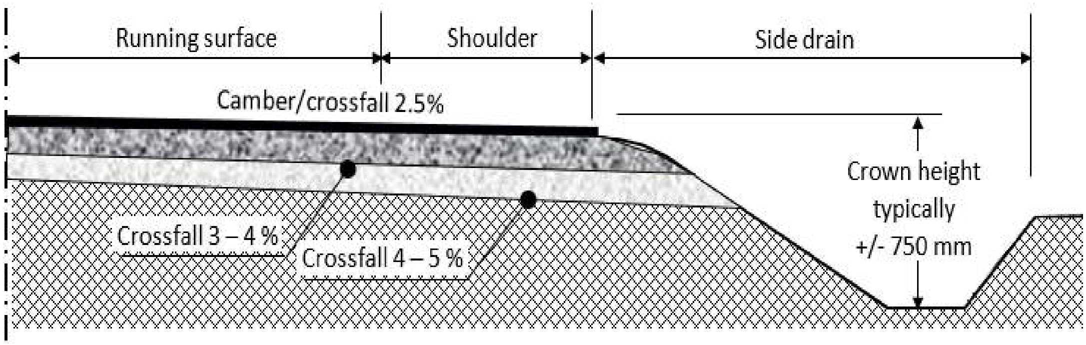


Figure 6-4: Illustrative drainage arrangements

Under no circumstances should the trench (or boxed-in) type of cross-section be used in which the pavement layers are confined between continuous impervious shoulders. As illustrated in Figure 6-5, this type of construction has the undesirable feature of trapping water at the pavement/shoulder interface and inhibiting flow into drainage ditches, which, in turn, facilitates damage to the shoulders and eventual failure under even light trafficking.

Figure 6-5: Infiltration of water through a permeable surfacing

If it is too costly to extend the base and subbase material across the shoulder, drainage channels at 3 m – 5 m intervals should be cut through the shoulder to a depth of 50 mm below the subbase level. These channels should be back-filled with a material of base quality but which is more permeable than the base itself, and should be given a fall of 1 in 10. Alternatively, a preferable option would be to provide a continuous layer of pervious material of 75 mm – 100 mm thickness laid under the shoulder such that the bottom of the drainage layer is at the level of the top of the subbase, with regular discharge points into the side drain.

6.4.4 Sub-surface drainage

Seepage may occur where the road is in cut and may result in groundwater entering the sub-base or subgrade layers. Inadequate surface or subsurface drainage can, therefore, adversely affect the pavement by weakening the soil support and initiating creep or failure of the downhill fill or slope. Localised seepage can be corrected in various ways, but seepage along more impervious layers, such as shale or clay, combined with changes in road elevation grades, may require subsurface drains as well as ditches. The depth of the seepage zone depends on several variables, including the depth of the water table, the type of soil, rock fracture and strata, etc. In practice, the seepage zone may be determined from test pits.

6.4.5 Drainage on gravel roads

The measures to ensure sufficient and adequate drainage on unpaved roads are, in principle, the same as those applicable on paved roads, including:

- Provision of an appropriate cross-section and crown height (see Chapter 8, Figure 8-1 and Table 8-1) as well as adequate camber or cross-fall (4-5%) to ensure rapid shedding of water from the road surface.
- Ensuring sufficient and appropriately positioned mitre drains and cross drainage.

The design and construction of the various components of the drainage system must be based on the current climatic and hydrological conditions in the area as well as the projected climate changes, which are dealt with below.

6.5 Adaptation to Climate Change

6.5.1 General

The projected climate change is likely to have several significant effects on the road infrastructure of most countries, but particularly on the low volume rural road network, which is more vulnerable than the higher-order network. These roads are often constructed to lower standards using local materials and labour and are thus more susceptible to climate damage than higher trafficked roads.

To reduce this impact, new roads must be designed to incorporate as many of the necessary climate adaptation measures as possible, but it may be neither practical nor economical to make every existing road fully resilient to climatic effects. Thus, it is important to identify those roads and/ or sections of roads that are not resilient and to prioritise them for adaptation measures. The priority would be based on, among other issues, the road classification and purpose, the number of people affected, and the availability of alternative routes. To implement the necessary adaptations to make roads more climate-resilient, and to assist with their prioritisation, it is necessary to carry out visual assessments of existing roads (in addition to the conventional routine assessments for pavement management purposes). Particular attention should be paid to the projected changing climatic conditions along those roads.

For existing roads and structures, it will not be possible, neither practically nor financially, to make every road and structure climate-resilient. Only those that are deemed to be particularly vulnerable should be identified and improved wherever possible. Thus, for existing LVR infrastructure, "retrofitting" the most vulnerable facilities to a climate-resilient condition is often required, but this is a costly option.

The most important and cost-effective countermeasure to meet the challenges of a changing climate is to provide regular and adequate routine and periodic maintenance, particularly addressing the surface condition and drainage. This will ensure that large parts of the road infrastructure network will be functional and will provide the intended service for most of the time.

To take account of future changes in climate, either in the design of new roads and structures, or in improving the resilience of existing infrastructure, the following should be noted:

1. The projected changes in climate along the road should be identified from the best available data, i.e., those with the most detailed and precise predictions. Such changes will generally include increased temperatures, decreased rainfall, more extreme rainfall events, and increased numbers of consecutive very hot days. More windy conditions should also be taken into account where appropriate.
2. The expected effects of the projected changes in climate on the infrastructure need to be assessed.
3. The road, road environment (earthworks and pavement drainage structures), and any larger drainage structures need to be assessed. This should follow a standard assessment protocol as outlined in the ReCAP or other relevant guidelines, and concentrate on those issues not normally assessed during standard visual assessments for Road or Bridge Management Systems.

One of the biggest problems is re-defining the storm return periods, although it is known that these will be reduced. It has been estimated that a current 1:100 return period may be as low as 1:18 by the end of the century in some countries. In the absence of any data for the specific project, it is recommended that the current return period be doubled for design purposes, i.e., a design specifying a 1: 50 storm should use the equivalent current 1: 100 storm data.

6.5.2 Engineering Adaptations

It is not possible to specify any specific adaptation measures for any specific problem. Each solution is unique and will depend on the topography, geology, geomorphology, drainage characteristics, structural design, etc. of the individual facility and location.

Adaptation measures need not be highly sophisticated, especially for low volume rural access roads, but should be the best solution that is cost-effective. Typically, it requires that the potential problems and their causes are fully identified, and that good, conventional engineering design decisions are taken. Assistance with this is provided in the ***AfCAP Engineering Adaptation Manual***.

Engineering adaptation may include measures such as:

- **Pavement sealing:** Particularly for steep gradients (> 8-10%).
- **Additional or enlarged culverts:** Additional or enlarged or improved existing cross culverts are considered essential to improve overall road drainage.

- **Side drainage:** Additional side drains and associated turnouts. Scour checks where necessary, lined drains required with gradients >6%.
- **Raised embankments**: Raising of earth embankments where the alignments are low and impacted by flooding and/or the weakening of the pavement by saturation.
- **Culvert or bridge abutment protection:** Gabion, concrete, masonry or bio-engineering protection where erosion of abutments is identified as a significant risk.
- **River/stream erosion protection:** Gabion, concrete, masonry or bio-engineering protection where erosion of the alignment by rivers or streams is identified as a significant risk.
- **Cut and fill slope protection:** Gabion, concrete, masonry or bio-engineering protection where erosion or deterioration of existing earthwork slopes is identified as a significant risk.
- **River/stream crossing:** Existing drifts and low-level structures might need to be replaced by more climate-resilient structures, such as vented fords or submersible bridges.

It is equally important that where innovative or unusual solutions are implemented, their cost-effectiveness against more conventional solutions should be assessed for future implementation.

6.5.3 Climate adaptation measures for gravel roads

The main climate impacts on gravel roads are related to the presence of excessive water on the road surface and within the pavement structure, which is one of the scenarios predicted in many tropical and subtropical countries. The impact is also strongly related to the duration and intensity of the precipitation. Increased temperatures will result in more rapid and prolonged drying out and shrinkage (and cracking) of the road structures where excessively plastic materials are used.

Adaptation measures need not be highly sophisticated, especially for low volume rural access roads, but should be the best solution that is cost-effective. Typically, it requires that the potential problems and their causes are fully identified, and that good, conventional engineering design decisions are taken.

To ensure adequate climate resilience, however, the gravel roads must not only be designed and constructed adequately but must be maintained to ensure that the road surface sheds water rapidly and effectively, without excessive erosion.

The resilience to extreme weather events and typical design considerations for gravel roads are dealt with below.

Apart from the obvious problems related to ***extreme precipitation*** events and erosion of unpaved roads, the impact of ***less rainfall*** on unpaved roads will be notable. Unpaved roads deteriorate quicker in the dry season when the moisture in the wearing course dries out, and the effective cohesion due to soil suction is lost. This results in greater dust emissions, the loss of the cohesiveness (in the dust) and the greater propensity to form loose material and corrugations.

Design considerations: A properly designed and constructed gravel road with appropriate wearing course materials, as described in this chapter, should be able to withstand even the most severe climatic effects, except for gravel roads on steep gradients where they could be impacted by erosion.

Excessive water (***increased precipitation or extreme events***) is really the only climatic attribute likely to have a significant effect on the performance of gravel roads. This water could arise from local precipitation or rainfall somewhere distant that flows down nearby rivers, causing flooding. The optimum solution in these areas is to place the road outside the normal flood limits, but this is usually impracticable.

In areas that are expected to have ***higher precipitation*** or those where more ***frequent extreme events*** are to be expected, it will be important to ensure that the minimum Shrinkage Products maintained and that the Grading Coefficients remain between the respective lower and upper limits shown, i.e., 16 and 34 for ASTM testing and 14 and 30 for the BS test methods. Materials with properties outside these limits will be particularly prone to erosion and damage under heavy precipitation.

Construction: It will be equally important that the roads are constructed properly. Construction requirements are primarily that excessive oversize material is removed (this interferes with compaction and results in roads more prone to erosion) and that the wearing course material (and shoulders) is compacted to a minimum of 98% heavy compaction effort. A good road cross-sectional shape (cross-falls of 4-6%) to allow efficient shedding of water is also essential and must be achieved during construction.

Maintenance: Regular and high-quality maintenance must be applied to the road. This must ensure that depressions (ruts or potholes) are not permitted to form in the wearing course, the shoulders are properly shaped to ensure that water runs off the wearing course into the side drains and that no windrows are left after blading that would impede removal of water from the wearing course surface.

Periodic maintenance should also be adapted to improve the resilience of the road network. During regravelling or ripping and reshaping, the material properties should be modified where necessary (typically by stone removal and/or blending of additional materials), and the construction quality can be improved where it is found to be deficient.

The generation of dust from unpaved roads is almost inevitable from most materials that have significant silt content. In areas where the ***dry/drought season*** becomes longer, it may be necessary to apply dust palliatives to selected sections of the road, such as those passing schools or clinics or where geometrics and sight distance is poor. Chemicals such as lignosulphonates and magnesium/calcium chlorides are usually cost-effective in these cases. The loss of dust also reduces the plasticity of the materials resulting in the quicker formation of corrugations, which need to be maintained more regularly. Generally, corrugations form more quickly than other defects, and dragging with tyre drags is a cost-effective means of maintaining these roads.

There will be areas where even properly engineered and constructed gravel roads are subject to on-going damage due to flooding, which may require costly and frequent maintenance. In these areas, consideration should be given to localised lengths of paved or even concrete roads. Where water flows over the road, measures to minimise turbulent flow should be installed.

Bibliography

Chaddock B C J (1992). ***A Review of the Effect of Sub-Base Permeability on Road Performance.*** TRL Working Paper WP/PE/111, TRL, Crowthorne, Berkshire, UK.

Dawson A R and A R Hill (1998). ***Prediction and Implications of Water Regimes in Granular Bases and Sub-Bases.*** Proceedings of the International Symposium on Sub-drainage in Roadway Pavements and Subgrades, Granada, Spain.

Gerke R J (1987***). Subsurface Drainage of Road Structures.*** **Special Report Number 35**. Australian Road Research Board, Australia.

Griffiths P J, Hird A B and P Tomlinson (2000). ***Rural Road Drainage for Environmental Protection.*** TRL Report PR/INT/197/00, Crowthorne, Berkshire, UK.

Le Roux A, Engelbrecht F, Paige-Green P, Verhaeghe B, Khuluse-Makhanya S, McKelly D, Dedekind Z, Muthige M, van der Merwe J and K Maditse (2016). ***Climate Adaptation: Risk Management and Resilience Optimisation for Vulnerable Road Access in Africa: Climate Threats Report.*** AfCAP Project GEN2014C.

Paige-Green P, Verhaeghe B and M Head (2017). ***Climate Adaptation: Risk Management and Resilience Optimisation for Vulnerable Road Access in Africa: Engineering Adaptation Guidelines.*** AfCAP Project GEN2014C.

Rolt J, Gourley C S and J P Hayes (2002). ***Rational Drainage of Road Pavements.*** TRL Report PR/INT/244/2002. TRL Ltd, Crowthorne, Berkshire, UK.

7. Structural Design: Paved Roads

7.1 Introduction

7.1.1 Background

The Dynamic Cone Penetrometer (DCP)-DN method of design was developed for upgrading unpaved LVRs to a paved standard as an alternative to the more traditional methods based on the use of the California Bearing Ratio (CBR). The main emphasis of the method is on using the existing road pavement structure without disturbing its inherent strength and only adding material of the required quality and thickness to carry the anticipated design traffic loading.

7.1.2 Purpose and Scope

The purpose of this chapter is to provide details on the application of the DCP-DN design method for the structural design of Low Volume Sealed Roads (LVSRs).

The chapter presents a detailed procedure for undertaking pavement design based on data obtained from a DCP survey and the use of the ReCAP LVR DCP v 1.00 software. It also covers the application of the method in various situations typically encountered in practice. Finally, the option of using alternative design catalogues to accommodate particular design requirements is addressed.

7.2 DCP-DN Design Procedure

7.2.1 General

Based on the pavement balance concept, as discussed in *Chapter 3: Appendix 3.1*, the standard DCP-DN design catalogue for different traffic load classes (TLCs) presented in Table 7-1 below, was developed to provide optimum pavement balance. The design catalogue is based on the anticipated, long-term in-service equilibrium moisture condition (EMC). If there is a risk of prolonged moisture ingress into the pavement, then the design should be based on the same DN values, but in a soaked condition.

Table 7-1: Standard DCP-DN Design Catalogue

Traffic Class MESA	TLC 0.03 0.01-0.03	TLC 0.1 0.03-0.10	TLC 0.3 0.1-0.3	TLC 0.7 0.3-0.7	TLC 1.0 0.7-1.0
0- 150 mm Base ≥ 98% Mod. AASHTO	DN ≤ 6.2	DN ≤ 4.4	DN ≤ 3.2	DN ≤ 2.5	DN ≤ 2.3
150-300 mm Sub-base ≥ 95% Mod. AASHTO	DN ≤ 13	DN ≤ 9	DN ≤ 6.6	DN ≤ 5.2	DN ≤ 4.7
300-450 mm Subgrade ≥ 95% Mod. AASHTO	DN ≤ 21	DN ≤ 15	DN ≤ 11	DN ≤ 8.7	DN ≤ 7.8
450-600 mm In situ material	DN ≤ 32	DN ≤ 23	DN ≤ 17	DN ≤ 13	DN ≤ 12
600-800 mm In situ material	DN ≤ 48	DN ≤ 34	DN ≤ 25	DN ≤ 19	DN ≤ 17
DSN_{800}	≥ 52	≥ 73	≥ 100	≥ 128	≥ 143

Source: Adapted from Kleyn and van Zyl (1988)

Using the pavement balance concept, as explained in *Chapter 3 – Approach to Design: Appendix 3.1*, it is possible to develop alternative design catalogues for different design situations while retaining the total bearing capacity expressed by the DSN_{800} value; for example, for areas with strong subgrades or for labour-based projects where thinner lifts for subbase and base may be warranted. An alternative DCP-DN catalogue and the method for developing new catalogue values for different pavement layer configurations are presented in Appendix 7.1.

7.2.2 Design procedure

A flow-chart for the DCP-DN design procedure is shown in Figure 7-1. The step-by-step procedure is explained below and illustrated by an example using the ReCAP LVR DCP software.

The software mimics the way the calculations and operations would be carried out manually, and, as shown in Appendices 7.2 and 7.3, it is possible to carry out all the steps in the design using spreadsheets.

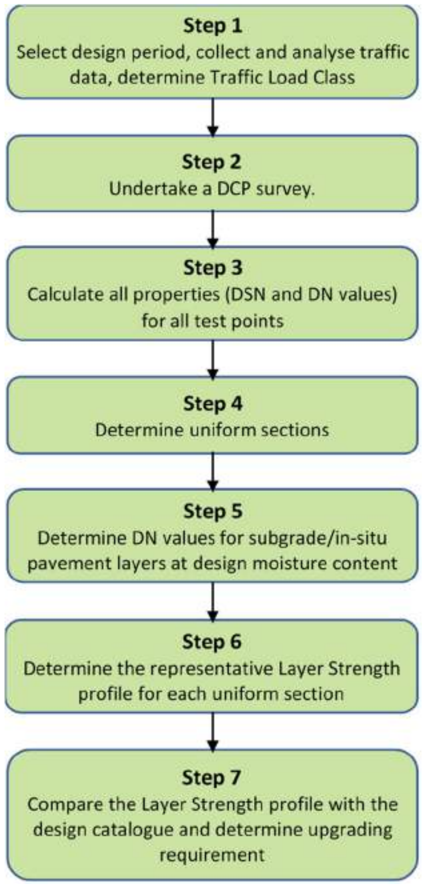


Figure 7-1: DCP-DN design procedure

Step 1: Determine the Traffic Load Class

Carry out a traffic survey and determine the TLC, as described in Chapter 4.

Step 2: Carry out a DCP survey

Carry out a DCP survey following the procedures described in Chapter 4.

Step 3: Calculate the DSN and DN values for all test points.

This is required for the determination of uniform sections. After entering all the DCP data program, the calculation of the following parameters is done automatically:

- The weighted average[1] of the DN value of each 150 mm layer down to a depth of 800 mm. This is the standard configuration of the ReCAP LVR DCP software, but the layer thicknesses can be varied if required, as described in Appendix 7.1.
- The number of blows (DSN_{450}) required to penetrate the top 450 mm of the pavement. This is the portion of the pavement that needs to be the strongest, and hence the DN for the top three 150 mm layers and the DSN_{450} provides a quick appreciation of the likely need for strengthening.
- The DSN_{800} is the total number of blows required for the DCP to penetrate to 800 mm depth and gives a broad measure of the overall strength of the pavement analogous to the AASHTO Structural Number. The DSN_{800} thus reflects the strength of the top 450 mm of the pavement as well as the contribution of the subgrade from a depth of 450 to 800 mm.

Step 4: Determine uniform sections.

The DCP results normally exhibit a fairly wide spread of DN values along the length of the road due to varying ground conditions. For an EOD, the road should be subdivided into uniform sections, each of which will be analysed separately, and may have different upgrading requirements.

The division of the road into uniform sections limits the variability within each section and thereby reduces the risk of design decisions based on an assessment of the average strength of the in-situ pavement. To avoid any distortion of the assessment of the representative strength in a uniform section, which may give rise to over- or under-design, "outliers" should be eliminated before determining the uniform sections.

Figures 7-2 and 7-3 illustrate the typical variability of the DCP results, and the extreme and average values, respectively.

[1] For difference between arithmetic and weighted average DN values, see Appendix 7.2.

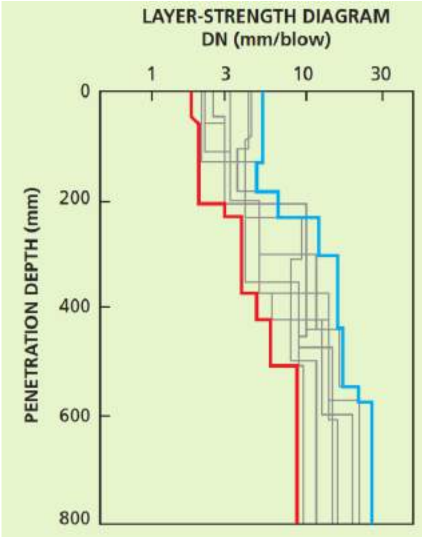


Figure 7-2: Collective DCP strength profile for a uniform section

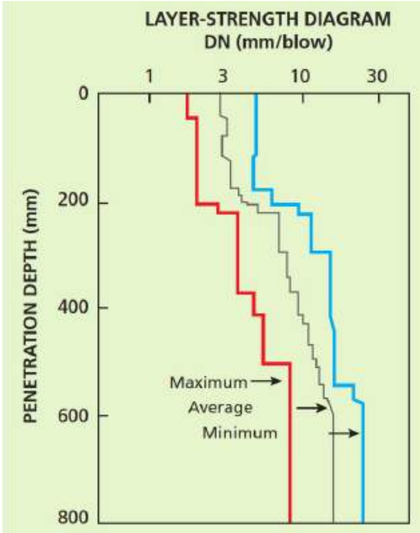


Figure 7-3: Average & extreme DCP strength profiles for a uniform section

The determination of uniform sections is carried out using the ReCAP LVR DCP software, as described below:

- By selecting the "Determine uniform sections" option in the "Analysis" menu, the screen, as shown in Figure 7-4, is used for the determination of uniform sections. Unless the file has been saved with sections already determined, only the Cusum[2] plots for the DNs and DSNs will appear, and the road will initially be displayed as one section.
- The tick-boxes above the diagram provide the option for (de)selecting one or more of the curves.
- By right-clicking in the diagram, the options are provided to "Add", "Adjust" or "Delete" sections. By hovering the cursor over the point where a section starts (or ends) and left-clicking, the user is prompted to enter a section-delimiter at that point. The section-delimiter will automatically "snap" to the chainage of the nearest DCP test point and the section chainages will be displayed on top of the screen.

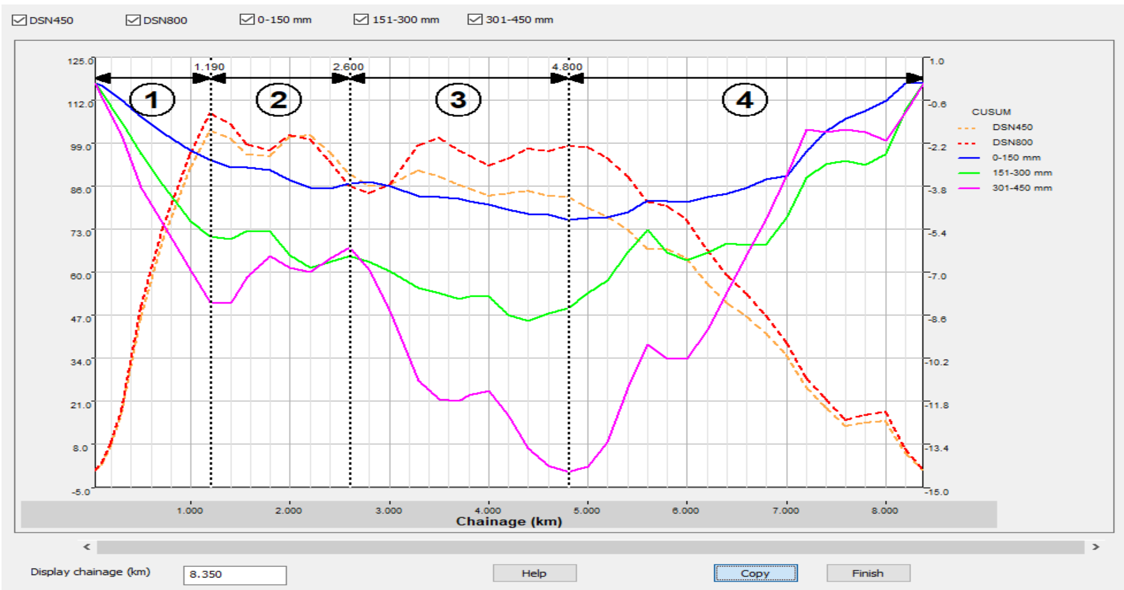


Figure 7-4: "Determination of uniform sections" screen

[2] Cumulative Sum (Cusum) analysis is a technique for analysing trends in a data set. See Appendix 7.2.

