Republic of Malawi

Ministry of Transport and Public Works

Low Volume Roads Manual

Volume 1
Pavement Design

July 2020

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Foreword

This Design Manual for Low Volume Roads applies to the pavement design of District, Community and Private roads in rural areas and lower-order road and street networks in urban environments in Malawi. The effective management of these essential components of the classified road network depends to a great extent on the adoption of appropriate and cost-effective standards that meet the needs of road users at minimum life-cycle costs.

The main purpose of the Manual is to provide all practitioners with comprehensive guidance on the wide range of factors that need to be addressed in a holistic and environmentally sustainable manner when designing unpaved roads or undertaking their upgrading to a paved standard. The Manual takes account of best practice developments in low volume roads technology that have evolved both regionally and internationally in the past few decades. The Ministry of Transport and Public Works therefore expects all practitioners in the roads sector to adhere to the standards set out in the Manual. This will ensure that a consistent, harmonised approach is followed in the design of low volume roads in the country.

The development of the Manual was overseen by a Technical Steering Committee comprising representatives from a wide range of stakeholder organisations from Government, the private sector and academia. By its very nature, the manual will require periodic updating to take account of the dynamic developments in low volume roads technology. The Ministry would, therefore, welcome comments and suggestions from any stakeholder as feedback on all aspects of the Manual during its implementation. All feedback will be carefully considered by professional experts in future updates of the Manual.

On behalf of the Ministry of Transport and Public Works, I would like to thank UK Aid through the Department for International Development (DFID) for its support towards the development of the Manual. I would also like to thank the Project Management Unit (PMU) of the Research for Community Access Partnership (ReCAP) and Infra Africa Development Consultants for their role in managing the project. In addition, I would commend all the roads sector stakeholders who contributed their time, knowledge and effort during the development of the Manual.

It is my sincere hope that this Manual will herald a new era in the more efficient and cost-effective provision of low volume roads in Malawi. In so doing, it will make a substantial contribution to the improvement of road infrastructure in our country and, in the process, enhance socio-economic growth and development, particularly in the rural areas of the country.

Mr Francis Chinsinga
Secretary for Transport and Public Works
Ministry of Transport and Public Works

Acknowledgements

The Ministry of Transport and Public Works acknowledges the support that was provided by the United Kingdom Department for International Development (DFID) for the preparation of this Pavement Design Manual for Low Volume Roads. The project was carried out under the aegis of the Research for Community Access Partnership (ReCAP) – a DFID-funded research programme that promotes safe and sustainable access for rural communities in Africa.

The development of the Manual was guided by a Technical Steering Committee comprising professionals from both public and private sector organizations as detailed below.

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The project was managed by Cardno Emerging Markets, UK, and was carried out under the general guidance of the AFCAP Technical Services Manager, East and Southern Africa, Eng. Henry Nkwanga.

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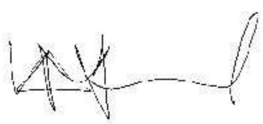
Peer Review

The manual was reviewed by Andrew Otto, UK Transport Research Laboratory.



Mr Kelvin N. Mphonda
Acting Director of Roads
Ministry of Transport and Public Works

Terminology

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

Pavement[1]

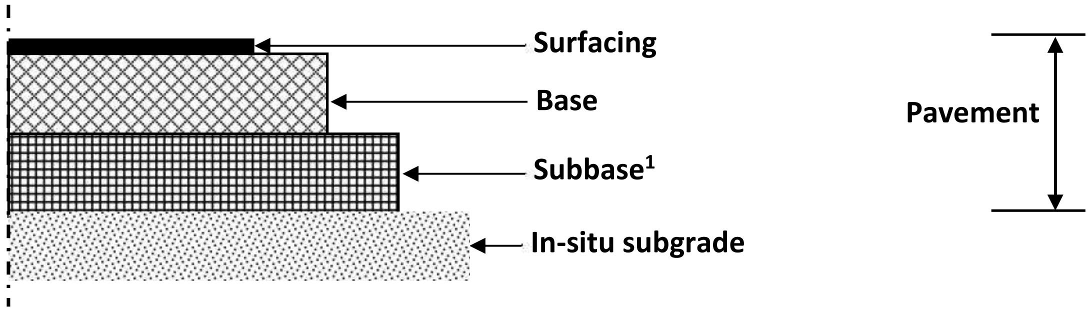


Figure 1: Main components of a LVSR pavement

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

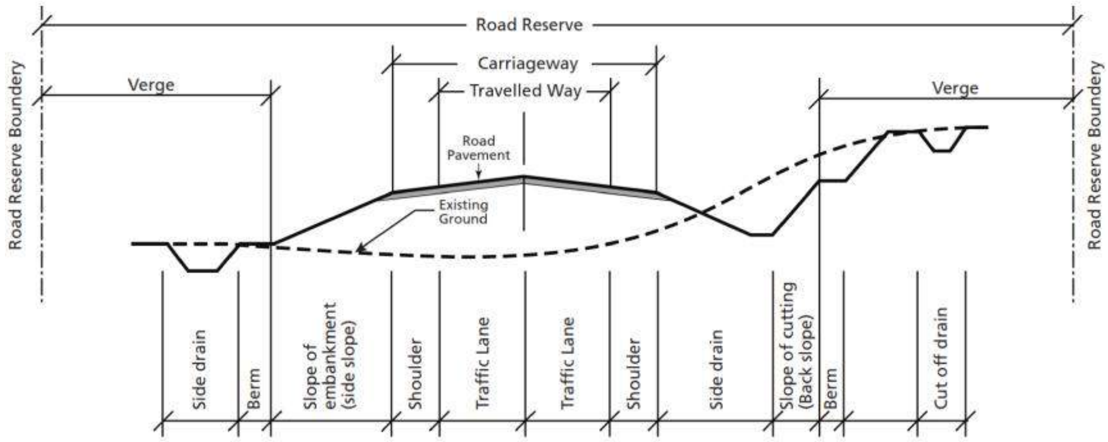


Figure 2: Cross-section elements

Drainage Elements

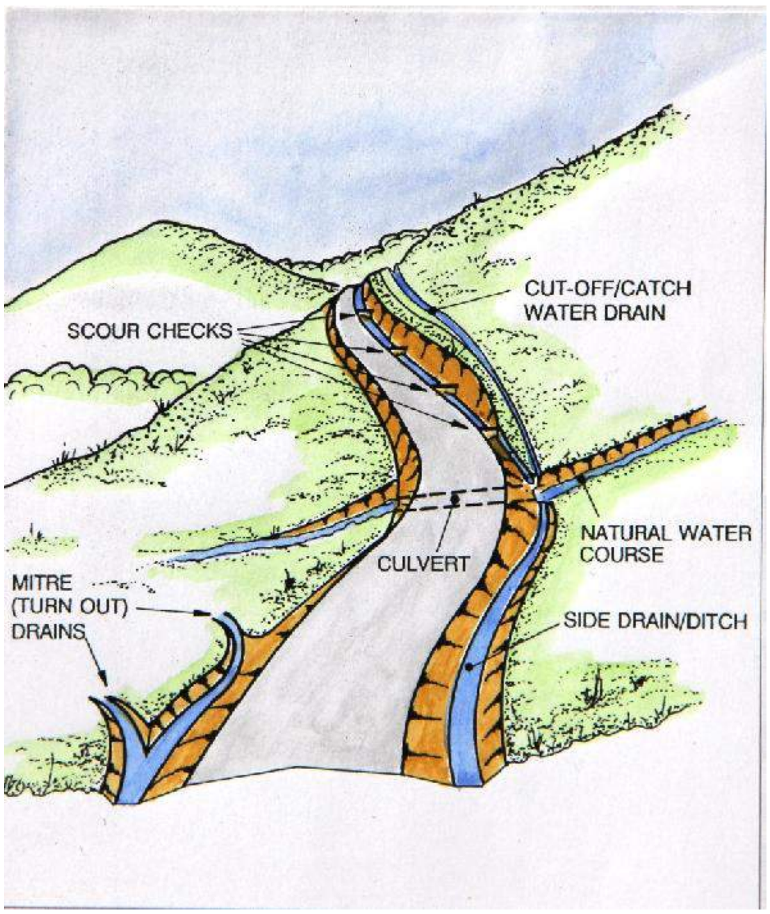


Figure 3: Main drainage elements

List of Abbreviations, Acronyms and Initialisms

AADT	Average Annual Daily Traffic
ASTM	American Society for Testing and Materials
BS	British Standard
CBR	California Bearing Ratio
CESA	Cumulative Equivalent Standard Axles
CSIR	Council for Scientific and Industrial Research
CUSUM	Cumulative Sum
DCP	Dynamic Cone Penetrometer
DES	Discrete Element Surfaces
DESA	Mean 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_{800}	Number of DCP blows required to penetrate top 800 mm of a pavement
DSS	Double Sand Seal
EF	Equivalence Factor
EIA	Environmental Impact Assessment
EIP	Environmental Impact Plan
EIS	Environmental Impact Statement
EMP	Environmental Management Plan
ENS	Engineered Natural Surface
ESA	Equivalent Standard Axle (80 kN)
EOD	Environmentally Optimized Design
ESIA	Environmental and Social Impact Assessment
ESP	Exchangeable Sodium Percentage
ETB	Emulsion Treated Base
FACT	Fines Aggregate Crushing Test
FHWA	Federal Highway Administration
FMC	Field Moisture Content
G60	Gravel with CBR of 60%
GB	Granular Base
Gc	Grading Coefficient
GM	Grading Modulus
GPS	Global Positioning System
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
MOTPW	Ministry of Transport and Public Works
MSL	Mean Sea Level
NMT	Non-motorised Traffic
NPV	Net Present Value
O/D	Origin & Destination
OMC	Optimum Moisture Content
ORN	Overseas Road Note
P075	Percentage 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
RA	Roads Authority
RED	Roads Economic Decision Model (a World Bank Model)
S2, S3	Subgrade Classes 2, 3 etc.
SA	South Africa
SAA	Sub-Saharan Africa
SADC	Southern African Development Community
SN	Structural Number
SNC	Modified Structural Number with subgrade contribution
SNP	Adjusted Structure Number
SOS	Single Otta Seal
Sp	Shrinkage Product
SS	Slow Setting

SSD	Single Surface Dressing
SSS	Single Sand Seal
TLC	Traffic Load Class
ToR	Terms of Reference
TRL	Transport Research Laboratory
TSC	Technical Steering Committee
UCS	Unconfined Compressive Strength
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

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Introduction

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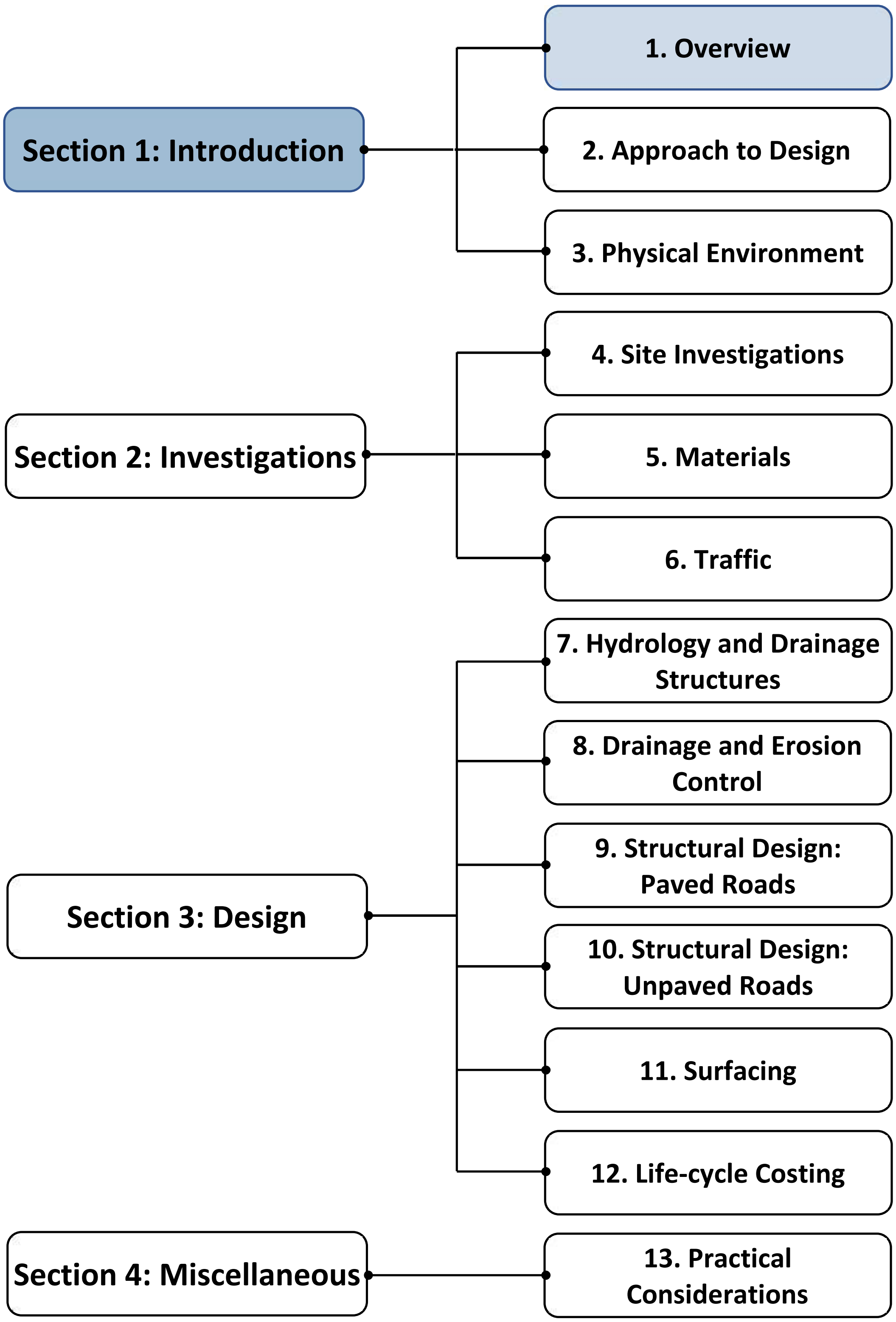

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1.1 Background

Low volume roads (LVRs) comprise a substantial proportion of the road network in Malawi and serve a large segment of the population that lives in the rural areas of the country where agriculture is the dominant economic activity. LVRs are viewed as a key driver for improving rural well-being, economic development, community livelihoods and food security. The attainment of this goal depends critically on the existence of sound rural road infrastructure. It is therefore important that roads agencies in Malawi are able to apply appropriate, sustainable, country-specific design standards and practices that are tailored to the rural road needs of the country.

The cost of providing rural road infrastructure based on traditional standards and design methods can be prohibitive. This is because these approaches, aimed at mobility, tend to be ill-matched to the dictates of access in the local road environment. As a result, they are generally far too costly for application to the rural road network in Malawi. This has led to a need to develop new Design Manuals for LVRs in Malawi that are tailored to the needs of the country and take account of the many advances in LVR technology that have taken place in recent times in the region and internationally.

1.2 Purpose

The main purpose of this Manual is to provide practitioners with the necessary guidance for undertaking a holistic, rational, appropriate, affordable and sustainable approach to the provision of both unpaved and paved LVRs in Malawi. Such an approach is aimed at minimising the life-cycle costs of road provision by taking account of the many local road environment factors that impact on the performance of LVRs. In so doing, the primary goal is to reduce the cost of providing low volume rural roads leading to:

Figure 1-1: Construction of a LVR in Malawi

- 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;
- reduced depletion of finite materials resources;
- enhanced socio-economic growth, development and poverty alleviation in Malawi.

The Manual draws on the outputs of a number of research and investigation projects that have been carried out in the region and internationally since the 1990s. The corroborative findings of these projects provide a wealth of performance-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 the Manual, which is expected to serve as a nationally recognized document, the application of which will harmonize approaches to the provision of LVRs in Malawi. The Manual is intended for use by all road sector practitioners such as the Roads Authority (RA), Local Councils, Consulting firms and Contractors.

1.3 Scope

The Manual caters for a range of road types, from basic earth tracks to bituminous sealed roads, that are typically found in Malawi. The environmentally optimised approach to the design of such roads is a key feature of the Manual that 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 country, it would be impractical and inappropriate to provide recipe solutions for specific situations. Instead, emphasis has been placed on guiding the practitioner towards evaluating alternative options and considering their pros and cons as a basis for decision making and application to region-specific situations. This is achieved by collating together in one document guidance in the application of tried and tested, new and innovative solutions in all aspects of LVR provision.

1.4 Development

The development of the Manual was overseen by a Technical Steering Committee (TSC) comprising the following stakeholder organisations.

- Malawi Roads Authority
- Ministry of Transport and Public Works – Roads Department
- Ministry of Local Government and Rural Development
- T2 Centre
- University of Malawi, The Polytechnic – Department of Civil Engineering
- Malawi Institution of Engineers
- Directorate of Road Traffic and Safety Services
- Association of Consulting Engineers

As a result of the high level of local participation in the development of the Manual, it has been possible to capture and incorporate a significant amount of local knowledge in the document.

1.5 Structure

The Manual is divided into four separate sections, each comprising chapters on various topics that are related to the design of LVRs as presented in Table 1-1.

Table 1-1: Structure and content of the Manual

Section	Chapter
A. Introduction	1. Overview 2. Approach to Design 3. Physical Environment
B. Investigations	4. Site Investigations 5. Materials 6. Traffic
C. Design	7. Hydrology and Drainage Structures 8. Drainage and Erosion Control 9. Structural Design: Paved Roads 10. Structural Design: Unpaved Roads 11. Surfacing 12. Life-cycle Costing
D. Miscellaneous	13. Practical Considerations
Appendix	Glossary of Terms

1.6 Benefits of Using the Manual

There are several benefits to be derived from adopting the approaches advocated in the Manual. These include providing LVRs that:

- Are less expensive in economic terms to build and to maintain through the adoption of more appropriate, locally-derived technology and design/construction techniques that are better suited to local conditions.
- Minimise adverse environmental impacts, particularly with regard to the use of non-renewable resources (gravel).
- Increase employment opportunities through the use of more appropriate technology, including the use of labour-based methods where feasible.
- Incorporate road safety measures to minimise road accidents.
- Take better account of the needs of all stakeholders, particularly the local communities served by these roads.
- Ultimately, facilitate the longer-term goal of socio-economic growth, development and poverty alleviation in Malawi.

1.7 Sources of Information

In addition to providing general information and guidance, the Manual also serves as a valuable source document because of its comprehensive lists of references from which readers can obtain more detailed information to meet their particular needs. A bibliography can be found 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.

1.8 Updating of the Manual

As LVR technology is continually being researched and improved, it will be necessary to update the Manual periodically to reflect improvements in practice. All suggestions to improve the Manual should be in accordance with the following procedures:

- Any proposed amendments should be sent to the Chief Executive Officer of the RA motivating the need for the change and indicating the proposed amendment.
- Any agreed changes to the Manual will be approved by the Chief Executive Officer of the RA after which all stakeholders will be advised accordingly.

1.9 Departure from Standards

There may be situations where the designer will be compelled to deviate from the standards presented in this Manual. An example of a Departure from Standard could be the use of a material specification that may be outside the limits given in the Manual. Where the designer departs from a standard, he/she must obtain written approval and authorization from the RA. The designer shall submit the following information to the RA:

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

The designer must submit all major and minor Departures from the Standards and his/her proposal for approval. If the proposed Departures from the Standards are acceptable, such departures will be approved by the Chief Executive Officer of the RA.

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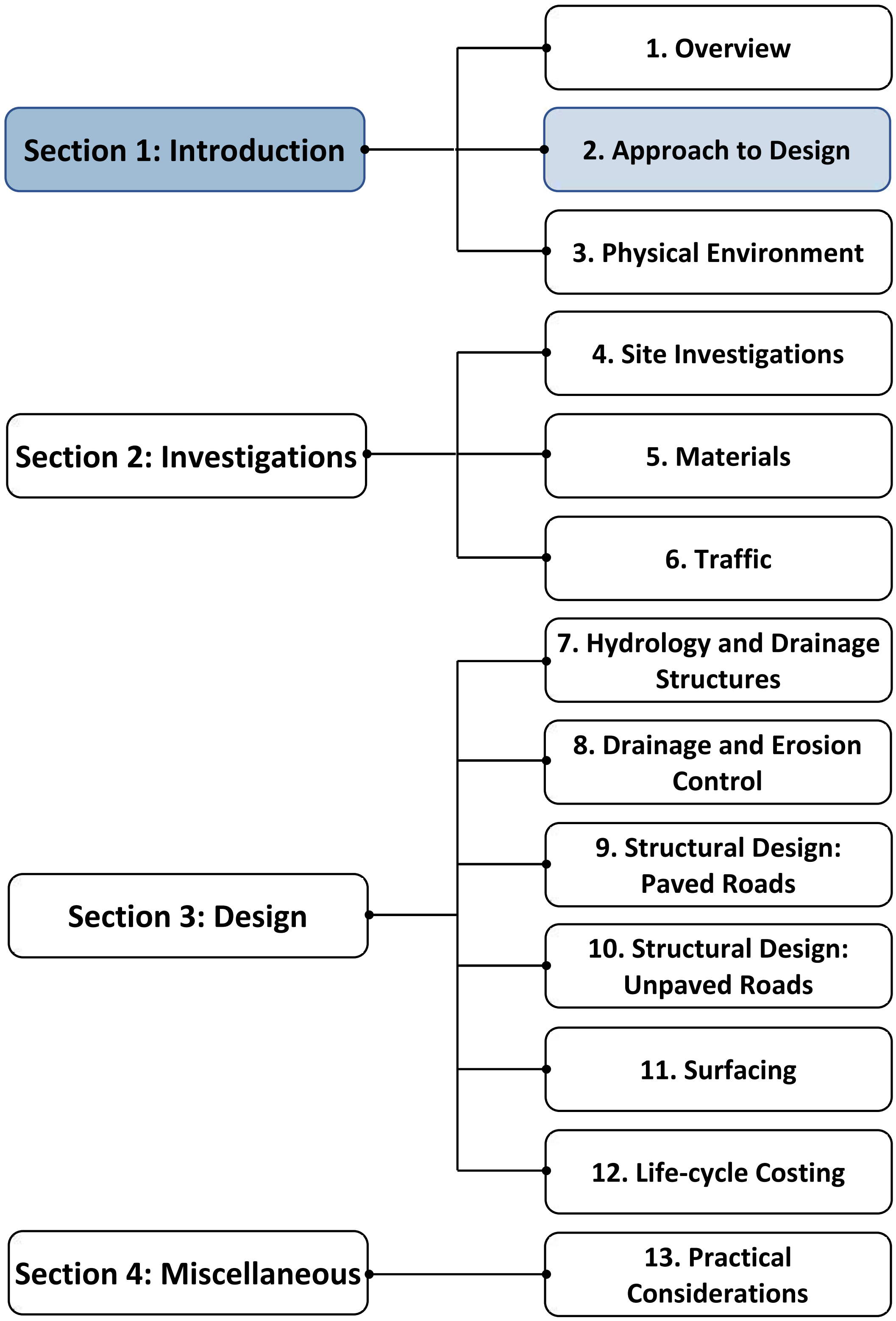
Low Volume Roads Manual
Volume 1 – Pavement Design



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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, such as those presented in the AASHTO design method, are based on technology and research carried out in external environments that are not reflective of those that prevail in these countries. While these "standard" approaches might still be appropriate for much of the main trunk road network, they remain conservative, inappropriate and too costly for application on much of Malawi's rural road network. Thus, in facing the challenges of improving and expanding the country's LVR network, more appropriate approaches need to be considered.

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 methods that need to be appreciated by the designer to provide designs that will meet with the multiple social, economic and environmental requirements of Malawi in a sustainable manner

2.1.2 Purpose and Scope

The main 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 LVR design philosophy.
- Various implementation considerations

Finally, a generic Table of Contents for a typical design report is presented in Appendix A.

2.2 Definition of Low Volume Roads

For pavement design purposes LVRs are defined as those roads that have a base year average annual daily traffic (AADT) of up to 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. Depending on the number and mass of the commercial vehicles in the traffic stream, the base year traffic for pavement design purposes, thus, could be somewhat more or less than 300 vpd for the same traffic loading. For geometric design purposes, however, the traffic at mid-life is required and this could exceed 300 motor vehicles per day. However, none of these figures provide a complete picture of the unique characteristics of LVRs in that there are many other aspects that need to be considered in their design, as discussed below.

2.3 Road Classification System

The type of LVRs being catered for in this Manual extends to all tiers of the road network as defined in the current Public Roads Bill, as follows:

1. **Main roads:**
    a. **Primary International:** Roads that link international centres. Connection between the national road system and those of neighbouring countries.
    b. **Primary National:** Roads that link cities, towns and centres of economic importance with each other and with major border posts or link to international roads.
2. **Secondary roads:**
    a. Secondary Arterial: Roads that link main centres of population and major towns to the international road network.
    b. Secondary Collector: Roads that link main centres of population, agricultural, commercial, recreational or major tourist areas or link to the national road network.

3. **District Roads:**
   - **a.** Roads that link district centres, villages, local centres of population and developed areas with each other or link to higher order roads of the road network.
4. **Community Roads:**
   - a. Roads that provide access to land adjacent to the collector network, or to villages or link to tertiary or higher order roads of the road network.
5. **Private Roads:**
   - a. Roads provided for use by private firms or persons

The classification system is also related to the administrative responsibilities for management of the road network, as follows:

- Main and Secondary Roads – Roads Authority
- District and Community Roads – District Councils
- Private Roads – Owner

In view of the above, the relationship between road class and road function is presented in Figure 2-1.