Step 5: Determine DN values for the subgrade and in-situ pavement layers (if any) at the design moisture content and density.

As described in Chapter 4, the method provides for two different options, as follows:

Option 1: In this option, which is recommended for most design situations, determination of the representative layer strength profile within each uniform section is carried out through laboratory testing of the in-situ pavement layers and subgrade materials at the anticipated, in-service equilibrium moisture content (EMC) and densities, as per the DCP-DN design catalogue. Determination of the in-situ moisture content in the pavement at the time of the DCP survey is therefore not required.

Option 2: This option may be considered in situations where the future moisture conditions along the alignment are likely to be fairly uniform, as would be the case with an improved gravel road having a functional and adequate drainage system.

For Option 1:

- Collect three bulk samples from each uniform section and thoroughly mix to produce one bulk sample on which all the tests are to be carried out in triplicate. The manner of carrying out the laboratory DN test is described in *Chapter 5 – Materials: Appendix 5.1.*

- Select the EMC to be used for the design based on engineering judgment, as influenced by a knowledge of the micro-climate and the likely in-service moisture and related drainage and maintenance conditions that have been established through site investigations. The DCP survey is particularly useful for the identification of sub-surface drainage problems (seepage, high water table, etc.), particularly if carried out towards, or at, the end of the wet season. It will, together with the excavation of test pits, provide a sound basis for understanding the environmental conditions and selecting the EMC for design.

Selection of design moisture content: For design purposes, the following is assumed:

- Raised formation level in areas with potential drainage problems; and
- Adequate drainage (crown height about 0.75 m depending on the gradient, whether lined/unlined drains, etc. See Section 6.4: Drainage); and
- A well-maintained, relatively impermeable surfacing, extending across the entire road width to the shoulder breakpoint (i.e., sealed shoulders).

Research has shown, with a high degree of probability, that under the above conditions:

- the EMC in the subgrade equilibrates below OMC in dry climates (annual rainfall < 500 mm) or at, or below, OMC in wet climates (annual rainfall > 500 mm);
- the EMC in the pavement layers is almost independent of climate with the average moisture content equilibrating below OMC.

On this basis, it is conservatively assumed that the EMC, in most cases, will be equivalent to OMC. Soaked designs for the pavement and subgrade could, of course, be warranted due to poor drainage, high water tables, the occurrence of flood plains, etc. Only in a dry climate with favourable drainage conditions could it be considered to base the design on the 0.75 OMC strength.

Selection of design density: For design purposes, the following is assumed:

- The subgrade and pavement layers are "compacted to refusal" without degrading the material by breaking down the coarse aggregates.
- The minimum densities will be achieved, as per the DCP-DN Design Catalogue.

Recommended laboratory DN testing program: Having selected the EMC of the subgrade and in-situ pavement layers, as described above, the risk associated with the design assumptions can be assessed by testing the material at a higher moisture content than that assumed. The recommended laboratory DN testing program is presented in Table 7-2:

Table 7-2: Practical schedule of laboratory DN tests

Climatic zone	Micro-climate	Moisture at testing	Laboratory DN tests		
			No @ compactive effort		
All	Risk of flooding, in marshy areas, poor drainage condition	OMC	3 @ Light	3 @ Intermediate	3 @ Heavy
		Soaked	1 @ Light	1 @ Intermediate	1 @ Heavy
Dry	Minimal risk of flooding, reasonable drainage conditions	0.75 OMC	3 @ Light	3 @ Intermediate	3 @ Heavy
		OMC	1 @ Light	1 @ Intermediate	1 @ Heavy

The output of the above testing program is shown in Figure 7-5. Two curves related to the moisture contents at which the material was tested provide the basis on which to evaluate the design subgrade strength as an input into Step 6 below for determination of the representative layer strength profile.

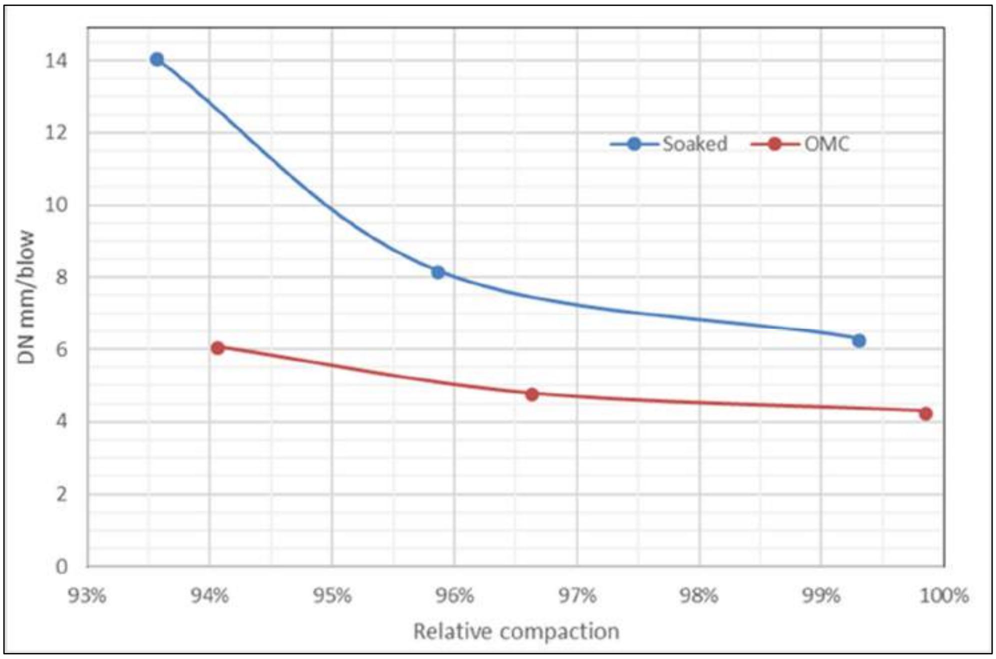


Figure 7-5: Typical result of the Laboratory DN test of subgrade samples

For Option 2:

Collect at least three representative samples from each uniform section to determine the in-situ moisture content in relation to the OMC of the in-situ layers. The prevailing FMC/OMC ratio at the time of the DCP survey enables the designer to assess whether the subgrade and pavement layers are likely to become wetter, remain more or less the same or become drier in-service after upgrading and improvement of the drainage system, taking into account the climatic zone, micro-climate, and drainage conditions for the respective sections in the same manner as for Option 1. On this basis, appropriate percentiles for each section and layer can be selected, as described in Chapter 4, Section 4.4.3, Moisture and drainage, Option 2, as an input into Step 6 below for the determination of the representative layer strength profile.

Step 6: For each uniform section, determine the representative Layer Strength Profile at the anticipated long-term moisture content and field density.

Having determined the uniform sections, the DCP Design Report will be displayed for the determination of the representative Layer Strength profile for each uniform section on-screen. The procedure will depend on the selected design option, as described above.

The top table in the DCP Design Report, as shown in Figure 7-6, displays the Weighted Average in-situ DN values from the DCP survey and cannot be manipulated.

The approach for determination of the representative Layer Strength Profiles in accordance with the selected design option is described below.

For Option 1:

- The middle table is interactive, providing the option for entering lab DN values at the selected EMC and field density determined through the laboratory DN test. In the example shown in Figure 7-6, the laboratory DN values to be used for the design have been entered for the two top layers.

Weighted Averages per in-situ DN values

Pavement Layer (mm)	Required DN value for TLC 0.3	Section no. 1	Section no. 2	Section no. 3	Section no. 4
		0.000 to 1.190 km	1.190 to 2.600 km	2.600 to 4.800 km	4.800 to 8.380 km
0-150	<= 3.2 (3.5)	2.4	4.0	4.0	5.8
150-300	<= 6.6 (7.5)	4.3	8.3	8.1	11
300-450	<= 11 (13)	8.3	16	12	19
450-600	<= 17	14	24	16	26
600-800	<= 25	20	31	20	30

Inadequate (non-compliance) in situ layer
Adequate (marginal compliance) in situ layer(s) that need to be improved
Adequate (full compliance) layer(s) in the upgraded pavement

DN values corrected for moisture content and density

Pavement Layer (mm)	Required DN value for TLC 0.3	Section no. 1	Section no. 2	Section no. 3	Section no. 4
		0.000 to 1.190 km	1.190 to 2.600 km	2.600 to 4.800 km	4.800 to 8.380 km
0-150	<= 3.2 (3.5)	**2.9**	**4.8**	**5.2**	**7.1**
150-300	<= 6.6 (7.5)	**5.0**	**10**	**11**	**13**
300-450	<= 11 (13)	8.3	16	12	19
450-600	<= 17	14	24	16	26
600-800	<= 25	20	31	20	30

Inadequate (non-compliance) in situ layer
Adequate (marginal compliance) in situ layer(s) that need to be improved
Adequate (full compliance) in situ layer(s)

Number of new layers required per section

1	2	3	4
0	1	1	1

DN values for the upgraded pavement structure

Pavement Layer (mm)	Required DN value for TLC 0.3	Section no. 1	Section no. 2	Section no. 3	Section no. 4
		0.000 to 1.190 km	1.190 to 2.600 km	2.600 to 4.800 km	4.800 to 8.380 km
0-150	<= 3.2 (3.5)	2.9	3.1	3.1	3.1
150-300	<= 6.6 (7.5)	5.0	4.8	5.2	7.1
300-450	<= 11 (13)	8.3	10	11	13
450-600	<= 17	14	16	12	19
600-800	<= 25	20	24	16	26

New base added with DN values <= 3.2
New subbase added with DN values <= 6.6

Figure 7-6: Determination of representative Layer Strength profile (Option 1)

For Option 2:

The use of percentiles[3] in Option 2 can be selected from the DCP System Configuration under the System menu by ticking the "Use percentiles/reliability level" tick-box, as shown in Figure 7-7.

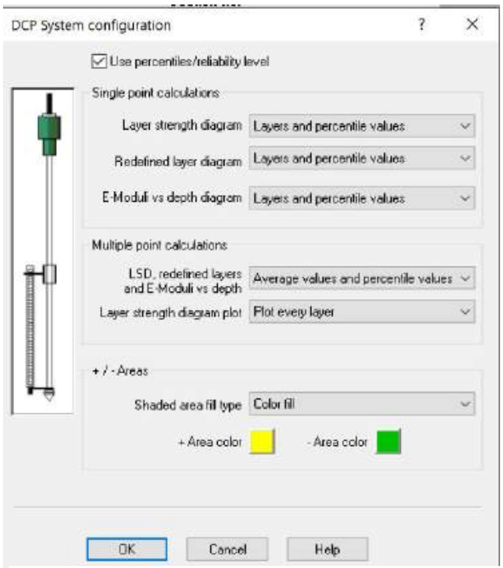


Figure 7-7: Selection of percentiles option from DCP System Configuration

- The middle table is interactive, providing the option for adjustment of DN values, which default to the 50th percentile. By clicking in each cell, the designer can select the appropriate percentile for moisture adjustment. In the example shown in Figure 7-8, the 80th percentile has been selected for the two top layers.

Weighted Averages per in-situ DN values

Pavement Layer (mm)	Required DN value for TLC 0.3	Section no. 1	2	3	4
		0.000 to 1.190 km	1.190 to 2.600 km	2.600 to 4.800 km	4.800 to 8.380 km
0-150	<= 3.2 (3.5)	2.4	4.0	4.0	5.8
150-300	<= 6.6 (7.5)	4.3	8.3	8.1	11
300-450	<= 11 (13)	8.3	16	12	19
450-600	<= 17	14	24	16	26
600-800	<= 25	20	31	20	30

Inadequate (non-compliance) in situ layer
Adequate (marginal compliance) in situ layer(s) that need to be improved
Adequate (full compliance) layer(s) in the upgraded pavement

DN values corrected for moisture content

Pavement Layer (mm)	Required DN value for TLC 0.3	Section no. 1	2	3	4
		0.000 to 1.190 km	1.190 to 2.600 km	2.600 to 4.800 km	4.800 to 8.380 km
0-150	<= 3.2 (3.5)	2.8 (80P)	4.6 (80P)	4.8 (80P)	6.6 (80P)
150-300	<= 6.6 (7.5)	4.9 (80P)	9.9 (80P)	9.7 (80P)	13 (80P)
300-450	<= 11 (13)	8.3 (50P)	16 (50P)	12 (50P)	19 (50P)
450-600	<= 17	14 (50P)	24 (50P)	16 (50P)	26 (50P)
600-800	<= 25	20 (50P)	31 (50P)	20 (50P)	30 (50P)

Inadequate (non-compliance) in situ layer
Adequate (marginal compliance) in situ layer(s) that need to be improved
Adequate (full compliance) in situ layer(s)

Number of new layers required per section

1	2	3	4
0	0	0	0

DN values for the upgraded pavement structure

Pavement Layer (mm)	Required DN value for TLC 0.3	Section no. 1	2	3	4
		0.000 to 1.190 km	1.190 to 2.600 km	2.600 to 4.800 km	4.800 to 8.380 km
0-150	<= 3.2 (3.5)	2.8 (80P)	4.6 (80P)	4.8 (80P)	6.6 (80P)
150-300	<= 6.6 (7.5)	4.9 (80P)	9.9 (80P)	9.7 (80P)	13 (80P)
300-450	<= 11 (13)	8.3 (50P)	16 (50P)	12 (50P)	19 (50P)
450-600	<= 17	14 (50P)	24 (50P)	16 (50P)	26 (50P)
600-800	<= 25	20 (50P)	31 (50P)	20 (50P)	30 (50P)

New base added with DN values <= 3.2
New subbase added with DN values <= 6.6

Figure 7-8: Determination of representative Layer Strength profile (Option 2)

[3] For explanation and calculation of percentiles, see Appendix 6.2

Step 7: Compare the representative Layer Strength profile with the design catalogue and determine upgrading requirement

- The representative Layer Strength profiles, whether determined through laboratory DN tests (option 1) or the selection of appropriate percentile values (option 2), are then compared with the TLC requirements, on the basis of which the designer can determine the number of layers to be added for each section, as shown in Figure 7-9 and 7-10 for Options 1 and 2, respectively.
- The layers in the bottom table are then moved down in steps equal to the number of new layers to be added. Empty cells are provided for entering DN values for the imported layers that have been determined from the laboratory DN test of the borrow pit materials to be used.

DN values corrected for moisture content and density

Pavement Layer (mm)	Required DN value for TLC 0.3	Section no. 1 0.000 to 1.190 km	2 1.190 to 2.600 km	3 2.600 to 4.800 km	4 4.800 to 8.380 km
0-150	<= 3.2 (3.5)	**2.9**	**4.8**	**5.2**	**7.1**
150-300	<= 6.6 (7.5)	**5.0**	**10**	**11**	**13**
300-450	<= 11 (13)	8.3	16	12	19
450-600	<= 17	14	24	16	26
600-800	<= 25	20	31	20	30

Inadequate (non-compliance) in situ layer
Adequate (marginal compliance) in situ layer(s) that need to be improved
Adequate (full compliance) in situ layer(s)

Number of new layers required per section			
0	1	1	1

DN values for the upgraded pavement structure

Pavement Layer (mm)	Required DN value for TLC 0.3	Section no. 1 0.000 to 1.190 km	2 1.190 to 2.600 km	3 2.600 to 4.800 km	4 4.800 to 8.380 km
0-150	<= 3.2 (3.5)	2.9	3.1	3.1	3.1
150-300	<= 6.6 (7.5)	5.0	4.8	5.2	7.1
300-450	<= 11 (13)	8.3	10	11	13
450-600	<= 17	14	16	12	19
600-800	<= 25	20	24	16	26

New base added with DN values <= 3.2
New subbase added with DN values <= 6.6

Figure 7-9: Determination of upgrading requirement using Option 1

DN values corrected for moisture content

Pavement Layer (mm)	Required DN value for TLC 0.3	Section no. 1 0.000 to 1.190 km	2 1.190 to 2.600 km	3 2.600 to 4.800 km	4 4.800 to 8.380 km
0-150	<= 3.2 (3.5)	2.8 (80P)	4.6 (80P)	4.8 (80P)	6.6 (80P)
150-300	<= 6.6 (7.5)	4.9 (80P)	9.9 (80P)	9.7 (80P)	13 (80P)
300-450	<= 11 (13)	8.3 (50P)	16 (50P)	12 (50P)	19 (50P)
450-600	<= 17	14 (50P)	24 (50P)	16 (50P)	26 (50P)
600-800	<= 25	20 (50P)	31 (50P)	20 (50P)	30 (50P)

Inadequate (non-compliance) in situ layer
Adequate (marginal compliance) in situ layer(s) that need to be improved
Adequate (full compliance) in situ layer(s)

Number of new layers required per section			
0	1	1	1

DN values for the upgraded pavement structure

Pavement Layer (mm)	Required DN value for TLC 0.3	Section no. 1 0.000 to 1.190 km	2 1.190 to 2.600 km	3 2.600 to 4.800 km	4 4.800 to 8.380 km
0-150	<= 3.2 (3.5)	2.8 (80P)	3.1	3.1	3.1
150-300	<= 6.6 (7.5)	4.9 (80P)	4.6 (80P)	4.8 (80P)	6.6 (80P)
300-450	<= 11 (13)	8.3 (50P)	9.9 (80P)	9.7 (80P)	13 (80P)
450-600	<= 17	14 (50P)	16 (50P)	12 (50P)	19 (50P)
600-800	<= 25	20 (50P)	24 (50P)	16 (50P)	26 (50P)

New base added with DN values <= 3.2
New subbase added with DN values <= 6.6

Figure 7-10: Determination of upgrading requirement using Option 2

7.2.3 Analysis of pavement balance

Having completed the design, each section can be analysed separately by selecting the "DCP Section analysis per section" from the Sections menu, as shown in Figure 7-11. The output from the analysis of Section 2 in the above example for Option 1 is shown in Figure 7-12.

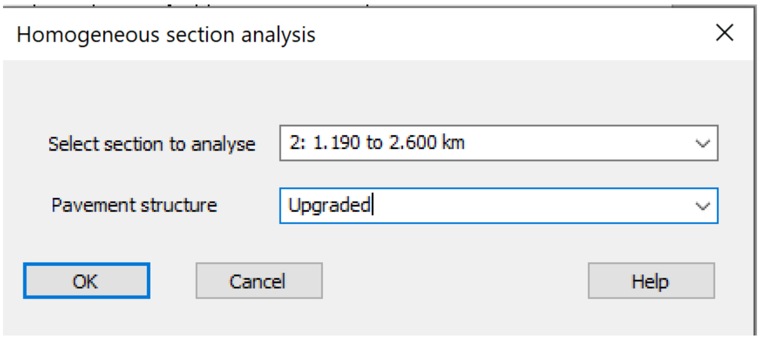


Figure 7-11: Homogeneous section analysis

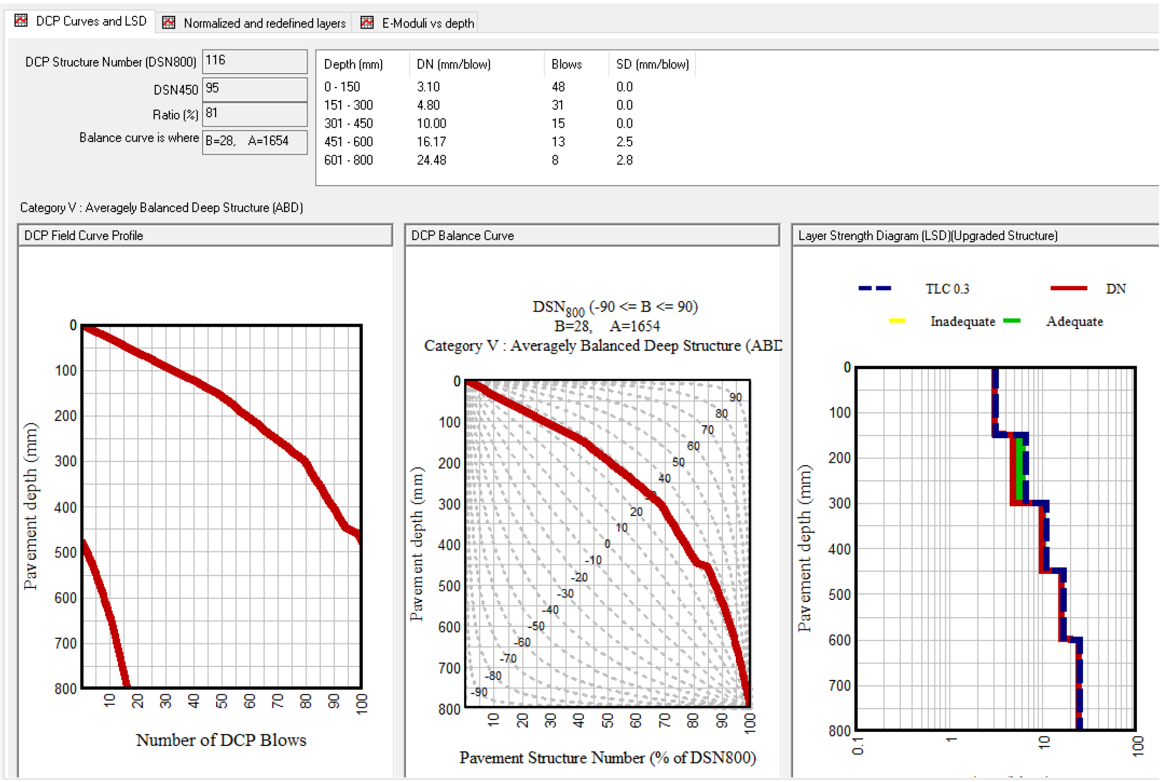


Figure 7-12: Output from the analysis of homogeneous sections

7.3 Application in Practice

7.3.1 General

In practice, the designer will inevitably be challenged with different scenarios as summarised below:

1. **Normal situation:** No alteration of the horizontal alignment or widening of the cross section is required.
2. **Road widening:** The cross section needs to be widened to accommodate the design traffic.
3. **New alignment:** A section of the road needs to be constructed on a new alignment for various reasons. This design scenario is equivalent to designing a road on virgin ground, i.e., where there was no road before.
4. **Sunken road profile:** The road is below the surrounding ground and needs to be raised to ensure adequate drainage.

5. **Embankment:** Over terrain susceptible to flooding.
6. **Section in high fill:** High fills may be required to improve the vertical alignment.
7. **Section in cut:** A cut through a crest is required to improve the vertical alignment.
8. **Section in cut/fill:** In sidelong ground, a cut is required on the high side, and material from the cut is used as fill on the low side.
9. **Coarse graded in-situ pavement:** The DCP cannot easily penetrate the pavement and would give an unreliable result.

The DCP-DN design method is based on the application of the DCP-DN design catalogue for the pavement design, and does not depend on the use of the DCP to undertake measurements as part of the survey along the alignment in the standard manner. The manner of addressing the various situations that can be encountered in practice is described below.

1. Normal situation

Figure 7-13 shows a typical gravel road that does not require widening. It has some residual wearing course gravel of varying thickness and may have a structural layer that overlies the formation, which has been constructed from the in-situ subgrade. Depending on the thickness of the residual wearing course gravel, it may be disregarded in the structural design (in which case the additional strength of this layer will be a bonus), or the layer may be augmented with a material of similar quality to provide a full structural layer.

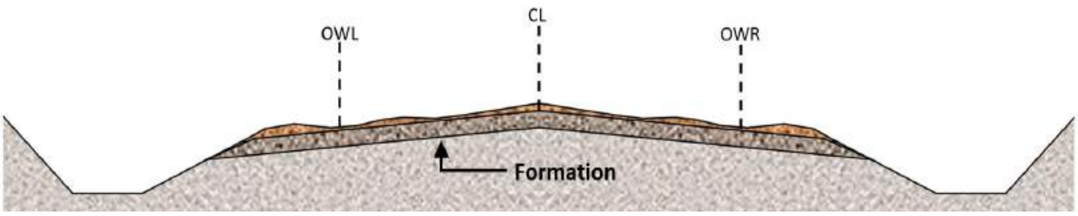


Figure 7-13: Typical, full-width gravel road

The design process is as follows:

a) Carry out the DCP survey with tests at regular intervals staggered at CL-OWL-OWR.

b) Determine uniform sections.

c) Determine the strength of subgrade and in-situ pavement layers.

d) Carry out the pavement design for each uniform section separately, to determine the upgrading requirements for the entire road. Contiguous uniform sections with the same upgrading requirements can then be enjoined.

e) Assess and select borrow pit materials for new pavement layers, if required, as described in *Chapter 5 – Materials*.

f) Assess the attainment of adequate drainage. If the h_{min} requirement (see *Chapter 6 – Drainage*) is not met, the formation may need to be raised, or additional layers are imported, ensuring that the material is at least of similar quality to that in place, or better.

2. Road widening

The situation is similar to the normal situation above, apart from the requirement to widen the road, say, from 4 m to 6 m sealed width, to cater for the forecasted traffic flow.

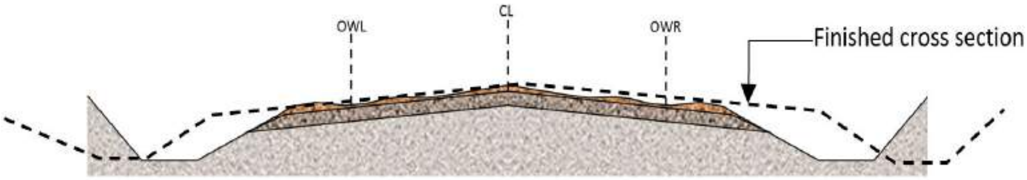


Figure 7-14: Cross section widening

Carry out steps a) to f) as described above.

g) Widening of the cross section will involve the removal of topsoil and vegetation to accommodate the additional width. Depending on the situation, the additional width can be constructed as a relatively narrow strip on both sides, or alternatively, as a wider strip on one side only by shifting the finished centre line to either side. From a practical perspective (e.g., traffic accommodation), the latter alternative may be preferable if the situation on the ground allows for it.

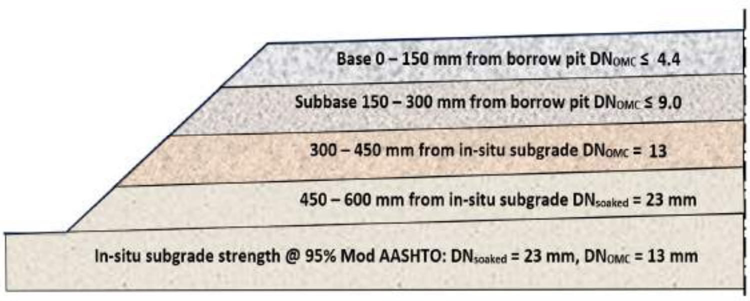
The widening (in benches) of the formation must be undertaken with a material of similar or better quality as that in the existing formation. This material can often be obtained from side borrow, or through importation from approved borrow pits.

The above approach applies where there is an existing engineered pavement that is worthwhile preserving. Otherwise, from the contractor's perspective, it may be easier, and perhaps more economical, to level the existing formation to form a roadbed over the required width, and then construct the subgrade formation and pavement on top, as described below for a road on a new alignment. Engineering judgement must be applied in selecting the most appropriate and economical solution.

3. New alignment

In this situation, there is no existing pavement structure, and all that is required is to determine the design strength of the subgrade, as described in Step 5 of Section 7.2.2. After determining the centre line for the section over the new alignment, representative bulk samples of the subgrade must be collected for testing in the laboratory.

After clearing of vegetation and topsoil to the required cleared width, the subgrade is levelled and compacted to refusal to serve as the roadbed on which the formation and pavement are constructed. The design strength of the subgrade will determine the number of layers that are required, as illustrated in Figure 7-15. In this example, the two lower layers constructed from the in-situ subgrade satisfy a soaked design strength of DN ≤ 23 mm. The same material can be used for the third layer, which in this case, is not anticipated to become soaked in service. Approved borrow pit materials with the required OMC design strength, as per the DCP-DN structural catalogue, must be imported for the subbase and base.



Traffic Class MESA	TLC 0.1 0.03-0.10
0- 150 mm Base ≥ 98% Mod. AASHTO	DN ≤ 4.4
150-300 mm Sub-base ≥ 95% Mod. AASHTO	DN ≤ 9.0
300-450 mm Subgrade ≥ 95% Mod. AASHTO	DN ≤ 15
450-600 mm In situ material	DN ≤ 23
600-800 mm In situ material	DN ≤ 34
DSN_{800}	≥ 73

Figure 7-15: Schematic example of pavement design on new alignment for TLC 0.1

4. Sunken road profile

Many low volume earth and gravel roads have, over time, developed a sunken profile, as shown in Figure 7-16, due to maintenance grading and erosion. Drainage on such sections, particularly where the longitudinal gradient is less than about 1%, is problematic as water will tend to pond in the side drains. In flat terrain, it may also be difficult to construct mitre drains to lead the water away from the road, either to natural watercourses or to soakaway ponds.

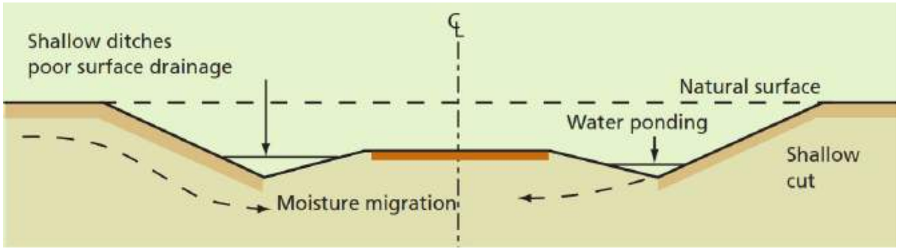


Figure 7-16: Sunken road profile

In such situations, the formation must therefore be raised to a level where functional, appropriately spaced mitre drains can be constructed. From a maintenance perspective, the mitre drains must not be too long and should have a gradient of about 2% to avoid silting up. In flat terrain, the subgrade formation should preferably be above the surrounding terrain.

The design approach for roads with a sunken profile will be as described above for roads on a new alignment.

5. Embankment

In terrain susceptible to flooding, such as wide flood plains and river estuaries, embankments are required, with the height of the embankment being determined by the maximum flood level. The added problem in such situations is that the soils may be of low strength and that considerable settlement and consolidation may take place after construction of the embankment. Specialist geotechnical advice may be required to determine the expected settlement and consolidation, which must then be taken into account in the determination of the constructed embankment height.

A DCP survey in such situations will have little value except to determine the depth of the low-strength soil down to stronger soil strata if it is within a depth of about 800-1000 mm.

The design of the embankment and pavement follows the principles for roads on new alignments, as described above. In the case of constructing the embankment on low-strength soils, a decision must be taken as to the most economical method of providing a roadbed of sufficient strength, either by replacing the low-strength soils to a certain depth (preferably with rockfill) or by treatment of the soils, e.g., by chemical stabilisation (lime or cement).

6. Section in high fill

The approach to the design of high fill sections is similar to the approach to the design of sections on a new alignment. Samples from the in-situ subgrade are taken for determination of the design subgrade strength. Depending on the subgrade strength, the material may be used to construct the formation while ensuring that the strength requirement of the DCP-DN design catalogue is met.

7. Section in cut

On LVRs, sections in cut, for example to improve the vertical alignment, will normally be over relatively short lengths. For cuts ≤ 0.5 m depth, a standard DCP survey will give information on the strength of the subgrade at the in-situ moisture content and density. For deeper cuts, which would be very rare, a DCP survey would have little value. In both cases, the material properties and design strength can be obtained from bulk samples taken at the level of the finished roadbed. The pavement design is then carried out as described above for a road on a new alignment.

8. Section in cut/fill

On roads in sidelong terrain, cut/fill sections may be required to attain the appropriate width. The properties and design strength of the cut material and subgrade at the roadbed level can be obtained from bulk samples along the section.

The fill section must be designed in a similar manner as described above for road widening, to attain a roadbed of equal strength over the full width of the road.

9. Coarse-graded in-situ pavement

It is not uncommon that gravel roads have been constructed with little regard for maximum aggregate size in the structural layer and/or the wearing course. A DCP survey can therefore be difficult, if not impossible, to carry out in such circumstances. The design of such sections can be carried out by either of the following options:

a) Excavate trial pits to determine the depth of the coarse-graded pavement layer(s). If the combined thickness of such layers is ≥ 300 mm, the strength is normally sufficient for any traffic load class. Standard materials tests and determination of design strength may be carried out on the – 20 mm fraction of the material, bearing in mind that the strength of this fraction of the material is likely to be significantly less than that of the coarse-graded material in the pavement. Such pavements can, therefore normally be accepted as they are as a foundation for a regulating base layer with the required properties, strength, and drainage attributes.

b) Remove the coarse layer at each test point and carry out the DCP survey in the normal manner. The thickness and position of the removed layers must be noted in the DCP survey form and a design DN value for the layer taken into account in the pavement design.

Bibliography

AfCAP. (2020). AfCAP LVR DCP v 1.04. http://www.research4cap.org/SitePages/Research.aspx

American Association of State Highway and Transportation Officials (1993). ***AASHTO Guide for Design of Pavement Structures***. Washington, D.C., USA.

ASTM D6591/D6591M-18 (2018). ***Standard Test Method for the Use of the Dynamic Cone Penetrometer in Shallow Pavement Applications.*** ASTM International, West Conshohocken, PA., USA.

Committee of Land Transport Officials (COLTO) (1996). ***Structural Design of Flexible Pavements for Inter-urban and Rural Roads. Technical Recommendations for Highways, TRH4***. Department of Transport, Pretoria, South Africa.

De Beer M, Kleyn E G and P F Savage (1988). ***Towards a Classification System for the Strength-balance of Thin Surfaced Flexible Pavements.*** DRTT Report 637, CSIR, Pretoria, South Africa.

De Beer M (1989). ***Dynamic cone penetrometer (DCP) - aided evaluation of the behaviour of pavements with lightly cementitious layers.*** Research Report DPVT 37, CSIR, Pretoria, South Africa.

Emery S J (1985). ***Prediction of Moisture Content for Use in Pavement Design.*** Ph.D. Thesis, Univ. of Witwatersrand, Johannesburg, South Africa.

Giummarra G (1995). ***Sealed local roads manual.*** ARRB Transport Research Ltd., Brisbane, Australia.

Gourley C S and P A K Greening (1999). ***Performances of low-volume sealed roads; results and recommendations from studies in southern Africa.*** TRL Published Report PR/OSC/167/99. TRL, Crowthorne, Berkshire, UK.

Kleyn E G and P F Savage (1982). ***The application of the pavement DCP to determine the bearing properties and performance of road pavements***, Proc Int Symp on Bearing Capacity of Roads and Airfields, Trondheim, Norway, 1982.

Kleyn E G and G D van Zyl (1988). ***Application of the DCP to Light Pavement Design***. First Int. Symposium on Penetration testing, Orlando, USA.

Ministry of Transport and Public Works, Malawi (2013). ***Design Manual for Low Volume Sealed Roads Using the DCP Design Method***. Lilongwe, Malawi.

Mitchell R L, C P van der Merwe C P and H K Geel (1975). ***Standardised Flexible Pavement Design for Rural Roads with Light to Medium Traffic***, Ministry of Roads and Road Traffic, Salisbury, Rhodesia.

Paige-Green P and G D van Zyl G (2019). ***Development of the DCP-DN Design Method,*** ReCAP Technical Report**,** www.afcap.org

Southern African Development Community (2003). ***Low volume sealed Roads Guideline***. SADC Secretariat, Gaborone, Botswana.

Transport Research Laboratory (1993). ***Overseas Road Note 31. A guide to the structural design of bitumen-surfaced roads in tropical and sub-tropical climates***. Overseas Center, TRL, Crowthorne, Berkshire, UK. (4th edition).

Transport Research Laboratory (1993). ***Measuring Road Pavement Strength and Designing Low Volume Sealed Roads Using the Dynamic Cone Penetrometer***. Unpublished Project Report UPR/IE/76/06. Project Record No. R7783. TRL Limited, Crowthorne, Berkshire, UK.

Van der Merwe C P (1999). ***Material and Pavement Structures for Low Volume Roads in Zimbabwe.*** Unpublished Report, Harare, Zimbabwe.

Wolff H, S J Emery, van Zyl G D and P Paige-Green (1995). ***Design Catalogue for Low-Volume Roads Developed for South African Conditions.*** Proc. Sixth Int. Conf. on Low-Volume Roads, Minneapolis, Minnesota, USA.

Appendix 7.1: Alternative DCP-DN design catalogues

Catalogue for areas with strong subgrades

As discussed above, the DCP-DN design method provides an option for using alternative design catalogues to accommodate particular design requirements.

The catalogue below may be used in areas with strong subgrades and provides for the use of slightly weaker material in the upper pavement layers than the standard catalogue. Thus, this option facilitates a wider use of naturally occurring materials without having to resort to modification or stabilisation to satisfy the more stringent requirements of the standard catalogue.

Table 7-3: DCP-DN Design Catalogue for BN_{100} = 24%

Traffic Class MESA	TLC 0.03 0.01-0.03	TLC 0.1 0.03-0.10	TLC 0.3 0.1-0.3	TLC 0.7 0.3-0.7	TLC 1.0 0.7-1.0
0- 150 mm Base ≥ 98% Mod. AASHTO	DN ≤ 8.5	DN ≤ 6.0	DN ≤ 4.4	DN ≤ 3.4	DN ≤ 3.1
150-300 mm Sub-base ≥ 95% Mod. AASHTO	DN ≤ 12	DN ≤ 8.8	DN ≤ 6.4	DN ≤ 5.1	DN ≤ 4.6
300-450 mm Subgrade ≥ 95% Mod. AASHTO	DN ≤ 17	DN ≤ 12	DN ≤ 8.9	DN ≤ 7.0	DN ≤ 6.3
450-600 mm In situ material	DN ≤ 23	DN ≤ 16	DN ≤ 12	DN ≤ 9	DN ≤ 8
600-800 mm In situ material	DN ≤ 30	DN ≤ 21	DN ≤ 16	DN ≤ 12	DN ≤ 11
DSN_{800}	≥ 52	≥ 73	≥ 100	≥ 128	≥ 143

Source: Adapted from Kleyn and van Zyl (1988)

User-defined Traffic Load Design curve

Using the Standard Pavement Balance Curve (SPBC) concept, it is easy to determine the DN requirements for pavement structures with layer thicknesses different from the layer configuration as per the standard DCP-DN Catalogue; for instance, if the subbase and base are to be constructed by labour-based methods and thinner lifts of, say, 100 mm is desirable.

The example below shows the method for determining the DN requirements for this pavement structure while ensuring that the DSN_{800} requirement of 73 for TLC 0.1 is satisfied.

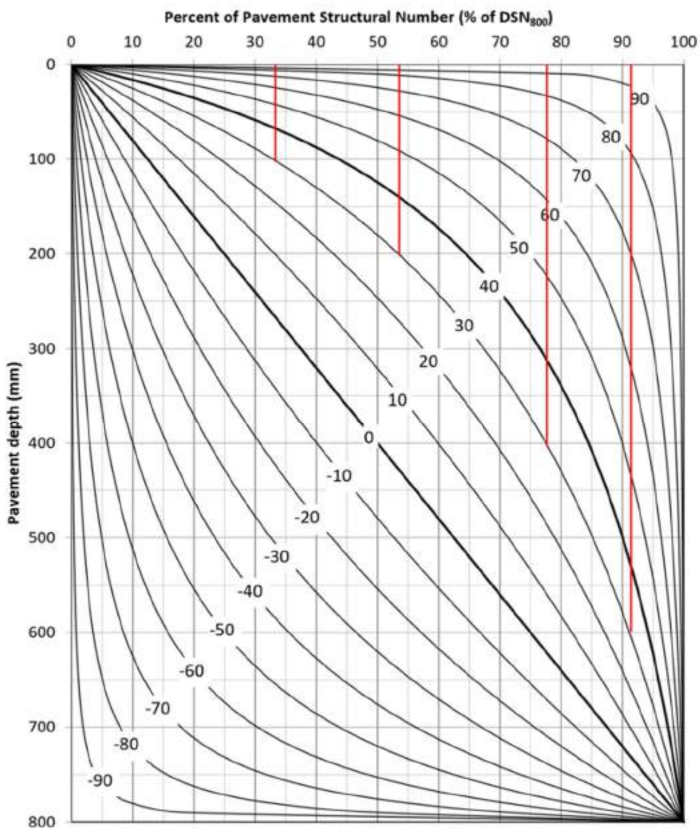


Figure 7-17: Determination of contribution to DSN800 of layers in new pavement structure

Step 1: Select the SPBC to be used as a basis for the determination of the new DN values. Optimum pavement balance is achieved with a Balance Number, BN100, between 30% and 40%. For this example, the SPBC for B=30 is chosen, with BN100 = 33%.

Step 2: Draw vertical lines to the horizontal axis from the points where the lower layer boundaries cross the selected SPBC and read off the horizontal axis the percentage contribution of each layer to the DSN_{800}, as shown in Figure 7-17.

The following steps are shown in Table 7-4:

Step 3: Calculate the number of blows required to penetrate each layer.

Step 4: Calculate the maximum DN per layer.

Table 7-4: Calculation of DN requirements in new pavement structure

Layer	Layer depth (mm)	Step 2 % of DSN_{800}	Step 3 Blows/layer	Step 4 Max DN per layer (mm)
Base	0-100	33%	73*33% = 24	100/24 = 4.2
Subbase	100-200	53-33=20%	73*20% = 15	100/15 = 6.8
Improved subgrade	200-400	78-53=25%	73*25% = 18	200/18 = 11
In-situ subgrade	400-600	92-78=14%	73*14% = 10	200/10 = 20
In-situ subgrade	600-800	100-92=8%	73*8% = 6	200/6 = 34

For the application of an alternative catalogue or TLC design curve different from those pre-defined in the ReCAP LVR DCP software, the user must define the new layer configuration and/or new DN values per layer in the “Traffic Load Design curves configuration” option under the System menu in the software.

Appendix 7.2 – Background calculations and operations

In the analysis of the DCP data, the software carries out some calculations and operations in the background, invisibly to the user. While it is possible to carry out the data analysis manually by the use of a spreadsheet, it is much more convenient and time saving for the designer to have all this carried out by the software. The following explanations are provided for the user to understand what is happening behind the scene to provide the various outputs as a basis for the design.

Weighted average DN

In the software, the <u>weighted average</u> DN values are used. The difference between the arithmetic and weighted average DN values is illustrated below:

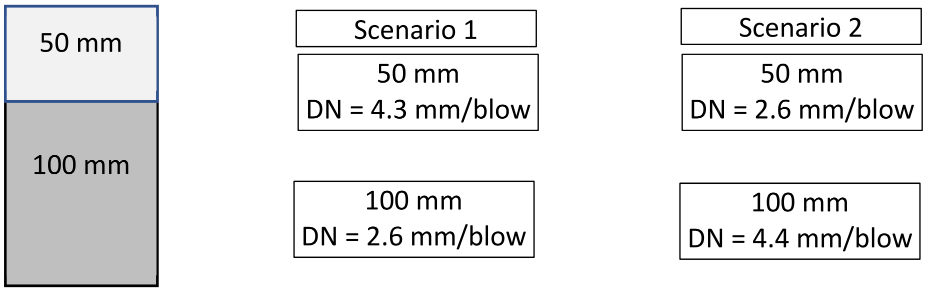


Table 7-5: Difference between arithmetic and weighted average DN values

Layer	Scenario 1	Scenario 2
Top 50 mm	DN = 4.4	DN = 2.6
Bottom 100 mm	DN = 2.6	DN = 4.4
Arithmetic average DN	= (4.4 + 2.6)/2 = 3.5	= (2.6 + 4.4)/2 = 3.5
Weighted average DN	= (4.4*50 + 2.6*100)/150 = 3.2	= (2.6*50 + 4.4*100)/150 = 3.8

The weighted average DN values take into account the relative contribution of the different portions of the layer to the strength of the whole layer. If the variation in the strength of the portions is relatively low, both methods will give acceptable results. On the other hand, with relatively large variations in the strength, as may be the case in the field, the arithmetic average DN could potentially give rise to under- or overdesign, and would be unacceptable. Thus, the weighted average DN value provides a more realistic DN value to be used for design.

Cumulative sum (Cusum) analysis

The software uses the Cumulative Sum (Cusum) technique to analyse various aspects of the DCP data, as follows:

- Determination of uniform sections.
- Determination of layer boundaries.
- Calculation of the A-value to categorise the pavement balance (see Chapter 3 – Approach to Design, Appendix 3.1).

The manner of carrying out a Cusum analysis is illustrated in Table 7-6. Note that the Cumulative Sum of the DSN – Average values will always be zero.

The start and end of uniform sections is determined as the points where the general direction of the curve changes, as indicated in the plot in Figure 7-18.

Procedure for carrying out a Cusum analysis:

Step 1: Calculate the Average DSN value

Step 2: Calculate the DSN-Avg for all the test points

Step 3: Calculate the cumulative sums of all the DSN-Avg values

Step 4: Plot the Cusum values in a graph as shown in Figure 6-22 to determine where the trend in the data changes, i.e. the points where the general direction of the curve changes. In this case, three distinct sections of the curve is identified. Minor "spikes" on the curve indicate variations in the DSN values from the average. Larger spikes would indicate points with extreme values. These should preferably be excluded from the analysis as they could unduly influence the analysis. Such points should rather be investigated separately to determine the likely cause of the variation.