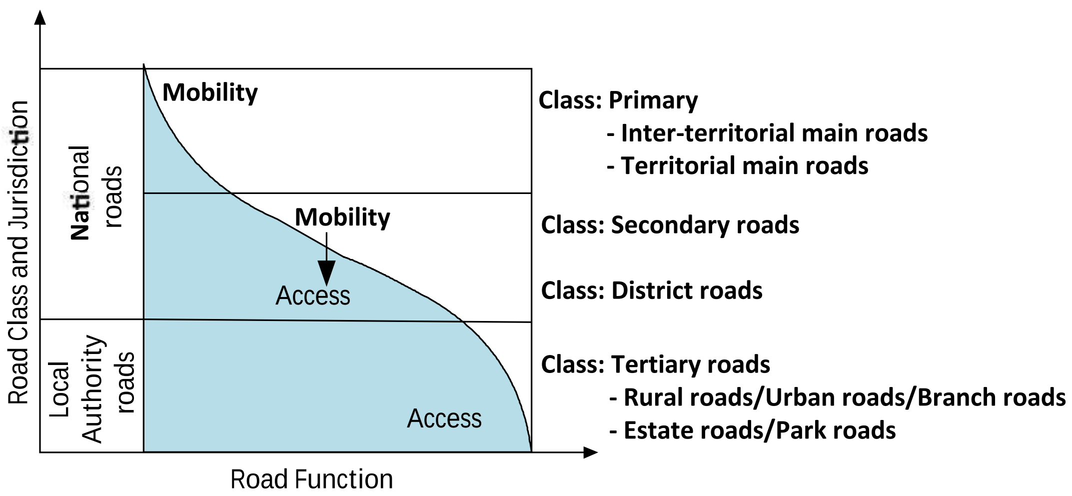


Figure 2-1: Relationship between road class and road function

A schematic diagram of the various road classifications is illustrated in Figure 2-2. The intent of the diagram is to illustrate the relative function of the road classifications in terms of primary, secondary and district and tertiary roads. This is a generic diagram, and, in practice, there will be many overlaps of function and clear distinctions may not always be apparent in functional terms alone. This hierarchy should not be confused with the division of administrative responsibilities, which may be based on other criteria.

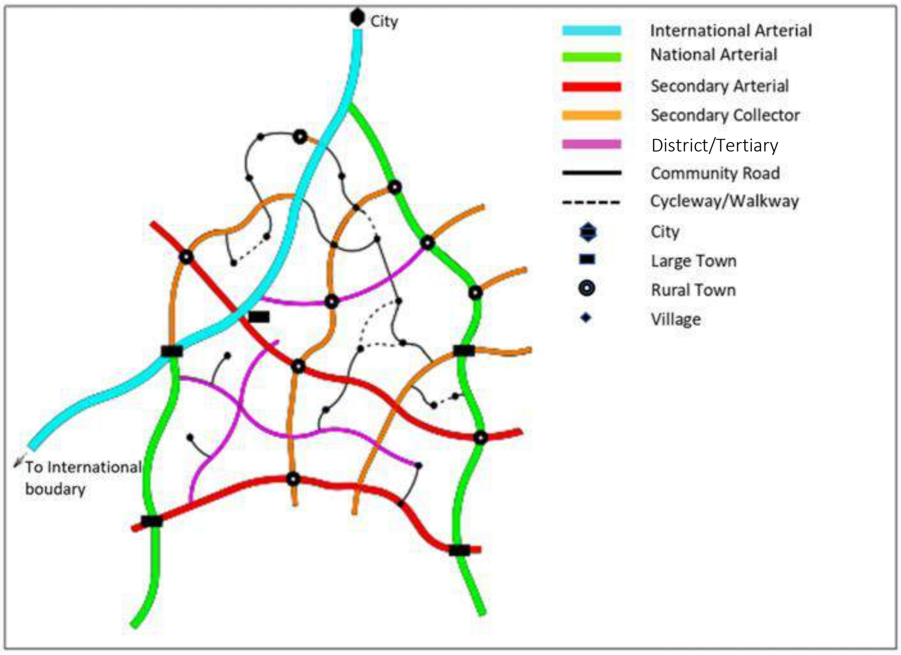


Figure 2-2: Road hierarchy and functions

Based on the specific function and typical characteristics of LVRs described above, it would be apparent that roads that do not generally fulfil these attributes would not fall under the heading of LVRs. For example, a functionally classified primary or secondary road carrying less than 300 vpd and less than 1 million MESA over its design life would not be classified as a LVR. This is because the level of serviceability that it would be expected to provide would be dictated by its function that is characterised by a relatively high design speed and corresponding geometric design requirements, which would be economically unjustifiable to apply to a LVR. Nonetheless, the pavement could be designed in accordance with the LVR design philosophy presented in this Manual.

2.4 Characteristics of Low Volume Roads

The following specific characteristics 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.
- The alignment may not necessarily be fully "engineered," especially at very low traffic levels, with most sections following the existing alignment and for which particular attention must be paid to road safety.
- A need to cater for non-motorized traffic, especially in urban/peri-urban areas, coupled with a focus on the adoption of a range of appropriate road safety measures.
- Variable travelling speeds that will seldom exceed about 60-80 km/h, as dictated by local vehicle characteristics and prevailing topography.

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 and that it can be relatively difficult to quantify the many other benefits that are of a broad socio-economic and environmental nature.

The unique characteristics of LVRs, as described above, challenge conventional engineering practice in several aspects, including materials and pavement design, geometric design, drainage and road safety for which particular attention has been paid in the development of the Manual.

2.5 Design Considerations

2.5.1 General

While 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. Thus, an optimum LVRR design requires:

- A full knowledge 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 thinking related to traditional road design.

In view of the many variables and interactions that influence the final choice of a LVR pavement it would be appropriate to adopt a "systems" approach to pavement design in which all influential factors are considered. Figure 2-3 shows schematically such a design system and illustrates the relationships between the input variables, design process and design outputs. For any particular set of inputs, alternative pavement designs can be produced and evaluated in terms of the engineering, operational and policy constraints which apply.

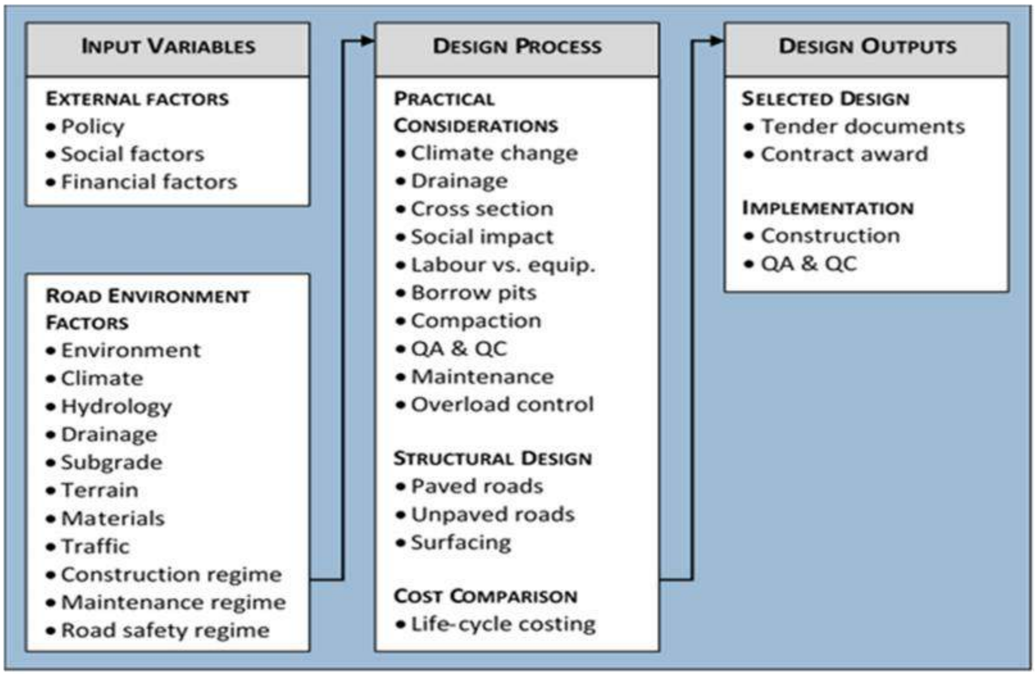


Figure 2-3: Pavement design system

2.5.2 Input Variables

Road environment factors

The term "road environment" is all-encompassing and includes both the natural or bio-physical environment and the human environment. It consists of 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 and intensity), local hydrology and drainage, terrain and gradient. Collectively, these will influence the performance of the road and the design approach needs to recognize such influence by providing options that minimize 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.

Environmental factors – primarily in terms of moisture and temperature - have a profound effect on pavement performance, as illustrated in Figure 2-4. Pavement deterioration of LVRRs is influenced mainly by how they respond to environmental factors, such as moisture changes in the pavement layers, fill and subgrade. In contrast, pavement deterioration on HVRs is influenced primarily by traffic. Thus, in the design of a LVRR, particular attention needs to be paid to the influence of moisture on the performance of the road, and to the adoption of appropriate drainage measures to mitigate against the adverse effects of moisture ingress into the pavement structure.

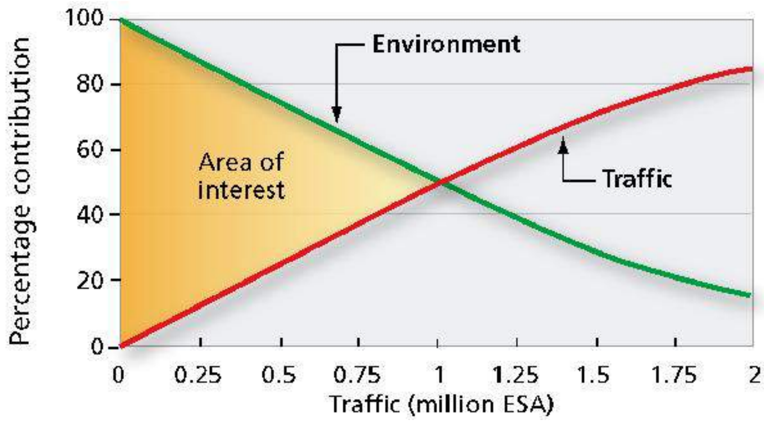


Figure 2-4: Traffic loading versus dominant mechanism of pavement distress (schematic only)

2.5.3 Design Process

External factors

There are a number of external factors outside the control of the designer which can influence the approach to design and which must be taken into account. These factors are discussed briefly below.

Policy: Government policy, national legislation and development planning dictate the underlying principles of LVR design. This includes, for example, environmental controls, road safety legislation, promotion where appropriate of the use of labour-based technologies to encourage local participation and the development of local contractors. National or local policies will provide guidelines, requirements and priorities for the decision-making processes. Such policies could typically include:

Design Issues: Examples of issues that could influence the design include:

- The type of technology to be adopted during construction. For example, if the national policy is to promote labour-based technology on certain classes of roads, then the design will have to be developed accordingly in terms of being conducive to the use of labour rather than plant.
- The occurrence of problem soils, such as expansive, collapsible or salt-laden soils, all of which require specialised design solutions, as discussed in *Chapter 5: Materials*.
- The constructability of the pavement layers in relation, for example, to the use of stabilized or unstabilised materials for which the former option would raise practical issues such as the ready availability of both specialized plant and the requisite construction skills.
- The feasibility of undertaking a staged construction approach for which there are several potential problems which should be carefully considered at the design stage, including:
    - A risk that the future upgrading does not take place at the appropriate time, thereby resulting in a more costly end product in life cycle terms.
    - The economic and social consequences of failure in the first stage
    - The cost of disruption to existing traffic flows during the second stage of construction.

- The choice of pavement or surfacing that may influence the design process. For example:
    - A need to use relatively low noise/smooth-textured surfacings in particular areas such as hospitals and residential developments.
    - A need to accommodate highly stressed sections of the pavement due to sharp turning/starting-stopping movements at intersections that require high stability surfacings.
- The availability of specific plant and materials which may preclude the adoption of certain types of pavements. For example, the unavailability of aggregates with the required crushing strength may rule out the use of certain types of surface treatments.
- The need to cater for situations where the pavement is likely to be exposed to soaked conditions for prolonged periods of time by the use, for example, of relatively moisture insensitive cement-, lime- or bitumen-bound materials as well as other measures to control the movement of water into or out of the pavement and subgrade.
- The need to promote internal pavement drainage by considering the relative permeability of specific combinations of pavement and shoulder materials and their configuration within the pavement cross section (i.e. avoidance of permeability inversion, where possible).
- Long term maintenance capability which, if not assured, may require the use of pavement/surfacing options with the least periodic maintenance interventions.

Construction issues: Examples of problems that could affect the construction process include:

- Construction of the pavement under traffic may be necessary where, for example, detours are not possible. This requirement will necessitate the use of materials that can be trafficked soon after construction and would preclude the use of relatively lengthy construction processes, such as stabilization.
- The availability of water which, if located at a significant distance from the road alignment, could adversely affect the construction period.
- Water-logged areas which may prevent access to the site during the rainy season.

Structural design

There are a number of methods available for the structural design of both paved and unpaved LVRs, including the DCP-DN, DCP-CBR and DCP-SN methods, as discussed in *Chapter 9: Structural Design: Paved Roads* and *Chapter 10: Structural Design: Unpaved Roads.* The design catalogues, associated with these methods are relatively easy to use as all the practical and theoretical work have been carried out and different structures are presented in catalogue form for various combinations of traffic, environmental effects, pavement materials and design options.

Cost comparisons

The optimum pavement design solution is that which satisfies the design requirements for the specified input at minimum cost, for the whole life of the pavement. The cost criterion to be applied for comparison of alternatives is minimum life-cycle total cost which allows for discounted future maintenance and rehabilitation costs. However, certain design constraints, sometimes of a policy nature, may need to be applied to allow for factors which influence maintenance, safety and road users. These constraints may need to override cost considerations and are discussed in *Chapter 12 – Life-Cycle Costing.*

2.5.4 Design Outputs

Practical considerations

Having chosen the selected design on the basis of a life-cycle cost analysis, there are several practical considerations that need to be considered after the completion of the pavement design, all of which could affect the final design and subsequent construction and maintenance. These include such factors as Engineering Adaptation to Climate Change, Pavement Cross Section and Quality Assurance and Control. These and other similar issues are discussed in *Chapter 13 – Practical Considerations.*

Implementation

The implementation of the selected pavement design is achieved through an efficient and effective construction process that relies on the adoption of a construction strategy that is appropriate to the social, economic and cultural environment of Afghanistan. Thus, the implementation needs to be underpinned by a rigorous quality control process that ensures that the assumptions made in the design process are achieved in practice.

2.5.5 Adoption of an Environmentally Optimised Design Approach

In order to obtain optimal results from investments in road infrastructure in Malawi, it is essential to adopt an approach that is guided by appropriate local standards and conditions. In this regard, international research has highlighted the 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 outlined below and illustrated in Figure 2-5. The essence of the EOD approach is that it is:

1. Task based: LVRs must suit their identified function and the nature of the traffic (the people as well as the vehicles) which will pass along them, by applying appropriate standards.
2. Environmentally compatible: Suitable for, and where necessary, adapted to the local road environment factors.
3. Local resource-based: The design of the LVR must be compatible with the construction materials that are readily available within appropriate specifications, and within the capacities of the engineers and technicians who will design the roads, and the contractors who will construct them, and within the means of the roads agency to maintain them, involving local communities, where possible.

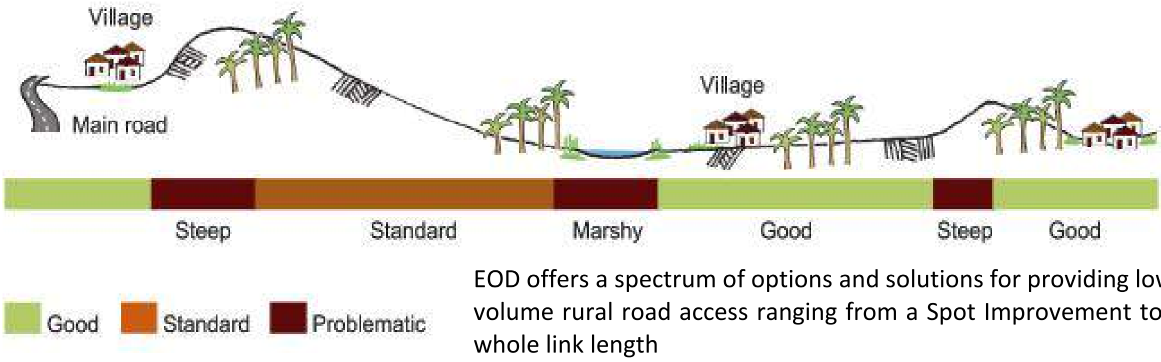
Figure 2-5: LVR implementation within an EOD context

EOD is a strategy for utilising the available resources of budget and materials in the most cost-effective manner to take account of the variable factors of traffic, terrain, materials and subgrade that may exist along an alignment. In order to be successful and sustainable, LVR technology needs to be implemented within the framework of an EOD strategy. Moreover, if the LVR project is to be sustainable in the long run, several strategic objectives should be satisfied, 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 during the 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.
- Maximum use of local labour and skills.
- Maximum use of materials that are locally available or those that can be processed locally.
- In-built maintenance considerations in the design such as provision of adequate drainage, resistance to soil erosion along the side slopes, adequate lateral support from shoulders etc. as would minimize 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 individual requirements, ranging from engineered natural surfaces to bituminous pavements. The chosen solution must be achievable with materials, plant and contractors available locally.

The EOD approach ensures that specifications and designs support the requirements of different road sections and assesses whether the same design is appropriate for both problematic and good areas, as is often not the case. An under-design of poor sections can lead to premature failure and an over-design will often be a waste of resources that would be better applied to the problematic sections. The EOD principle is illustrated in Figure 2-6.



EOD offers a spectrum of options and solutions for providing low-volume rural road access ranging from a Spot Improvement to a whole link length

Figure 2-6: Environmentally optimized and spot improvement approach

2.5.6 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 are essential aspects of the road environment assessment. The challenge is to adopt appropriate design and material standards that deliver acceptable solutions. In this regard, there is now 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 5 – Materials* and *Chapter 9 – Structural Design: Paved Roads.*

2.5.7 Surface Improvement Technology

Earth and gravel roads are particularly vulnerable to the effects of the road environment. A range of more durable surfacing options, other than gravel or earth, 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 11 – Surfacings.*

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.

2.5.8 Upgrading Stages of a Low Volume Road

The decision as to when an 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. In practice, a spot improvement strategy could be adopted as discussed in Section 2.4.4 above. Thus, over a period of time, a road will often undergo a number of improvements or upgrading iterations during its use. Figure 2-7 illustrates the various upgrading stages of LVRs. A life-cycle cost analysis, as discussed in *Chapter 12 – Life-Cycle Costing*, should be undertaken to determine when to upgrade from one standard to a higher one.

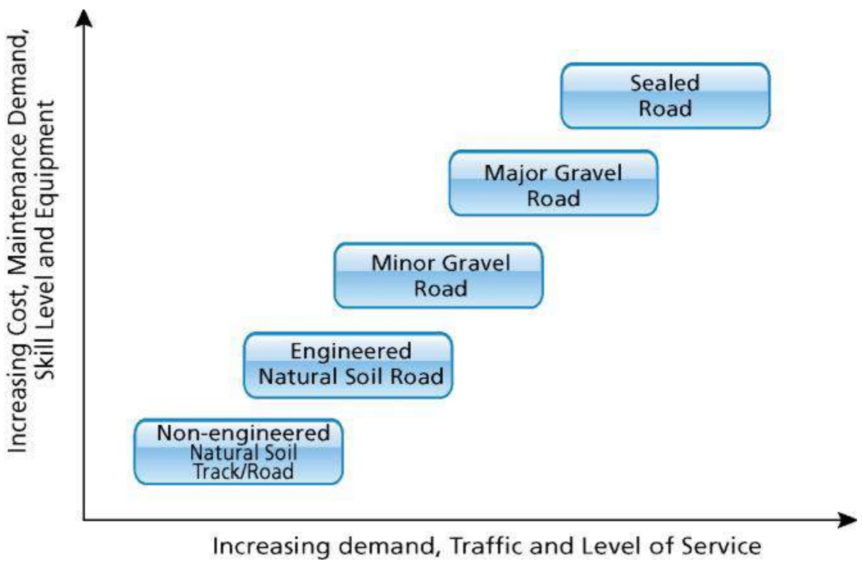


Figure 2-7: Upgrading stages of low volume roads

2.5.9 Risk Factors

The departure from well-established, conservative, material quality specifications may carry some increased level of risk of failure for an 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 minimize them. These factors are summarized below:

- Quality of the materials (strength and moisture susceptibility)
- Construction control (primarily compaction standard and layer thicknesses)
- Environment (particularly drainage)
- Maintenance standards (drainage, surfacing and shoulders)
- Vehicle loads (overloading)

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 in a timely manner and vehicle overloading is reasonably well controlled.

The above factors are discussed in more detail in *Chapter 13 – Practical Considerations.*

2.6 Project Implementation

2.6.1 Level of Influence of Key Activities

The four major stages of LVR provision are as follows:

- Planning
- Design
- Construction
- Maintenance

The above stages have important but changing impacts on the completed project in terms of a "level of influence" concept. This concept shows how the effect on the total life-cycle costs of a project decreases as the project evolves, as illustrated in Figure 2-8 in which the lower portion of the figure presents in bar chart from the length of time each major activity acts over the life of a pavement. The upper portion shows a plot of increasing expenditures of the project activity and their related decreasing influence on the life-cycle cost of the project.

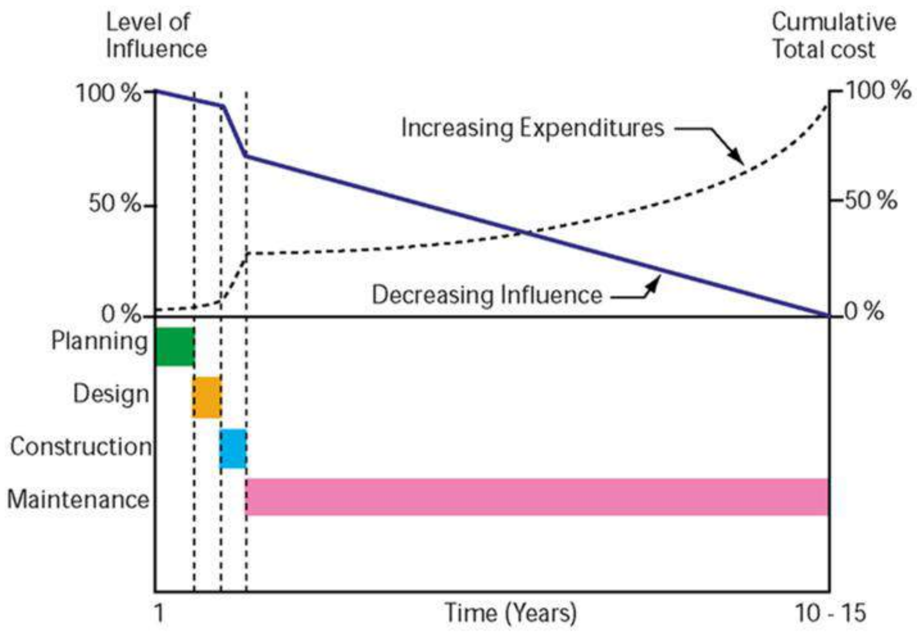


Figure 2-8: Level of influence of activity in relation to life-cycle cost of LVR provision

2.6.2 Implementation Implications

As illustrated in Figure 2-8, at the beginning of the project cycle, the roads agency controls all factors (100 per cent influence) in determining future expenditures. The key challenge is how to ensure that the key components of LVR provision are optimised so as to minimise the life-cycle cost of LVR provision.

The key message from Figure 2-8 is that:

- Expenditures incurred during the planning and design phases of a project are relatively small compared with total expenditure and are incurred during a relatively short period of the project's life. However, in terms of decisions and commitments made during the early phases of the project, they exert a significant influence on the downstream phases of the project (construction, maintenance, operations).

  This emphasizes the importance of employing a broadly-based, holistic approach to the planning of LVR's with the main stakeholders being involved in the decision-making process. In addition, the designs employed (geometric and pavement) should be appropriate and relevant to the road environment factors that impact on the road.

- The capital costs for construction are a fraction of the maintenance and operating costs associated with a pavement life cycle. However, the decisions made during the *design* and *construction* phases can significantly impact on the maintenance and operating costs of the road.

  This emphasizes the importance of adopting appropriate construction techniques, ensuring a high degree of quality assurance and control in the use of local materials and complying fully with the project standards and specifications.

- The maintenance phase of the project occupies a significant number of years in the life of the project and the type and cost of maintenance required are influenced significantly by the preceding planning, design and construction phases.

  This emphasizes the importance of ensuring that the preceding project phases are properly carried out and that the maintenance phase is prolonged as much as possible to extend the useful life of the road and the period of time during which benefits are incurred.

2.7 Sustainability

2.7.1 General

In addition to ensuring that the design developed is technically appropriate and is within the budgetary constraints, the design engineer needs to bear in mind other factors that could influence the success of the LVR design approach and its long-term sustainability. This requires the adoption of a broadly focused, context-sensitive approach in which a number of other influential factors are considered, as illustrated in Figure 2-9.

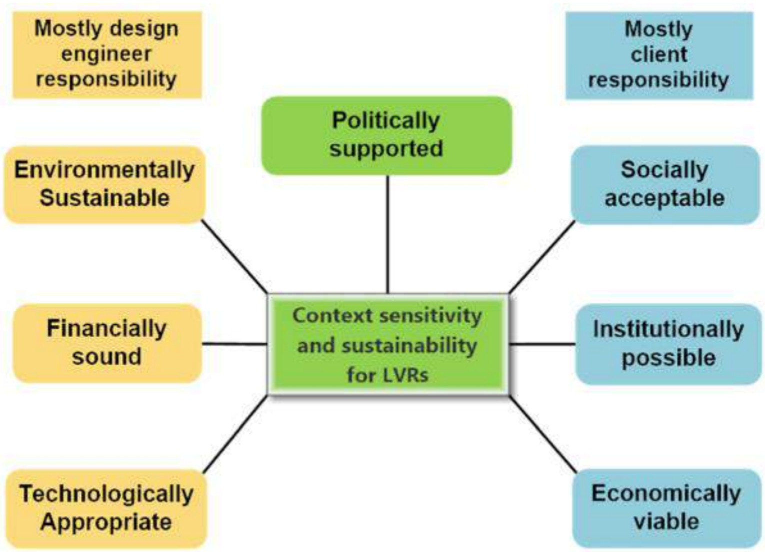


Figure 2-9: Framework for sustainable provision of LVRs

2.7.2 Strategy for Ensuring Sustainability

In terms of ensuring sustainability in LVR provision, such roads need to be framed under a national policy driven by Government and should be supported at the highest level. The cross-sectoral influence of LVR provision and its role in underpinning other sectoral development strategies and poverty alleviation programmes should be highlighted, quantified and disseminated.

There is a need to maintain a dialogue with political and public stakeholders in order to highlight the advantages of design approaches and alternative, often unfamiliar, solutions adopted for LVR provision. The language used for advocacy should be carefully chosen and should avoid negative connotations such as "low standard", "low cost", "marginal" and "relaxed". Instead, the language used should emphasise the provision of LVRs that are "fit for purpose" in terms of providing appropriate all-season access for rural populations at minimum life-cycle costs, "affordable" within the often resource-constrained budgets of rural roads *agencies* and "environmentally sustainable" in terms of the design of roads that are compatible with the local road environment.