Table 7-6: Cusum analysis of DSN800

Test no	Chainage km	DSN800	DSN - Avg	Cusum	Sections
1	0.03	115	7	7	Section 1
2	0.1	161	53	61	
3	0.2	180	72	133	
4	0.3	225	117	251	
5	0.5	302	194	445	
6	0.72	130	22	468	
7	1	245	137	605	Section 2
8	1.19	96	-12	594	
9	1.4	87	-21	573	
10	1.57	56	-52	522	
11	1.8	142	34	556	
12	2	119	11	567	
13	2.2	83	-25	543	
14	2.4	64	-44	499	
15	2.6	77	-31	469	
16	2.8	117	9	478	
17	3	121	13	492	
18	3.3	173	65	557	
19	3.5	64	-44	514	
20	3.7	114	6	520	
21	3.8	67	-41	480	
22	4	117	9	489	
23	4.2	119	11	500	
24	4.4	125	17	518	
25	4.6	82	-26	492	
26	4.8	148	40	533	Section 3
27	5	63	-45	488	
28	5.2	119	11	500	
29	5.4	40	-68	432	
30	5.6	97	-11	422	
31	5.8	106	-2	420	
32	6	66	-42	379	
33	6.2	57	-51	328	
34	6.4	81	-27	301	
35	6.6	78	-30	272	
36	6.8	71	-37	235	
37	7	62	-46	190	
38	7.2	45	-63	127	
39	7.4	106	-2	126	
40	7.6	46	-62	64	
41	7.8	183	75	140	
42	8	42	-66	74	
43	8.2	63	-45	30	
44	8.38	78	-30	0	
	Average DSN	108			

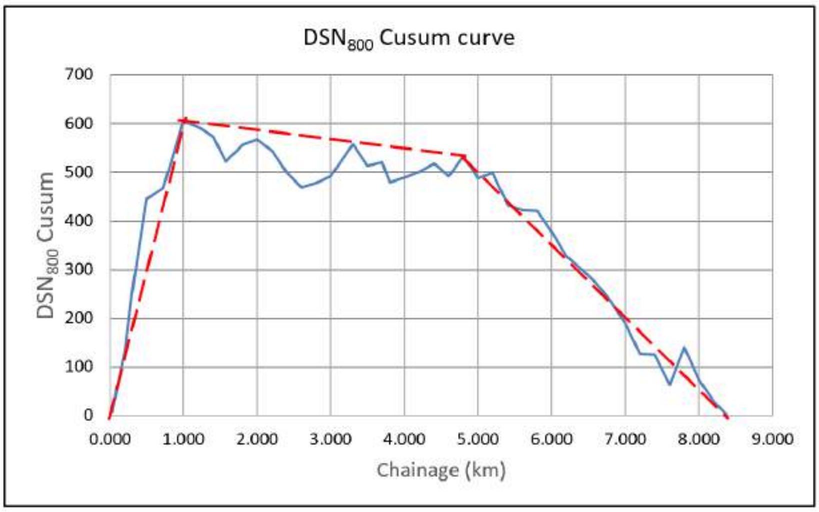


Figure 7-18: Plot of DSN800 Cusum

Calculation of Percentile (P) values

The software assumes a normal distribution of the data readings. This is used for calculating percentiles (P) of the DCP data, as follows.

A normal distribution of the DN values is illustrated in Figure 7-19.

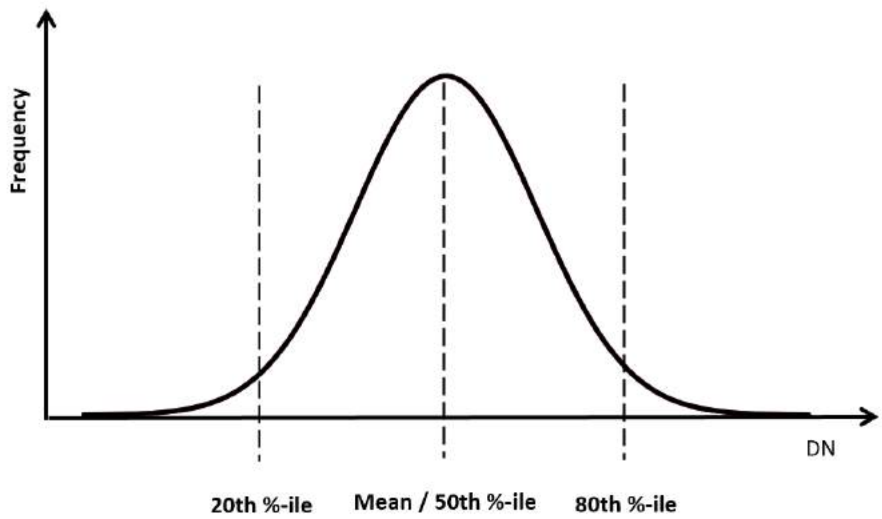


Figure 7-19: Normal distribution of DN values

The percentile values are calculated using the "z"-value for a Gaussian distribution. The z-values are given in Table 7-7.

Table 7-7: Normal distribution Z-Values

Reliability Level	Percentile	z-Value
A	95	1.64
B	90	1.28
C	80	0.84
D	50	0
E	85	1.04
F	75	0.67
G	70	0.52
H	65	0.39
I	60	0.25
J	55	0.13

The Percentile (P) = Mean DN +/- (Standard Deviation DN * z)

Calculation of percentile values can also be carried out in Excel, but the results will differ from those calculated in the software. Table 7-8 shows a calculation of the 80th percentile in the software and illustrates the difference in results between this method and the results calculated in Excel (depending on which Excel function is used).

Table 7-8: Example for calculating 80th Percentile (i.e., 80P) for 11 DN Values

<table>
<tr><th>DN No.</th><th colspan="3">DN value</th></tr>
<tr><td>1</td><td colspan="3">1</td></tr>
<tr><td>2</td><td colspan="3">1.16667</td></tr>
<tr><td>3</td><td colspan="3">1.2</td></tr>
<tr><td>4</td><td colspan="3">1.8</td></tr>
<tr><td>5</td><td colspan="3">2</td></tr>
<tr><td>6</td><td colspan="3">2</td></tr>
<tr><td>7</td><td colspan="3">2</td></tr>
<tr><td>8</td><td colspan="3">3</td></tr>
<tr><td>9</td><td colspan="3">3.8</td></tr>
<tr><td>10</td><td colspan="3">4.2</td></tr>
<tr><td>11</td><td colspan="3">8</td></tr>
<tr><td rowspan="2"></td><td rowspan="2">Software calculations</td><td colspan="2">Excel calculations</td></tr>
<tr><td>PERCENTILE.INC</td><td>PERCENTILE.EXC</td></tr>
<tr><td>Weighted average</td><td>2.742</td><td></td><td></td></tr>
<tr><td>Standard Deviation</td><td>2.034</td><td></td><td></td></tr>
<tr><td>80th %-ile (80P)</td><td>4.451</td><td>3.8</td><td>4.04</td></tr>
</table>

Application of percentiles

The percentile values to be used for the design depend on the anticipated long-term in-service moisture content in the various pavement layers in relation to the moisture condition at the time of the DCP survey as well as the design traffic loading, as shown in Table 7-9. Note that all pavement layers must be evaluated independently as the changes in moisture content may not be the same for all layers, including the subgrade.

Table 7-9: Application of percentiles

<table>
<tr><th rowspan="2">Anticipated long-term in-service moisture content in the pavement layer</th><th colspan="2">Percentile value of DN values in uniform section</th></tr>
<tr><th>Design Traffic < 0.5 MESA</th><th>Design Traffic 0.5 – 1.0 MESA</th></tr>
<tr><td>Drier than at time of DCP survey</td><td>20</td><td>30</td></tr>
<tr><td>Same as at time of DCP survey</td><td>50</td><td>50</td></tr>
<tr><td>Wetter than at time of DCP survey</td><td>80</td><td>90</td></tr>
</table>

Appendix 7.3 – Data analysis by use of a spreadsheet

Appendix 7.2 shows how the DCP data are analysed in the software. It is also possible to analyse both field and laboratory test data manually in a spreadsheet, as shown below.

Analysis of DCP field test data

Table 6-11 shows the complete DCP readings for one test point. Approximate Weighted Average DN values can be calculated by means of the Excel “SUMPRODUCT” function, as shown:

=SUMPRODUCT(array1,array1)/(layer thickness)

where array1 = penetration per reading over the layer thickness

array2 = DN per reading over the layer thickness

Layer thickness = cumulative penetration spanning the upper and lower layer boundaries

The colour shading indicates the ranges over which the Weighted Average DNs have been calculated.

Table 7-10: Analysis of data for one DCP test point

Blows per reading	Cumulative blows	Readings	Penetration per reading	Cumulative penetration	DN (mm/blow)	Weighted average DN per layer	
0		132		0			
5	5	141	9	9	1.80		
5	10	146	5	14	1.00		
5	15	151	5	19	1.00		
5	20	159	8	27	1.60		
5	25	163	4	31	0.80		
5	30	167	4	35	0.80		
5	35	167	0	35	0.00		
5	40	171	4	39	0.80		
5	45	174	3	42	0.60		
5	50	177	3	45	0.60	0-150 mm 1.93	
5	55	182	5	50	1.00		
5	60	188	6	56	1.20		
5	65	197	9	65	1.80		
5	70	205	8	73	1.60		
5	75	215	10	83	2.00		
5	80	227	12	95	2.40		
5	85	240	13	108	2.60		
5	90	253	13	121	2.60		
5	95	267	14	135	2.80		
5	100	281	14	149	2.80		
5	105	293	12	161	2.40		
5	110	305	12	173	2.40		
5	115	319	14	187	2.80		
5	120	331	12	199	2.40		
5	125	346	15	214	3.00		150-300 mm 3.91
5	130	361	15	229	3.00		
5	135	378	17	246	3.40		
5	140	399	21	267	4.20		
5	145	425	26	293	5.20		
5	150	450	25	318	5.00		
5	155	494	44	362	8.80	300-450 mm 10.82	
5	160	542	48	410	9.60		
5	165	608	66	476	13.20		
5	170	668	60	536	12.00		450-600 mm 20.05
5	175	788	120	656	24.00		
5	180	915	127	783	25.40	600-800 mm 26.88	
5	185	1042	127	910	25.40		

In this manner, a Layer Strength Diagram for each test point can be determined. After the determination of uniform sections, the Weighted Average DN values for each layer and uniform section can be calculated and entered into an Excel table, as shown in Figure 7-6, for completion of the design following the outlined design procedure.

Analysis of DCP laboratory test data

Table 7-10 shows a data sheet for recording and analysing DCP laboratory test data. The Weighted Average DN value for the sample is calculated using the Excel "SUMPRODUCT" function, as shown:

=SUMPRODUCT(array1,array1)/(penetration depth)

where array1 = DN per 'n' blows
array2 = average DN per blow
penetration depth = last reading – zero reading

Table 7-11: Data sheet for recording and analysis of laboratory DCP test data

No of blows n	DCP Reading	DN per n blows	Avg. DN per blow
0	98		
5	120	22	4.40
5	138	18	3.60
5	154	16	3.20
5	170	16	3.20
5	187	17	3.40
5	205	18	3.60
3	218	13	4.33
Penetration depth		120	
Weighted Average DN			3.69

The value obtained in this manner is likely to differ on the conservative side from the "best fit" value estimated by the software, as shown in *Chapter 5 – Materials*, Appendix 5.1. As is often the case, the first and last average DN value is higher than the values from the middle of the sample. These values may, therefore, be disregarded in the calculations.

For the determination of a representative DN value, an average of three results for each compactive effort and moisture content should be calculated.

8. Structural Design: Gravel Roads

8.1 Introduction

8.1.1 Background

The majority of the rural road network in developing countries consists of unpaved roads. Although often rudimentary, these roads provide communities with access to important services (schools, clinics, hospitals and markets) and are the basis of a thriving market and social environment.

Although it would be desirable to upgrade many of these roads to a sealed standard, a large network of important unpaved earth and gravel roads will remain for the foreseeable future. Thus, it is important that these roads are designed and maintained in the most cost-effective manner.

Unpaved roads are defined in this Manual as any road that is not surfaced with a non-structural "waterproof" bituminous surfacing or structural surfacing such as concrete, interlocking blocks, cobblestones or similar.

Unpaved roads will typically carry a maximum of about 200 - 300 vehicles per day (with less than 10 % being heavy), but in areas where materials are poor, upgrading to paved standard can often be economically justified at traffic volumes much lower than this.

8.1.2 Purpose and Scope

The purpose of this chapter is to provide details for the design of gravel roads in an economical and sustainable manner such that the appropriate levels of quality are produced.

The chapter covers the design of gravel roads based on the DCP-DN design method. Material selection and thickness design are treated in detail.

8.2 Design of Gravel Roads

8.2.1 General

Roads described as gravel roads imply that several factors have been taken into account in their design and construction. These include:

- Material of a selected quality is used to provide an all-weather wearing course.
- The structure of the road and strength of the materials is such that the subgrade is protected from excessive strains under traffic loads.
- The shape of the road is designed to allow drainage of water (mainly precipitation) from the road surface and from alongside the road.
- The necessary cross and side drainage are installed.
- The road is constructed to acceptable standards, including shape, compaction and finish.

Although an all-weather wearing course is provided, the road may not necessarily be passable at all times of the year as a result of low-level water crossings being flooded periodically.

8.2.2 Pavement Structure

A gravel road consists of a wearing course and a structural layer (base), which covers the in-situ material. In many cases, the same material could be used for both the structural layer and the wearing course. The minimum thickness of the structural layer is maintained in service by providing a wearing course throughout the design life of the road, which should under no circumstances be allowed to become thinner than 50 mm.

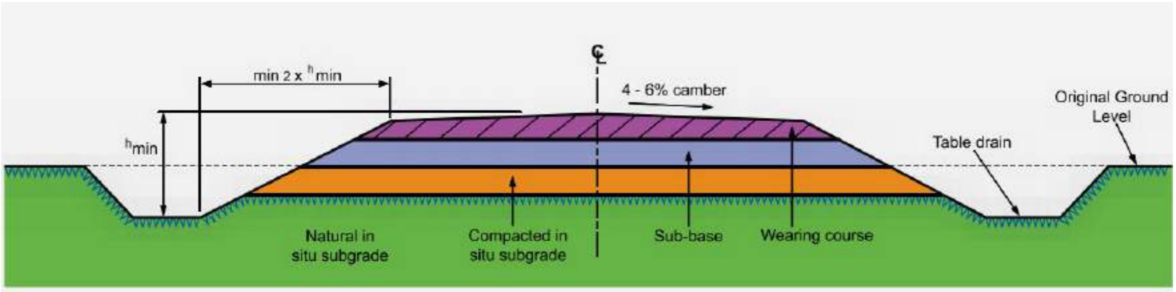


Figure 8-1: Typical gravel road cross section in flat terrain

To achieve adequate external drainage, the road must also be raised above the level of existing ground such that the crown of the road is maintained at a minimum height (h_{min}) above the drain inverts. The minimum height is dependent on the climate and road design class, as shown in Table 8-1.

Table 8-1: Minimum height (h_{min}) of road crown above drain invert

Traffic volume (vpd)	Climate factor (Weinert N value)	
	Wet (N<4)	Dry (N>4)
	h_{min} (mm)	
> 300	550	450
200-300	500	400
100-200	450	350
50-100	400	300
< 50	350	250

8.2.3 Materials

Material selection is the most critical aspect of gravel road design. The use of incorrect materials in the wearing course will result in roads that deform, corrugate, become slippery when wet, lose gravel rapidly, and generate excessive dust. Table 8-2 summarises the required properties of suitable wearing course gravels.

Table 8-2: Specification requirements for wearing course materials for unpaved roads

Maximum nominal size	37.5 mm
Minimum percentage passing 37.5 mm	95
Shrinkage product (S_P)	100 – 365 (240)
Grading coefficient (G_c)	16 – 34
Min DN value (mm/blow)	13.5 at 95% AASHTO T180 compaction (soaked)
Treton Impact value (%)[1]	20 – 65

Source: Paige-Green P (1989)

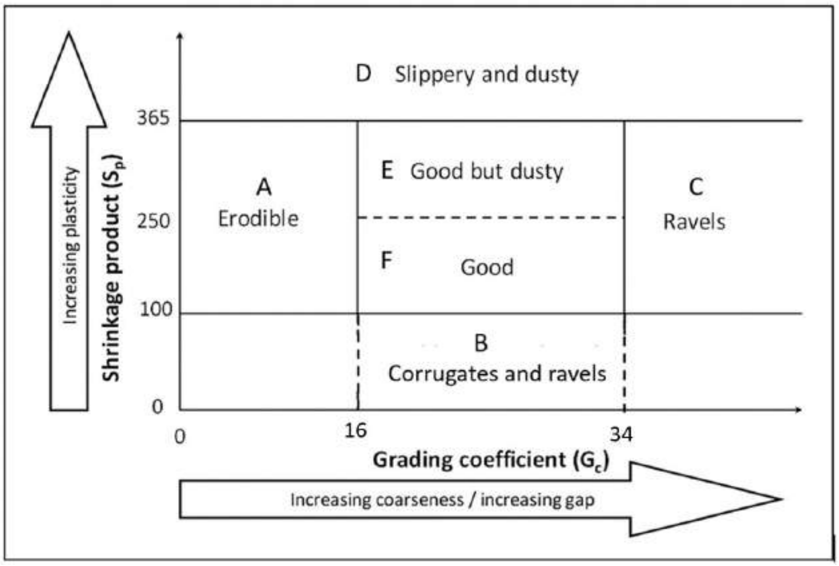
The Treton Impact Value is not a standard test but is described in TMH 1 (1985). It is a simple test and makes use of equipment that can be easily manufactured (see Appendix 8.1). The Treton Impact Value differentiates between aggregate particles that will perform well (20 to 65), aggregates that are too soft and will disintegrate under traffic (> 65), and aggregates that are too hard to be broken down by conventional or grid rolling during construction and will result in stony roads if large particles are not removed. No correlation currently exists with a similar BS Aggregate Impact Test.

The recommended grading and cohesion (shrinkage) specifications for gravel wearing course materials can also be shown diagrammatically concerning their predicted performance defined by the values of the Shrinkage Product and Grading Coefficient, as shown in Figure 8-2, where:

Shrinkage Product (SP) = Bar Linear Shrinkage$_{0.425}$ x $P_{0.425}$

Grading Coefficient (GC) = ($P_{26.5}$ - $P_{2.0}$) x $P_{4.75}$/100

The manner of determining the Bar Linear Shrinkage is shown in Appendix 8.2.



Source: Department of Transport, South Africa (1990)

Figure 8-2: Chart showing performance of unpaved road materials

In the chart presented in Figure 8-2, the 6 zones indicated (A to F) show the expected performance of materials as follows:

- Zone A: Fine-grained material prone to erosion.
- Zone B: Non-cohesive materials that lead to corrugation and ravelling/loosening.
- Zone C: Poorly-graded materials that are prone to ravelling.
- Zone D: Fine plastic material prone to slipperiness and excessive dust.
- Zone E: Good performance, but dusty in dry environments.
- Zone F: Optimum materials for best performance.

Requirements for both material and aggregate strength are provided. The material strength is specified as the soaked DCP-DN value (13 mm/blow), which initially appears very low, but the investigation of many roads in various countries has shown that material with a strength as low as this will not shear or deform under the passage of an 80 kN axle load (20 kN single tyre load), even when soaked. Materials of significantly higher quality than this should be preserved for later use in paved roads.

Figure 8-2 can be used to identify potential problems that could affect the road should the materials not fall into Zone E or F. These can be taken into account, and engineering judgement used to override the limits where necessary. For instance, in arid areas where rainfall is rare, the need to limit the upper shrinkage limit can be re-evaluated. Consideration may be given to using a high plasticity material in these areas with appropriate warning signs, provided that the road has no steep grades or sharp bends. Similarly, roads with light, slow-moving traffic are unlikely to corrugate, and non-cohesive materials could be considered under these conditions or if the application of regular light surface maintenance is possible.

In-situations where natural materials are scarce, performance results have shown that blended materials can work well. Successful blends can be obtained through:

- mixing non-plastic sand with clayey sand;
- mixing non-plastic sand with high PI calcrete; and/or
- mixing clayey material with low plasticity gravels (derived from granite and limestone).

Before blending, laboratory tests should be performed to ensure that the blends produce the required DN values and that the blended materials meet the selection criteria specified in Table 8-2. The laboratory testing should use various blend ratios to determine which are best, and these ratios must be carefully adhered to and controlled during construction. The use of material not complying with the specifications can result in severe deformation, rutting and impassability when wet.

Figure 8-3 illustrates the importance of ensuring that the material properties are as close as possible to the ideal.

Transverse erosion in fine material

Longitudinal erosion in fine material

Corrugation and ravelling

Ravelling

Dusty and rough due to oversize material

Slippery when wet

Good

Good, but dusty

Source: Jones and Paige-Green, 2015

Figure 8-3: Examples of gravel wearing course performance

8.2.4 The DCP-DN design procedure

General

The mechanism of deterioration of unpaved roads differs from that of paved roads and is directly related to the number of vehicles using the road rather than the number of equivalent standard axles. The traffic volume is therefore used in the design of unpaved roads, as opposed to the paved roads, which require the conversion of traffic volumes into the appropriate cumulative number of equivalent standard axles.

Unlike paved roads, any minor deformation of the support layers beneath the gravel wearing course does not unduly influence the performance of the road. The reason for this is that in paved roads, the cumulative deformation in the subgrade ultimately leads to rutting of the bituminous surfacing over the design or service life of the road, whereas in unpaved roads, any minor rutting or deformation (excluding serious shear failures) is rectified during routine grader maintenance and traffic wander. Even shear failures, although undesirable, are usually repaired (at least temporarily) during routine grader maintenance.

The need to invest in a series of structural layers is thus seldom warranted for unpaved roads. However, several decisions are required during the design to satisfy the following requirements:

- The wearing course must be raised above the surrounding natural ground level to avoid moisture accumulation (see Table 8-1) and to allow pipes and culverts for cross-road drainage to pass beneath/through the road.
- The material imported to raise the formation should be of a specified quality.
- Very weak or volumetrically unstable subgrade materials must be taken care of by removing, treating or covering with an adequate thickness of stable material – heave and collapse are seldom significant problems on unpaved roads, being smoothed out during routine maintenance.
- Should re-gravelling operations be delayed until the gravel has completely worn away (which is a regular occurrence in many countries), a "buffer" layer of suitable quality material should be in place to avoid vehicles travelling on very weak material.
- The maintenance capacity and frequency are thus important considerations in the pavement design.

If it is likely that the road will be upgraded to paved standard within 6 years to 10 years after construction, selected materials complying with the requirements for lower layers in the paved road design standards should be used.

Determination of subgrade strength

For the design of the pavement structure, it is necessary to assess the subgrade conditions for gravel roads and to base the pavement structure on these in order to obtain a balanced design. In a similar manner to the method described in *Chapter 6 – Structural Design: Paved Roads*, the subgrade should be divided into uniform sections based on a DCP survey. Slightly different methods will be used for the design of a new road as compared to the improvement of an existing earth road.