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Appendix A: 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:

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

Low Volume Roads Manual
Volume 1 – Pavement Design

- **Section 1: Introduction**
  - 1. Overview
  - 2. Approach to Design
  - 3. Physical Environment
- **Section 2: Investigations**
  - 4. Site Investigations
  - 5. Materials
  - 6. Traffic
- **Section 3: Design**
  - 7. Hydrology and Drainage Structures
  - 8. Drainage and Erosion Control
  - 9. Structural Design: Paved Roads
  - 10. Structural Design: Unpaved Roads
  - 11. Surfacing
  - 12. Life-cycle Costing
- **Section 4: Miscellaneous**
  - 13. Practical Considerations

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3.1 Introduction

3.1.1 Background

The physical environment of the project site exerts a great influence on the design and performance of LVRs. Thus, for a designer, it is essential to have a comprehensive understanding of the various factors that make up the physical environment to cater for them appropriately in the design of the LVR. The subgrade soils along the alignment of a road, for example, 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.

3.1.2 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. The chapter discusses both the physical features and climate of Malawi and indicates their potential impact on the design process.

3.2 Physical Features

3.2.1 General

The following physical features are considered in this section:

- Topography
- Geology
- Soils
- Hydrology
- Vegetation

3.2.2 Topography

A major part of Malawi (>40 %) is classified as 'plain' with slopes of less than 10 %, and about 10 % is classified as dissected plain. Hills and ridges make up about 13% of the territory.

The Malawi topography consists of five zones:

(1) High Plateaus located more than 1500 m above mean sea level (MSL) covered by more erosion resistant underlying strata;

(2) High Hill Country, mountainous areas that rises abruptly from the plateau, located at 1400 m - 1500 m above MSL covered by a more erosion resistant underlying strata;

(3) Plains/Plateaus that are gently undulating surfaces with broad valleys and large level areas on the interfluves generally located at 600 m - 1400 m above MSL. They are extensively covered by a thick weathered material;

(4) Rift Valley Escarpment that fall steeply from the plateau areas where erosion significantly strips away the weathering products; and,

(5) The Rift Valley floor (hosting the alluvial plain) located below 500 m above MSL, which is gently sloping and of very low relief.

Table 3-1 summarises the physiographic zones in Malawi.

Table 3-1: Physiographic zones in Malawi

Physiographic Zone	Erosional Surface Equivalent	Altitude (Metres)	Slopes (Degrees)	Examples
High Plateau	Gondwana	1500 - 2400	5-15	Nyika, Viphya, Zomba and Mulanje
Hill Country and Mountainous	Post Gondwana	1400 -1500	Moderate to Steep	Dissected Nyika, Viphya, Zomba and Mulanje
Plains/ Plateaus	African	600-1400	2-5	Lilongwe-Kasungu and Phalombe
Rift Valley Escarpment	Post African	500-600	Steep	Thyolo and Livingstonia Escarpment
Rift Valley Floor	Quaternary	30-500	Flat	Karonga, Nkhotakota, Salima and Lower Shire

Detailed topographic mapping is available for most areas of Malawi in a scale of 1:50 000 from the Department of Surveys, Ministry of Lands, Housing and Urban Development.

The diverse features of the topography, in terms of whether the terrain is flat or mountainous, impact on a number of technical and economic (cost) aspects of LVR design including:

- geometric design in terms of horizontal, vertical alignment and road cross- section;
- drainage and anti-erosion measures;
- traffic safety measures; and
- choice of road surfacing.

3.2.3 Geology

The greater part of the geological setting of Malawi is characterized by crystalline metamorphic and igneous rocks of Pre-Cambrian to Lower- Palaeozoic Age which are referred to the Malawi Basement Complex, which were later overlain by Karoo sedimentary rocks and intruded by basaltic/dolerite dykes and sills. The basement complex that covers approximately 70 % of Malawi's landscape, has undergone a prolonged structural and metamorphic history. The Karoo Supergroup along the Malawi Basement Complex, is cut by the Mesozoic alkaline igneous rocks. The Post-Basement Complex development of the country was dominated by epeirogenic movements, faulting and the formation of the Malawi Rift Valley.

Large tracts of plains are covered by various superficial deposits. The major lithological units of the basement complex are syenitic granites. The Rift Valley has modified and interrupted this landscape.

At various localities in the north and south of the country, these rocks are overlain from Permo-Triassic to Quaternary. Large inselberg of these rocks rises above the plateau as a result of epeirogenic events and form a distinctive feature of the local geology. The Permo-Triassic period was later followed by Upper Jurassic – Lower Cretaceous period which saw the intrusion of syeno-granitic and nepheline syenite rocks that were later intruded by volcanic rocks infilled by carbonatite and alkaline dykes, charconoctic and ultra-basic gneisses, schistis, granulalite and quartzites. These rocks occur widely throughout southern Malawi and have been grouped as Chilwa Alkaline Province. The same period saw sedimentary deposition characterized by Dinosaur Beds. The above rocks have been overlain by Tertiary – Pleistocene rocks characterized by consolidated to semi consolidated beds grouped into Timbiri, Chiwondo, Chitimwe and Alluvial.

Table 3-2 shows the lithostratigraphic units in Malawi.

Table 3-2: Lithostratigraphic units in Malawi

Lithostratigraphic Units	
Precambrian – Lower Palaeozoic	**Basement Complex** • Mafic, Ultramafic meta-igneous • Charnokitic suite • Biotite and Hornblende gneisess • Quartz feldspathic granulites, gneisses and quartzites • Calc silicate rocks and marbles • Mica schists and gneisses • Granitic and pegmatitic gneisses • Aegerine and nepheline gneisses • Conglomerates, quartz sandstones, • Phillites and siltstones (Mafingi group) • Phyllonites, quartzite, granites
Permian – Triassic	**Karoo System** • Conglomerates • Sandstone • Mudstones • Carbonaceous Shales • Basalt/Dolerite dykes and sills
Upper Jurassic – Lower Cretaceous	**Chilwa Alkaline Province** • Syenites • Carbonatites **Sedimentary** • Dinosaur beds
Tertiary - Pleistocene	**Various Beds (Miocene)** • Pebbly sandstones • Conglomerates • Marls • Sands • Gravel
Recent	• Lacustrine • Alluvial • Colluvial

Source: Malunga (2018)

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 would usually be gravelly with low plasticity compared with those derived from basic volcanic rocks, which would have higher clay contents with high plasticity.

Figure 3-1 shows a geological map of Malawi. Large scale geological maps from the Geological Surveys Department should be obtained for the relevant project area as a basis for planning site investigations and materials prospecting.

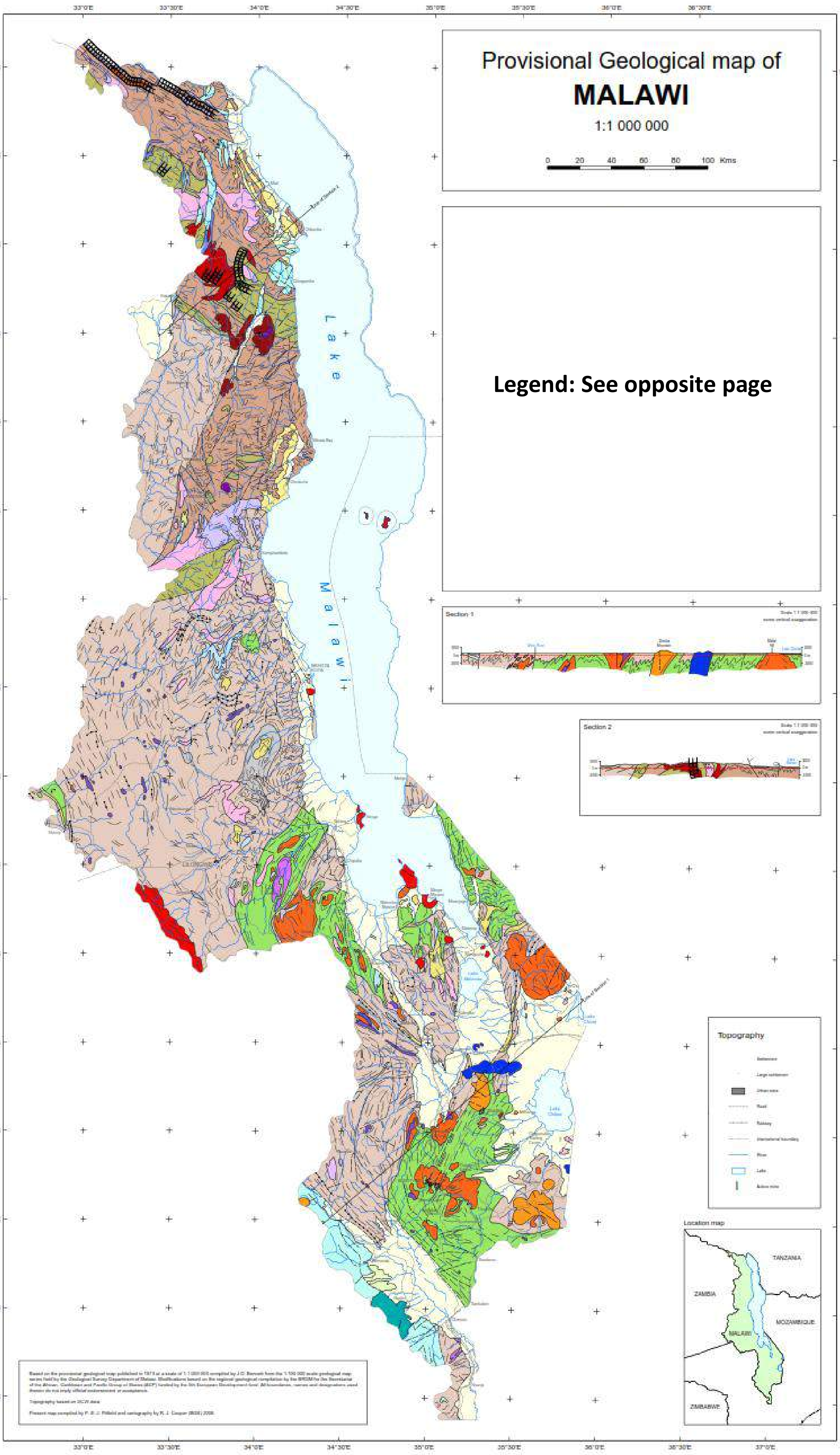
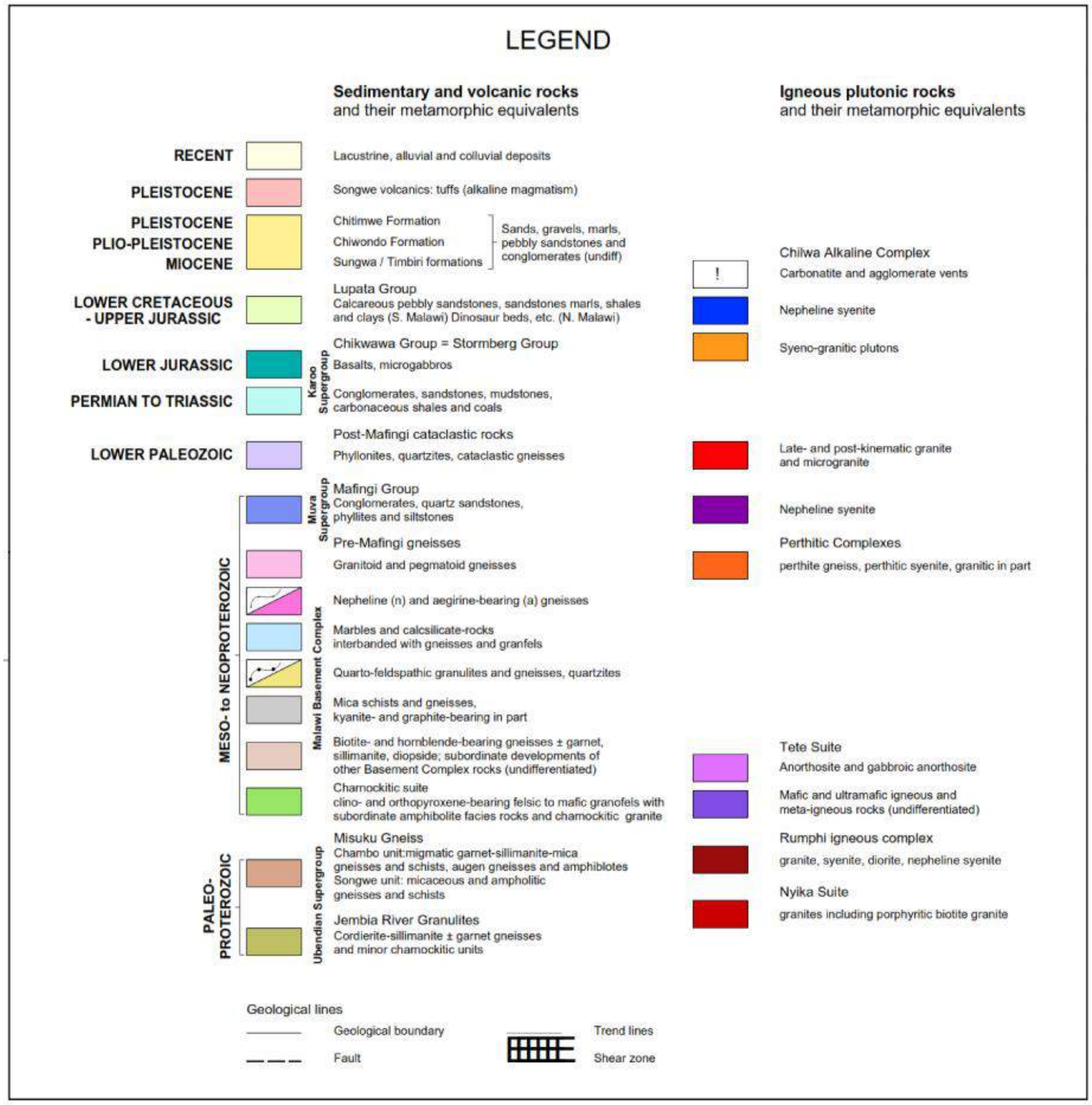


Figure 3-1: Geological map of Malawi



3.2.4 Soils

The country has four main soil classes, namely:

Latosols: These are red-yellow soils which include the ferruginous soils of Lilongwe plain and some parts of Southern Region. The weathered ferrallitic (plateau or sand-veld) soils some with a high lateritic content. Ferrallitic soils cover large parts of the plains along the western border of the country. They are also found in high rainfall areas such as Nkhata Bay.

Lithosols: The most widely spread of the lithosol group are the shallow stony soils that are associated with steep slopes.

Calcimorphic soils: This soil group includes the alluvial soils of the lacustrine and riverine plains; the vertisols of the Lower Shire Valley and the Phalombe plain; and the mopanosols in the Liwonde and Balaka areas.

Hydromorphic soils: These are grey soils of the hydromorphic group which are in found either seasonally or permanently wet areas, as in Lake Chilwa plain and Lower Shire Valley, and localized marshy areas known as "dambo". Most of the soils in the Rift Valley are of alluvial origin. In the hilly places the soils are shallow.

Figure 3-2 shows a soils map of Malawi.

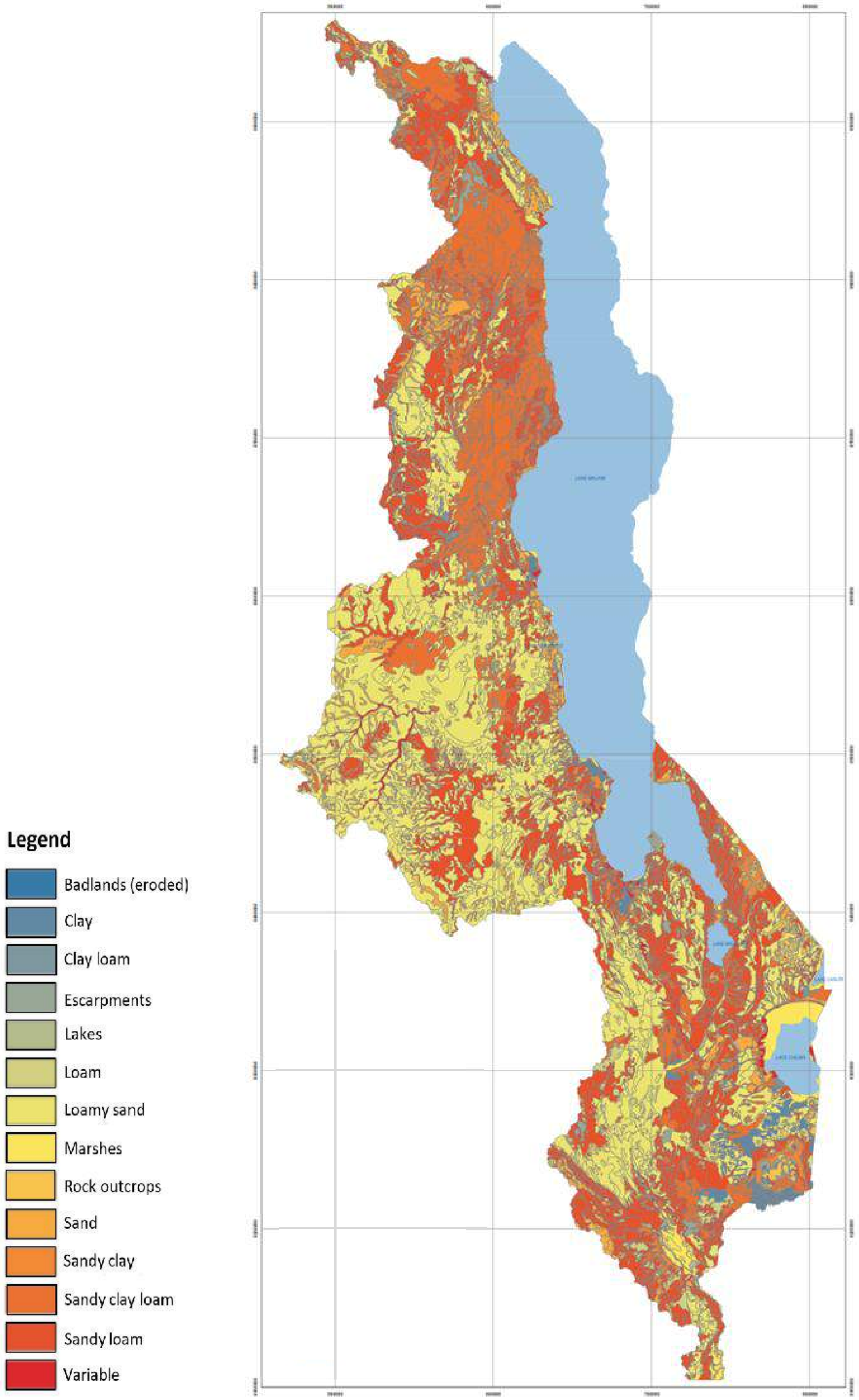


Figure 3-2: Soils map of Malawi

3.2.5 Hydrology

The hydrological systems in Malawi comprise of a network of river systems such as the Shire, Songwe, Ruo, Bua, Linthipe, Lilongwe, Rukuru and lakes such as Lake Malawi, Lake Chilwa, Lake Chiuta and Lake Malombe. The Songwe River is located in the border of Malawi and Tanzania. The river is divided into middle part and lower part with the lower part being a flood plain. The middle part of the river often changes its course during the flood. The North Rukuru River basin is one of the areas vulnerable to flood in Malawi, often occurring due to levee breach in the river.

There are two major drainage systems. The first is the Lake Malawi system, which is part of the Zambezi River basin, with the Shire River as the only outlet of the lake. About 91 % of the country is located in the Zambezi River basin. The second is the Lake Chilwa system, which is shared with Mozambique. Lake Chilwa is an endorheic basin draining rivers originating from the eastern slopes of the Shire Highlands, the Zomba Plateau and the northern slopes of the Mulanje Massif.

The National Water Resources Management Policy developed in 1994 outlines the policy and strategies for water resources management in Malawi. The country is divided into 17 Water Resources Areas (WRAs), which are subdivided into 78 Water Resources Units (WRUs).

3.2.6 Vegetation

The wide variation in physiography, climate has given rise to a large variety of vegetation types in Malawi. The dominant vegetation is Miombo woodland characterized by broadleaved Brachystegia species. It is relatively moist woodland that intergrades into savannah. In southern Malawi, the relatively dry, broadleaved mopane woodland is more common, often intergrading into savannah vegetation.

The vegetation of Malawi has been classified as follows:

1. Montane evergreen forest;
2. Montane grassland;
3. Semi-evergreen forest
   a. Closed canopy woodland of wetter uplands (tall Brachystegia spp);
   b. Open canopy woodland of plateaux Brachystegia / Julbernadia / Isoberlinia);
   c. Open canopy woodland of hills and scraps (Brachystegia spp);
   d. Open canopy woodland fertile areas (Piliostigma / Acacia / Combretum); and
   e. Mixed thicket/woodland of drier upland.
4. Woodlands
   a. Mopane woodland;
   b. Woodland soft fertile areas (Adansonia / Cordyla / Falderbia albida);
   c. Thicket/savanna of poorer areas (Combretum/Acacia); and
   d. Woodland savanna of poorer areas (mixed species).
5. Sand dune vegetation
   a. Grasslands (seasonally wet);
   b. Grasslands (perennially wet/swamp);
   c. Lakes (fresh water); and
   d. Somewhat saline lakes (without outlet).

3.3 Climate

3.3.1 General

Malawi's climate is influenced by the country's proximity to Lake Malawi that covers almost two-thirds of its length. The climate is tropical continental with two distinct seasons; the rainy season from November to April and the dry season from May to October. However, from May to July, it is relatively cool and in some high-altitude areas, drizzles (Chiperoni rains) are common.

Climatic data is available from the Malawi Meteorological Services Department, which maintains a network of 22 full meteorological stations, 21 subsidiary agrometeorological stations, strategically located in the eight Agricultural Development Divisions, and over 400 rainfall stations.

3.3.2 Rainfall

Annual rainfall in Malawi ranges from 400 mm to 1,800 mm, depending on the topography and climatic conditions. Its distribution is influenced by topography (orographic effects) and proximity to the lake. Much higher rainfall is experienced in uplands as compared to lowlands. Least rainfall is registered in rain shadow areas such as in the Shire Valley, west of Shire Highlands and Zomba plateaux (e.g. Lake Chilwa area) and north-west of both Viphya and Nyika plateaux. Highest rainfall is experienced in high altitude areas, e.g. in the Mulanje, Nyika and Viphya plateaux.

Average annual rainfall is shown in Figure 3-4.

Figure 3-3: Main meteorological stations in Malawi

3.3.3 Temperatures

Malawi experiences a tropical continental climate, with a cool dry season from May to August, a hot dry season from September to November, and a fairly hot wet season from December to April. The mean annual minimum and maximum temperatures for Malawi range from 14°C to 32°C. Temperatures are influenced by variations in relief.

There are three temperature zones:

- The Shire Valley and the Lake Malawi littoral experience mean annual temperatures above 27°C;
- The plateau areas are characterised by mean annual temperatures in the range of 24°C to 29°C. An example of this temperature zone is the Central Region Plateau, comprising the Lilongwe Plain.
- The higher plateaus and mountain areas such as Zomba, Nyika and Mulanje and these experience mean annual temperatures of 14°C to 18°C.

The highest temperatures occur at the end of October or early November, but after that the rains bring moderating effects. The coldest months are June and July. Highest temperatures are recorded in the Shire Valley and along the Lake Malawi shores while the lowest are recorded over the high attitude areas particularly the Shire Highlands, the Viphya and Nyika plateaux, Dedza and Mulanje mountains and other high-altitude areas. Frost is rare but has at least been recorded at Chitedze, Lilongwe, Dedza, Bvumbwe, Mimosa, Chichiri and Mzimba meteorological stations.

Knowledge of the day- and night-time temperatures in a project area is important for selection of appropriate bituminous binders, which will affect the performance of bituminous surfacings.

3.3.4 Climatic Zones

Table 3-3 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.

Figure 3-4 shows that most of the country is within the moderate zone with Weinert N values between 2 and 4 and smaller areas in the south-east, the extreme north and along shores of Lake Malawi being in the wet zone with Weinert N values < 2.

Table 3-3: Guideline for selection of climatic zone

Description		Weinert N value	Thorntwaite Moisture Index, 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

Source: Modified from TRL report R6990 – Rational Road Drainage Design

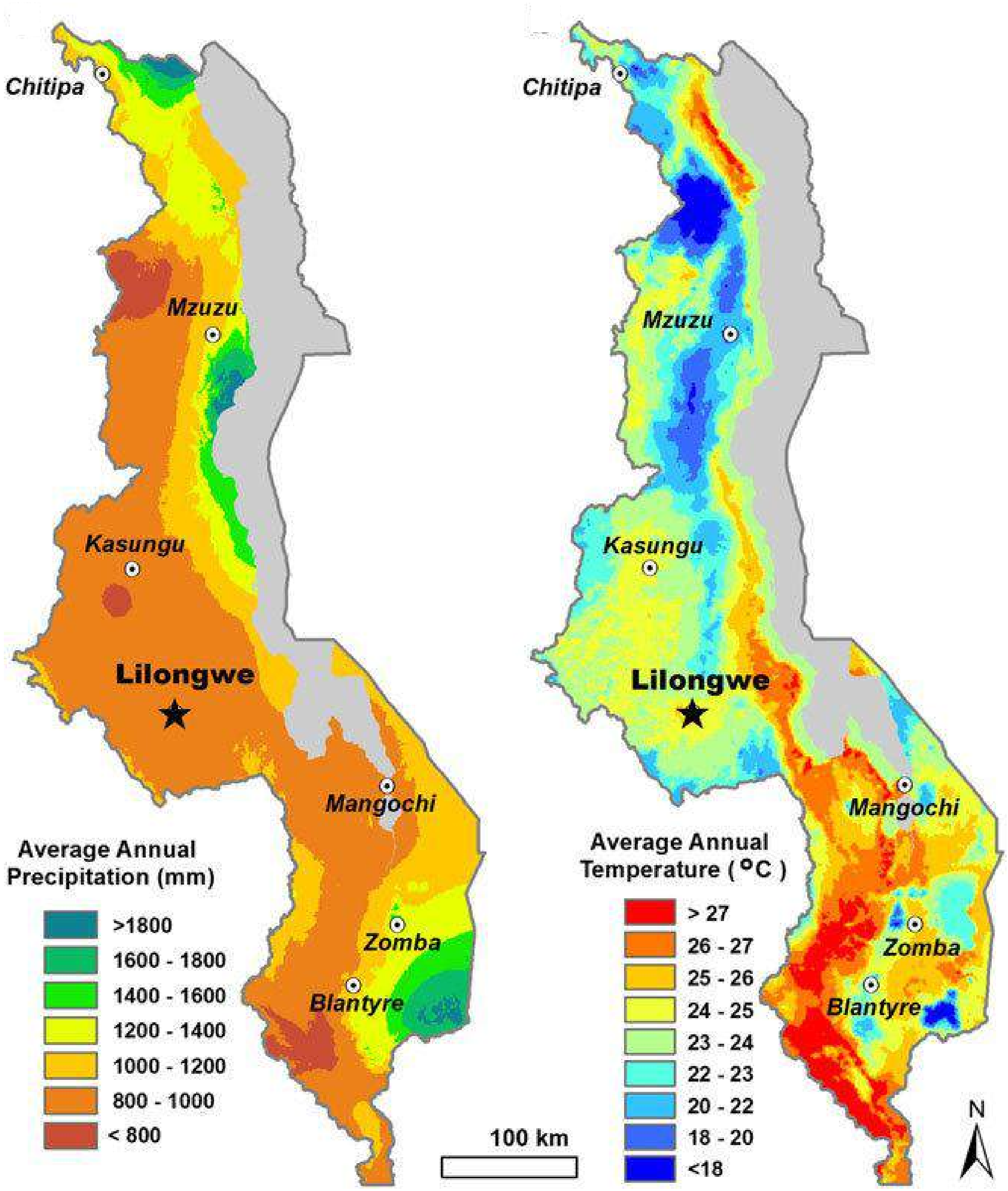


Figure 3-4: Rainfall and temperature Map of Malawi

3.3.5 Climate Change

The change in the climate of Malawi during the 21st century is predicted to be marginal in terms of rainfall. Malawi is located between a large part of the southern African interior projected to become generally drier, and a large part of East Africa (from northern Mozambique in the south to the Horn of Africa) that is projected to become generally wetter. As a result, some uncertainty surrounds the projected rainfall for Malawi, with some models projecting general rainfall increases and more extreme rainfall events, but with other models projecting the opposite.

In order to sensitise practitioners to the realities of climate change in future, and the impact on the planning and design phases of LVRs, particularly as regards long-term designs for bridges with 50 or 100-year design lives, various projections relating to temperature and rainfall are presented in Figures 3-5 to 3-8.

Figure 3-5 shows the expected 90th, 50th and 10th percentiles of the predicted temperature changes for the period 2071 – 2100 relative to 1961-1990 based on downscaled analyses for Africa with the detail for Malawi enlarged on the right. Increases in temperature (up to 4.5°C) are expected in the central west areas with slightly lower increases in the remainder of the country.

In terms of the rainfall (Figure 3-6), there is a 90 per cent chance that the average annual rainfall will increase over the entire country by up to about 80 mm and a 10 per cent chance that it will decrease by about 80 mm over most of the country. There is also likely to be a small increase in the number of extreme rainfall events causing flash floods (Figure 3-7). Extreme rainfall events are defined as 20 mm of rain occurring within 24 hours over an area of 0.5 degrees longitude by 0.5 degrees latitude (about 2500 km2).

Precise assessments of other climate change indicators (e.g. wind) in Africa are not yet complete and are often limited to mean temperature and precipitation. However, with drier and hotter conditions, increased convection currents can be expected in the air, with higher wind speeds being common.

Figure 3-8 provides the projected changes for the average value of the Keetch-Byram drought index. This shows a small increase over most of Malawi, with a maximum in the central west areas.

Together with these changes, significant increases in the number of very hot days (> 35°C) can be expected.

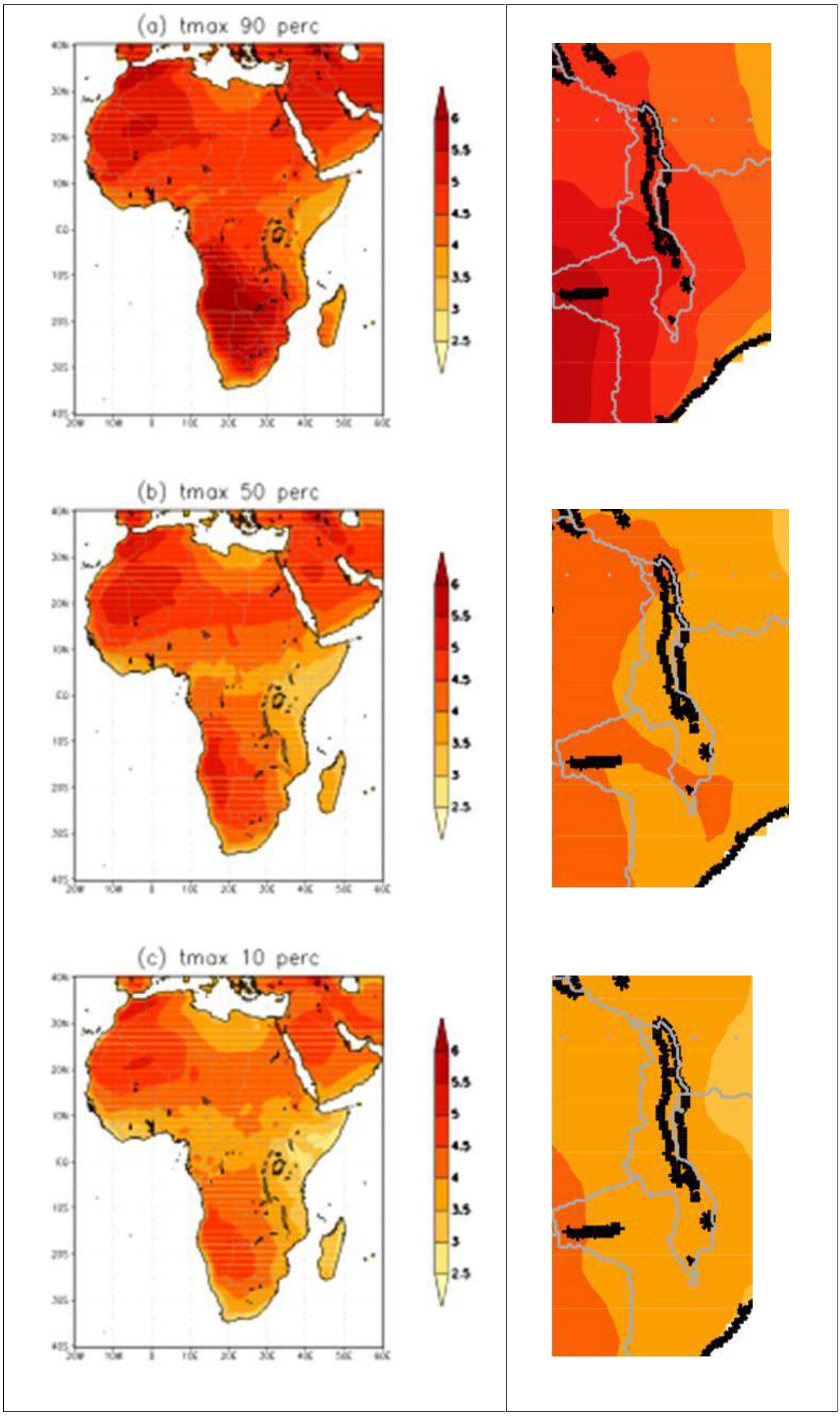


Figure 3-5: Predicted changes in temperature for the period 2071-2100 relative to 1961- 1990 under a low mitigation scenario

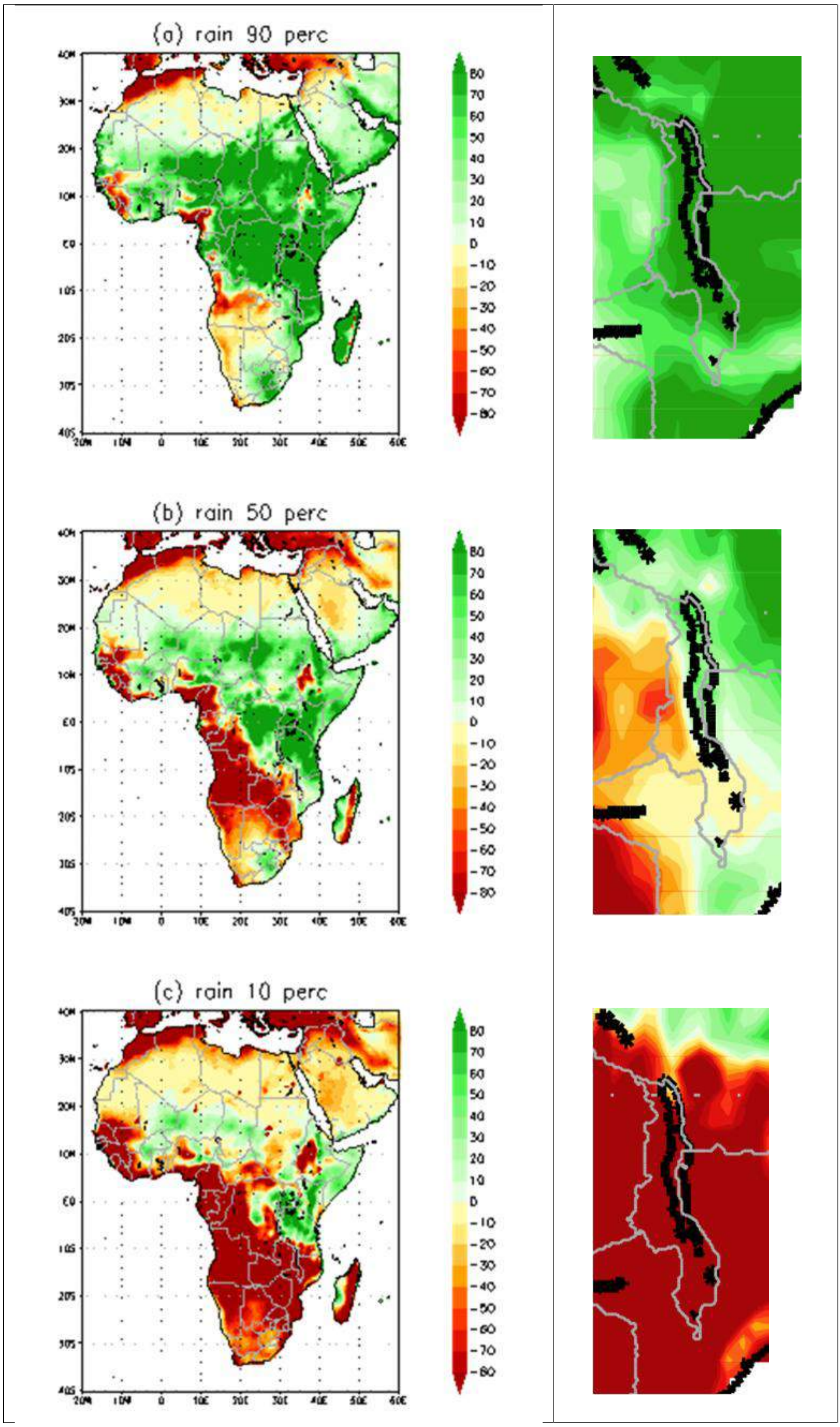


Figure 3-6: Predicted changes in rainfall for the period 2071-2100 relative to 1961- 1990 under a low mitigation scenario

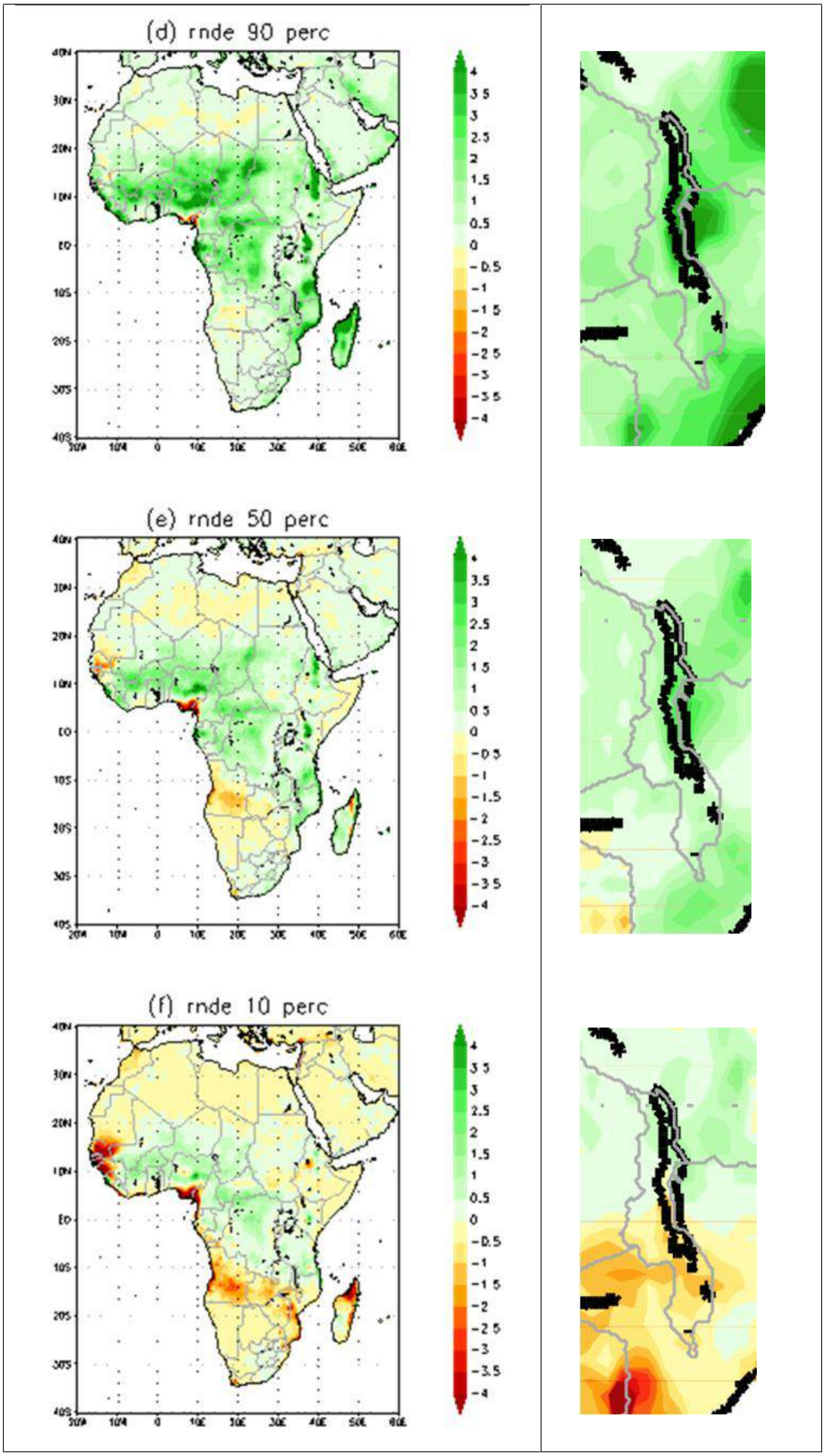


Figure 3-7: Predicted changes in number of extreme rainfall events for the period 2071-2100 relative to 1961- 1990 under a low mitigation scenario

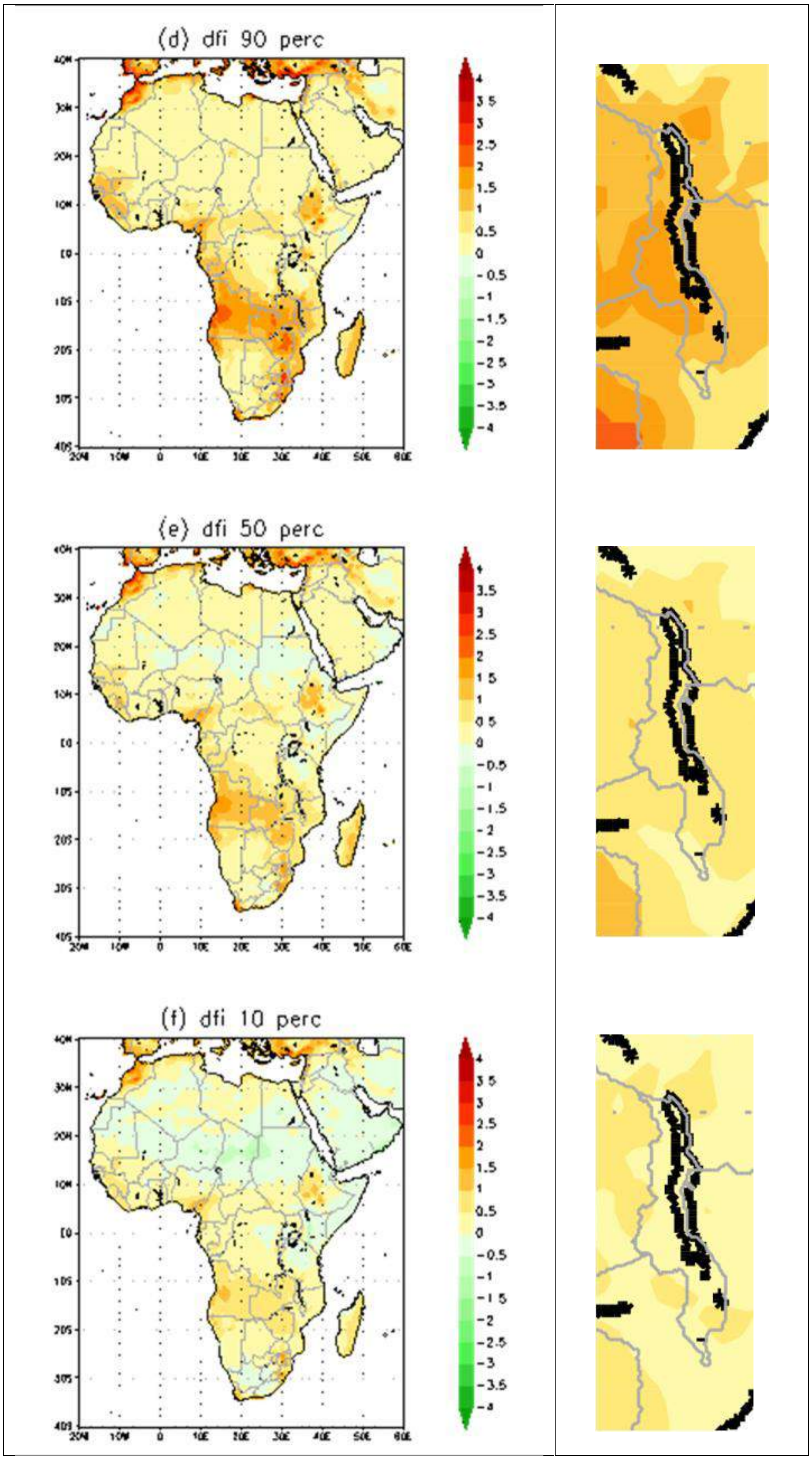


Figure 3-8: Predicted changes in the Keetch Byram Drought Index for the period 2071-2100 relative to 1961- 1990 under a low mitigation scenario

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Section 2

Investigations

Low Volume Roads Manual
Volume 1 – Pavement Design

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  - 4. Site Investigations
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4.1 Introduction

4.1.1 Background

Site investigation and surveys are a vital and integral part of selecting the road alignment and undertaking the design and construction of the road. Surveys provide essential information on the characteristics of the soils along the possible alignments, the hydrology of the area, the availability and properties of construction materials as well as the topography, land use, environmental and socio-political issues. Survey information is required to:

- Assess the condition, level of accessibility and inventory details of any existing road or track.
- Select the route/alignment of the road.
- Identify the best location of water crossings and drainage structures.
- Provide information for the design of the road pavement, bridges and other structures.
- Identify any areas that might require a specialist geotechnical investigation.
- Identify areas of potentially problematic soils requiring additional investigation and treatment.
- Identify and assess suitable, locally available borrow and construction material.

All projects are unique, and it is imperative that all factors that can affect the design of a LVR are investigated to the required level of detail. However, not all road projects would require the same level of surveys as, for example, in the case of the following:

- A completely new road;
- A new road that follows the general alignment of an existing track or trail;
- Upgrading a lower class of road to a higher class; or
- Rehabilitation or improvement of an existing road including spot improvements (to the original service level).

Site investigations for an entirely new road, greenfield sites, are very comprehensive because none, or very little, information will be available beforehand and collecting it usually requires a range of skills. In contrast, for brownfield sites, such as rehabilitation or improvement and upgrading an existing road to an all-weather access standard, the required site investigations are considerably simpler because much of the information is already available. For LVRs in Malawi, it is very unlikely that an entirely new road will be needed where no existing track or road already exists. However, survey techniques described are also applicable to greenfield sites.

4.1.2 Purpose and Scope

The purpose of this chapter is to provide guidance on the appropriate type and level of site investigations that are required for design of LVRs. It focusses on 'engineering' or, more precisely, 'geotechnical engineering' investigations. However, other information, as described below, is also required from what can generically be described as site surveys.

- Community consultation surveys on the details of the project.
- Hydrological surveys are required to estimate the water flows that determine the drainage design of the road and the design of water crossings (refer to *Chapter 7 – Hydrology and Drainage Structures*).
- Traffic surveys are required to estimate future traffic (including non-motorised road users, number and type of vehicles and traffic loading) that will use the road over its design life (refer to *Chapter 6 - Traffic*).
- Surveys to evaluate environmental impact and how to control them.

This chapter provides practitioners with the necessary tools to develop suitable site investigation programmes and to identify the need for, and approach to, more detailed geotechnical investigations. The chapter focuses primarily on-site investigations for existing roads (brownfield projects) and not on new roads, as it is unlikely that entirely new District or Tertiary roads will be constructed on completely new alignments.

It is not the purpose of the chapter to describe individual site investigation techniques in detail. Where additional information on the type, use and interpretation of site investigation techniques is needed, the reader should refer to appropriate internationally recognized standards.

4.2 Preliminary Site Investigations

4.2.1 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.

4.2.2 Types and Scope of Site Investigations

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

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

For LVRs, investigations should employ relatively 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 simple material testing procedures to assess the grading and plasticity of in-situ soils and borrow materials. More sophisticated 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 frequency and type of testing necessary for the specific road project and to assess when samples should be taken for laboratory testing in accordance with the appropriate standard.

New roads

As indicated above, it is not likely that entirely new, District or Tertiary roads will be constructed on new alignments. However, short sections of existing roads may be re-aligned for various reasons. For such “greenfield” sections, more emphasis should be placed on 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 to a paved road, route selection is not required, although some minor realignments may be required. For such a situation, standard inventory and condition data collection, in addition to subgrade materials testing, as described in Section 4.4 below, will be essential to inform the design as described in *Chapter 9 – Structural Design: Paved Roads* and *Chapter 10 – Structural Design: Unpaved Roads.*

4.2.3 Desk Study and Initial Survey

The desk study and initial survey should cover all of the aspects of site investigation required in the detailed survey, but only to the extent required to plan the detailed survey that will provide all the information required for design.

Desk study

Desk studies are much less expensive than site investigations, therefore, by making use of existing information, the project can (at the very least) be improved, the cost of new site investigations reduced, and the effectiveness and efficiency of carrying out the required new site investigations can be considerably enhanced. However, care is required to ensure that any existing data are reliable. In particular, old data might be out of date (e.g. traffic data) or incomplete (e.g. hydrological data).

Sources of information typically include:

- Available historical data from previous construction and maintenance activities may be available. These should be collected for review. Any sections of poor alignment and potential accident black spots should be identified for attention in the design;
- Aerial photographs and satellite images (e.g. Google Earth). These provide a very useful source of information, including road environment factors such as the alignment of the road, catchment areas, drainage patterns, low-lying areas, locations of settlements, land use etc.;
- Previously collected information on the location and variety of materials used in constructing the gravel road;
- Geological maps;
- Topographical maps;
- Social/economic reports;
- Population census data; and
- Climatic data and climate change projections.

The scope and level of detail of desk studies obviously depends on the type of project, the type of information under consideration (e.g. geotechnical, hydrological, traffic, environmental, social) and the amount of information that is available.

Initial survey

An initial site visit and assessment is essential for planning a cost-effective detail survey.

During the initial survey, consultations with the local community are essential for a variety of reasons, particularly with regard to levels and frequency of flooding and, in flat terrain, the extent and direction of flood flows. They may also be able to provide information about local materials and labour/skills resources and any seasonal accessibility problems.

Following an existing road or track as the basic route, potential problems would become readily apparent, for example:

- Inadequate water crossings
- Poor or dangerous alignment
- Problem subgrades
- Areas likely to flood
- Possible slope instabilities

Furthermore, the sources of materials for the existing road may still be available and there are unlikely to be any major problems of land use. However, encroachment into the road reserve, often affecting drainage, is common and it is good practice to request local authorities to warn farmers not to cultivate any land within the road reserve at least a year in advance of the roadworks.

Minor realignments may also be necessary and thorough site investigations are essential to obtain all data that are required for a professional engineering project.

The activities typically associated with the desk study and initial survey are listed in Table 4-1.

Table 4-1: Activities for desk study and initial survey

Stage of Design	Aspect	Activity
Desk Study	Technical	Review of topographical, geological, soils mapping including satellite imagery. Preliminary review of alignment and identification of potential problems (wet areas, steep slopes).
	Socio-economic	Review of demographic data, land use data, environmental data.
	Financial	Review of historic data for roads of similar type in similar terrain. Rough cost estimation per km.
Initial survey	Technical	Drive-through to ascertain overall accessibility, terrain, drainage.

4.2.4 Site Visit for General Assessment

The site visit for undertaking a general assessment of the project road typically consists of the activities discussed below.

Consultations with local communities

For the successful implementation of LVR projects, it is vital that the communities that are directly affected by the project are involved at an early stage in the planning process to ensure that their needs are catered for in the final design and to engender a sense of community ownership, which is likely to affect the project implementation and future road maintenance in a positive way. Apart from providing information related to the design of the road, the communities should also have a say on the choice of technology, i.e. the use of labour-based or capital-intensive methods of construction. Such consultations are best undertaken through a series of meetings with a representative group of key stakeholders within which there is a good gender balance since women's and children's perception of access problems is often different from men's perception. Comprehensive community consultations are thus a vital part of the site investigations and must be carried out in conjunction with the technical surveys.

Visual assessment of road condition

The nature of the structural survey necessary for full engineering design depends on the condition of the existing road pavement. The condition assessment, which is part of the initial and detailed assessments, forms the foundation of the rehabilitation investigation and design process. The aim is to determine uniform pavement sections that are similar in condition and have similar structural or functional improvement requirements.