Design procedure

The general procedure for the design of gravel roads is the same as for paved roads described in Chapter 6, with the following modifications:

- At least 5 DCP tests to 800 mm depth should be carried out per kilometre of road.
- **For an existing road:** Tests should alternate between the outer wheel tracks in each direction.
- **For a new road:** Tests should alternate with 2.0 m offsets to the left and right of the center-line after removing the upper soil layer containing humus, vegetable matter, or any other undesirable materials.

- **For both existing and new roads:** If the subgrade conditions appear to be highly variable, the frequency of testing should be increased, even up to one test per 50 m, if necessary.
- Determine the Weighted Average DCP penetration (DN value) rate for the upper 150 mm and the 150 -300 mm layers of the existing structure or the exposed subgrade (DN_{150} and $DN_{150-300}$).
- Determine the DCP structural number (DSN_{800} or number of blows to penetrate 800 mm).
- Determine uniform sections based on the DN_{150}, $DN_{150-300}$ and DSN800 curves using the ReCAP LVR DCP software. If the uniform sections delineated by these three parameters differ significantly, it is necessary to look at the individual DCP profiles and decide whether the differences are significant. Low DSN_{800} values indicate weak support, while high DN_{150} values indicate that the upper 150 mm of the road is weak.
- Once the uniform sections have been defined, the subgrade can be classified in terms of its required strength to carry the expected traffic.

  DCP testing is carried out at in-situ moisture and density conditions. It is recommended that the testing is done at the end of the wet season when the subgrade is probably in or near its worst moisture condition, but this is not always possible. The same procedure as described in Chapter 6 should be applied for the determination of the characteristic subgrade strength of the two upper layers, DN_{150}, $DN_{150-300}$, for each uniform section at the anticipated field density and long-term moisture content. For gravel roads, the soaked Laboratory DN values should normally be applied.
- Compare the relevant subgrade strength profiles with the necessary design given in Table 8-3, for the specified traffic categories.
- For new roads, it should be borne in mind that the upper 150 mm layer will at least be ripped and recompacted as the in-situ material and that formation material will usually be imported to raise the level of the road above natural ground level.

It is not possible to determine the DSN_{800} based on the Laboratory DN_{150} and $DN_{150-300}$. However, as long as these satisfy the criteria in Table 8-3, the DSN_{800} requirement will also be above the lower limit, unless the layers below 300 mm depth are particularly weak.

It should be noted that the upper two layers are critical, the underlying layers being given values in an attempt to improve the pavement balance. It can be seen that the in-situ strengths of the third layer (300 mm – 450 mm) and below range from 19 mm to 50 mm/blow, which are likely to occur in most situations. If these do not compare adequately (low DSN_{800}), additional thickness of material at the surface may be necessary. It should also be borne in mind that in most cases, some formation material is likely to be placed on this in-situ profile, with this imported material having an in-situ DN value of between 14 mm and 25 mm/blow depending on the traffic.

Table 8-3: Gravel road pavement design for different traffic categories (DN)

Layer, depth and DCP Structural Number	Traffic (Heavy vehicles/day)			
	≤ 2	2 – 6	7 – 20	21 -60
	Soaked DN (mm/blow)			
150 mm Wearing course (Table 7-2)	13	13	13	13
Formation or upper 150 mm ≥ 95% AASHTO T180 Density	25	19	14	13
In-situ (rip & compact) 150 mm -300 mm ≥ 93% AASHTO T180 Density	33	25	19	13.5
300 – 450 mm	50	33	25	19
450 – 600 mm	50	50	33	25
600 – 800 mm	50	50	50	33
DSN_{800}	21	25	33	41
Heavy vehicles are defined as those vehicles classified as Heavy Goods Vehicles (HGV)				

If the in-situ profiles compare adequately with the layer strength diagrams, the wearing course layer can be placed on top. This would normally consist of 150 mm of specified material, as shown in Table 8-2 and Figure 8-2, but if the potential for delayed maintenance (i.e., re-gravelling) exists, an additional 50 mm should be added as a buffer layer.

The minimum strength of the support layer beneath the wearing course need not be very high and actually becomes equal to the required minimum strength of the wearing course for higher traffic. This material may not have the necessary cohesive or grading properties to provide the necessary performance as a wearing course but must always be present. If this material complies with the requirements of Zone E or F in Figure 8-2, the total thickness of the upper 150 mm formation and the wearing course can be reduced to 225 mm.

It should be pointed out that the design is based on the number of heavy vehicles per day and not cumulative axle loads as traditionally used for paved roads. This is a result of the mode of distress being related to shear failure of the layers under loading as opposed to cumulative deformation with time, which is removed during routine maintenance and re-gravelling. The reliability of the design is thus accepted as being slightly lower as the repair of any possible failures is much less disruptive than traditional paved road repairs.

Wearing course thickness design

This must take into account the fact that gravel will be lost from the road continuously. Other than the road user costs, this is the single most important reason why gravel roads are expensive, and often unsustainable, in whole-life cost terms, especially when traffic levels increase.

Reducing gravel loss by selecting better quality gravels, modifying the properties of poorer quality materials, and ensuring high levels of compaction is one way of reducing long term costs. Gravel loss (gravel loss in mm/year/100 vpd) is a function of several factors: climate, traffic, material quality, road geometrics, maintenance frequency and type, etc., and can be predicted using various models. These, however, often need regional calibration, but an approximate estimate can be obtained from Table 8-4.

Table 8-4: Typical estimates of gravel loss

Material Quality Zone	Material Quality	Typical gravel loss (mm/year/100 vpd)
Zone A	Satisfactory	20
Zone B	Poor	40
Zone C	Poor	40
Zone D	Marginal	20
Zone E	Good	15

The gravel losses shown in Table 8-4 hold only for the first phase of the deterioration cycle lasting possibly two or three years. Beyond that period, as the wearing course is reduced in thickness, other developments, such as the formation of ruts or heavy grader maintenance, may also affect the loss of gravel material. However, the rates of gravel loss given above, can be used as an aid to the planning for re-gravelling in the future.

The rates of gravel loss increase significantly on gradients greater than about 6% and in areas of high and intense rainfall and should be allowed for in the estimation of required re-gravelling operations. Spot improvements should be considered in these sections.

Re-gravelling should take place before the underlying layer is exposed. The re-gravelling frequency, R, is typically in the range 5 - 8 years, provided the wearing course is compacted to specification.

The optimum wearing course thickness is expressed as R x GL, where:

R = re-gravelling frequency in years and GL = expected annual gravel loss (mm/yr/100vpd).

Where suitable sand is available adjacent to the road, the application of a sand cushion (25 to 40 mm) on top of the wearing course allows low-cost regular maintenance of the road and preserves the wearing course from wear and material loss as long as the sand covers the road.

8.3 Treated Gravel Roads

It is often difficult to locate suitable materials for unpaved roads or costly to haul them from some distance away. Numerous proprietary chemicals are being marketed that claim to improve almost any soil to a quality suitable for road construction. These chemicals can have mixed results and are very material dependent.

There are essentially two uses of these chemicals – those used for dust palliation and those used for soil stabilization/improvement. Despite these main uses, there can be some overlap in that, for example, dust palliatives may strengthen the upper part of the treated layer and reduce gravel loss. The chemical products can be applied by surficial spraying or mixing in. Again, certain products are better and more cost-effectively mixed-in (at greater cost) than being sprayed on the surface of the road.

No general guidance on the use of the chemicals can be given as the types, actions and uses can differ widely. However, the following aspects should be considered before using any chemical:

- Is the use of the chemical going to be cost-effective and give some kind of financial, social or environmental benefit that is value for money? Are proprietary products more cost-effective than generic products? Will it be more cost-effective to import better material from further away?
- Does the chemical consistently increase the strength of the material, if it is to be used as a stabiliser? This can be checked in a laboratory using the DN test. However, it has been found that the application rate is critical and that some materials react better with chemicals. This may vary considerably within a material source, and ongoing testing of the compatibility between material and chemical must, therefore, be carried out.
- Products used for dust palliation are best tested on short sections of a road before full-scale use. It is very difficult to test their effectiveness in the laboratory as a result of the speed and abrasion of vehicles that generate dust.
- Many of the chemical products are costly, and where used, it may often be more cost-effective to place a bituminous surfacing on the material to conserve it for the full life of the road than to allow it to be lost in the normal gravel loss process. The gravel loss may be reduced, but the road is still an unpaved road and will still be subjected to traffic and environmental erosion and material loss.
- Potential environmental impacts.

Bibliography

Ahlvin R G and G M Hamitt (1975). ***Load-supporting Capability of Low-volume Roads.*** Special Report 160. Transportation Research Board. National Academy of Sciences, Washington D C., USA.

ARRB (2000). Unsealed roads manual: ***Guideline to good practice.*** ARRB Transport Research Ltd., Australia.

ASTM D6591/D6591M-18 (2018). ***Standard Test Method for the Use of the Dynamic Cone Penetrometer in Shallow Pavement Applications.*** ASTM International, West Conshohocken, PA., USA.

Department of Transport, South Africa (1990). ***Draft TRH 20: Structural Design, Construction and Maintenance of Unpaved Roads***. Pretoria, South Africa.

Jones D and P Paige-Green (2015). ***Limitations of Using Conventional Unpaved Road Specifications for Understanding Unpaved Road Performance***. 11th TRB Low Volume Road Conference, Pittsburgh, USA 12th – 15th July 2015.

Ministry of Works, Tanzania (1999). ***Pavement and Materials Design Manual***. TANROADS, Dar es Salaam, Tanzania.

Netterberg F and P Paige-Green (1988). ***Wearing courses for unpaved roads in southern Africa: A review.*** Proc. 1988 Annual Transportation Convention, S.443, Vol. 2D, Pretoria, South Africa.

Paige-Green P (1989). ***The influence of geotechnical properties on the performance of gravel wearing course materials.*** Ph.D. Thesis, University of Pretoria, Pretoria, South Africa

Appendix 8.1: Method of test for Treton Impact Value

Step 1

From the field sample, screen out a sufficient quantity (at least 200 g) of the -19.0 + 16.0 mm fraction. If the aggregate is noticeably variable as regards type or hardness, each type should be tested and reported separately. In this case, an estimate should be made of the percentage of each type.

Step 2

Select 15 to 20 of the most cubical pieces. Weigh the aggregate pieces to an accuracy of 1 g, and place them as evenly spaced as possible on the anvil shown in the Figure in such a manner that their tops are approximately in the same horizontal plane.

Step 3

Place the cylinder over the anvil and tighten the clamp screws. Place the hammer in the cylinder so that the top of the hammer is level with the top of the cylinder and let it drop ten times from this position.

Step 4

Remove the cylinder and sieve all the aggregate on the anvil and base plate thoroughly through a 200 mm sieve. Weigh the aggregate retained on the sieve to the nearest 0.1 g and record the mass. The test should be carried out in triplicate. (If any individual result differs from the others by more than five units, further tests should be carried out.)

Step 5

Calculate the Treton value to the first decimal place as follows:

$$\text{Treton value} = (A - B)/A \times 100$$

where

A = the mass of the stone particles before tamping (g)

B = the mass of the stone particles retained on the 2.0 mm sieve after tamping (g).

Report the value to the nearest whole number.

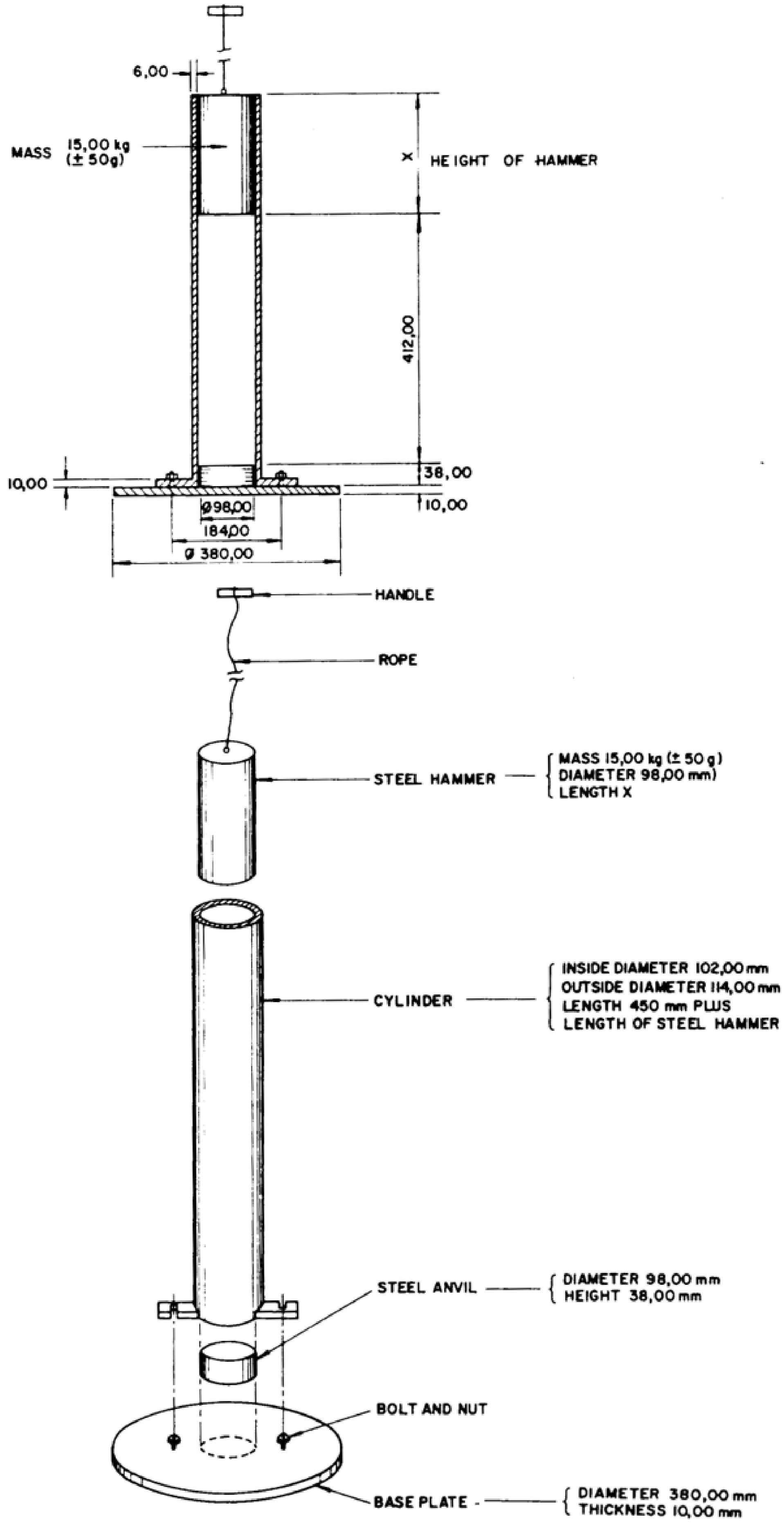


Figure 8-4: Treton Impact test apparatus

Appendix 8.2: Method of test for Bar Linear Shrinkage

Step 1

The interior of a clean, dry shrinkage mould is sprayed evenly with the silicone lubricant. The shrinkage moulds have internal dimensions of 150 ± 0.25 mm long x 10 ± 0.25 x 10 ± 0.25 mm and made of 10 mm thick stainless-steel bar, open on two sides.

Step 2

The fines (fraction passing 0.425 mm) saved during a grading analysis are used for this test. Add water from a squeeze bottle to the fines and mix thoroughly until the consistency is at the liquid limit.

Step 3

The lubricated mould should be placed on the plate provided, and one half should be filled with the moist soil by taking small pieces of soil on the spatula and pressing the soil down against the one end of the mould and working along the mould until the whole side is filled and the soil forms a diagonal surface from the top of one side to the bottom of the opposite side (see Figure a).

The mould is now turned around and the other portion is filled in the same manner (see Figure b). The hollow along the top of the soil in the mould is then filled so that the soil is raised slightly above the sides of the mould (see Figure c). The excess material is removed by drawing the blade of the spatula once only from the one end of the mould to the other. The index finger is pressed down on the blade so that the blade moves along the sides of the mould (see Figure d). During this process, the wet soil may pull away from the end of the mould, in which case it should be pushed back gently with the spatula. On no account should the surface of the soil be smoothed or finished off with a wet spatula.

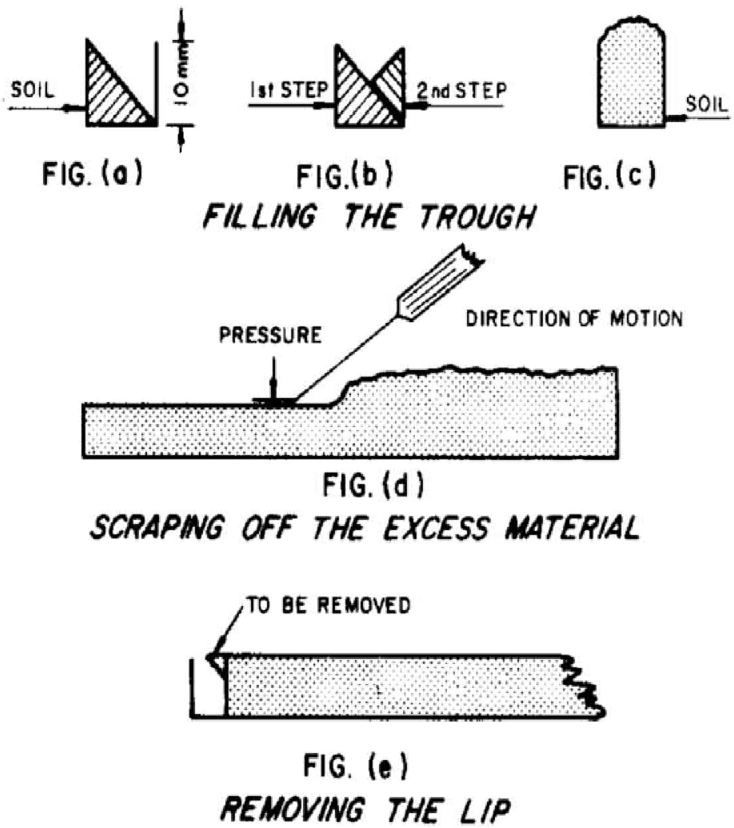


Figure 8-5: Preparation of material for the bar linear shrinkage test

Step 4

The filled mould is placed in the drying oven and dried at a temperature of between 105 and 110°C until no further shrinkage can be detected. As a rule, the material is dried out. The mould with the material is taken out of the oven and allowed to cool.

Step 5

It may be found that the ends of the dry soil bar have a slight lip or projecting piece at the top. These lips should be removed by abrading with a sharp, narrow spatula so that the end of the soil bar is parallel to the end of the mould (see Figure e). If the soil bar is curved, it should be pressed back into the mould with the finger-tips so as to make the top surface as level as possible.

The loose dust and sand, removed from the ends, as well as any loose material between cracks should be emptied out of the mould by carefully inverting the mould whilst the material is held in position with the fingers. The soil bar is then pressed tightly against one end of the mould. It may be noticed that the soil bar fits better at the one end than at the other end. The bar should be pressed tightly against the end at which there is a better fit. The gap between the soil bar and the end of the mould is measured by means of a good pair of dividers, measuring on a millimeter scale, to the nearest 0.5 mm and recorded.

Step 6

The bar linear shrinkage (BLS) is calculated from the measured shrinkage LS (in mm) as follows:

$$BLS = LS \times 0.67\ (\%)$$

Note: After the test, the soil bar should be examined to ensure that the corners of the mould were filled properly and that no air pockets were contained in the soil bar. If air pockets were contained, the material should be retested.

9. Surfacing

9.1 Introduction

9.1.1 Background

The surfacing of any road plays a critical role in its long-term performance. It prevents gravel loss, eliminates dust, improves skid resistance, and reduces water ingress into the pavement. The latter attribute is especially important for LVRs where moisture sensitive materials are often used.

There are several surfacing options, both bituminous and non-bituminous, that are available for use on LVRs. They offer a range of attributes that need to be matched to such factors as expected traffic levels and loading, locally available materials and skills, construction and maintenance regimes, road safety concerns, and the environment. Careful consideration should, therefore, be given to all of these factors in order to make a judicious choice of surfacing to provide satisfactory performance and minimize life cycle costs.

9.1.2 Purpose and Scope

The main purpose of this chapter is to provide a broad overview of:

- The various types of bituminous and non-bituminous surfacings that are potentially suitable for use on LVRs.
- The performance characteristics and typical service lives of the various types of surfacings.
- The factors that affect the choice of surfacings.

Thick bituminous surfacings (> 30 mm), due to their relatively high cost, are generally not appropriate for use on LVRs and are not considered in this chapter.

9.2 Bituminous Surfacings

9.2.1 General

The term "bituminous surfacings" applies to a wide variety of different types of road surfacings, all of which are generally comprised of an admixture of varying proportions of sand, aggregate and bitumen. Such surfacings may be constructed in a variety of ways depending on their particular function and serviceability requirements – single/multiple, thin/thick, flexible/rigid, machine laid/plant processed, etc. Some types, e.g., surface treatments and thin asphalt concrete (<30 mm), do not add any structural strength to the pavement, whilst others, e.g., thick asphalt concrete (> 30 mm) do provide a structural component to the pavement structure. Ultimately, the type of surfacing chosen should be carefully matched to the specific circumstances.

9.2.2 Main Types

Terminology for different surfacing types varies for different countries within the region. For purposes of this Manual, Figure 9-1 illustrates the main types of bituminous surfacings that are potentially suitable for use on LVRs.

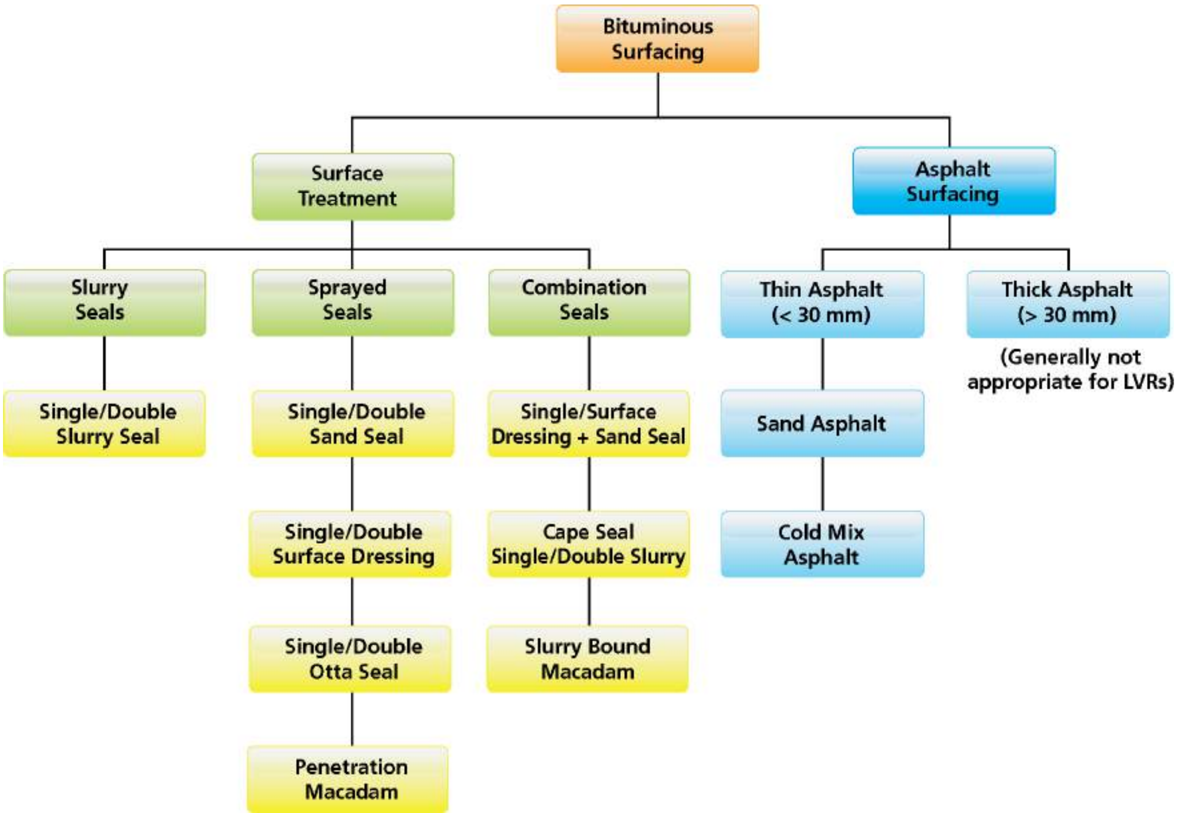


Figure 9-1: Terminology and categorisation of bituminous surfacings

Some of the typical types of bituminous surfacings used on LVRs are shown in Figure 9-2.

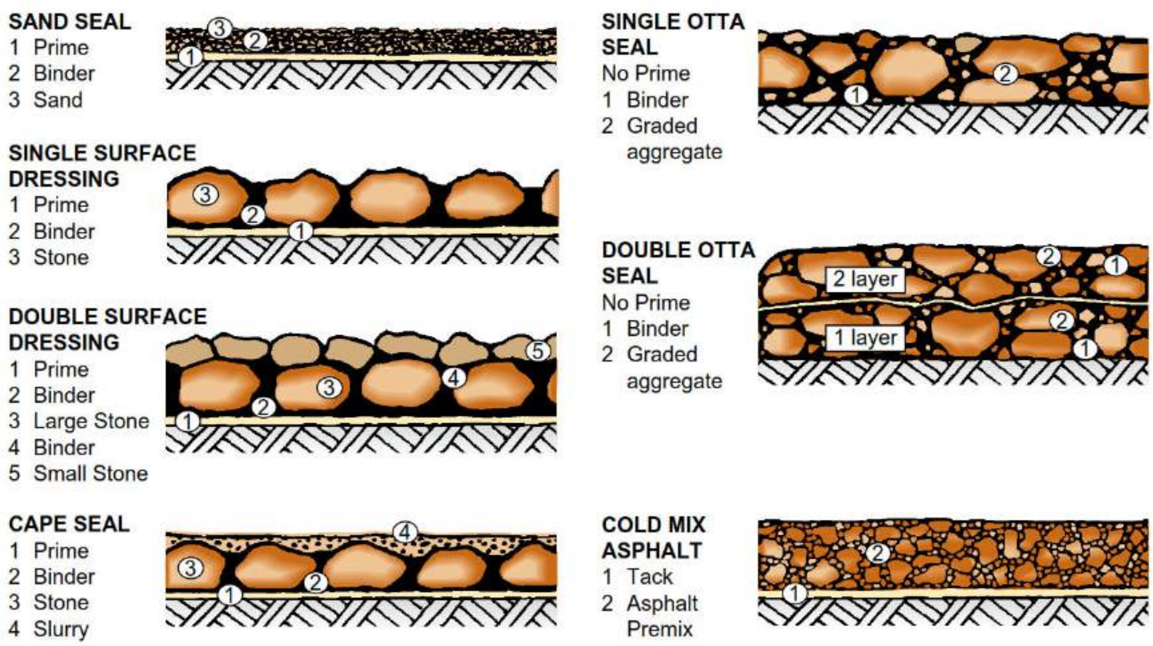


Figure 9-2: Common types of bituminous surfacings

9.2.3 Performance Characteristics

The various types of bituminous surfacings may be placed in two categories as regards their mechanism of performance, which is illustrated in Figure 9-3.

Figure 9-3: Different performance mechanisms of bituminous surfacings

Category A (e.g., Sand Seal, Otta Seal, Cold Mix Asphalt): These seal types, like hot mix asphalt, rely to a varying extent on a combination of mechanical particle interlock and the binding effect of bitumen for their strength. Early trafficking and/or heavy rolling are necessary to develop the relatively thick bitumen film coating around the particles.

Category B (e.g., Surface Dressing): These seal types rely on the binder to "glue" the aggregate particles to the primed base course. Where shoulder to shoulder contact between the stones occurs, some mechanical interlock is mobilised. Under trafficking, the aggregate is in direct contact with the tyre and requires relatively high resistance to crushing and abrasion to disperse the stress without distress. Should the bitumen/aggregate bond be broken by traffic, or should there be poor aggregate/binder adhesion, insufficient material strength, or oxidation and embrittlement of the binder, then "whip-off" of the aggregate is almost inevitable.

Table 9-1 indicates the relative difference in the required properties between the various surfacing types.

Table 9-1: Differences in required properties of main types of bituminous surfacings

Parameter	Category A	Category B
Aggregate Quality	Less stringent requirements in terms of aggregate strength, grading, particle shape, binder adhesion, dust content, etc. Allows extensive use to be made of natural gravels.	More stringent requirements in terms of strength, grading, particle shape, binder adhesion, dust content, etc. Allows limited use to be made of locally occurring natural gravel.
Binder type	Relatively soft (low viscosity) binders or emulsion are required.	Relatively hard (high viscosity) binders are normally used.
Design	Empirical approach. Relies on guideline and trial design on site. Amenable to design changes during construction.	Rational approach. Relies on confirmatory trial on site. Not easily amenable to design changes during construction.
Construction	Less sensitive to standards of workmanship. Labour-based approaches relatively easy to adopt if desired.	Sensitive to standards of workmanship. Labour-based approaches less easy to adopt if desired.
Durability of seal	Enhanced durability due to the use of relatively soft binders and, in the case of the Otta Seal, a dense seal matrix.	Reduced durability due to use of relatively hard binders and open seal matrix.

9.2.4 Typical Service Life

The life of a surface treatment depends on a wide range of factors such as the quality of the design, climate, pavement strength, binder durability, the standard of workmanship, adequacy of maintenance, etc. As a result, the service life of the surfacing can vary widely. In general, however, thin single seals have much shorter service lives (generally < 7 years) than double/combination seals (generally > 7 years). Typical service lives of bituminous surfacings are presented in Table 9-2.

Table 9-2: Typical lives of bituminous surfacings

Type of surfacing	Typical service life (years)
a) Thin/single seal	
• Single Sand Seal	2 – 3
• Double Sand Seal	3 – 6
• Single Slurry Seal	3 – 5
• Single Surface Dressing	5 – 7
b) Thick/double seal	
• Single Surface Dressing + Sand Seal	6 – 8
• Double Surface Dressing	8 – 10
• Cold Mix Asphalt	8 – 10
• Single Otta Seal	8 – 10
• Single Otta Seal + Sand Seal	10 – 12
• Cape Seal (13 mm + Single Slurry)	10 – 12
• Cape Seal (19 mm + Double Slurry)	12 – 15
• Double Otta Seal	15 – 18
• Penetration Macadam	8 – 12
• Slurry Bound Macadam	8 – 10
• Sand Asphalt	8 – 10
• Thin Asphalt (< 30 mm)	8 – 10

9.2.5 General Characteristics

The general characteristics of the different types of bituminous surfacings are summarised in Table 9-3.

Table 9-3: General characteristics of bituminous surfacings

Surfacing	Characteristics
Sand Seal	• Empirical design. • Consists of a film of binder (cutback bitumen or emulsion) followed by a graded natural sand or fine sand-sized machine or hand-broken aggregate (max. size typically 6 mm – 7 mm), which must then be compacted. • Single sand seals are not very durable, but performance can be improved with the application of a second seal after 6 months -12 months, depending on traffic. Should then last for another 6 – 7 years before another seal would be needed. • Especially useful if good aggregate is hard to find. • Very suitable for labour-based construction, especially where emulsions are used, and requires simple construction plant. • Need to be broomed back into the “worn” wheel tracks. There is an extended curing period (typically 8 weeks – 12 weeks) between the first and second seal applications to ensure the complete loss of volatiles and thus prevent bleeding. During this period, the sand may need to be broomed back into the “worn” wheel tracks.

Table 9-3 Continued: General characteristics of bituminous surfacings

Surfacing	**Characteristics**
Slurry Seal	• Rational design with both simplified and detailed approaches. • Consists of a mixture of fine aggregates, Portland cement, emulsion binder and additional water to produce a thick creamy consistency which is spread to a thickness of 5 mm - 15 mm. • Can be used on LVRs carrying only light traffic. More typically used for re-texturing surface dressings prior to resealing or for constructing Cape seals. • Very suitable for labour-based construction using relatively simple construction plant (concrete mixer) to mix the slurry. • Thin slurry (5 mm) is not very durable; performance can be improved with the application of a thicker (15 mm) slurry.
Otta Seal	• Empirical design. • Consists of a low viscosity binder (e.g., cutback bitumen, MC 3000 or 150/200 penetration grade bitumen) followed by a layer of graded aggregate (crushed or screened) with a maximum size of up to 19 mm, (normally 16 mm). • Thickness about 16 mm for a single layer. • Due to the fines in the aggregate, requires extensive rolling to ensure that the binder is flushed to the surface. • May be constructed in a single layer or, for improved durability, with a sand seal over a single layer or in a double layer. • Fairly suitable for labour-based construction but requires relatively complex construction plant (bitumen distributor + binder heating facilities) and extended aftercare (replacement of aggregate and rolling).
Penetration Macadam	• Empirical design • Constructed by first applying a layer of rolled coarse aggregate (e.g., 40/60 mm) followed by the application of emulsion or penetration grade binder. Next, the surface voids in the coarse aggregate layer are filled with finer aggregate (e.g., 10/20 mm aggregate) to lock in the coarse aggregate followed by an additional application of emulsion binder, which is then covered with fine aggregate (e.g., 5/10 mm) and rolled. • Very suitable for labour-based construction as aggregate and emulsion can be laid by hand. • Produces a stable interlocking, robust layer after compaction, but the cost is relatively high for LVRs due to the very high rate of application of bitumen (7/9 kg/m²). Not considered appropriate for use on LVRs.
Single Surface Dressing + Sand Seal	• Partly rational (surface dressing) and partly empirical design. • Consists of a single 13 mm or 9.5 mm surface dressing followed by a single layer of Sand Seal (river sand or crusher dust). • The primary purpose of the sand seal is to fill the voids between the chips to produce a tightly bound, close-textured surfacing. • Fairly suitable for labour-based construction and, when emulsion is used, requires relatively simple construction plant. • More durable than a Single Surface Dressing.
Double surface Dressing	• Partly rational (surface dressing) and partly empirical design. • Usually consists of a single 19 mm or 14 mm surface dressing followed by a single layer of aggregate of 9 or 7mm. • The primary purpose of the second layer is to fill the voids between the chips to produce a tightly bound, close-textured surfacing. • Fairly suitable for labour-based construction and, when emulsion is used, requires relatively simple construction plant. • More durable than a Single Surface Dressing.

Table 9-3 Continued: General characteristics of bituminous surfacings

Surfacing	Characteristics
Cape Seal	• Partly rational (surface dressing) and partly empirical (slurry seal) design. • Consists of a single 19 mm or 13 mm surface dressing followed by two layers or one layer respectively of slurry. The primary purpose of the slurry is to fill the voids between the chips to produce a tightly bound, dense surfacing. • Fairly suitable for labour-based construction and, when emulsion is used with the surface dressing; can be constructed with relatively simple plant. • Produces a very durable surfacing, particularly with the 19 mm aggregate + two slurry applications (life of 12 – 15 years).
Slurry Bound Macadam	• Empirical design. • Consists of a layer (about 20 mm - 30 mm thick) of single size aggregate (typically 13 mm or 19 mm), static roller compacted and grouted with bitumen emulsion slurry before final compaction with light pedestrian roller (vibrating at low amplitude and high frequency). A fine slurry is normally applied after the curing of the penetration slurry. • Acts simultaneously as a base and surfacing layer. • Very suitable for labour-based construction as aggregate and emulsion can be laid by hand. • Produces a stable interlocking, robust layer after compaction, but the performance is sensitive to single-sized aggregate and all voids being filled with slurry. The cost is relatively high for LVRs due to the high rate of application of bitumen and may not be appropriate for use on LVRs.
Sand Asphalt	• Empirical design. • Consists of 30 mm – 50 mm thick admixture of sand and bitumen, mixed at high temperature (130°C – 140°C) which is spread and rolled when the temperature has reduced to 80 degrees Celsius. • Performance not yet proven, so not considered for use on LVRs.
Cold Mix Asphalt	• Empirical design. • Consists of an admixture of graded crushed aggregate (0-6/6-10 mm) and a stable, slow-breaking emulsion which is mixed by hand or in a concrete mixer. After mixing the material is spread on a primed road base and rolled. Thickness about 20 mm. • Very suitable for labour-based construction; requires very simple construction plant; reduces the potential hazard of working with hot bitumen; does not require the use of a relatively expensive bitumen distributor.
Thin Asphalt < 30 mm	• Rational design. • Consist normally of 4.74 mm crushed aggregate mixed in asphalt hot mix plant and placed by a paver.

9.2.6 Suitability of Surface Treatment on LVRs

The choice of the appropriate surfacing type in a given situation will depend on the relevance or otherwise of several factors, including the following:

- Traffic (volume and type).
- Pavement (type – strength and flexural properties).
- Materials (type, quality and availability).
- Environment (climate – temperature, rainfall, etc.).
- Operational characteristics (geometry – gradient, curvature, etc.).
- Safety (skid resistance - surface texture, etc.).
- Construction (techniques and contractor experience).
- Maintenance (capacity and reliability).
- Economic and financial factors (available funding, life cycle-costs, etc.).
- Other external factors.

The suitability of various types of surfacings for use on LVRs, in terms of their efficiency and effectiveness in relation to the operational factors outlined above, is summarized in Table 9-4.

Whilst not exhaustive, the factors listed in the table provide a basic format that can be adapted or developed to suit local conditions and subsequently used to assist in making a final choice of surfacing options. These options can then be subjected to a life cycle cost analysis and a final decision made with due regards to prevailing economic factors and be compatible with the overall financial situation.

Table 9-4: Suitability of various surfacings for use on LVRs

SSS-Single Sand Seal, **DSS**-Double Sand Seal, **SLS**-Slurry Seal, **SSD**-Single Surface Dressing, **SOS**-Single Otta Seal, **DSD**-Double Surface Dressing, **DOS**-Double Otta Seal, **CS**-Cape Seal 13/19mm+Single/Double SLS, **CMA**-Cold Mix Asphalt

Surfacing attributes	Thin seal/phased strategy				Double/Combination seal strategy							
	SSS	DSS	SLS	SSD	SSD+SS	DSD	SOS	SOS+SS	DOS	CS 13mm	CS 19mm	CMA
Ease of design	Very good	Very good	Very good	Reasonable	Reasonable	Reasonable	Poor/not suited	Poor/not suited	Poor/not suited	Reasonable	Reasonable	Good
Ease of construction	Very good	Very good	Very good	Reasonable	Reasonable	Reasonable	Poor/not suited	Poor/not suited	Poor/not suited	Reasonable	Reasonable	Very good
Service life	Poor/not suited	Poor/not suited	Poor/not suited	Poor/not suited	Reasonable	Good	Reasonable	Very good	Very good	Good	Very good	Good
Suitability for LBM	Very good	Very good	Very good	Good	Good	Good	Reasonable	Reasonable	Reasonable	Good	Good	Very good
Risk of poor mtce capability	Poor/not suited	Poor/not suited	Poor/not suited	Poor/not suited	Reasonable	Reasonable	Reasonable	Very good	Very good	Very good	Very good	Good
High skid resistance	Poor/not suited	Poor/not suited	Poor/not suited	Very good	Good	Very good	Reasonable	Reasonable	Reasonable	Good	Very good	Reasonable
Early road marking	Reasonable	Reasonable	Very good	Very good	Reasonable	Very good	Poor/not suited	Poor/not suited	Poor/not suited	Very good	Very good	Very good
Suitability for turning actions	Poor/not suited	Poor/not suited	Poor/not suited	Poor/not suited	Reasonable	Reasonable	Good	Good	Good	Good	Good	Good
Sensitivity to material quality	Reasonable	Reasonable	Reasonable	Poor/not suited	Poor/not suited	Poor/not suited	Very good	Very good	Very good	Poor/not suited	Poor/not suited	Good
Constr. sensitivity to gradient (>8%)*	Poor/not suited	Poor/not suited	Poor/not suited	Poor/not suited	Reasonable	Reasonable	Poor/not suited	Poor/not suited	Poor/not suited	Reasonable	Reasonable	Good

Very good | Good | Reasonable | Poor/not suited

9.3 Non-bituminous Surfacings

9.3.1 General

There are many situations in which bituminous surfacings are unsuitable for use on LVRs, for example, on very steep grades (>8%), very flexible subgrades, or in marshy areas. In such circumstances, some type of more rigid, structural/semi-structural, surfacing would be more appropriate. There are a number of such surfacings that are potentially suitable for use on LVRs as described below.

While these non-bituminous surfaces have the potential to provide all-season accessibility, some have safety concerns. Design engineers should use their professional judgement to weigh up the benefits of improving access with the drawbacks of increased safety risks. Examples of such safety risks, as well as potential mitigations, are described in Section 9.3.7.

9.3.2 Main Types

The main types of non-bituminous surfacings are summarized in Figure 9-4.

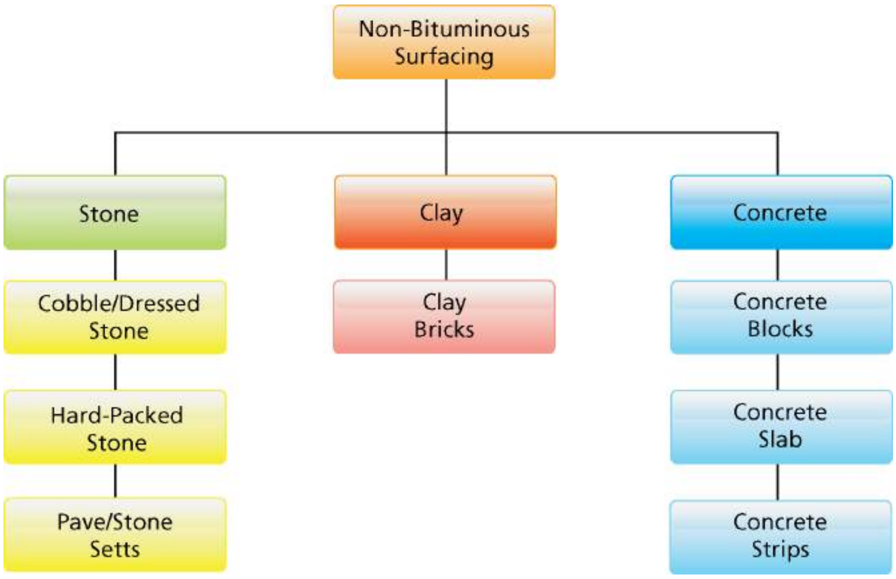


Figure 9-4: Terminology and categorisation of non-bituminous surfacing types

Some of the typical types of non-bituminous surfacings are shown in Figure 9-5.

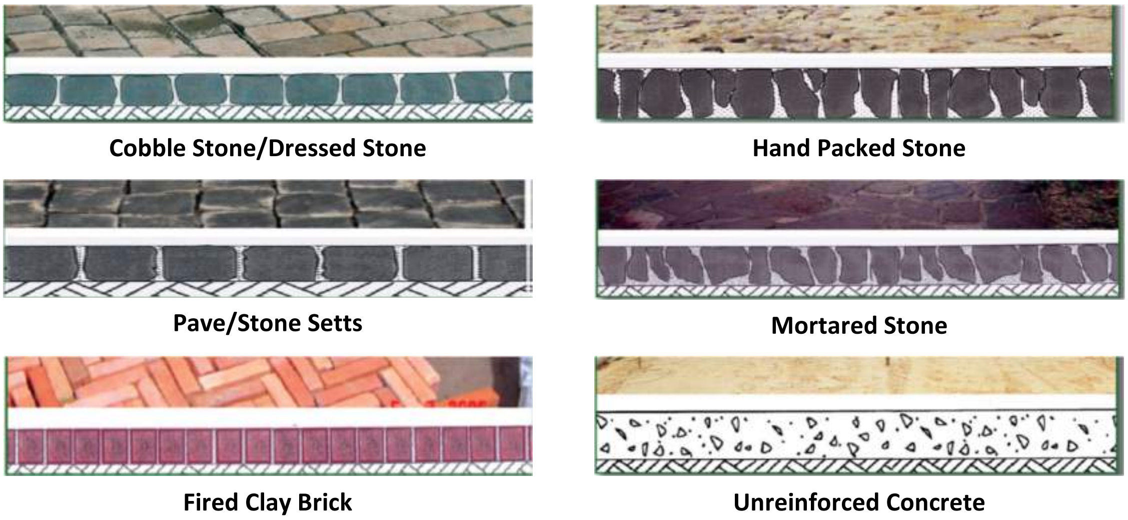


Figure 9-5: Common types of non-bituminous surfacings

9.3.3 Performance Characteristics

The non-bituminous surfacings described above all act simultaneously as a surfacing and base layer and provide a structural component to the pavement because of their thickness and stiffness. They all require the use of a sand bedding layer, which also acts as a load transfer layer for the overlying construction. In some cases, they act additionally as a drainage medium.

In some circumstances (e.g., on steep slopes in high rainfall areas and in areas with weak subgrades and/or expansive soils), it may be advantageous to use mortared options. This can be done with Hand-packed Stone, Stone Setts (or Pavé), Cobblestone (or Dressed Stone), and Fired Clay Brick pavements. The construction procedure is largely the same as for the un-mortared options except that cement mortar is used instead of sand for bedding and joint filling.

The behaviour of mortared pavements is different from that of sand-bedded pavements and is more analogous to a rigid pavement than a flexible one. There is, however, little formal guidance on mortared option, although empirical evidence indicates that inter-block cracking may occur.

All the non-bituminous surfacings are well suited for use on steep grades in situations where the more traditional types of bituminous surfacings would be ill-suited.

9.3.4 Typical Service Lives

The service life of a non-bituminous surfacing is relatively much longer than for a bituminous surfacing. This is due largely to the superior durability of the surfacing material, mostly natural stone, which is very resistant to the environment. Provided that the foundation support and road drainage are adequate, non-bituminous surfacings require relatively little maintenance and will last almost indefinitely on LVRs as long as they are well constructed and maintained. Thus, for life-cycle costing purposes, the service life of a non-bituminous surfacing can generally be assumed to be at least as long as the design life of a typical LVR pavement.

9.3.5 General Characteristics

The general characteristics of a range of non-bituminous surfacings that may be considered for use on LVRs are summarised in Table 9-5.