A visual assessment of the road is required to determine its general condition. The visual survey identifies any weak areas and isolated failures that require rectifying before the pavement layer(s) and surfacing are constructed. The following defects should be noted along the length of the road for inclusion on a strip map as indicated below:

- Rutting.
- Shear deformation.
- Potholes (structural and not surface).
- Oversize material (if the road is gravel surfaced).

It is important to distinguish between those defects caused by inadequate structural capacity of the existing pavement, if any, and those caused by poor drainage, particularly in the shoulders or outer wheel-path. Whereas the former will probably require increasing the structural capacity of the existing pavement, for example, by importing one or more new pavement layers, the latter defects could be rectified by improving the drainage without importing new layers. A spot improvement approach where isolated problem areas are rectified individually rather than taking them as representative of the section as a whole is often adopted based on the severity and extent of the problem areas. This requires that a DCP survey be carried out in a discriminating manner.

Drainage and erosion

On existing roads, the road pavements and other infrastructure such as drainage systems will have been in place for some time. It is important to ensure that the drainage system is functioning well and can be expected to be adequate for future climatic conditions. As the upgrading of major types of drainage structures, such as bridges and large culverts, is generally expensive, existing infrastructure should be used as much as possible. Removing, altering or adjusting existing infrastructure will add costs to project. Where required, however, additional longitudinal and cross drainage infrastructure should be provided to ensure effective drainage of the road which critically affects its performance and, ultimately, its life. The initial survey should identify the problem areas that require more detailed analysis.

Geometric design and road safety

Geometric characteristics of the road, in terms of its horizontal and vertical alignment, will normally be retained for the upgraded road with only small improvements unless the design speed is dramatically increased. Nonetheless, any hazardous locations or obvious geometric shortcomings, particularly as they affect road condition (access) and safety, such as steep gradients or sharp bends combined with poor sight distance, livestock crossing locations, should be noted for possible improvement including appropriate measures for producing a safer road environment.

In general, full-scale topographic surveys (e.g. road corridor surveys) may not be necessary to carry out the geometric design of LVR road improvements or upgrading assessments. In many cases, depending on the type of project, simple topographic surveys can be achieved with the use of a handheld GPS device which is sufficiently accurate for preparing the line diagram and cross-referencing road inventory and road works. Where drainage may be a problem, for example, at low-lying points on the road, cross-sections will be required along the road alignment, downstream of the structures or crossing and through the riverbed using topographic survey instruments.

Materials

An assessment must be made of the source and availability of all materials required to upgrade the road including the surfacing, pavement layers, structural concrete and water for construction, as well as the cost implications. Every effort should be made to obtain materials that are as close as possible to the road alignment to reduce haulage costs.

Traffic assessment

Although collection and analysis of traffic data is an integral part of the site investigations, the subject is too broad to be dealt with in detail in this chapter. The reader is referred to Chapter 6 for guidance on the traffic assessment for design purposes.

Climate

Characteristics of the climate, such as historical annual rainfall data, should be obtained, if available. The rainfall data required partly depends on the method to be used for designing the drainage (see *Chapter 7 – Hydrology and Drainage Structures*). The projected future climatic conditions and expected changes should be analysed, based on the best modelling available in the area.

Hydrological data

Hydrological data is necessary to design water crossings or to improve them, particularly if there is visual evidence that their capacity is insufficient. Such data will also provide valuable information on the moisture regime in which the road will operate. This information will alert the designer to the potential sources of moisture infiltration into the road pavement and the measures that should be taken to mitigate such entry.

Table 4-2 summarises the activities to be considered during the site visit, while Section 4-3 provides more detail on the activities required.

Table 4-2: Summary of activities for the site visit

Aspect	Activity
Capturing local knowledge	Consultation with local residents on social and economic conditions in project area, climatic data, historical flood vents, sources of construction material, conditions of various sections of the road, common livestock crossing sections, etc.
Technical	Visual inspection to: • 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 Drainage (catchment) areas of main river systems and extent of flooding of water crossings and low-lying areas. • Location of all possible bridge sites and water crossings requiring more than a small culvert. • Identification of slope stability and potential landslide problems. • Identification of other possible major hazard areas such as poorly drained soils, problem soils, springs, and erosion in river courses. • Assessment of Extent of erosion problems with road drainage requirements. • Identification of possible sources of water for construction. • Identification of possible sources of construction materials. • Assessment of land acquisition/site clearance problems. • Assessment of Traffic (observation).
Environmental	Many common environmental issues associated with major roads are unlikely to be significant for LVRs, but 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. However, Malawian laws and regulations must be complied with. See *Chapter 13 – Practical Considerations*.
Financial	Cost estimation can be refined with more available information on the site conditions.

4.3 Detailed Site Investigations

4.3.1 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 a LVR project, these activities may also be combined in one site investigation. The sections below describe the activities to be carried out.

4.3.2 Feasibility and Detailed Design Study

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

a) Structural assessment

The site investigation should establish the condition of the existing pavement structure in order to maximise its use for the new pavement structure so that it can carry the expected future traffic. The DCP provides an efficient and inexpensive way to examine structural properties of each layer. How to undertake a DCP survey is described in Section 4.4.2.

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 existing structure to cope with floods
  - Structural condition
  - Width
  - Erosion
- Bridges (if any)
- Surface drainage:
  - Standing water due to rutting, etc.
- Drainage channels:
  - Adequacy of side drains (shape of drain, ponding, silting, scour, erosion)
  - Catchwater drains and cut-off drains (shape of drain, ponding, silting, erosion)
  - Mitre drains (frequency, shape of drain, ponding, silting, erosion)
- Down chutes (condition, erosion)

Erosion is closely related to drainage and depends on soil type, grade, climate and site conditions and is a function of the volume and velocity of water in the channel. A general assessment of erosion potential is needed for embankments, cuttings, road reserve and borrow areas, leading to design of anti-erosion 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 specifications (refer to *Chapter 5 - Materials*).

4.4 Site Investigation Methods

4.4.1 General

The engineering design requires sufficient design data for preparation of the tender and draft contract documents. The quality and level of the site investigations should not be compromised to provide cost savings, nor should the level of investigation be necessarily reduced simply because the road is classified as an LVR. This Chapter describes and summarises the principal methods available.

4.4.2 Characterisation of Subgrade and In-situ Materials

DCP survey

General - All the design methods described in this manual makes use of the DCP for characterising the strength of the subgrade or existing pavement layers. The strength of using the DCP to characterise the pavement (or virgin ground for greenfield projects), is that the DCP “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 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.

DCP survey for new roads – The design of a new road where none existed before is strongly dependent on the characteristics of the subgrade and, therefore, its potential performance. A good subgrade must be strong enough to resist shear failure and must have adequate stiffness to minimize vertical deflection. The stronger the subgrade, the thinner overlying pavement needs to be. The designer usually has very little choice about the subgrade along most of the road alignment. It is therefore essential that its characteristics are determined in some detail. In situations where the subgrade materials are unsuitable, either cost effective methods of improving the existing conditions must be identified (e.g. improving drainage or stabilisation) or the road alignment must be altered to avoid such areas completely.

Figure 4-1: Team of three persons required for DCP testing

DCP survey on existing roads – For upgrading an existing road, it is equally important to determine the characteristics of the existing pavement layers because these will be utilised in the new pavement structure. An adequate structural survey of the existing road is therefore essential. The most cost-effective method for obtaining sub-surface information to a depth of approximately 800 mm is by using the DCP.

The DCP is light and portable and DCP tests are quick and simple to carry out. The advantage of the DCP is that information can be gathered without disturbing the in-situ material. Using this test, the strength characteristics and thickness of the subsurface materials at field moisture and density conditions can be obtained directly. The DCP is also useful for quality control testing during construction.

The DCP survey must be carried out along the full length of the road with each measurement being taken to a depth of at least 800 mm.

The frequency of the DCP measurements depends on the variability in road conditions and level of confidence required. Where obvious changes of 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 4-3.

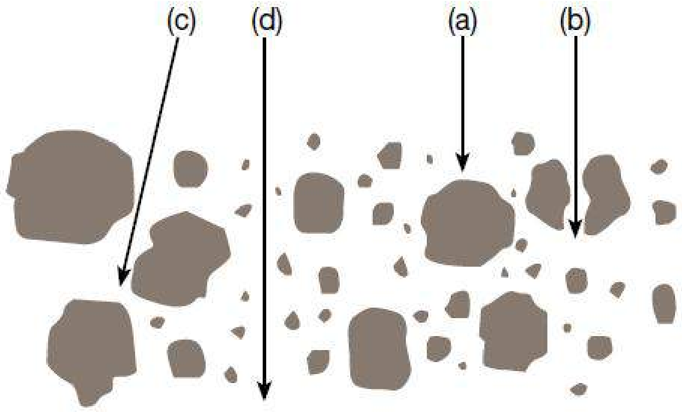
Table 4-3: Frequency of DCP testing

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

Because the DCP survey is so quick and easy to carry out, it is in most cases better to increase the frequency to, say, one test every 100 m, in the first instance rather than having to go back into the field later to do additional tests. However, the initial analysis of the DCP results may identify problematic sections that require re-testing at closer intervals of, say, 25 m - 50 m intervals.

The tests should be staggered across width of the road at outer wheel-tracks (left and right) and the centre line. 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 10 tests should be taken in each uniform section hence additional tests may be required after analysing the first set.

Care must be exercised in carrying out the DCP survey by discarding any measurements which could produce anomalous results. Such results could arise, for example, where large stones occur in the pavement layer as shown in Figure 4-2. However, in these cases the in situ strength is likely to be considerably higher than similar materials without the oversize material, and will usually provide strong subgrades.



(a) Cone cannot penetrate.	(b) Cone breaks 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 low reading.	(d) Normal result.

Figure 4-2: Effects on the DCP measurement when large stones are present

Interpretation and analysis of DCP data

The analysis of the DCP test results is done automatically in the respective DCP software programs (AFCAP LVR DCP programme for the DCP-DN method and UK DCP programme for the DCP-SN/CBR method).

However, the interpretation of the data and variations caused by, for instance weak or hard interlayers or a sudden drop in strength or increase in DN values, cannot be automated and may require further investigations and good engineering judgement on how to deal with the localised 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 while the latter can often be remedied simply by ensuring that the level of the pavement is raised, and that proper drains are constructed.

For the analysis of the DCP test results with any of the design methods, it is therefore good practice to exclude "outliers", i.e. very weak or very strong points in order not to distort the determination of representative values within the uniform sections. Weak points should 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" as described above, the DCP data is used to determine uniform sections using the CUSUM method described in *Chapter 9 - Structural Design: Paved Roads*. The different design methods use different parameters for the analysis, but the basic procedure remains the same for all.

Applicability of the DCP for site investigations

For the DCP-based design methods described in this Manual, the DCP is used to characterise the subgrade. It should be noted, however, that the use of the DCP for determination of subgrade strength would not be appropriate if the proposed road is in deep cut or high fill, where the final formation level (selected subgrade or capping layer) is either below or above the currently exposed soil surface.

In the case of elevated roads, the material to be used for the embankment would need to be tested to determine its properties and strength (DN or CBR) at varying densities and moisture contents. Fills can then be designed in accordance with the relevant catalogue to ensure that all the layers comply with the specifications of the respective design method. This will allow designers to go straight to design catalogues for contractual quantities. In general, this applies when the formation needs to be raised by more than approximately 0.5 m above the existing ground, e.g. on severely sunken sections that need to be raised to achieve adequate drainage, on embanked sections traversing swampy areas or when the vertical alignment needs to be improved.

In the case of deep cuts, the properties and strength of the formation level material can be determined either from test pits or, if not feasible, after cutting to level. This may give rise to contractual problems which should be provided for with appropriate clauses and pay items in the contract documentation.

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

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 for characterising 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 will give a good indication of the subsurface conditions and variability with depth of the subgrade.

The determination of uniform sections, as described in Chapter 9, limits the variability of the subgrade. For gravel or earth roads in 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 to a depth of 450 mm below the surface, are deemed to be sufficient for determination of the design subgrade strength (keeping in mind that the in-situ moisture content normally increases, resulting in higher DN values with depth in uniform subgrade material). Engineering judgement must be used to locate them 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 changed, deeper test pits may be required. These will also be useful for investigation of the water table level.

The location of each test pit should be precisely determined on the route alignment and all layers, including topsoil, should be accurately described and their thicknesses measured. All horizons below the topsoil should be sampled. This will also provide a proper assessment of the materials excavated in cuts that are to be used in embankments. The samples should be taken over the full depth of the layer by taking vertical slices of materials.

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 pavement layers. This is especially true in areas where a thick layer of problem soils and 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 remaining gravel wearing course on the road of, say, less than 75 mm thickness, if any at all, to the strength of the in-situ pavement and thus to the measured in-situ DN values, can be disregarded and should not be mixed with the subgrade samples. This gravel 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 and thus give 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 well be augmented by 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 Central Materials Laboratory have long-standing experience and performance data on specific soil types in the local climate and topographic conditions. Use of this information can supplement and reduce (but not replace) the overall requirement for subgrade evaluation. The approach involves the assessment of subgrades based on local geology, topography and drainage, together with regular routine soil classification tests.

The subgrade design strength is determined through the laboratory DN test of the subgrade, taking care not to include the gravel wearing course. If there are two (or more) distinctly different subgrade material types as indicated in Figure 4-3, both layers must be tested separately.

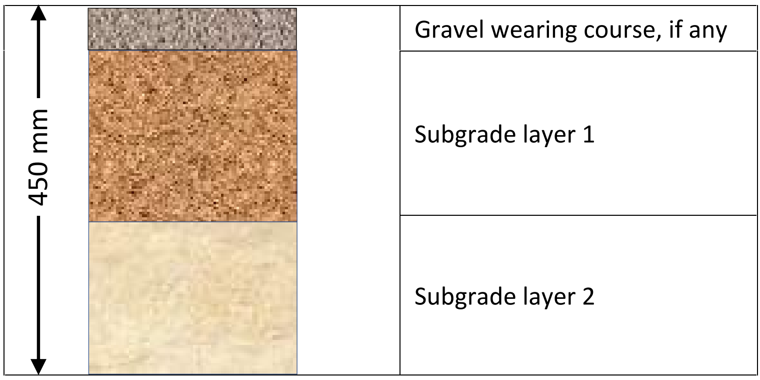


Figure 4-3: Typical soil profile in test pit

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 4-4.

Table 4-4: Standard laboratory testing of test pit samples

Laboratory testing of test pit samples	DCP-DN Test Methods	DCP-CBR / SN Test Methods
Soil Profile: - Overburden - Layer / horizon thickness - Visual description - In-situ moisture content - In-situ density	 SANS 2001 – GR20 SANS 3001 – NG5	 BS 1377 – 4 BS 1377 – 4
Index tests	SANS 3001 – GR1, GR12	BS 1377 – 2
Compaction (Density/Moisture relationship)	SANS 3001 – GR30	BS 1377 – 4
Strength	Laboratory DN	CBR & Swell (BS 1377-4)

Figure 4-4: Excavation of test pit by labour

Figure 4-5: Logging of subgrade layers

The soil profile shall be described, and index and compaction tests be carried out for the material from each test pit to ascertain the representative nature of the materials. Then, the bulk samples shall be mixed for determination of the subgrade design strength, as described in *Chapter 9 – Structural Design: Paved Roads,* Table 9-2.

4.4.3 Identification and Treatment of Problem Soils

General

Soils which can cause foundation problems and decrease the performance of roads are common. These soils are collectively called problem soils and comprise:

- Expansive soils
- Dispersive soils
- Saline soils
- Micaceous soils
- Low-strength soils
- Collapsible soils

The preliminary identification of such soils is crucial during the site investigation so that appropriate additional investigations can be included prior to the final designs being developed. Those areas with particularly problematic soils often require specialist investigations and testing. Failure to recognise problem soils at the design stage could result in claims and cost overruns if identified later during construction or have a detrimental impact on the long-term performance of the road.

In assessing the appropriateness of the measures available for dealing with the above problem soils, a careful balance has to be struck between the cost of the measures and the benefits to be derived. This would normally require that a life-cycle analysis be carried out to determine whether the costs of the measures would be at least off-set by the benefits (see *Chapter 12 – Life Cycle Costing*).

Expansive soils

Causes – Expansive soils are those containing smectite (montmorillonite) clays, which are mostly derived from the chemical weathering of basic rock forming minerals. Probably the worst expansive clays occur on deeply weathered gabbros, basalts and dolerites in tropical and sub-tropical areas. Expansive clays are also commonly found in transported soils derived locally or from some distance from weathered basic igneous rocks. Smectites can also form from the alteration under alkaline conditions of other silicate minerals low in potassium, as long as calcium and magnesium are present, and leaching is impeded. Although the expansive potential of a soil can be related to many factors, it is primarily controlled by the quantity and type of clay minerals (e.g. smectites). In Malawi, where vertisols occur they are likely to be associated with the occurrence of expansive soils, such as in the Badlands areas of the country as shown in Figure 3-2 in *Chapter 3 – Physical Environment.*

Recognition – Expansive soils are those which exhibit particularly large volumetric changes (swell and shrinkage) following variations in moisture contents. The mechanism of expansion is that of seasonal wetting and drying caused by fluctuating movement of the water table. Soils at the edge of the road wet up and dry out at a different rate to those under a paved surface, thus bringing about differential movement. It is this movement rather than the low soil strength which brings about failure. Such failure typically takes the form of associated longitudinal crack development, occurring first in the shoulder area and developing subsequently in the carriageway, as well as general unevenness of the pavement surface, arcuate cracking and settlement near trees and transverse humps and cracks at culvert sites.

Volume changes in expansive soils are confined to the upper few metres of a soil deposit where seasonal moisture content varies due to drying and wetting cycles. The zone within which volume changes are most likely to occur is defined as the active zone. The active zone can be evaluated by plotting the in-situ moisture content with depth for samples taken during the wet and dry seasons. The depth at which the moisture content shows no seasonal variation is the limit of the active zone. This is also referred to as the depth of seasonal moisture change.

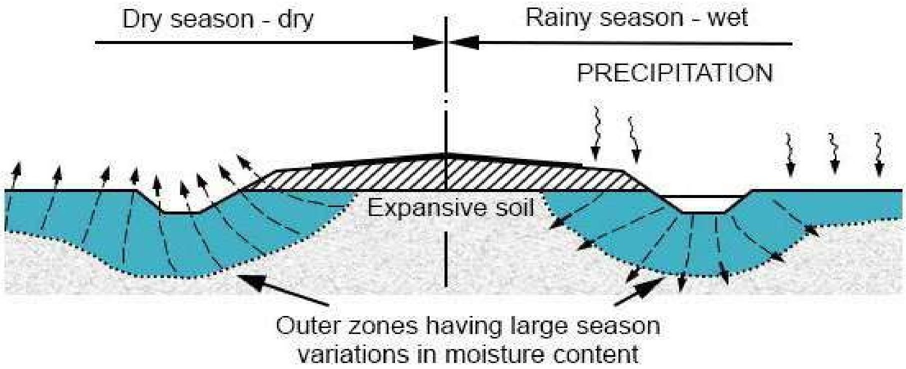
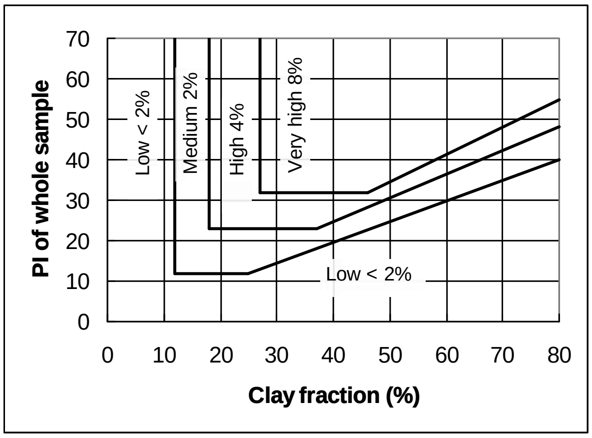


Figure 4-6: Moisture movements in expansive soils under a paved road

The simplest way of identifying the presence of expansive soils is through field observations where the surface expression of cracking in dark grey, black or sometimes red soils is evident as shown in Figure 4-7. However, the presence of a thick non-expansive transported or topsoil cover can sometimes mask these cracks and excavation of a test pit, in which cracking and slickensiding of the material will be observed, is necessary. The identification of smectite in subgrade soils is best done using X-ray diffraction.

Figure 4-7: Cracking on expansive clays

By their nature, smectites will tend to be more plastic than other clay minerals and a measure of the plasticity index, or better still the activity (ratio of plasticity index to clay fraction) is a good indication of the presence of smectites. One of the earliest methods of indicating potentially expansive soils is shown in Figure 4-8, based on the clay fraction of the soil (minus 2 µm) and the standard Plasticity Index (PI), which remains very useful for the preliminary identification of expansive soils. It should be noted that the estimates for the degree of swell using this technique do not take into account the initial moisture content of the material, assuming that they move from a state of dryness normally used in the laboratory to wet. It is known that an equilibrium moisture content develops under a road structure and the moisture fluctuation in this zone is minimal. However, from beneath the outer wheel track of roads with unsealed shoulders to the edge of the fill, significant and variable moisture fluctuations occur. It is unlikely that the initial moisture content in these zones is, however, particularly dry.



Source: Van der Merwe, 1976

Figure 4-8: Identification of potentially expansive clay

An indication of potentially expansive soils can also be obtained from land type soil maps where materials identified as "vertic" soils will always have expansive characteristics, while soils with a high base status (or eutrophic) and clay content should be investigated more thoroughly, as they have the potential to be expansive.

Countermeasures – Although the estimation of potential heave is imperative for structures on expansive clay, it is not as critical for subgrades under roads. It is more important to identify the possible existence of the problem and the potential for differential heave along the road and take the necessary precautions. These will generally be based on the expected degree of swell determined from Figure 4-8.

If the calculated potential heave exceeds 25 mm to 50 mm countermeasures should be installed. If there is likely to be significant differential movement as a result of variable material properties or thicknesses, changing loading conditions or localised drainage differences, the countermeasures will need to take this into account to avoid localised sections of road with poor riding quality.

Where culverts or small bridge structures are involved, it is usually necessary to quantify the potential movement more accurately. This is best done using oedometer testing of specimens cut from block samples. Correct orientation of the block samples is imperative as expansive clays tend to be highly anisotropic with significantly lower swells in the horizontal direction. This testing needs to be carried out in conjunction with good estimates of the potential changes in in situ moisture content from season to season.

Solutions that can be considered for LVRs over expansive clays include:

1. Flattening of embankment side slopes (between 1V: 4H and 1V:6H).
2. Remove expansive soil and replace with inert material (between 0.6 m and 1 m depending on depth of clay).
3. Retain the road over the clay as an unpaved section.
4. Pre-wetting prior to construction of the fill or formation (to OMC).

5. Placing of uncompacted pioneer layers of sand, gravel or rockfill over the clay and wetting up, either naturally by precipitation or by irrigation (100 mm to 500 mm depending on clay thickness and potential swell).
6. Lime stabilization of the clay to change its properties (expensive – up to 6% lime may be required).
7. Blending of fine sand with the clay to change its activity (blend ratio to be determined by laboratory experimentation).
8. Sealing of shoulders (not less than 1 m wide).
9. Compaction of thin layers of lower plasticity clay over the expansive clay to isolate the underlying active clays from significant moisture changes.
10. Use of waterproofing membranes and/or vertical moisture barriers, which are generally geosynthetics (only limited success has been achieved using these methods.

Figure 4-9 provides a preliminary indication of possible counter-measure options (numbered as above) as a function of potential expansiveness. It should be noted that usually a combination of these is most effective and all should go together with careful design and construction of side-drains, which should preferably be sealed.

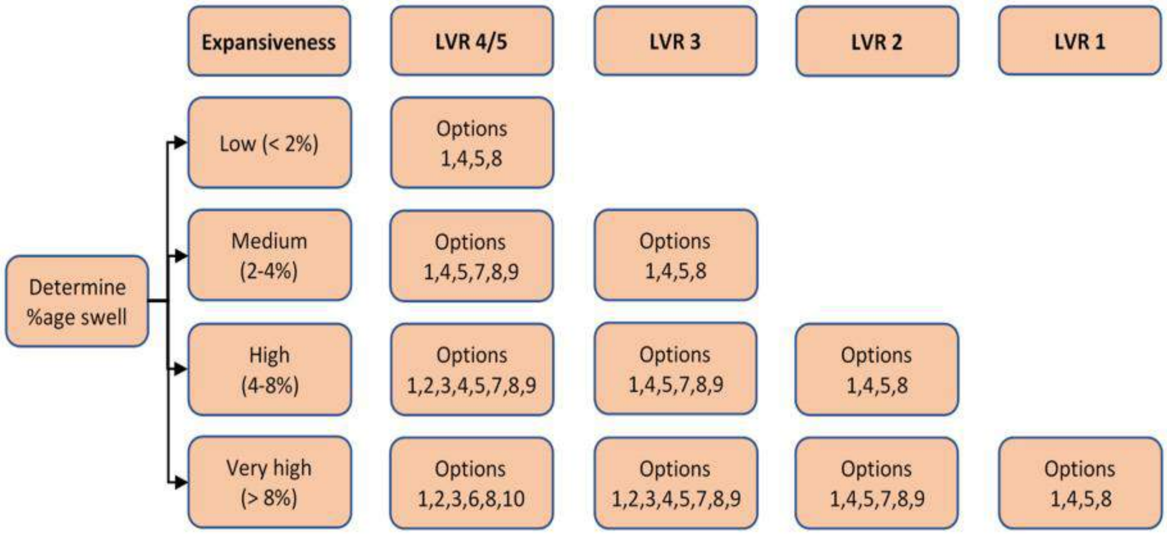


Figure 4-9: Possible solutions for the construction of roads on active clays

In many cases for LVR1 class roads, it may be more economic to retain the road as a gravel road over the expansive clay sections and apply the necessary maintenance.

One of the most important considerations is to try and minimise the zone of seasonal moisture movement beneath the road, as shown in Figure 4-10, and to increase the zone of moisture equilibrium. A combination of slope flattening, material replacement, sealed shoulders and lined side drains as shown in Figure 4-11 is usually the most cost-effective means of achieving this, but the design of counter-measures needs to be specific to any situation.