Table 9-5: General characteristics of non-bituminous surfacings

Surfacing	Characteristics
Cobble Stone/ Dressed Stone	• Consists of a layer of roughly rectangular dressed stone laid on a bed of sand or fine aggregate within mortared stone or concrete edge restraints. Individual stones should have at least one face that is fairly smooth, to be the upper or surface face when placed. • Each stone is adjusted with a small (mason's) hammer and then tapped into position to the level of the surrounding stones. Sand or fine aggregates is brushed into the spaces between the stones and the layer then compacted with a vibratory tamper or vibratory plate compactor. • Cobble Stones generally 150 mm thick, Dressed Stones generally 150 mm -200 mm thick. • Joints sometimes mortared.
Hand Packed Stone	• Consists of a layer of large broken stone pieces (typically 150 mm to 300mm thick) tightly packed together and wedged in place with smaller stone chips rammed by hand into the joints using hammers and steel rods. The remaining voids are filled with sand or gravel. • Hand-packing achieves a degree of interlock, which should be assumed in the design. • Requires a capping layer when the subgrade is weak and a conventional sub-base of G30 material or stronger. • Normally bedded on a thin layer of sand (SBL), which normally is compacted by a vibratory tamper or vibratory plate compactor. • An edge restraint or kerb constructed, for example, of large or mortared stones improves durability and lateral stability.
Pave/Stone Setts	• Consists of a layer of roughly cubic (100 mm) stone setts laid on a bed of sand or fine aggregate within mortared stone or concrete edge restraints. • Individual stones should have at least one face that is fairly smooth to be the upper or surface face when placed. • Each stone sett is adjusted with a small (mason's) hammer and then tapped into position to the level of the surrounding stones. • Sand or fine aggregate is brushed into the spaces between the stones and the layer is then compacted, normally by a vibratory tamper or vibratory plate compactor.
Fired Clay Brick	• Consists of a layer of high-quality bricks, typically each 10 cm x 20 cm and 7 cm -10 cm thick, laid by hand on a sand bed with joints also filled with a sand and lightly compacted or bedded and jointed with cement mortar. • Kerbs or edge restraints are necessary and can be provided by sand-cement bedded and mortared fired bricks. • Normally laid in herringbone or other approved pattern to enhance load spreading characteristics. • Un-mortared brick paving is compacted with a plate compactor and jointing sand is topped up if necessary. For mortar-bedded and joint-fired clay brick paving, no compaction is required.

Surfacing	Characteristics (Cont'd)
Concrete Blocks	• Consists of pre-cast concrete blocks in moulds typically 10 cm x 20 cm x 7 cm thick. • Laid by hand, side-by-side on a 3 cm to 5 cm sand bed with gaps between blocks filled with fine material and lightly compacted to form a strong, semi-pervious layer with a vibrating plate compactor. • Well suited to labour-based construction with a modest requirement for skilled workforce.
Non-reinforced Concrete (NRC)	• Involves casting slabs of 4.0 m to 5.0 m in length between formwork with load transfer dowels between them to accommodate thermal expansion. • Provides a strong, durable pavement with low maintenance requirements. • More suited to areas with good quality subgrade; in areas of weakness, reinforcement may have to be considered. • Suited to small contractors as concrete can be manufactured using small mixers.
Lightly Reinforced Concrete	• Similar to NRC but with light mesh reinforcement, which provides added strength to counteract the wheel loading as traffic moves onto the end slab from the adjacent surfacing. • Well suited in areas of relatively weak subgrade to improved strength, preventing excessive stress and cracking • Using mesh reinforcement 6 mm @ 200 mm is a good practice independent from the subgrade condition.
Concrete Strips	• Consists of parallel 0.9 m wide, 3.0 m (max) in length and 0.20 m in thickness unreinforced concrete strips spaced at a distance from centre-to-centre at 1.55 m so that both sets of vehicle wheels would run on the strips. The end of the strip on a downward slope should be thickened to act as a dowel. • Strips contain transverse concrete strips between the wheel tracks to help stop excessive erosion down the centre of the strips.

9.3.6 Suitability for Use on LVRs

Non-bituminous surfacings of one type or another are particularly suitable for use on LVRs in the following situations:

- On relatively steep gradients where high tyre traction is required.
- In high rainfall areas where slipperiness may be a problem on steep grades.
- On severely stressed sections, such as near marketplaces and at traffic check points.
- In locations where oil spillage is likely to occur.
- In junctions with heavy turning vehicles.
- In parking bays with prolonged static loading.
- When very low maintenance capability is likely.
- When very long service life is required.
- Where natural stone is in plentiful supply.

9.3.7 Safety Risks Associated with Non-Bituminous Surfacings

Of the non-bituminous surfaces, the concrete strips pose the greatest safety concerns, especially for motorbikes when they are forced to leave or re-join a strip, for example, when encountering a four-wheeled vehicle or when overtaking another motorcycle. In order to mitigate the potential road safety risks associated with the use of concrete strips, the following provisos should be applied:

- Low traffic situations with maximum 50 four-wheeled vpd.
- Relatively short, straight sections of road.
- The width of the road, including shoulders, is sufficient to allow a motorcycle to pass a four-wheeled vehicle safely.

- Ditch side slopes (not less than 1V:3H).
- The un-surfaced part of the road is adequately maintained to prevent edge-drops developing, and to keep them clear of vegetation and loose and oversize material.
- The gravel area between the two strips should be maintained to prevent edge-drops developing, and to prevent the transverse concrete strips or chevrons from becoming a hazard.

Other concrete surfaces also pose safety risks. Mitigations include ensuring that:

- Their width should be sufficient to allow a motorcycle to pass a four-wheeled vehicle safely.
- Shoulders should be maintained to prevent edge-drops developing.
- The surface should be scoured (roughened) to provide adequate texture, thereby increasing skid resistance, but the scouring should not leave the surface overly rough as this can impart vibrations through the hands of motorcyclists, creating the risk of loss of control.
- In a transition between a concrete surface and an earth or gravel surface, the end of the concrete surface should be bevelled downwards to reduce the risk of erosion creating a drop-down from the concrete to the earth.
- Where two different types of surfacing adjoin each other, there is a need to ensure that this point does not occur where it cannot be seen by a motorcyclist, such as at the brow of a hill or on a sharp curve.

Bibliography

Cook J, Petts R C and J Rolt (2013). ***Low Volume Rural Road Surfacing and Pavements: A Guide to Good Practice.*** AFCAP Report GEN/099, Crown Agents, UK.

Ethiopian Roads Authority (2013). ***Best Practice Manual for Thin Bituminous Surfacings.*** Addis Ababa, Federal Democratic Republic of Ethiopia.

Ministry of Transport, Vietnam: RRST, July 2007. ***Guidelines: Rural Road Pavement Construction – RRST Options.*** Intech Associates/TRL in association with ITST. Vietnam.

National Association of Australian State Road Authorities (1986). ***Principles and Practice of Bituminous Surfacing Vol. 1: Sprayed work.*** National Association of Australian State Road Authorities, Sydney, Australia.

Overby C (1999). ***Guide to the use of Otta Seals. Publication No.93.*** Norwegian Public Roads Administration, Oslo, Norway.

SABITA (2012). ***Bituminous Surfacings for Low Volume Roads and Temporary Deviations. Manual 28.*** Cape Town, South Africa.

Shell Bitumen (2015). ***The Shell Bitumen Handbook***. London, UK.

The South African Road Agency Ltd, (2007). ***Design and Construction of Surfacing Seals. Technical Recommendations for Highways, TRH 3***. Pretoria, South Africa.

Transport Research Laboratory (2000). ***A Guide to Surface Dressing in Tropical and Sub-tropical Countries. Overseas Road Note No 3, (2nd Edition).*** TRL, Crowthorne, Berkshire, UK.

TRL (2003). ***Manual for Labour Based Construction of Bituminous Surfacings on Low Volume Road.*** Report R7470. TRL, Crowthorne, Berkshire, UK.

10. Life-cycle Costing

10.1 Introduction

10.1.1 Background

There are always several potential alternatives available to the designer in the design of new roads or the rehabilitation of existing ones, each capable of providing the required performance. For example, as illustrated in Figure 10-1, for a given analysis period, one alternative might entail the use of a relatively thick, more expensive pavement with fewer interventions (Alternative A). In contrast, the other option may involve the use of a relatively thin, inexpensive pavement, which requires multiple strengthening interventions (Alternative B).

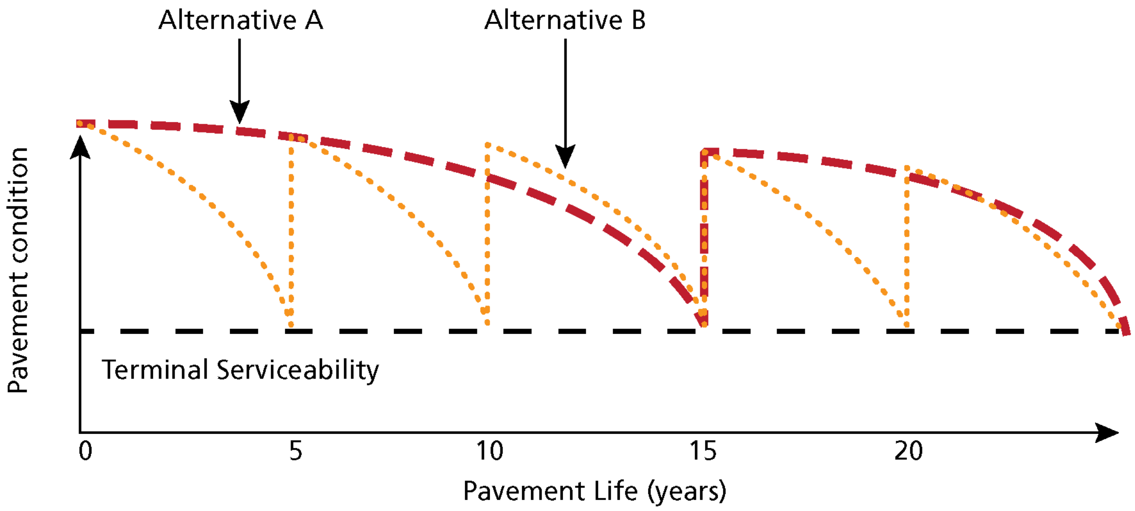


Figure 10-1: Alternative pavement options

10.1.2 Purpose and Scope

The primary purpose of this chapter is to outline the procedure to be followed in undertaking a life-cycle cost (LCC) analysis to compare alternative pavement/surfacing/upgrading options over their design lives to arrive at the most cost-effective solution. The chapter outlines the method of carrying out an LCC analysis, including the necessary inputs to the analysis.

The chapter focuses on the LCC analysis of alternative road surfacing options as well as the upgrading of unpaved roads to a paved standard. However, the principles of LCC analysis can also be applied to comparing road projects involving alternative alignments, or alternative maintenance strategies, etc., which are outside the scope of this Guideline.

10.2 Economic Analysis Methods

10.2.1 General

There are several methods for undertaking an economic comparison of alternative designs such as the Net Present Value (NPV), Internal Rate of Return (IRR), or Benefit-Cost Ratio (BCR). However, the NPV method is generally preferred over other methods of evaluating projects. One of its main advantages is that it can be used to evaluate both independent and mutually exclusive projects whilst the IRR method cannot be relied upon to analyse mutually exclusive projects, as this method can lead to conflicts in the ranking of projects. Thus, where there are no significant uncertainties or budget constraints, it is generally the preferred criterion for choosing between alternative projects

10.2.2 Use of Alternative Economic Tools

The choice of tool for undertaking an economic comparison of alternative designs will depend on the nature of the impact triggered by the investment intervention. Such interventions would typically fall into the following three categories:

- **Category 1**: Investments in this category would typically yield predominantly social rather than economic benefits, and traffic levels would be relatively low, up to about 75 vpd. Thus, a least-cost or cost-effectiveness approach is usually adopted, as investment models are generally not appropriate for such roads, which would typically be unpaved. This approach would entail a simple comparison of the initial construction costs of alternative projects.
- **Category 2**: Investments in this category would typically give rise to a mix of economic, social, and environmental impacts, depending on the function of the road and level of traffic carried, the latter being typically of the order of 75 – 300 vpd. The design alternatives could be between unpaved roads of different standards or between an unpaved road and a paved road, and the riding quality (roughness) of the alternatives could be significant. In this case, a life-cycle cost analysis could be undertaken using either a simple spreadsheet based on the NPV criterion or an economic model, such as the World Bank's Road Economic Decision Model (RED).
- **Category 3**: Investments in this category would give rise to predominantly economic impacts in the form of reduced transport costs, as well as environmental impacts. Traffic levels would typically be > 300 vpd. In such a case, a life-cycle cost analysis could be undertaken using the RED model.

10.2.3 Use of the NPV for Undertaking a LCC Analysis

In this approach, the NPV is simply the discounted monetary value of expected net benefits (i.e., benefits minus costs) and may be calculated as follows:

$$NPV = C + \sum M_i (1 + r)^{-X_i} - S(1 + r)^{-Z}$$

Where NPV = net present worth of costs
C = present cost of initial construction
M_i = cost of the i^{th} maintenance and/or rehabilitation measure
r = real discount rate
X_i = number of years from the present to the i^{th} maintenance and/or rehabilitation measure within the analysis period
Z = analysis period
S = salvage value of the pavement at the end of the analysis period expressed in terms of present values

The NPV is computed by assigning monetary values to benefits and costs, discounting future benefits (b_i) and costs (c_i) using an appropriate discount rate, and subtracting the sum of total discounted costs from the sum of total discounted benefits. A positive NPV indicates that the project is economically justified at the given discount rate and, the higher the NPV, the greater will be the benefits of the project.

10.3 Life-Cycle Cost Analysis

10.3.1 Principal Components of a LCC Analysis

The principal components of an LCC analysis are illustrated in Figure 10-2. They include the initial investment or construction cost and the future costs of maintaining or rehabilitating the road, as well as the benefits due to savings in user costs over the analysis period selected. An assessment of the residual value of the road is also included to incorporate the possible different consequences of construction and maintenance strategies for the pavement/surface options being investigated.

When all cost items escalate at the same rate, inflation is taken into account by calculating all future expenditures at current unit costs. The real discount rate is then used to determine the present value of these future costs by multiplying the discount rate by the general inflation rate.

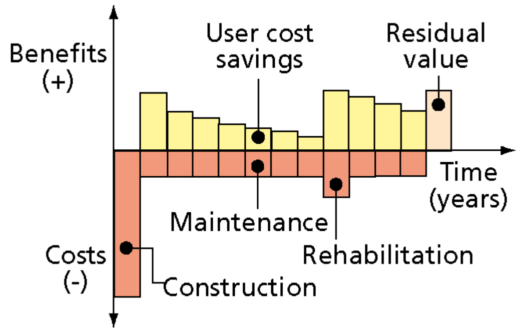


Figure 10-2: Distribution of costs and benefits during the life-cycle of a road

Where a new pavement is to be constructed in which the difference in vehicle operating costs (VOC) benefits is considered negligible, then only the initial construction costs and the future maintenance costs of each type need to be considered.

The components of an LCC analysis associated with a particular design alternative are listed below and illustrated in Figure 10-3.

- Analysis period
- Structural design period
- Construction/rehabilitation costs
- Maintenance costs
- Road user costs
- Salvage value
- Discount rate

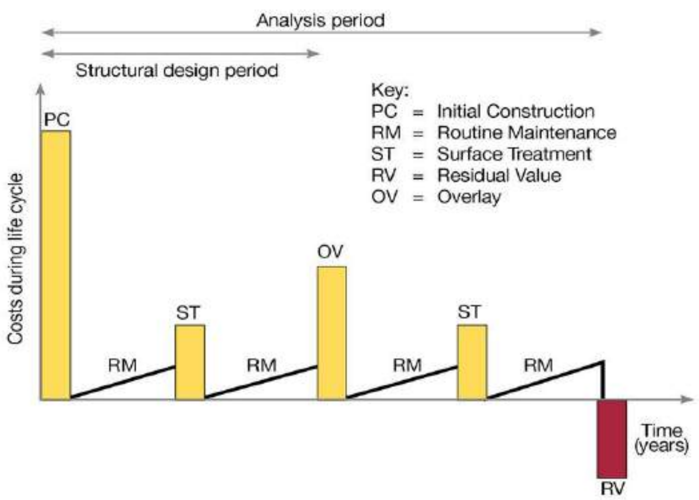


Figure 10-3: Components of a typical life cycle cost analysis

10.3.2 LCC Procedure

The procedure that is followed typically in undertaking an LCC analysis of mutually exclusive projects, i.e., the selection of one project precludes the selection of the other project, is:

1) Establish alternative project options
2) Determine analysis period
3) Estimate agency (construction and maintenance) costs
4) Estimate road user costs
5) Develop expenditure stream diagrams (similar to Figure 10.3)
6) Compute NPV of both options
7) Analyse results, including sensitivity analysis, if warranted
8) Decide on the preferred option, i.e., the option with the highest NPV.

In view of the uncertainty of future costs, e.g., hauling distances for gravel, bitumen prices, etc., there would be merit in undertaking a sensitivity analysis of the main parameters in the LCC analysis.

Analysis period
This period is the length of time for which comparisons of total costs are to be made. It should be the same for all alternative strategies and should not be less than the longest design period of the alternative strategies.

Structural design period
This is the design life of the road, at which time it would be expected to have reached its terminal serviceability level and to require an appropriate intervention such as an overlay.

Construction costs
Unit costs for alternative pavement designs will vary widely depending on such factors as locality, availability of suitable materials, the scale of project, and road standard. Other factors that would typically warrant consideration include:

- Land acquisition costs.
- Supervision and overhead cost.
- Establishment costs.
- Accommodation of traffic.
- Relocation of services.

Maintenance costs
The nature and extent of future maintenance will be dependent on pavement composition, traffic loading and environmental influences. An assessment needs to be made of future routine and periodic maintenance as well as rehabilitation requirements.

Road User Costs
These are the costs that each driver will incur in using the road system. They typically comprise vehicle operating costs (fixed costs, fuel, tires, repair and maintenance, and depreciation costs), the costs of accidents and congestion, and travel time costs. VOCs are related to the roughness of the road in terms of its International Roughness Index (IRI) and will change over the life of the road due to changes in surface conditions and traffic. Relationships can be developed for main vehicle types that relate VOCs to variations in road surface conditions (IRI) under local conditions.

Road user costs are normally excluded from the LCC analysis that is confined to comparing alternative pavement/surfacing options, as the pavement options are considered to provide "equivalent service" during the analysis period. However, when evaluating the viability of upgrading a gravel road to a paved standard, the savings for the road user (primarily vehicle operating costs) on the latter versus the former option can be significant and are treated as benefits which should be incorporated as one of the components in the LCC analysis (ref. Figure 10-7). Such user costs would also include accident and time costs, which should be considered, where available.

Salvage value
The value of the pavement at the end of the analysis period depends on the extent to which it can be utilized in any future upgrading. For example, where the predicted condition of the pavement at the end of the analysis period is such that the base layer could serve as the subbase layer for the subsequent project, then the salvage value would be equal to the cost in current value terms for construction in future to subbase level discounted to the evaluation year.

Discount rate
This rate is used as the means for comparing future expenditure in terms of present values when evaluating alternative options. The rate is dependent on various factors, including the effective rate of borrowing money and the rate of return that money can earn if invested. The discount rate in a country is usually recommended by the Ministry of Finance and is typically of the order of 6 to 8 percent.

10.4 Selection of Design Standard

10.4.1 General

The selection of an appropriate pavement design standard requires an optimum balance to be struck between construction/rehabilitation, maintenance and road user costs, such as to minimize total life cycle costs, as illustrated in Figure 10-4. The optimum standard varies in relation to traffic level and the associated relative mix of construction, maintenance and user costs. Such an analysis can be undertaken using an appropriate techno-economic model, such as the World Bank's HDM4.

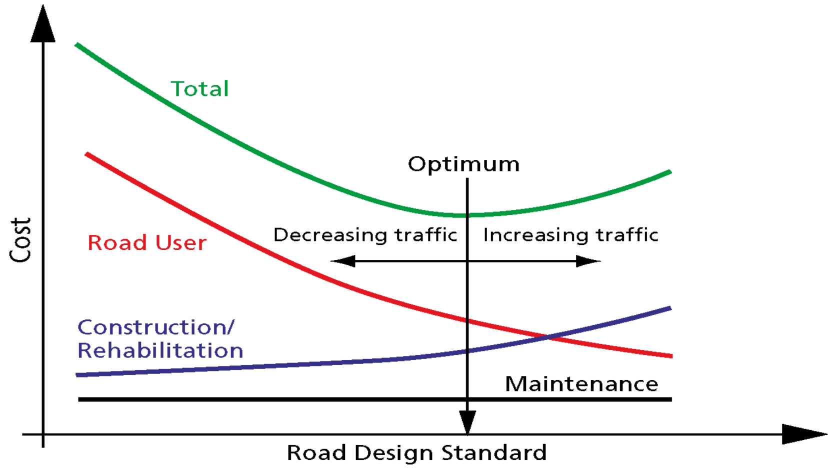


Figure 10-4: Economic analysis of optimum road design standard

The optimum road design standard varies in relation to traffic level and the associated relative mix of construction, maintenance and user costs. Thus, the optimum road design standard, in terms of the pavement structural capacity, for a relatively low-traffic pavement would incur lower initial construction costs but, within its life cycle, this would be balanced by higher maintenance and VOC. Conversely, a high-traffic pavement would incur higher initial construction costs but lower maintenance and VOC.

10.4.2 Gravel versus paved road option

A typical situation faced by a road agency is – when is it economically justified to upgrade a gravel road to a paved standard. As illustrated in Figure 10-7, the gravel and paved road options would have a different relative mix of construction, maintenance and road user costs. In such a situation, the LCC analysis should be undertaken to determine the viability of upgrading a gravel road to a paved standard.

Figure 10-5: Gravel road option

Lower construction costs, higher maintenance and road user costs.

Figure 10-6: Paved road option

Higher construction costs, lower maintenance and road user costs.

The typical components of the LCC analysis are illustrated in Figure 10-7 and could be undertaken using an appropriate appraisal model such as the World Bank's Roads Economic Decision (RED) model, which performs an economic evaluation of road investment options using the consumer surplus approach and which is customized to the characteristics and needs of low-volume roads. Using RED, the VOC relationships may need to be calibrated for local conditions. The option with the higher NPV would be the preferred one.

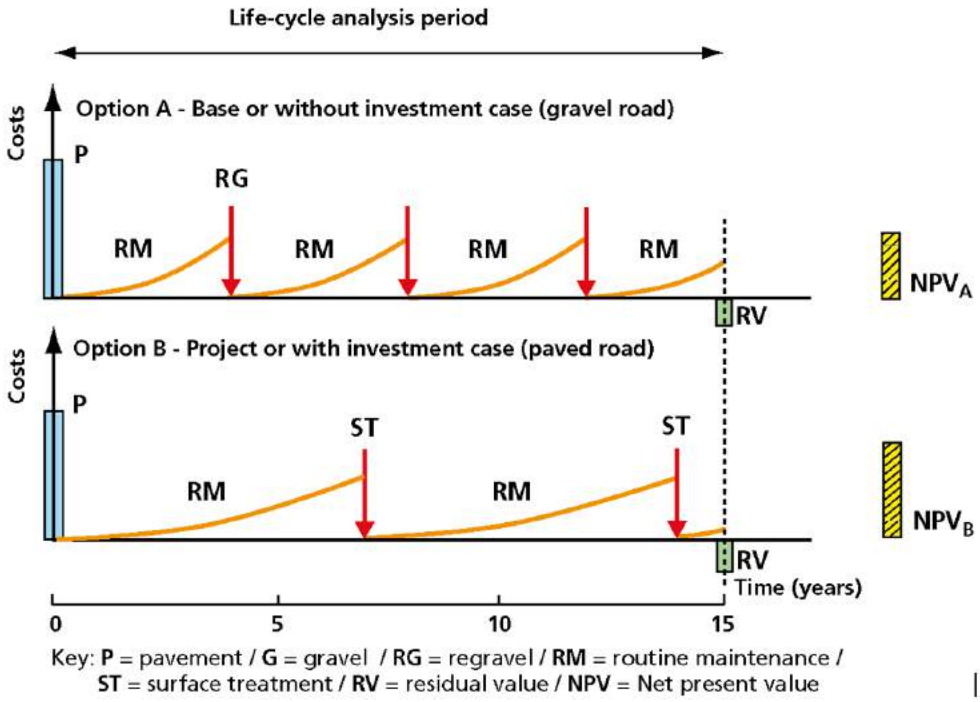


Figure 10-7: Typical components of a LCC: Gravel versus Paved road

In very general terms, the upgrading of a gravel road to a paved standard would be economically justified when the net present value (NPV) of the sum of savings in VOCs and maintenance costs, relative to the well-maintained gravel road, is at least as great as the NPV of upgrading costs to paved standard. Where not captured in the investment appraisal model, the inclusion of socio-economic benefits would need to be evaluated separately after the economic appraisal has been carried out.

10.4.3 Factors Affecting Traffic Threshold for Upgrading

The new approaches to the design of LVRs have resulted in a reduced threshold for upgrading gravel roads to a paved standard from a traditional, rule-of-thumb figure of > 300 vpd to typically in the range 100 – 200 vpd depending on road environmental conditions. Some of the factors influencing this reduced traffic threshold for upgrading are given in Table 10-1.

Table 10-1: Factors influencing the traffic threshold for upgrading

Parameter	Impact
Use of more appropriate pavement designs	Reduced costs
Use of more appropriate geometric design	Reduced costs
Increased use of natural/unprocessed gravels	Reduced costs
Quantified impacts of depleted gravel resources	Reduced costs
Benefits of non-motorized transport	Increased benefits
Quantified adverse impacts of traffic on gravel roads	Increased benefits
Reduced environmental damage	Increased benefits
Quantified assessments of social benefits	Increased benefits

The impact of these factors is illustrated conceptually in Figure 10-8, which reflects the outcome of recent research and which indicates that, in principle, in some circumstances, bitumen sealing of gravel roads may be economically justified at traffic levels of less than 100 vpd. This is in contrast to the previously accepted figures, which indicated a first-generation bitumen surface at a traffic level of over 200 vpd.

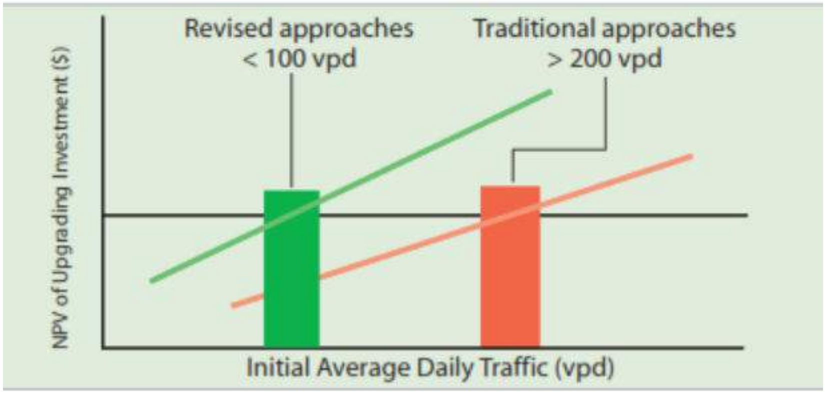


Figure 10-8: Break-even traffic levels for paving a gravel road: Traditional versus revised approaches.

10.4.4 Selection of Surfacing Option

An LCC analysis entails comparing the construction and maintenance costs of the alternative surfacing options over the life of the road for which the main inputs would typically include:

- Assumed service life of surfacing.
- Construction cost for surfacing options.
- Maintenance cost for surfacing options.
- Discount rate.

The analysis assumes that the vehicle operating costs imposed by the various options are similar due to very small differences in their roughness levels.

Typical example

Figure 10-9, Table 10-2 and Table 10-3 illustrate the manner of undertaking an LCC analysis for two typical types of bituminous surfacings by comparing the PV of all costs and maintenance interventions that occur during a given analysis period using a 12% real discount rate.

As indicated in Table 10-2 and Table 10-3, the Single Otta Seal + Sand Seal Option has the lower PV of costs and is the preferable option on economic grounds. This example is a hypothetical one used for illustrative purposes only and does not necessarily reflect a real-life situation.

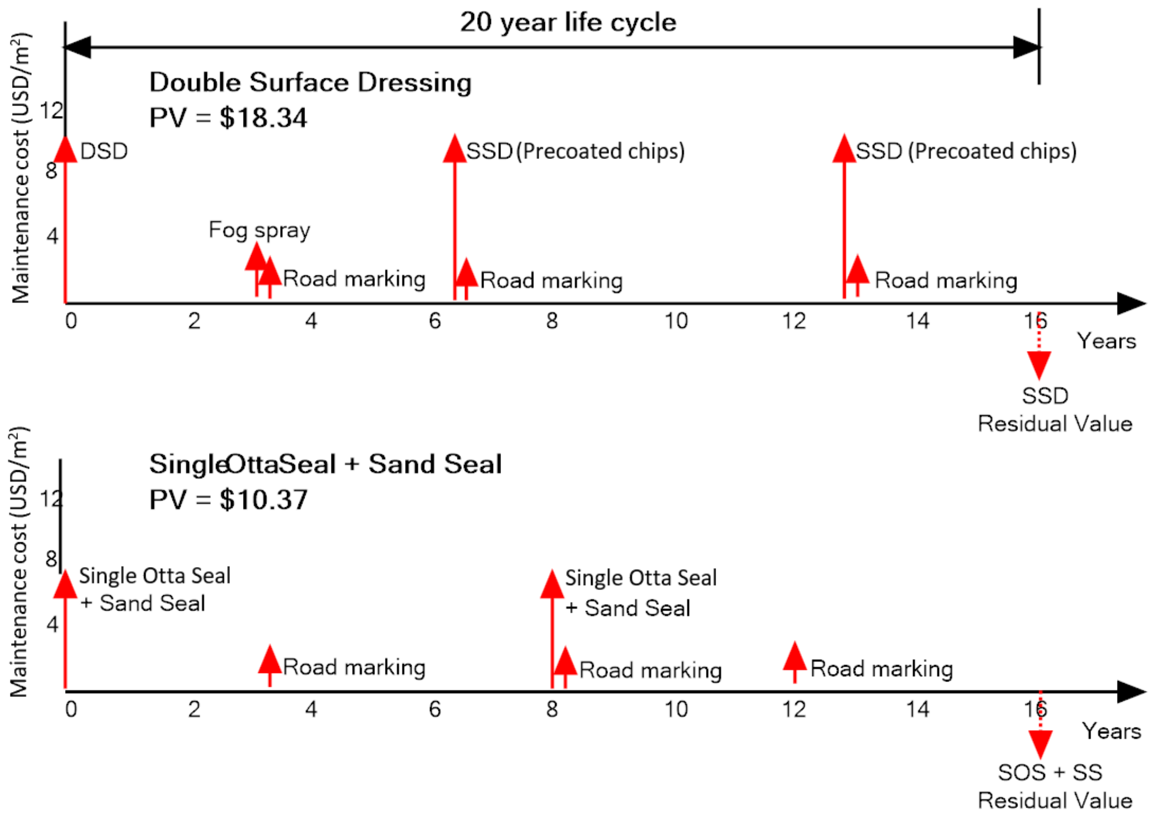


Figure 10-9: LCC comparison between a single Otta Seal + Sand Seal and a DSD

Table 10-2: Life-cycle cost analysis for Double Surfacing Dressing

Activity	Years after construction	Base Cost/m² ($)	12% Discount Factor	PV of Costs/m² ($)
1. Construct Double Chip Seal	-	10.00	1.000	10.00
2. Fog spray	4	02.00	0.636	1.27
3. Road marking	4	0.96	0.636	0.61
4. Single Chip Seal (pre-coated)	8	10.00	0.404	4.04
5. Road marking	8	0.96	0.404	0.39
6. Fog spray	12	2.00	0.257	0.51
7. Road marking	12	0.96	0.257	0.25
8. Single Chip Seal (pre-coated)	16	10.00	0.163	1.63
9. Road marking	16	0.96	0.163	0.16
10. Residual value of surfacing	20	(5.00)	0.104	(0.52)
				Total 18.34/m²

Table 10-3: Life-cycle cost analysis for Single Otta Seal + Sand Seal

Activity	Years after construction	Base Cost ($)	12% Discount Factor	PV of Costs ($)
1. Construct single Otta Seal + sand Seal	-	7.25	1.00	7.25
2. Road marking	5	0.96	0.567	0.54
3. Single Otta reseal	10	7.25	0.332	2.41
4. Road marking	10	0.96	0.332	0.32
5. Road marking	15	0.96	0.183	0.16
6. Residual value of surfacing	20	(3.00)	0.104	(0.31)
				Total 10.37/m²

Bibliography

Archondo-Callao, R S (1999). ***Roads Economic decision Model (RED) for economic evaluation of low volume roads.*** Sub-Saharan Africa Transport Policy Program (SSATP) Technical Note No. 18. World bank, Washington, D.C., USA.

ACPA (2002). ***Life Cycle Cost Analysis. A Guide for Alternative Pavement Designs***. American Concrete Pavement Association, Stokie, Illinois, USA.

Baum W C (1982). ***The Project Cycle***. International Bank for Reconstruction and Development, The World Bank, 1818 H Street, N.W. Washington, D.C. 20433, USA.

Goldbaum J (2000): ***Life Cycle Cost Analysis State-of-the-Practice***. Report No. CDOT-R1-00-3. Aurora, CO, USA.

Ozbay K et al (2003). ***Guidelines for Life Cycle Analysis. Final Report***. Report No. FHWA-NJ-2003-012. Trenton, NJ, USA.

Peterson D (1985). ***Life Cycle Cost Analysis of Pavements. Synthesis of Highway Practice***. NCHRP Report 122, Washington, D.C., USA.

Transportation Research Board (1985). ***Life-cycle Cost Analysis of Pavements***. NCHRP Synthesis of Highway Practice 122, Washington, D.C., USA.

Transport Research Laboratory (1988). ***Overseas Road Note 5: A guide to road project appraisal***. Crowthorne, Berkshire, UK.

Walls J and M R Smith (1998). ***Life Cycle Cost Analysis in Pavement Design – Interim Technical Bulletin***. Federal Highway Administration, Washington, D.C., USA

Glossary of Terms

Aggregate (for construction)
A broad category of coarse particulate material including sand, gravel, crushed stone, slag and recycled material that forms a component of composite materials such as concrete and asphalt.

Asphalt
A mixture of inert mineral matter, such as aggregate, mineral filler (if required) and bituminous binder in predetermined proportions.

Atterberg limits
Basic measures of the nature of fine-grained soils which identify the boundaries between the solid, semi- solid, plastic and liquid states.

Base Course
The upper layer of the road pavement.

Binder, Bituminous
Any bitumen-based material used in road construction to bind together or seal aggregate or soil particles.

Binder, Modified
Bitumen based material modified by the addition of compounds to enhance performance. Examples of modifiers are polymers, such as PVC, and natural or synthetic rubbers.

Bitumen
A non-crystalline solid or viscous mixture of complex hydrocarbons that possesses characteristic agglomerating properties, softens gradually when heated, is substantially soluble in trichlorethylene and is obtained from crude petroleum by refining processes.

Bitumen, Cutback
A liquid bitumen product obtained by blending penetration grade bitumen with a volatile solvent to produce rapid curing (RC) or medium curing (MC) cutbacks, depending on the volatility of the solvent used. After evaporation of the solvent, the properties of the original penetration grade bitumen become operative.

Bitumen, Penetration Grade
That fraction of the crude petroleum remaining after the refining processes which is solid or near solid at normal air temperature and which has been blended or further processed to products of varying hardness or viscosity.

Bitumen Emulsion
An emulsion of bitumen and water with the addition of an emulsifier or emulsifying agent to ensure stability. For conventional bitumen emulsion used in road works the bitumen is dispersed in the water. An invert bitumen emulsion has the water dispersed in the bitumen. In the former, the bitumen is the dispersed phase and the water is the continuous phase. In the latter, the water is the dispersed phase and the bitumen is the continuous phase. The bitumen is sometimes fluxed to lower its viscosity by the addition of a suitable solvent.

Bitumen Emulsion, Anionic
An emulsion where the emulsifier is an alkaline organic salt. The bitumen globules carry a negative electrostatic charge.

Bitumen Emulsion, Cationic
An emulsion where the emulsifier is an acidic organic salt. The bitumen globules carry a positive electrostatic charge.

Bitumen Emulsion Grades
Premix grade: An emulsion formulated to be more stable than spray grade emulsion and suitable for mixing with medium or coarse graded aggregate with the amount smaller than 0.075 mm not exceeding 2%.

- Quick setting grade: An emulsion specially formulated for use with fine slurry seal type aggregates, where quick setting of the mixture is desired.
- Spray grade: An emulsion formulated for application by mechanical spray equipment in chip seal construction where no mixing with aggregate is required.
- Stable mix grade: An emulsion formulated for mixing with very fine aggregates, sand and crusher dust. Mainly used for slow-setting slurry seals and tack coats.

Blinding

a) A layer of lean concrete, usually 5 to 10 cm thick, placed on soil to seal it and provide a clean and level working surface to build the foundations of a wall, or any other structure.

b) An application of fine material e.g. sand, to fill voids in the surface of a pavement or earthworks layer.

Borrow Pit
An area where material is excavated for use within another location

Brick (fired clay)
A hard, durable block of material formed from burning (firing) clay at high temperature.

California Bearing Ratio (CBR)
The value given to an ad-hoc penetration test where the value 100% applies to a standard sample of good quality crushed material

Camber
The road surface is normally shaped to fall away from the centre line to either side. The camber is necessary to shed rainwater and reduce the risk of passing vehicles colliding. The slope of the camber is called the Crossfall. On sharp bends the road surface should fall directly from the outside of the bend to the inside (superelevation).

Carriageway
The road pavement including the traffic lanes and the road shoulders.

Cape Seal
A single application of binder and stone followed by one or two applications of slurry.

Cement (for construction)
A dry powder which on the addition of water and other additives, hardens and sets independently to bind aggregates together to produce concrete.

Chippings
Clean, strong, durable pieces of stone made by crushing or napping rock. The chippings are usually screened to obtain material in a small size range.

Chip Seal, Single
An application of bituminous binder followed by a layer of stone or clean sand. The stone is sometimes covered with a fog spray.

Chip Seal, Double
An application of bituminous binder and stone followed by a second application of binder and stone or sand. A fog spray is sometimes applied on the second layer of aggregate.

Cobble Stone (Dressed stone)
Cubic pieces of stone larger than setts, usually shaped by hand and built into a road surface layer or surface protection.

Collapsible soil
Soil that undergoes a significant, sudden and irreversible decrease in volume upon wetting.

Compaction
The process whereby soil particles are densified, by rolling or other means, to pack more closely together, thus increasing the dry density of the soil.

Complementary Interventions
Actions that are implemented through a roads project which are targeted toward the communities that lie within the influence corridor of the road and are intended to optimise the benefits brought by the road and to extend the positive, and mitigate the negative, impacts of the project.

Concrete
A construction material composed of cement (commonly Portland cement) as well as other cementitious materials such as fly ash and slag cement, aggregate (generally a coarse aggregate such as gravel or crushed stone plus a fine aggregate such as sand), water, and chemical admixtures.

Concrete Block Paving
A course of interlocking or rectangular concrete blocks placed on a suitable base course and bedded and jointed with sand.

Crossfall
See Camber

Crushed Stone
A form of construction aggregate typically produced by mining a suitable rock deposit and breaking the rock down to the desired size using crushers.

Curing
The process of keeping freshly laid/placed concrete or stabilised soil moist to prevent excessive evaporation with attendant risk of loss of strength or cracking. Similarly, with cement or lime stabilised layers.

Cut-off/Catchwater Drain
A drain constructed uphill from a cutting face to intercept surface water flowing towards the road.

Layer-Strength Diagram (LSD)
Is a visual representation of the DN in depth through the pavement structure.

DCP Structure Number (DSN)
Is the number of DCP blows required to penetrate through a pavement structure or layer. For example, the DSN_{800} of the pavement is the number of blows required to penetrate the pavement to a depth of 800 mm.

DCP Number (DN)
Defined as the penetration of the DCP cone through a specific pavement layer as measured in mm per blow.

Dispersive Soil
Soil in which the clay particles detach from each other and from the soil structure in the presence of water and go into suspension.

Drainage
Interception and removal of ground water and surface water by artificial or natural means.

Dressed Stone
See Cobble Stone

Embankment
Constructed earthworks below the pavement raising the road above the surrounding natural ground level.

Expansive Soil
Typically, a clayey soil that undergoes large volume changes in direct response to moisture changes.

Filler
Mineral matter composed of particles smaller than 0.075 mm.

Formation
The shaped surface of the earthworks, or subgrade, before constructing the pavement layers.

Grading Modulus (GM)
Is obtained from the equation GM = 300 – [(P2,00 mm + P0,425 mm + P0,075 mm)] / 100 where P2,00 mm, etc., denote the percentage. retained on the indicated sieve size.

Gravel
A naturally-occurring, weathered rock within a specific particle size range. In geology, gravel is any loose rock that is larger than 2 mm in its largest dimension and not more than 63 mm.

Hand Packed Stone
A layer of large, angular broken stones laid by hand with smaller stones or gravel rammed into the spaces between stones to form a road surface layer.

In-situ
Taken in position (i.e. test undertaken on the material within its natural state, rather than a sample taken for a lab test).

Labour Based Construction
Economically efficient employment of as great a proportion of labour as is technically feasible throughout the construction process to produce the standard of construction as demanded by the specification and allowed by the available funding. Substitution of equipment with labour as the principal means of production.

Laterite
Residual deposits formed under tropical climatic conditions. Laterite consist of iron aluminium oxides.

Lime
Lime in a material derived from the burning of limestone or chalk. It is normally obtainable in its 'hydrated' form (slaked) as Calcium Hydroxide. It can be used for the drying, improvement and stabilisation of suitable soils, as an anti-stripping agent in the production of bituminous mixes and as a binder in masonry or brick work mortars.

Low Volume Road
Roads carrying up to about 300 vehicles per day and less than about 1 million equivalent standard axles over their design life.

Macadam
A mixture of broken or crushed stone of various sizes (usually less than 6cm) laid to form a road surface layer. Bitumen macadam uses a bituminous binder to hold the material together. Tarmacadam uses tar for the same purposes. Bound macadams are usually expensive for use on LVR.

Otta Seal
Sprayed bituminous surfacing using graded natural gravel rather than single-sized crushed rock.

Pavé
See Sett

Paved Road
A road that has a bitumen seal or a concrete riding surface

Pavement
The constructed layers of the road on which the vehicles travel.

Penetration Macadam
A pavement layer made from one or more applications of coarse, open-graded aggregate (crushed stone, slag, or gravel) followed by the spray application of bituminous binder. Usually comprising two or three applications of stone each of decreasing particle size, each grouted into the previous application before compaction of the completed layer.

Plasticity Index (PI):
LL – PL, an indication of the clay content of soils; the larger the PI, the larger the clay content.

Plasticity Modulus
The product of Plasticity Index (PI) and percentage fraction passing 425-micron sieve.

Prime Coat
A coat of bituminous binder applied to a non-bituminous granular pavement layer as a preliminary treatment before the application of a bituminous base or surfacing. While adhesion between this layer and the bituminous base or surfacing may be promoted, the primary function of the prime coat is to assist in sealing the surface voids and bind the aggregate near the surface of the layer.

Reinforced Concrete
A mixture of coarse and fine stone aggregate bound with cement and water and reinforced with steel rods or mesh for added strength.

Rejuvenator
A material (which may range from a soft bitumen to petroleum) which, when applied to reclaimed asphalt or to existing bituminous surfacing, has the ability to soften aged, hard, brittle binders.

Reseal
A surface treatment applied to an existing bituminous surface.

Resilient Modulus
A measure of the stiffness of a material. Is an estimate of the Modulus of Elasticity of a soil which is the stress divided by strain for rapidly applied loads experienced by pavements.

Road Maintenance
Suitable regular and occasional activities to keep pavement, shoulders, slopes, drainage facilities and all other structures and property within the road margins as near as possible to their as constructed or renewed condition. Maintenance includes minor repairs and improvements to eliminate the cause of defects and avoid excessive repetition of maintenance efforts.

Seal
A term frequently used instead of “reseal” or “surface treatment”. Also used in the context of “double seal” and “sand seal” where sand is used instead of stone.

Selected layers
Pavement layers of selected gravel materials used to bring the subgrade support up to the required structural standard for placing the sub-base or base course.

Sett (Pavé)
A small piece of hard stone trimmed by hand to a size of about 10cm cube used as a paving unit.

Shear Strength
Defined as the resistance to shear stress, at failure, on a surface within a soil mass

Shoulder
Paved or unpaved part of the roadway next to the outer edge of the pavement. The shoulder provides side support for the pavement and allows vehicles to stop or pass in an emergency.

Site Investigation
Collection of essential information on the soil and rock characteristics, topography, land use, natural environment, and socio-political environment necessary for the location, design and construction of a road.

Slope
A natural or artificially constructed surface at an angle to the horizontal.

Slurry (Slurry Seal)
A mix of suitably graded fine aggregate, cement or hydrated lime, bitumen emulsion and water, used for filling the voids in the final layer of stone of a new surface treatment or as a maintenance treatment.

Slurry-Bound Macadam
A surfacing layer constructed where the voids in single-sized stone skeleton are filled using bituminous slurry.

Stiffness
The relationship between stress and strain in the elastic range of a stressed material. Is a measure of how well a material is able to return to its original shape and size after being stressed.

Sub-base
The layer in the road pavement below the base course.

Subgrade
The native material underneath a constructed road pavement.

Surface Dressing
A sprayed or hand applied film of bitumen followed by the application of a layer of stone chippings, which is then lightly rolled.

Surface Treatment
A general term incorporating chip seals, micro surfacing, fog sprays or tack coats.

Surfacing
The layer with which traffic makes direct contact.

Travelled Way
Comprising the traffic lanes on which vehicles are travelling.

Unpaved road
Earth or gravel road

Waterbound Macadam
A pavement layer constructed where the voids in a large single-sized stone skeleton are filled with a fine sand.

Wearing Course
The upper layer of a road pavement on which the traffic runs and is expected to wear under the action of traffic.