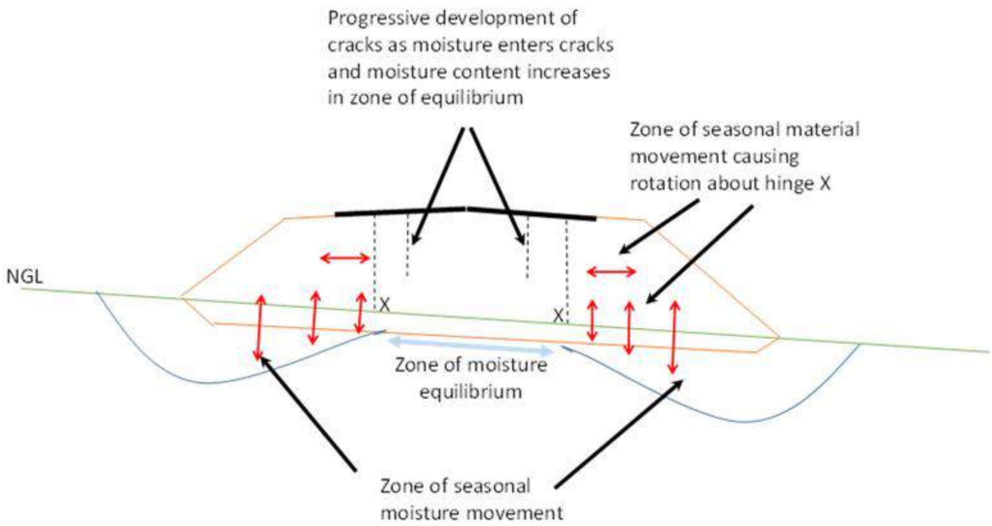


Figure 4-10: Typical moisture movement regime under roads on expansive clays

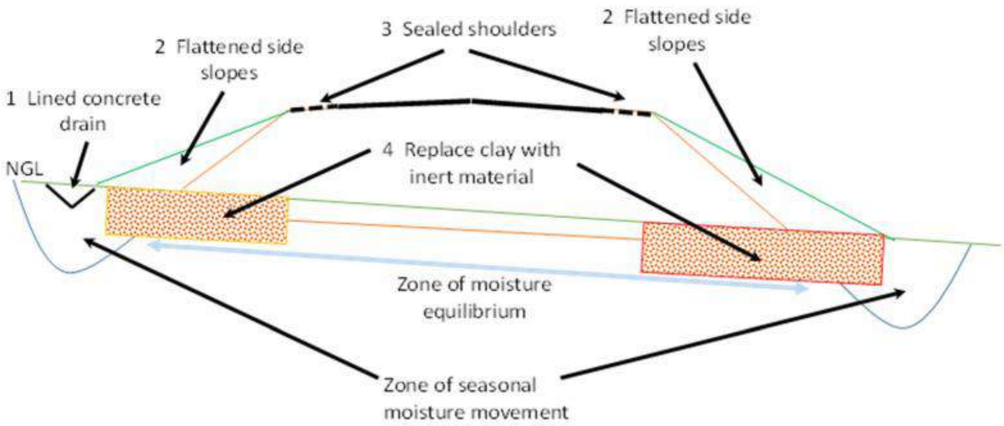


Figure 4-11: Some countermeasures used to increase zone of moisture equilibrium

Expansive clays are often thick and laterally widespread, and this makes the implementation of countermeasures costly. The most successful technique for counteracting subgrades susceptible to high movement is to remove the expansive clay beneath the road structure and replace it with a raft of inert material. This would typically involve the excavation and removal of between 600 mm and 1 500 mm (or even deeper in some cases) of material over the entire footprint of the road prism (or at least beneath shoulders and side slopes) combined with drainage structures that remove all water from adjacent to the fill slopes and culverts. Removal of material results in the reduction of the swell potential as well as slightly increasing the load on the expansive subgrade with a usually denser, better compacted material. Unfortunately, this is often impracticable or uneconomic for LVRs, unless the problem is localised. More frequently, expansive materials cover a wide area and the importation of substitute material involves the haulage of large quantities of inert material over long distances.

The recommended, and probably most economical solution specifically for LVRs showing high to very high potential swell, is to partially remove the clay from the subgrade and replace it with a less active material, increase the fill height using inactive material to provide a greater load on the clay, seal the shoulders of the road and flatten the fill slopes using the material removed from the subgrade and side drains. This has the effect of moving the zone of seasonal moisture fluctuation away from the pavement structure and inducing movements and cracking in the more flexible fill slopes rather than in the stiffer pavement structure.

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Particular attention should be paid to culverts. The clay beneath them must be replaced with an inert material, all joints must be carefully sealed to avoid leakage and inlets and outlets well graded to avoid ponding of water.

It is also good practice to remove and control the re-establishment of “water loving” trees. The roots of such trees seek water beneath the pavement and remove it from the clay, causing significant depressions in the road during the dry season, which may or may not recover in the wet season. This is usually associated with arcuate and/or longitudinal cracking.

Dispersive, erodible and slaking soils

Causes – Dispersive soils are those soils that, when placed in water, have repulsive forces between the clay particles that exceed the attractive forces. This results in the colloidal fraction going into suspension and in still water staying in suspension. In moving water, the dispersed particles are carried away. This obviously has serious implications in earth dam engineering but is of less consequence in road engineering except when used in fills. Dispersive soils often develop in low-lying areas with gently rolling topography and relatively flat slopes. Their environment of formation is also usually characterised by an annual rainfall of less than 850 mm.

Figure 4-12: Crumb test

Dispersive, erodible and slaking soils are similar in their field appearance (highly eroded, gullied and channelled exposures), but differ significantly in the mechanisms of their actions. Fortunately for road builders, only the (probably less common) dispersive soils present problems of any consequence. Figure 4-13 shows a typical dispersive soil with definite evidence of piping.

Figure 4-13: Piping in dispersive soils

Erodible soils will not necessarily disintegrate or go into dispersion in water. They tend to lose material as a result of the frictional drag of water flowing over the material that exceeds the cohesive forces holding the material together.

Figure 4-14: Gulley formation in erodible soil (Ntcheu District, Central Region)

Slaking soils disintegrate in water to silt, sand and gravel sized particles, without going into dispersion. The cause of this process is probably a combination of swelling of clay particles, the generation of high pore air pressures as water is drawn into the voids in the material and softening of any incipient cementation.

Slaking and erodible soils when occurring as subgrades or even when used in fills are unlikely to cause significant problems unless rapid flows of water through the fill or subgrade occur. Problems are thus mostly associated with poor culvert and drainage design. The inclusion of dispersive soils in the subgrade or fill on the other hand has been seen to lead to significant failures through piping, tunnelling and the formation of cavities in the structure. It is therefore important to identify dispersive soils timeously.

Recognition – The testing and recognition of dispersive soils requires various soil engineering and pedological laboratory tests. These include:

- Determination of the Exchangeable Sodium Percentage (ESP).
- Pinhole test.
- Cation Exchange Capacity (CEC).
- Crumb test.
- Double hydrometer test.
- Sodium Absorption Ratio (SAR) and the pH.

The crumb test on undisturbed lumps of material is usually the best first indication but is not always fool proof. Dispersive soils tend to produce a colloidal suspension or cloudiness over the crumb/lump during the test, without the material necessarily disintegrating fully. Disintegration of the crumb in slaking soils is very rapid and forms a heap of silt, sand and gravel. Erodible soils do not necessarily always disintegrate in the crumb test as they require a frictional force of moving water to loosen the surface material, without any of the loose material remaining in suspension.

Soils with a low sodium component have also been seen to be highly dispersive. These materials usually contain significant quantities of lepidolite (a purple lithium mica). Lithium is of course the most reactive metal in the alkali series (Li > Na > K > Mg, etc.) and this should be investigated where the sodium content is low but dispersion seems to be prevalent.

It is not very important (or even really possible) to quantify the actual potential loss of dispersive material from subgrades and fills as the process is time related and, given enough time, all of the colloidal material could theoretically be dispersed and removed, leading to piping, internal erosion and eventually loss of material on a large scale. It is, however, important to identify the presence of dispersive soils, and their differentiation from erodible and slaking materials, so that the necessary precautions can be taken if they affect the constructed pavement.

Countermeasures – The countermeasures for avoiding dispersive soil damage in the road environment are relatively simple:

- Avoid its use in fills as far as possible.
- Remove and replace it in the subgrade.
- Manage water flows and drainage in the area well.

As the presence of sodium as an exchange cation in the clays is the major problem, treatment with lime or gypsum will allow the calcium ions to replace the sodium ions and reduce the problem. The use of gypsum is recommended over lime as lime may lead to soil stabilisation with its associated cracking, allowing water to move through the cracks.

It is also important that the material is compacted at 2% to 3% above optimum moisture content to as high a density as possible.

To avoid problems with slaking and erodible soils, the drainage must be well controlled. Covering of the soils with non-erodible materials and careful bioengineering, assisted by geosynthetics where necessary, is usually effective. Once erosion has occurred, the channels and gullies should be back-filled with less erodible material and the water flows redirected.

Saline soils

Causes – Unlike dispersive soils that are affected by the presence of excessive cations of sodium attached to clays, saline materials are affected by the combination of specific cations and anions in the form of soluble salts, independent of clays. These can be a major problem on road projects where migration of soluble salts to beneath bituminous surfacings (Figure 4-15) leads to weakening of the upper base and blistering and disintegration of the surfacings. Soluble salts, particularly sulphates, and their acids can also have a serious detrimental effect on the stability/durability of chemically stabilized materials and concrete.

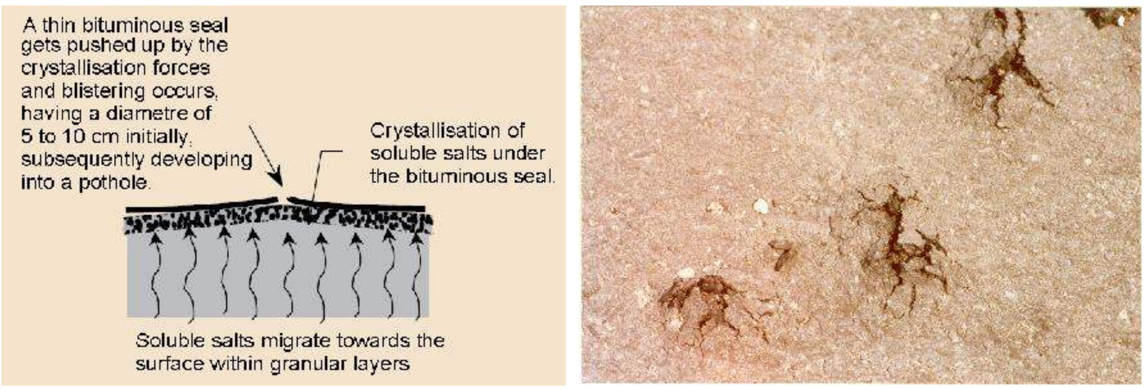

Figure 4-15: Mechanism of salt damage

Salts can originate from the in situ natural soils beneath the structures as well as from imported material for the pavement layers or from saline construction water. Only the presence of soluble salts in subgrade materials is considered in this report as the materials for other layers can be controlled provided the problem is identified timeously.

Subgrade materials in areas where the land surface shows some depression resulting in seasonal accumulation of water are particularly prone to the accumulation of salts leached from the surrounding areas. In other flat areas, capillary rise of groundwater and precipitation in saline soils can result in the upward migration of salts to or near the soil surface.

Recognition – In some cases, the visible presence of crystallised salt deposits at the soil surface is a certain indication of the need for additional investigation for possible salt problems. This is often associated with the presence of animals licking the soil surface. In most other cases, the presence of salt is best confirmed by using laboratory test methods.

Photo: Rolf D.Vogt, University of Oslo

Figure 4-16: Salt deposition on the soil surface in the Lower Shire region

In the conventional road engineering context, the identification of possible soluble salt problems is based on the pH and conductivity of the materials. It should be noted that the results of the electrical conductivity and pH tests can vary significantly depending on the pre-treatment, the moisture content at which the measurements are made and particularly on the material size fraction tested.

Limits for the use of saline materials are generally based on work in specific countries and their applicability to other areas is unknown. In general, an electrical conductivity on the passing 6.7 mm fraction in excess of 0.15 S/m (or an electrical resistance of less than 200 Ω on the minus 2 mm fraction) should raise concern and indicate the need for further investigation. Similarly, soluble salt contents in excess of 0.5% should be a cause for possible concern and lead to additional investigations.

In Malawi, saline soils are not very widespread. They generally occur in low lying areas of the medium to low rainfall regions. They are often associated with the Mopane vegetation. Some Dambo soils contain surface efflorescences of salt. Dambo soils are mostly widespread in the lower-lying swampy areas. Saline soils are typically more prevalent in the semi-arid climate in lower Shire valley area.

Countermeasures – The following measures should be considered:

- As soluble salt problems arise from the accumulation and crystallization of the salts under the road surfacing and in the upper base layer, minimisation of salts in the pavement layers and subgrade should be attempted.
- If the surfacing is sufficiently impermeable (coefficient of permeability, k in nanometre/ second)/ surfacing thickness, T in mm or k/T < 30 (µsec)-1) to avoid water vapour passing through it, crystallization will not occur beneath the surfacing.
- Construction should proceed as fast as possible to minimise the migration of salts through the layers. Only impermeable primes should be used, e.g. bitumen emulsions. Figure 4-17 provides an indication of the allowable delay between priming and sealing for various material subgrade salinity.
- The addition of lime to increase the pH to in excess of 10.0 will also suppress the solubility of the more soluble salts.

Even for the lowest classes of road (LVR1 and LVR2), the effects of excessively saline materials can lead to a rapid and total loss of the bituminous seal and precautions should thus be taken for all road classes. The use of non-bituminous surfacings should be considered over saline materials.

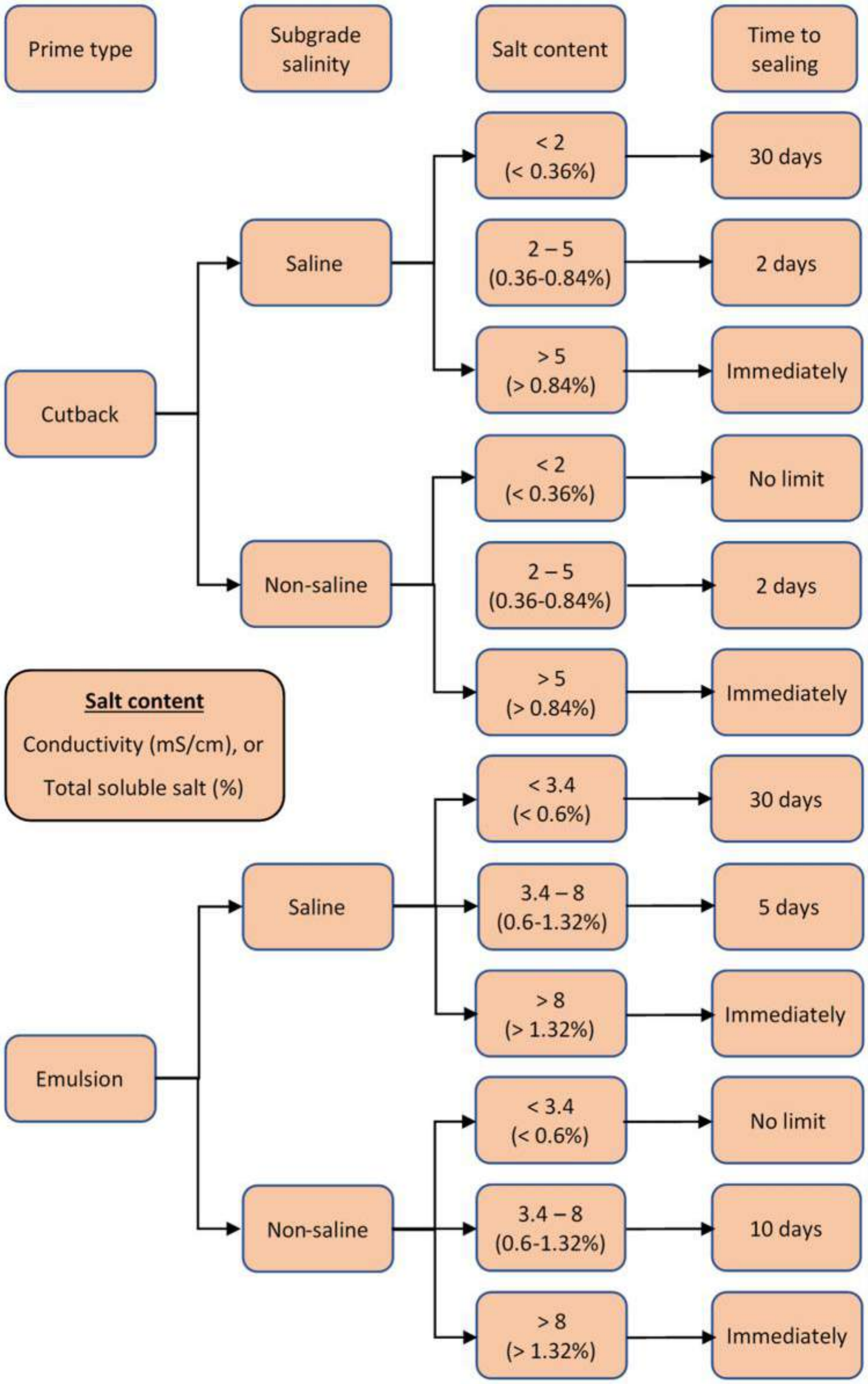

Figure 4-17: Permissible intervals between priming and surfacing

Micaceous soils

Causes – Micaceous soils contain large quantities of mica (muscovite and biotite) and occur in such materials as weathered granite, gneiss, mica schist and phylitte. Such soils belong to a group of minerals characterised by their extreme platy (flake-like) cleavage and can be distinguished by the presence of fine grains of mica which are flat and shiny. They are fine grained and contain about 50% of silt and clay. A characteristic of this soil is that the maximum CBR does not coincide with the optimum moisture content or maximum dry density.

Recognition – Micaceous soils often cause problems with compaction. The reason for this is because of the flaky, soft but strong, elastic, resilient nature of the mica particles. During compaction they are believed to act as small leaf springs, bending rather than breaking and then springing back once the compacting force is removed, resulting in a loss of density. Moreover, their flaky shape would also tend to increase the air voids content. This effect may be sufficiently rapid to be reflected as an inability to achieve compaction or may manifest itself later during service, causing rutting.

Figure 4-18: Micaceous soil with large flakes of mica (Chitipa District, Northern Region)

Other deleterious effects reported to be imparted by mica are that it:

- Reduces the apparent plasticity (i.e. that which is measured by the Atterberg limits), but increases the effective plasticity making materials weaker and difficult to compact;
- Can cause surfacing failure due to "sponging" of the base;
- Increases the PL more rapidly than the LL so that a negative PI might be obtained at high mica contents;
- Reduces the CBR and UCS;
- Increases the OMC;
- Causes large density gradients in the CBR mould.

There is no simple test to determine mica content. A good rule of thumb is – if the mica can readily be seen with the naked eye (platy, shiny particles within the soil mass) then it is most likely to behave as described above.

More quantitative approaches may be adopted for detecting and measuring mica content. They include macroscopic examination to determine the lithology and X-Ray Diffraction analysis to determine the quantity of mica present in the different fractions of the sample.

Countermeasures - Typical limits specified for the mica content of a soil is 2% free mica by mass or 4% free mica by volume for muscovite or biotite.

Other methods for dealing with micaceous soils include:

- Removing the micaceous soil layer to below the material depth in the subgrade;
- Stabilizing the micaceous soil with lime or cement with the qualification that, as a precaution, should mica be readily visible, then the soil should preferably not be stabilized.

Low strength soils

Causes – Widespread problems result from the presence of very soft alluvial clays in marshy areas. Soft clays are generally, but not necessarily, saturated and normally consolidated to lightly over-consolidated (as a result of fluctuating water tables). The materials thus have low shear strengths, are highly compressible and their low permeabilities result in time-related settlement problems. In addition, the frequent occurrence of organic material in the clays affects their behaviour and the determination of their properties.

The presence of these materials is associated with large mature river systems such as the Rukuru and Shire systems. The shear strength of these clays would normally be between 10 kPa and 40 kPa, making them impossible to or difficult to walk on. Soft clays are seldom uniform with depth and are usually interlayered with silts and sands, which provide more permeable drainage paths than would be determined from oedometer testing of undisturbed clay samples. However, the depths and strengths of the materials are such that inspection of the materials in test pits or auger holes is not recommended.

Recognition – The in-situ condition of these materials is one of their most important properties that need to be considered and testing of disturbed samples will usually provide results that are meaningless. It is thus better to use in situ test methods such as Standard Penetration Testing (SPT), vane shear or Cone Penetration Testing (CPT) to determine the depths, presence of silt or sand layers, strengths and if possible, permeabilities. If these can be identified to a reasonable degree of confidence, estimates of the quantity and rate of settlement and the potential stability of embankments over the materials can be made.

Countermeasures – Road embankments built on soft clays thus need careful control during their construction to avoid stability failures as pore water pressures increase under the applied loads. It is recommended that embankments in these areas are constructed slowly, layer by layer, while monitoring pore water pressures and additional layers are only added once the pore water pressures have dissipated adequately. Despite these measures, long-term settlement often continues, and problems are often encountered with large differential settlements between the approach fills founded on the clays and bridges founded on piles. These long-term differential settlements require ongoing maintenance to provide an adequate performance of the road.

The use of the wide range of geosynthetic products as separation layers and to facilitate and accelerate drainage has contributed to improved construction over such areas in the past decade or two, and specialist advice in this respect should be obtained.

Collapsible soils

Causes – Collapsible soils result from a unique condition in which "bridges" of fine materials (usually clays or iron oxides) within a framework of coarser and harder particles (mostly quartz) become weak when wet and collapse under load. The important condition is that the material must be in a partially saturated condition and then wetted up and loaded simultaneously, which is a common situation beneath road structures.

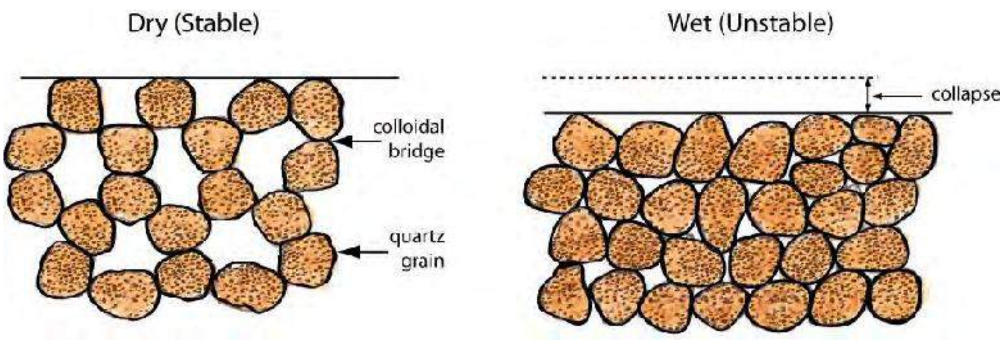

Figure 4-19: Graphical illustration of mechanism of collapse

Collapsible materials can occur on both residual and transported materials but are not widespread in Malawi. Many granites and feldspathic sandstones when weathered result in the feldspar altering to kaolinite with the quartz particles staying intact. This forms a honeycomb type of structure, which, when wetted up and loaded, results in shearing or "collapse" of the clay bridges and a settlement or reduction in volume of the material. Certain basalts and dolerites with dry densities of 1200 kg/m³ to 1300 kg/m³ have also shown collapse potential.

Recognition – Indications of the possibility of collapsible materials are:

- A very low density, because of the large number of voids separating the quartz framework.
- Densities of less than about 1600 kg/m3 (mostly in the range 1000 kg/m³ to 1585 kg/m³).
- The presence of "pinholing" or voiding observed during the soil profiling.
- Usually more than 60% of the mass of the material lies in the 0.075 mm to 2 mm range and less than 20% is finer than 0.075 mm.
- When the material excavated from a pit is insufficient to fill the pit again (the collapse structure will be disturbed, and the material will decrease in volume).

Countermeasures – If potentially collapsible soils are identified, specialist assistance should be sought to avoid excessive rutting. The deformation that is likely to affect very lightly trafficked roads will seldom have a major impact on their performance.

The result of collapse of the subgrade is mostly manifested by the development of a deeply rutted and often uneven road surface and significant deterioration of the riding quality of the road.

Wet areas/high water tables

Causes – It is possible that some non-clayey areas have a water table close to the natural ground surface, which makes the placement of road structures difficult and can affect their structural integrity. Unlike the clay areas, the problem is not the low strength or settlement potential, but the effect of the water (and high pore-water pressures under traffic loading) on the pavement structure.

High water tables result in a steady, high in-situ moisture content, but it is also possible that fluctuating high moisture content conditions within the pavement sub-structure may occur as a result of seasonal precipitation. A good understanding of the moisture conditions and environment needs to be defined during any investigation involving subgrade materials.

Recognition – It is usually easy to recognise potential wet conditions, which are characterised by areas of standing water, specific types of vegetation (reeds, papyrus grasses, etc.), localised muddy conditions and often the presence of crabs and frogs. Site investigations during the rainy season will also provide ready information.

Figure 4-20: Typical road condition in a wet/marshy area

Countermeasures – The treatment of wet areas for roads can be costly if the aim is to reduce the water tables using sub-surface drainage systems. These would seldom be warranted for low volume roads.

The only cost-effective measures for low volume roads are to raise the level of the road to at least 750 mm above the natural ground level, with a permeable gravel or rock fill layer (at least 100 mm to 150 mm thick) on the natural formation (after removal of the topsoil and vegetation). Properly designed and graded side drains should also be constructed to avoid the presence of standing water adjacent to the road.

The installation of sub-soil drainage systems is seldom warranted for low volume roads because of the cost and the ongoing need to maintain them diligently. However, in cases where they are considered to be essential, they should be designed by a drainage/ground-water specialist.

Sinkholes

In areas with carbonate rocks (limestones), the potential for dissolution of the rock material to form voids beneath the ground surface always exists. Care should thus be taken in such areas to ensure that no large voids occur beneath the road. Specialist advice should be sought in areas known to have such dissolution features.

Figure 4-21: Sinkhole

4.4.4 Location and Characteristics of Construction Materials

General

Sources of road-building materials must be identified within an economic haulage distance and they must be available in sufficient quantity and of sufficient quality for the purposes intended. Previous experience in the area plus local knowledge, may assist with locating such soils, but additional survey is usually required.

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

The investigation of construction materials often requires an extensive programme of site and laboratory testing, especially if the materials are of marginal quality or occur only in small quantities. The site investigation must identify and prove that there are adequate and economically viable reserves of natural construction materials. The uses of construction materials required are summarised below and discussed in more detail in *Chapter 5 – Materials*:

- Common embankment fill
- Capping layer / imported subgrade
- Sub-base and road-base aggregate
- Road surfacing aggregate
- Paving stone (e.g. for cobblestone pavements)
- Aggregates for structural concrete
- Filter/drainage material
- Special requirements (e.g. rock-fill for gabion baskets)

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 should be given to:

- Eliminate the need for regravelling by using a surfaced road
- For a surfaced road consider:
    - Modifying the material (e.g. mechanical or chemical stabilization);
    - Material processing (e.g. crushing, screening, blending);
    - Innovative use of non-standard materials;
    - Recycling.

Materials investigations should also take into account any future needs of the road. This is particularly important in the case of gravel roads where re-gravelling is normally needed regularly to replace material lost from the surface. Sources of good materials could be depleted resulting in increased haul distances and subsequent costs. Furthermore, good quality material may be required at a later stage in the road's life when the standard needs to be improved to meet increased traffic demands.

Figure 4-22: Sufficient construction material of the required quality must be located

A comprehensive list of the location of potential borrow pits and quarries is needed, along with an assessment of their proposed use and the volumes of material available. Apart from quality and quantity of material, the borrow pits and quarries must 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; and
- Located such that their exploitation will not lead to any complicated or lengthy legal problems and will not unduly affect the local inhabitants or adversely affect the environment.

Exploration of an area to establish availability of materials 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 variations, and possible flow of surface water into the excavation ground; and
- Assessment of the property of soils and rocks for the purposes intended.

Records of roads already built with the material can be 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. Construction records are often available with regional road authorities, or by road design consultants and contractors.

Fill

In general, location and selection of fill material for low volume roads poses 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 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 the impacts of winning fill in agriculturally productive areas where land expropriation impacts can be high.

Improved subgrade

The subgrade can be made of the same material as any fill. Where in-situ and alignment soils are weak or problematic, the import of improved subgrade 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, import of strong (CBR>9) subgrade materials can provide economies because pavement thickness design can be reduced (refer to *Chapters 9 and 10 – Structural Design*). Where improvement is necessary or unavoidable, mechanical and chemical stabilisation methods can 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 treatments 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. Base and subbase materials are expected to meet the specifications for the design method being used (refer to *Chapter 5 - Materials*).

The minimum thickness of a deposit normally considered workable for excavation for materials for subgrade, sub-base and base is of the order of one metre. However, thinner horizons could also be exploited if there are no alternatives. The absolute minimum depends on material availability and the thickness of the overburden. 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 cobble stone. In any area, a relatively fresh rock can be encountered at some depth as there is a gradual transition from one weathering state to the other. 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.

Figure 4-23: Variable degrees of weathering in quarry

Locating and testing construction materials

Field surveys and possibly laboratory testing programmes should be used to identify and locate potential construction materials. This information will guide the verification process undertaken by the design engineer in preparation of the detailed design.

For projects involving the use of aggregate processing there may be an additional requirement to undertake quality assurance/laboratory tests on trials of the product produced using the expected processing procedures (e.g. crushed aggregate for surfacing).

Projects involving significant fill and aggregate requirements will require mass-haul diagrams to be drawn that augment cost-benefit decisions for using any alternative materials or treatments, for example modifying the design requirements by modifying the material (stabilisation) or by additional material processing (e.g. crushing and screening).

The minimum frequency of testing of borrow pit material needs to strike a balance between cost, time and statistical validity as shown in Table 4-5. Where possible, the location and testing of borrow pit material should be done by traditional methods using established, full laboratory facilities.

Table 4-5: Material testing frequency

Project	No. per km			Minimum number of DN values per uniform section	
	Indicator tests	CBR tests	DCP tests	For statistical validity	Absolute
New road	≥ 3	≥ 2	≥ 5	8	5
Existing gravel road	≥ 2	≥ 1	≥ 5	8	8

The frequency of testing will depend on the variability of the material in that the more homogeneous the material, the less testing will be required. However, it is important to carry out sufficient tests to quantify the variability of the material within the pit during the site investigation stage and prior to construction. For LVRs, irrespective of the testing techniques and methods used, it is recommended that test samples are taken from at least five randomly selected locations per borrow pit (covering the full depth of the layer to be used) to quantify the variability. The variability provides an indication, for process control, of the variation in material quality that can be expected during construction.

Water sources

Water is a vital construction resource and can be a significant problem in arid countries. Many projects have been delayed because of an underestimate of the quantity of water that is conveniently available for construction. Suitable sources of water must therefore be identified at the design stage and due attention should be given to the phasing of construction if best use is to be made of the limited natural moisture in the materials.

In certain areas, water may be scarce for construction purposes and, in particular for providing proper moisture content during compaction of the soils and pavement layers. It is important to search for water sources, their yields and the distances from the construction site. In regions where water is scarce, a separate and dedicated hydro-geological study may be needed. Alternatively, dry compaction could be considered for some types of materials in certain layers. Data from the field reconnaissance can indicate if surface water is a critical problem.

Water sources for construction need to be chemically analysed for salinity (to assess the concentration of chloride and sulphate) which could be deleterious to performance of concrete and bituminous materials.

4.4.5 Geotechnical Investigations

General

The geotechnical investigation is a more sophisticated exercise than the activities described previously in this chapter, during which aspects such as the types and extent of excavations, foundation works, and control of problem subgrades is assessed. This usually requires sub-surface investigations and the development of geological and geotechnical models related to the road and structures. The information will allow the engineer to:

- Design stable and safe road pavements, bridges and other structures;
- Identify areas for specialist geotechnical investigation (deep cuts and high fills); and
- Identify areas of potentially problematic soils requiring additional investigation and treatment.

In many cases, especially smaller projects, the geotechnical investigation may be included as an integral part of the site investigation, whereas in other situations (mountainous regions, areas with particularly poor subgrade or drainage conditions etc.), independent and more comprehensive geotechnical investigations may be required.

Geotechnical investigations are progressive in nature, requiring more sophisticated and costly investigations as the project progresses when they are seen to be necessary. The primary steps are to:

- Understand the engineering objectives;
- Adapt the investigation to the project scope, local conditions and expected soil profile;
- Identify potential problems at an early stage;
- Investigate those sites identified as potentially problematic;
- Foresee potential difficulties, risks and consequences of failure;
- Facilitate an adequate and cost-effective design; and
- Identify the need for additional investigations.

Each geotechnical investigation is unique, depending on the specific ground conditions and pavement/structure, and should be planned as such. It is thus not possible to give a general step-by-step procedure applicable to all investigations.

The choice of methods for geotechnical investigation is determined by the type of road project and the nature of the issues likely to arise from the site conditions, geology, terrain and climate. The primary objective of such investigations is to obtain sufficient information such that the overlying structures are not subject to any unacceptable deformations related to ground subsidence or movement. The methods used should also be available locally and should be accompanied by experienced interpretation.

A wide variety of techniques is used for geotechnical investigations as presented in Table 4-6, but relatively simple and standard techniques should be used as much as possible. More sophisticated and expensive techniques should only be employed when a significant geotechnical problem is encountered with potentially severe consequences should failure occur. Under such circumstances, it is advisable to seek specialist assistance. Ground investigations need to be carefully planned and must take into account the following:

- The nature of the ground;
- The nature and phase of the project; and
- The project design requirements.

Results from the Desk Study and the initial assessment (walk-over survey) as described in Section 4.2.3 should be used in the planning of cost-effective ground investigations.

Error! Reference source not found. outlines the key objectives of ground investigations which may be undertaken using a variety of sampling and testing techniques, outlined in Table 4-6.

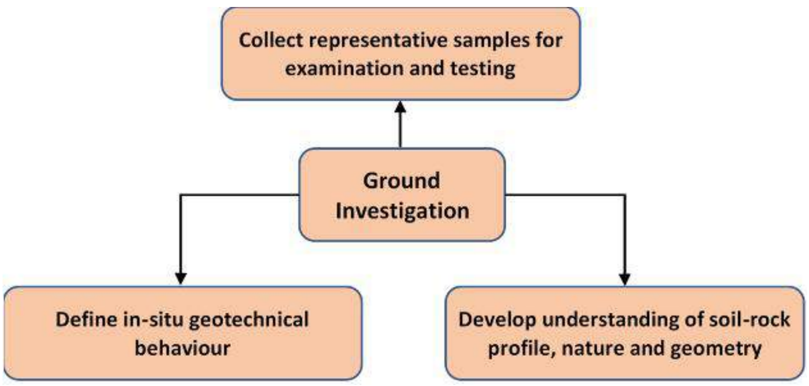

Figure 4-24: Objectives of ground investigation

Table 4-6: Ground investigation techniques

Ground Investigation			
Technique	**Purpose**	**Advantages**	**Frequency**
DCP survey	In-situ strength Characteristics.	Light and portable, gives information on state of any pavement layers present. Can test both road and shoulder. Test quick and simple.	A minimum of 5-20 DCP tests/km should be used for LVRs.
Vane shear test	In-situ shear strength in clays.	Especially good at assessing soft clays. Equipment is easily portable.	Where soft clays are present, 4-10 tests/km should be used.
Cone Penetration Test	In situ strength and compressibility of soils.	Good reliable information in soft to stiff clays and loose to dense sands.	Used in areas under moderate to high embankment and for structure foundation investigations.
Test Pits and trenches	Provides a ground profile and samples for testing subgrade and potential fill material.	Gives an accurate picture of the ground profile.	Dependent on DCP testing. Pits should be at least 0.5m below the natural subgrade level. In cuts this can be reduced to 0.3m. For a new alignment, pits should be at least 2 m deep unless rock is present.
Auguring and Boring	Provides in situ information on material present.	Can be used in areas where trial pits are not possible. Can extend to great depth.	Should be used in landslide zones, unconsolidated soils and where existing pavement layers are present.
SPT	Provides in situ strength parameters in most materials and can be used in weak rocks.	Used in conjunction with auguring or boring holes.	Used for structure foundation investigation and high earthworks.
Seismic hammer	Can differentiate between loose unconsolidated sediments and intact rock.	Light and portable. A sledgehammer and geophones provide a cheap option.	Can use for key areas where rock head is uncertain and critical for design.

Earthworks – Cut and fill Investigation

General – In order to comply with horizontal or vertical geometric guidelines and thus permit reasonable access for users, LVR alignments in hilly or mountainous areas may require the construction of cuts or embankment earthworks. On low plain areas liable to flood, it may also be necessary to raise roads on embankments. In general terms, these earthworks should be designed to minimise subsequent slope failure by implementing designs and construction procedures that are compatible with the engineering properties of the excavated soil-rock or the placed fill, whilst at the same time taking into account the impact of these earthworks on existing slopes or foundations.

The aim of any low-cost approach to earthworks design is to excavate to safe slope angles without having to resort to extensive use of support structures. However, the interaction of LVR route alignment and the geometry or instability of the natural slopes may be such that construction to recognised safe angles is not an economical or engineering feasibility. Engineered stabilisation may have to be considered, particularly in areas of identified natural hazard. If temporary road closures and debris clearance can be tolerated and allowed for in maintenance, then a steeper slope may be more economic.

A particular difficulty in steep terrain is the disposal of excess material (spoil), therefore every effort should be made to balance the cut and fill. Where this is not possible, suitable stable areas for the disposal of spoil must be identified. Spoil can erode or may become very wet and slide in a mass. Material is carried downslope and may cause scour of watercourses or bury stable vegetated or agricultural land. Material may choke stream beds causing the stream to meander from side to side, undercutting the banks and creating instability.

High level embankment foundation investigation should, as a minimum, consider; the range of materials and settlement potential; side-slope stability; groundwater; moisture regime and drainage requirements; erosion resistance; haul distance; and environmental impact.

Cut-slopes – Where possible, LVR cut slopes are generally designed on precedent or modified precedent principles (i.e. past experience), based on past experience with similar soil and rock materials. Cut slopes greater than 3 m – 6 m in height may require a more detailed engineering geological assessment depending on the complexity of the ground conditions. This would include an assessment of the type of the soil-rock materials and their mass structure.

The slope angles indicated in Table 4-7 have been provided as a general guide for LVRs. Note that these angles cannot be applied without due consideration of the ground conditions.

Table 4-7: Recommended cut-slope gradients

Soil/Rock Classification		**Slopes (V:H) for Various Cut Heights**		
		< 5 m	**5-10 m**	**10-15 m**
Hard rock (without adverse structure)		1:0.3 – 1:0.8		
Soft rock		1:0.5 – 1:1.2		
Sand	Loose, poorly graded	1:1.5		
Sandy soil	Dense or well graded	1:0.8 – 1:1.0	1:1.0 – 1:1.2	-
	Loose	1:1.0 – 1:1.2	1:1.2 – 1:1.5	-
Sandy soil mixed with gravel or rock	Dense, well graded	1:0.8 – 1:1.2		1:1.0 – 1:1.2
	Loose, poorly graded	1:1.0 – 1:1.2		1:1.2 – 1:1.5
Cohesive soil		1:0.8 – 1:1.2		–
Cohesive soil mixed with rock or cobbles		1:1.0 – 1:1.2	1:1.2 – 1:1.5	–

Cuttings in strong homogenous rock masses can often be very steep where adverse structure is not present, but in weathered rocks and soils it is necessary to use shallower slopes. In heterogeneous slopes, where both weak and hard rock occur, the appropriate cut-slope angle can be determined on the basis of the location, nature and structure of the different materials and the variations in permeability between the different horizons. One of the most effective ways to decide upon a suitable cut slope is to survey existing cuttings in similar materials along other roads or natural exposures in the surrounding areas. Generally, new cuttings can be formed at the same slope as stable existing cuttings if they are in the same material with the same overall structure. In rock excavations, persistent joint, bedding or foliation surfaces may determine the final cut slope profile.

Excavation of rock slopes should be undertaken in such a way that disturbance, for example due to blasting, is minimised. It should also be undertaken in a manner to produce material of such size that allows it to be placed in embankments in accordance with the requirements.

Cut slope profiles can be single-sloped, or benched. Single-sloped profiles are usually cut in uniform soil or rock materials or excavations less than 5 m - 10 m. Benched slopes are generally used in deeper cuts or where layered soil rock profiles are encountered. The construction of benches should be considered to intercept falling debris and control the flow of water. There is no hard rule regarding the dimension of benches, but a preliminary approach is to provide bench widths that are one third of the height of the cut immediately above. Outward sloping benches are generally not recommended because this may concentrate and erode channels through the bench if the bench is in weathered rock or soil. If the bench is in strong, un-weathered rock then this erosion will not occur, and outward sloping benches are permitted. In weaker materials the water should be encouraged to drain along the bench to a discharge point rather than over it. Maintenance of these drains is important to prevent water accumulating on the bench.

Embankments – Embankments may be required to:

- Raise the road above flood level on low-lying flat ground;
- Reduce steep gradients and minimise excess spoil in hilly terrain; and
- Facilitate suitable access in steep hilly or mountainous terrain.

Embankment design must accommodate two related elements; the design of the embankment itself using available materials and the strength or compressibility of its foundation. Embankment slopes should be designed taking into account both elements; typical angles for embankment fill on sound foundations are presented in Table 4-8.

Table 4-8: Recommended embankment slope gradients

<table>
<tr><th rowspan="2">Fill materials</th><th colspan="4">Embankment Side-slope (V:H) for Various Heights</th></tr>
<tr><th>< 5 m</th><th>5 -10 m</th><th>10-15 m</th><th>15-20 m</th></tr>
<tr><td>Well graded sand, gravels, sandy or silty gravels</td><td colspan="2">1:1.5 – 1:1.8</td><td>1:1.8 – 1:2.0</td><td></td></tr>
<tr><td>Poorly graded sand</td><td colspan="3">1:1.8 – 1:2.0</td><td></td></tr>
<tr><td>Weathered rock spoil</td><td colspan="3">1:1.5 – 1:1.8</td><td>1:1.8 – 1:2.0</td></tr>
<tr><td>Sandy soils, hard clayey soil and hard clay</td><td colspan="2">1:1.5 – 1:1.8</td><td>1:1.8 – 1:2.0</td><td>--</td></tr>
<tr><td>Soft clayey soils (not recommended)</td><td colspan="2">1:1.8 – 1:2.0</td><td>-</td><td>--</td></tr>
</table>

Fill slopes over 3 m in height or any embankment on soft soils, in unstable areas, or those on expansive clays may require site-specific geotechnical assessment depending on specific ground conditions. Fill placed near or against a bridge abutment or foundation, or that can impact on a nearby structure, may require specific stability analysis.

For embankments founded on soft soils, the most usual design option in low-cost road engineering is excavation down to satisfactory strength materials where possible. The option of route re-alignment to avoid soft soils areas should also be considered. Where these approaches are not feasible, detailed geotechnical analysis will be required.

The overall stability of a fill slope on a hillside may be difficult to assess. Before constructing a fill slope on side-long ground, it is necessary to terrace or step the formation in order to prevent a possible slip surface from developing at the interface between the fill and the natural ground. The potential for failure along a deeper surface in the ground beneath should be considered, although this rarely happens since the strength of soils tends to increase with depth. Problems can occur when strata or foliations in the rock masses beneath the fill are dipping parallel to the ground slope, or where the groundwater table is at or very close to the surface.

Cut-fill cross sections – Cut-fill cross sections are a combination of excavation into hillside above the alignment and placement of the excavated fill on the "down" side. Although the cut-fill option is attractive in terms of cut-fill balance and is a common situation in many hilly or mountainous access routes, it is also a frequent cause of access failure unless adequate design and construction precautions are adopted as shown in Figure 4-25. Most importantly, the fill must be placed and compacted in benches to ensure that it keys in with the natural ground to minimise the chance of slope failure.

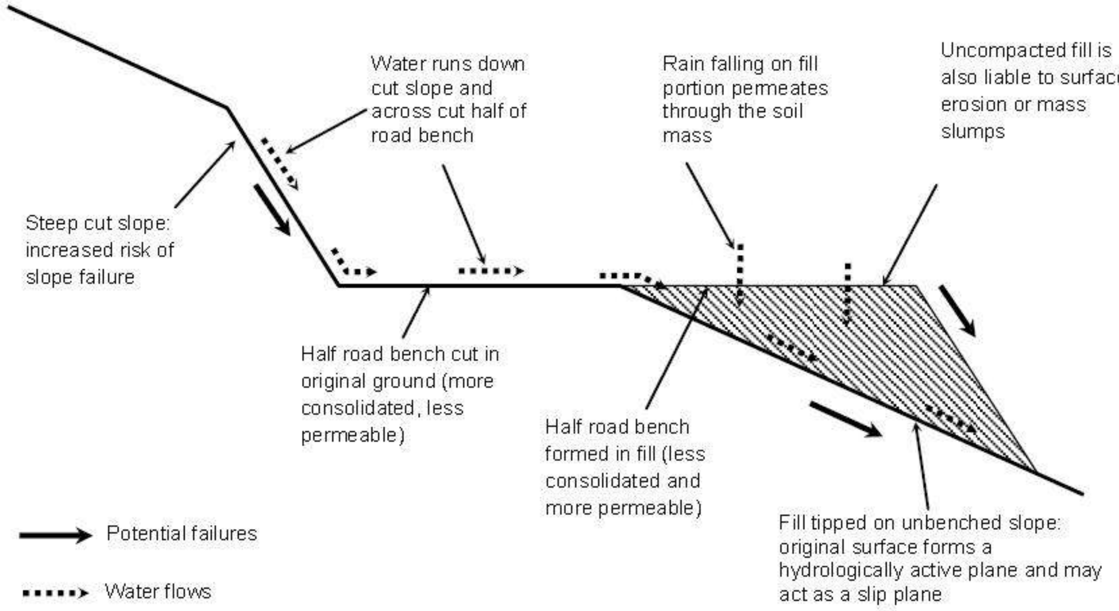

Figure 4-25: Cut-fill situations

Key requirements for an adequate design are:

- Suitable cut slope excavation, as shown in Figure 4-25.
- Key-in of the fill section to the natural slope by filling and compacting in benches.
- Adequate drainage to prevent pore pressure build-up or lubrication of the cut-fill interface and erosion or permeation of water into the slope.
- Specification for compaction of fill in layers and not simply dumped over the alignment edge.
- Specification of complete removal of vegetation and organic material prior to construction.
- Prevention of erosion on slopes immediately below the embankment.

Slope protection and stabilisation

Various techniques are used to protect and stabilise slopes associated with roads and prevent the occurrence or recurrence of landslides, especially along low volume roads. These are summarised in Table 4-9 and Table 4-10.

The use of these techniques depends on site-specific conditions such as the size of the slide, soil type, road use, existence of alternative routes and the causes of failure. Appropriate site investigations may be required to define the slope problem accurately within the overall geotechnical environment.

Table 4-9: Stabilisation options for problems above the road

Instability	Stabilisation options	Drainage options	Protection options
Erosion of the cut slope surface	None	Usually none; Occasionally a cut-off drain above the cut slope can reduce water runoff; however, these are difficult to maintain and can contribute to instability if blocked or otherwise disturbed.	In most cases, bioengineering is adequate, usually grass slip planting. Where gullies are long, or slopes are very steep, small check dams may be required. Sometimes a revetment wall at the toe helps to protect the side drain.
Failures in cut slope	Reduce the slope grade and if this is feasible, then add erosion protection. A retaining wall to retain the sliding mass; For small sites where the failure is not expected to continue, a revetment might be adequate.	A subsoil drain may be required behind a wall if there is evidence of water seepage; Herringbone surface drains may be required if the slope drainage is impeded.	Bioengineering can be important to prevent surface erosion and increase the resistance of the surface soil. Will have no effect on deeper failure prevention or stabilisation.
Failures in cut slope and hill slope	Reduce the slope grade, and if this is feasible, then add protection; A retaining wall to retain the sliding mass. This may need to be quite large, depending on the depth of the slip plane.	A subsoil drain may be required behind a wall if there is evidence of water seepage; Herringbone surface drains may be required if the slope drainage is impeded.	Bioengineering can be important to prevent surface erosion and increase the resistance of the surface soil. Will have no effect on deeper failure prevention or stabilisation
Failures in hill slope but not cut slope	Reduce the slope grade, and if this is feasible, then add protection. A retaining wall to support the sliding mass, as long as foundations can be found that do not surcharge or threaten the cut slope.	A subsoil drain may be required behind a wall if there is evidence of water seepage; Herringbone surface drains may be required if the slope drainage is impeded.	Bioengineering can be important to prevent surface erosion and increase the resistance of the surface soil. Will have no effect on deeper failure prevention or stabilisation.
Deep failure in the original ground underneath the road	Consider re-alignment of road away from instability. If slow moving, short term option may be to repave or gravel the road.	Ensure road-side drainage is controlled.	Bioengineering will not be effective.

Table 4-10: Stabilisation options for problems below the road

Instability	Stabilisation options	Drainage options	Protection options
Erosion of the fill slope surface.	None.	Ensure road-side drainage is controlled.	Bioengineering a key option.
Failures in fill slope	Re-grade or remove, replace and compact fill. Before replacing fill, cut steps in original ground to act as key between fill and original ground; A new road retaining wall may be the only option.	Ensure road-side drainage is controlled.	Bioengineering can be important to prevent surface erosion and increase the resistance of the surface soil. Will have no effect on deeper failure prevention or stabilisation.
Failure in fill slope and original valley slope.	Re-grade or remove, replace and compact fill. Before replacing fill, cut steps in original ground to act as key between fill and original ground. A new road retaining wall may be the only option.	Ensure road-side drainage is controlled.	Bioengineering can be important to prevent surface erosion and increase the resistance of the surface soil. Will have no effect on deeper failure prevention or stabilisation.
Failure in original valley slope.	Re-grade if sufficient space between road and valley side. A new road retaining wall may be the only option.	Ensure road-side drainage is controlled.	Bioengineering can be important to prevent surface erosion and increase the resistance of the surface soil. Will have no effect on deeper failure prevention or stabilisation.
Removal of support from below by river erosion.	May need extensive river training works to prevent further erosion.	None.	Slope protection (walls and riprap etc.) may be necessary – possible with additional bioengineering options.

Slope drainage

Slope stability is greatly influenced by hydrology, either by the erosive impacts of surface water or the changes in pore pressure resulting from rainfall infiltration and concentration within the slope mass. Water may decrease pore suction in the underlying soil or increase pore water pressure, thereby reducing the effective strength and hence the stability of the slope. The construction of surface and sub-surface drainage structures is therefore often vital to ensure that excess water can be intercepted and conveyed to a safe location where it will not create instability problems.

Principal earthwork drainage options include the following measures which are discussed in more detail in *Chapter 8 – Drainage and Erosion Control*:

- Cut-off drains
- Herringbone (or chevron) drains
- Counterfort drains
- Horizontal drains
- Lined channels or cascades
- Scour checks

Groundwater

If groundwater is not identified and adequately addressed early, it can significantly impair constructability, road performance and slope stability. Claims related to unforeseen groundwater conditions often form a significant proportion of contractual disputes. Many of these claims originate from a failure to record groundwater depths and conditions during site investigation.

The presence of groundwater is normally identified from test pits and trenches. If these pits and trenches are left open for 24 hours, high water table will be indicated by the presence of water in the pits/trenches. Open pits and trenches should be marked by warning tape to protect the general public. In addition, moisture content samples from the various layers excavated will indicate the moisture regime at the time of sampling.

Water crossings

General – The objective of a site investigation for possible structures is to provide a clear picture of the ground conditions to enable a suitable design of the structure to be carried out. The level of site investigation clearly depends on the type and complexity of the proposed structure. When bridges and large causeways are considered, site investigations should be undertaken by a suitably qualified foundation/structural engineer, and a suitably designed geotechnical survey should be carried out.

The ground underneath a proposed structure should have an adequate bearing capacity to support the load of the structure itself and the vehicles which pass over it. If the soil has insufficient strength, it will compress and the structure will subside, possibly leading to failure or at least functional inadequacies.

The bearing capacity will depend on a range of different factors including; the proportions of gravel, sand and clay; organic and other material in the soil; the mineralogy of the clay materials; and the level of the water table. As the type of soil may change with depth, it is necessary to dig trial pits at the proposed site to determine the bearing capacity at the proposed foundation level. By identifying and sampling the material excavated from different depths of the trial pits, the bearing capacity of the soil can be determined. Bearing capacities are particularly important in the design of structures where large localised loads are expected, (e.g. bridge abutments and piers). The soil must have a high bearing capacity to support these loads. Where the bearing capacity of the soils is deficient then engineering solutions such as piling may be necessary. Such investigations would help in determining the side friction or end bearing capacity for the design of piles.

Areas requiring water crossings or where water will naturally cross over the road if not already catered for must be identified during the site investigation. Those areas that will necessitate the provision of large culverts or bridges need to be identified, as they will require detailed geotechnical investigations for their foundations.

The sub-surface investigation for the final design stage is typically performed prior to defining the proposed structural elements or the specific locations of culverts, embankments or other structures. Accordingly, the investigation process includes techniques sufficient to define soil and rock characteristics and the centreline subgrade conditions.

For small, simple structures such as drifts, culverts and vented fords, it is normally sufficient to ensure that the proposed foundation material consists of well drained, firm (compacted) material. This will require the excavation and description of a number of test pits (usually to slightly weathered or hard rock) at critical points under the structures with simple material descriptions and strength testing (e.g. DCP or plate loading) where necessary. These will allow material types, depths and estimated strengths to be determined for use in the design.

Weathered rock, clays and silts that are at least “firm”, or sands and gravels that are at least “loose”, will be suitable for design purposes. Such conditions can also be determined on site by checking for footprints when walking on the proposed location. If more than a faint footprint is left, it will be necessary to improve the ground before construction commences. Additional useful information for design can usually be obtained from similar structures in the area.

The number of trial pits that should be dug will depend on the complexity of the structure and the uniformity of the soil. Table 4-11 gives a guide to the number and depth of trial pits that should be dug for different structures. If the ground conditions are known to vary over the proposed site, or two trial pits show markedly different results, then further trial pits should be dug as appropriate. The trial pit depth is only given as a guideline figure. If the soil conditions are very poor, it may be necessary to increase their depth or carry out deeper investigations using boring or drilling. Where bedrock exists close to the ground surface, this offers the best foundation.

Table 4-11: Guideline for number of trial pits for structure foundations

Structure	Number	Location	Depth
Drift	Not required.		
Culvert	1	At outlet.	1.5 metres.
Vented drift	2 (only 1 required if ford is shorter than 15 metres).	At each end of the vented section preferably one on the upstream, and one on the downstream side.	1.5 metres.
Large box culvert (> 3 metres width)	2+ (additional pits at each pier location required).	At each abutment and each pier.	2.5 metres (deeper in poor ground conditions).
Bridge	2+ (additional pits at each pier location if required).	At each abutment and each pier.	To firm strata (minimum of 3 metres).

If the ground conditions are poor at the proposed or expected level of the structure's foundation, it will be necessary to continue excavation to firm material that can provide sufficient bearing capacity. For larger structures, a range of foundations could be used depending on the materials on site. It is useful to carry out a geophysical survey (seismic or resistivity) to identify the general strata in the area and to provide a basis for siting further exploratory points. This can substantially reduce the number of boreholes or deep auger holes required.

Scour – Scour is the erosion of material from the riverbanks and bed due to water flow as shown in Figure 4-26. Damage due to scour is one of the most likely causes of structural failure. Minimising or eliminating the effects of scour should therefore receive adequate attention when designing any structure. Scour can occur during any flow, but the risk is generally greater during floods. There are three major types of scour to be considered and the potential for these should be assessed during the geotechnical investigation:

a) River morphology: This is long-term changes in the river due to bends and constrictions in the channel affecting the shape and course of the channel.

b) Construction scour: this is the scour experienced around road structures where the natural channel flow is restricted by the opening in the structure. The speed of the water increases through the restriction and results in more erosive power, removing material from the banks and bed.

c) Local scour: occurs around abutments and piers due to the increased velocity of the water and vortices around these new unnatural obstructions.

The proposed site of the structure and the watercourse upstream and downstream must be inspected for evidence of existing scour, erosion or deposition in the watercourse and banks. However, it is difficult to accurately predict the level of scour that may be experienced for a particular design as the changes on the flow characteristics of the water depend on the actual design as well as the stream channel geometry and water flow rates. The prediction of scour depth can be done using ORN 9 (TRL, 2000). However, the geotechnical investigation should provide the engineer with a basic knowledge of the scour characteristics of the materials.

Figure 4-26: Typical bridge scour

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Chapter 5 – Materials

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5.1 Introduction

5.1.1 General

Naturally occurring soils, gravel soil mixtures and gravels occur extensively in many parts of Malawi. These unprocessed materials are a valuable resource as they are relatively cheap to exploit compared, for example, to 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 minimize 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, 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 technique. In addition, it is important to recognise that the specifications for materials must be coupled to the pavement design method being used.

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 economic and sustainable manner, and to ensure that satisfactory levels of quality are attained.

The scope of the chapter covers the range of construction materials required for all classes of LVRs. The benefits of using locally occurring materials are highlighted and the relevant properties are discussed. Means of locating and improving local material are also briefly addressed.

Material requirements for the DCP-DN and DCP-SN/CBR methods of design are discussed as it is common practice to make use of more than one design method and to compare the results to check the reasonableness of the designs.

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, gravity). The use of expensive aggregate derived from the crushing of hard rock for the structural layers in LVRs should be minimised, such materials typically being used solely for bituminous surfacings or concrete structures.

5.2.2 Materials in Malawi

Malawi has a wide range of geologically-related material types as shown on the Geological Map of the country in the Appendix to *Chapter 3 – Physical Environment*. The lithology of Malawi is characterized by Precambrian metamorphic and igneous rocks which form part of the polyphase East African Orogen. Rift-related sedimentation and igneous activity during the late Palaeozoic (Karoo System) and the late Mesozoic (Chilwa Province) have produced a great variety of rocks that underwent strong chemical weathering and erosion when the entire region received its final shape by peneplanation and fluvial incision during the Cenozoic under (sub) tropical climatic conditions.

5.2.3 Weathered and Residual Materials

Weathering of rocks causes the chemical 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 a natural gravel for road construction. This material is of particular interest for use in LVRs as it 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.

Weathering effects generally decrease with depth, although zones of differential weathering can occur in many outcrops.

Figure 5-1: Variable weathering in borrow source

Table 5-1 presents a system for describing and classifying the states of weathering in borrow or quarry materials. In this table, the degree of weathering is divided into grades that reflect definable physical changes that could result in modified engineering properties. The general descriptions cover ranges in bedrock conditions and are intended for a rapid assessment of the use of borrow and quarry materials for different purposes in pavement construction.

Table 5-1: Weathering classification system

<table>
<tr><th rowspan="2">Term</th><th rowspan="2">Grade symbol</th><th colspan="6">Diagnostic features</th></tr>
<tr><th colspan="3">Rock material</th><th colspan="3">Rock mass</th></tr>
<tr><td>Fresh</td><td>IA</td><td rowspan="2">No visible sign of weathering</td><td rowspan="4">Not friable</td><td rowspan="6">Texture preserved</td><td>No visible sign of weathering</td><td rowspan="5">Not to slightly decomposed</td><td rowspan="6">Structure preserved</td></tr>
<tr><td>Faintly weathered</td><td>IB</td><td>Weathering restricted to surfaces of major discontinuities</td></tr>
<tr><td>Slightly weathered</td><td>II</td><td>Slightly weathered</td><td>Penetrative weathering on open discontinuity surfaces</td></tr>
<tr><td>Moderately weathered</td><td>III</td><td rowspan="4">Weathered</td><td rowspan="4">Weathered throughout</td></tr>
<tr><td>Highly weathered</td><td>IV</td><td>Partly friable</td></tr>
<tr><td>Completely weathered</td><td>V</td><td rowspan="2">Friable</td><td rowspan="2">Wholly decomposed</td></tr>
<tr><td>Residual soil</td><td>VI</td><td>Texture destroyed</td><td>Structure destroyed</td></tr>
</table>

Source: Knill et al, 1970.

Weathering tables may generally be applicable to all rock types. However, they are easier to use in igneous and metamorphic rocks that contain ferromagnesian minerals. Weathering in many sedimentary rocks will not always conform to the criteria in Table 5-1. In addition, weathering descriptions and categories may have to be modified to reflect site-specific conditions, such as fracture openness and filling, and the presence of groundwater.

The properties of the final residual material will, however, depend on the mineralogy of the parent rock and with the diverse range of rock types present in Malawi, a wide range of natural gravels can thus be expected. These can vary from non-plastic quarzitic gravels through to highly plastic and expansive swelling clays. Only some of these are suitable as selected road construction materials as discussed in *Section 5.4 – Materials Prospecting.*

5.2.4 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 sorting of different size fractions may occur. Many of these transported materials are also suitable for construction of roads.

Figure 5-2: Typical deposit of transported sandy material

Transported soils are often localised, occurring only in small deposits, but sources large enough to be considered for road construction are often associated with large rivers, arid areas with wind-blown sands and at the foot of escarpments and mountain ranges.

5.2.5 Pedogenic Materials

Pedogenic materials are a unique type 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:

- a relative accumulation of the cementing material resulting from the leaching or washing out of soluble bases leaving material rich in the cementing materials; or
- by an absolute 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.

These materials, collectively known as pedocretes, 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. Do note, however, that their unique properties usually require special sample preparation methods.

Figure 5-3: Typical laterite borrow pit showing variable nature of the material

The dominant pedocrete in Malawi is laterite, which is generally a good construction material for all layers up to 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 a deep and irregular weathering profile as shown in Figure 5-3.

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 with an adequate fines content. However, when judging the gradation of a 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 kaolinite content and sesquioxide content control the cohesion and the internal friction angle of laterites, respectively. The cohesion increases with increasing kaolinite content, and the internal friction angle increases with increasing sesquioxide content. Thus, if there is a significant difference in the specific gravities of the coarse and fine fractions, the grading should be calculated by use of both volume and mass proportions.

The requirements for selection and use of lateritic gravels for pavement layers are different to 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 done with circumspection.

Pedocretes are thought to have the property of self-stabilization (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 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-stabilization". The major benefits are that in the long-term, the materials probably become less susceptible to moisture-related damage.

5.2.6 Summary of Typical Material Properties

Table 5-2 summarises the typical properties of the residual gravels obtained from the 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 5-2: 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, coral-rock, dolomite,	Mixed gravels	Low	Medium to high
Diamictite	Tillite, greywacke	Mixed gravels	Low to high	Low to high
Pedogenic	Calcrete, laterite, silcrete	Mixed gravels	Low to high	Low to high

5.3 The 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 task and environment. Conventional specifications 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 work 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 regular maintenance as discussed in other chapters.

The benefits of utilising locally available materials arise from:

- a reduction in haulage costs;
- less damage to existing pavements from extended haul;
- stimulation of the local economy and local enterprise;
- road designs compatible with local maintenance capabilities; and
- generally, reduced whole life costs.

When material reserves are limited or of marginal quality, their relevant usage is a priority and it is important to ensure that the materials are neither sub-standard nor wastefully above the standards demanded by their engineering task. Hence, it is necessary to derive locally relevant specifications and to either adapt the designs or modify the materials to suit.

5.3.2 Optimum Utilisation of Local Materials

The potential benefits of innovatively using natural and alternative materials for road construction are illustrated in Figure 5-4, which shows that continued use of existing material standards and standard materials will result in depletion of potential sources at time t_1.

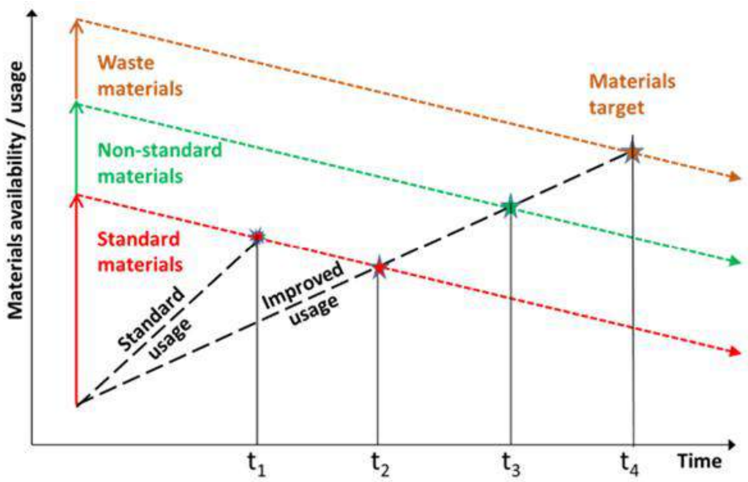

Figure 5-4: Impact of increasing the use of non-traditional road construction materials

The increased availability of natural materials can only be achieved by changing the required materials standards to those more appropriate for the road category, i.e. by not using material that is suitable for high quality roads in lower volume roads that do not require such high material standards. This will also reduce the cost of construction materials and will result in the potential to build more roads (shown as improved material usage in Figure 5-4).

By increasing the use of alternative materials, the period for availability of standard materials will be extended from t_1 to t_2. By also using non-standard materials not complying with current specifications, the total period for availability of materials will be extended further to t_3. By also making use of by-products which are currently considered as waste materials, the materials availability period will be extended to t_4, which must be the target for effective materials usage.

If the project is in an area where good quality construction materials are scarce or unavailable, consideration should be given to the following options as described in more detail in Sections 5.8.2, 5.8.3 and 5.8.4:

- Modifying the material (e.g. mechanical or chemical stabilisation).
- Processing the material (e.g. crushing, screening, blending).

Ultimately, the cost of haulage over a long distance to obtain good quality materials should be compared with the above options, on which basis the most cost-effective option should be chosen.

Figure 5-5: Coarse river gravels can be crushed for concrete or surfacing aggregate

Many material specifications, such as the SATCC Standard Specifications for Road and Bridge Works and TRH 14: Guidelines for Road Construction Materials, provide limits to material characteristics in various pavement layers. In addition to controlling the material strength, as indicated by soaked CBR values, there are additional criteria for other properties such as grading, plasticity and particle strength. Many local materials may comply with the material strength criterion but not the other requirements and would thus be rejected. In LVRs, it is the strength (or ideally stiffness) that is critical and provided that this is mobilised (and retained under the expected prevailing conditions in the short and long term), there is no reason why the material should not perform satisfactorily.

5.3.3 Beneficial Characteristics of Local Materials

Despite the innumerable differences that exist among local materials, there are some dominant characteristics that affect pavement performance which should be appreciated in order 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 manner in which they derive their strength in terms of the following intrinsic properties:

- Inter-particle friction;
- Cohesive effects from fine particles;
- Soil suction forces; or
- Physio-chemical (stabilisation) forces.

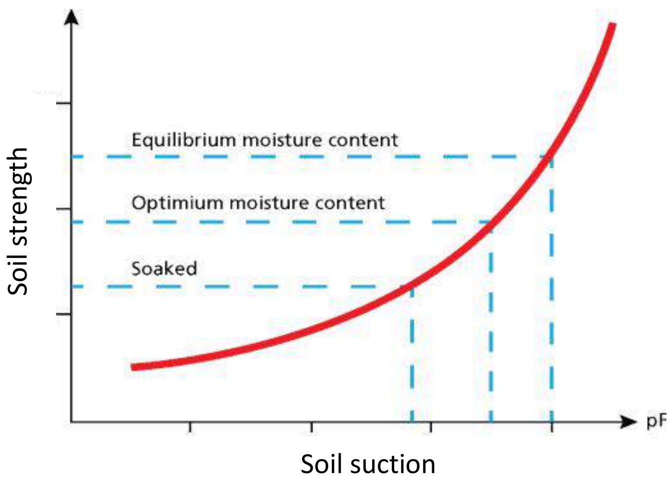

Figure 5-6: Illustrative soil strength – soil suction relationship

The influence of moisture on each of the above components of shear strength will significantly influence the manner in which they can be incorporated within a pavement. For example, unbound / unprocessed materials (e.g. calcrete or laterite) are highly dependent on suction and cohesion forces for development of shear resistance that will only be generated at relatively low moisture contents. Special measures therefore must be taken to ensure that moisture ingress into the pavement is prevented. Otherwise suction forces and shear strength will be reduced as illustrated in Figure 5-6.

Table 5-3 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 5-3: 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 reduces volume of moisture sensitive, soft and poorly graded gravels		Material strength maintained even in wetter state
Appropriate use	Low traffic loading in 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			

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 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 maximised when the aggregate is hard, durable and well graded. Very high levels of saturation, typically 80-100% will be 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, generally 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 so as to ensure that it operates as much as possible at an unsaturated moisture content. The beneficial effect of so doing is illustrated in Table 5-4 which shows the variation of a material's strength (CBR or DN) with moisture content.

Table 5-4: Variation of CBR and DN with moisture content

Laboratory Soaked CBR (%)	Approximate Laboratory Unsoaked CBR (%) / DN (mm/blow)* at varying FMC/OMC Ratios		
	1.0	**0.75**	**0.50**
80	96 / 3.1	151 / 2.2	205 / 1.7
45	69 / 4.1	109 / 2.8	148 / 2.2
25	54 / 4.9	85 / 3.4	115 / 2.7
15	50 / 5.2	79 / 3.6	108 / 2.9
10	37 / 6.6	59 / 4.6	80 / 3.6

Source: Paige-Green et al (1999) *Kleyn correlation

If, through effective drainage, the materials in the road pavement can be maintained at a field moisture content that does not rise above OMC in the rainy season, then more extensive use can be made of local, relatively plastic materials that might otherwise not be suitable if they were to become soaked in service.

5.3.4 Materials Selection

General selection criteria

The criteria used for selecting road materials for incorporation in a LVR need to take account of their actual engineering purpose within the pavement. This requires consideration of the following factors:

- A knowledge of the key engineering properties of the material.
- The task required of the material.
- The governing road environment.
- Future alterations to the road environment.

Requirements for pavement materials

To perform satisfactorily, pavement materials, particularly in the base, must possess a number of attributes which must be satisfied with regard to their selection for LVRs. In addition to the physical properties governing performance, other aspects also need to be satisfied, as presented in Table 5-5:

Table 5-5: Fundamental pavement material selection factors

Strength	Aggregate particles need to be resistant to any loads imposed during construction and the design life of the pavement.
Mechanical stability	The aggregate as a placed layer must have a mass mechanical interlocking stability sufficient to resist loads imposed during construction and the design life of the road.
Durability	Aggregate particles need to be resistant to mineralogical change and to physical breakdown due to any wetting and drying cycles imposed during construction or in service.
Impermeability	Impermeability of the base is generally desirable to prevent ingress of water.
Availability	Reserves must be within physically and economically feasible haulage distance.
Workability	The material must be capable of being placed and compacted by the available plant.
Environmentally compliant	The material reserves must be capable of being won and hauled within any governing environmental impact regulations.

Attainment of the above selection factors would normally lead to the following key attributes of any pavement, namely:

- Adequate bearing capacity under any individual applied load.
- Adequate bearing capacity to resist progressive failure under repeated individual loads.
- The ability to retain that bearing capacity with time (durability).
- The ability to retain bearing capacity under various environmental influences which relates to material moisture content and, in turn, to climate, drainage and moisture regime.

Both the mechanical stability and durability of a pavement material are strongly correlated to its strength. Thus, strength is one of the most important parameters affecting the performance of a LVR.

When very high moisture contents cannot be prevented, an open-graded permeable material may be advantageous to reduce the development of excess pore pressures.

Segregation of material within the base can be of concern, particularly if oversize material is permitted or if extensive water binding is used during compaction.

5.4 Materials Prospecting

5.4.1 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 interpretation of maps and other information is a key aspect of the process.

5.4.2 Stages in Prospecting

The various stages in the materials prospecting process are shown in the flow chart in Figure 5-7.

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.

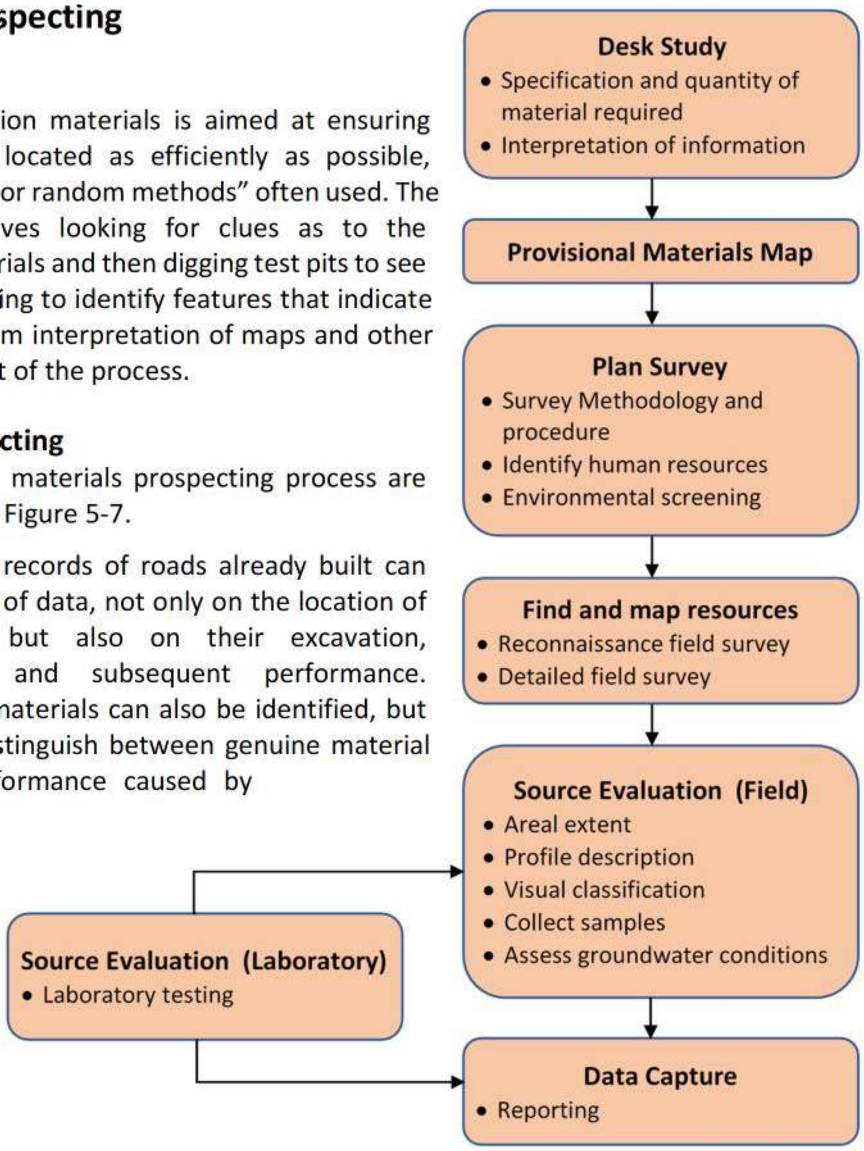

Figure 5-7: Flowchart of prospecting procedure

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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 by in-house resources. Due regard must be given to environmental considerations that impact the viability of potential material sources.

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.

5.4.3 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 variations, and possible flow of surface water into the excavation ground.
- Assessment of the property 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 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 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.

5.5 Materials Sampling and Testing

5.5.1 General

A high proportion of the LVR construction costs are 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 successful construction of LVRs is therefore that a materials sampling and testing programme is designed and implemented as appropriate to the circumstances for each individual project.

5.5.2 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 5-6 gives a guide to the required sample sizes for the most common soils tests applicable for LVRs.

Table 5-6: Sample requirements for soil testing

Test		Minimum mass required (kg)		
		Fine	Medium	Coarse
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	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		80.00	
Aggregate strength	Aggregate Crushing Value (ACV)		2.00	
	Aggregate Impact Value (AIV)		2.00	
	Los Angeles Abrasion (LAA)		5.00-10.00	

Source: Guidelines on the Selection and Use of Construction Materials (TRL, undated)

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 is changing, and correct sampling procedures must be followed to ensure that the samples are representative of the material to be tested.

Table 5-7: Guideline for materials sampling frequency

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

Potential borrow pits shall be surveyed by trial pit excavation and sampling at the detailed design stage (*Chapter 4 – Site Investigations*). The survey shall prove sufficient quantities for all pavement layers and the sampling frequency shall be *at least* as indicated in Table 5-7 per DN or CBR test.

5.5.3 Testing Programme

The quality of the testing programme depends upon the procedures in place to ensure that tests are conducted properly using suitable equipment that is mechanically sound and calibrated correctly. The condition of test equipment and the competence of the laboratory staff are therefore crucial. There needs to be a robust Quality Assurance (QA) procedure (overseen by a competent geotechnical engineer) in place that will reject data that does not meet acceptable standards of reliability. There should be no compromise on the QA procedure or quality of testing data just because the project is perceived as a LVR.

The laboratory testing programme should be part of a comprehensive programme designed by the engineer to provide all of the information needed to adequately define the nature, use and volumes available of construction materials. In this regard, the early phases of the programme will generally concentrate on gaining clues to unusual soil behaviour, e.g. swelling or collapse potential. Bearing in mind the difficulties of sample recovery, statistical sample sizes and the cost of laboratory testing, most testing programmes will be based around relatively simple classification tests that can be done quite quickly. More sophisticated tests will only be used if absolutely necessary.

At the stage of final design, there is always the problem that natural-occurring materials show high variability in their properties and therefore obtaining design parameters at the ideal level of statistical reliability is very difficult. As a result, considerable engineering judgement and skill is required.

5.5.4 Materials Testing Protocols

Material specifications are based on one particular test protocol. It is thus important that the materials are tested in accordance with the relevant test protocol and that tests from different protocols are not used for the same material sample as this could lead to conflicting results and disagreement with the contractor over the compliance with the specifications.

For the DCP-DN method of pavement design the SANS test protocol applies, but for the DCP-SN/CBR method the BS test protocol must be applied.

There are significant, inherent differences between similar soil tests using different testing standards as a result of which the test results are not comparable. Some of the most significant differences in test methods which yield very different results include:

- The Liquid Limit (LL) and, hence, Plasticity Index (PI) of soils determined from the BS LL which, all other factors being equal, yields LL and PI results 4 units higher than the ASTM/AASHTO-type LL device.
- There are significant differences in grading results depending on whether dry or wet sieving procedures are followed. Although both procedures are catered for in the standard test procedures, dry sieving should only be used for materials containing little or no silt or clay.
- CBRs determined by TMH1 and AASHTO methods are on average about 20% lower than for the same material tested by the BS method.

5.5.5 Standard Tests

The testing programme must ensure that all the materials used in the construction of the roads are in accordance with the specifications for each design method. Typically, determination of the basic material properties, i.e. the material grading, Atterberg limits, compaction characteristics and strength (DN or CBR) are the primary laboratory tests. These tests shall be carried out on samples obtained from the road, typically from uniform sections determined from a DCP survey, or from proposed borrow sources to give an early indication of the potential suitability of the material.

While the different design methods have different material specifications, the following laboratory tests are common to all the design methods described in this manual.

Grading and Atterberg Limits – The designer needs to be intimately familiar with the basic properties of the materials and how the material behaves under the influence of moisture at varying densities. Determination of the grading and Atterberg Limits is therefore a requirement in all design methods.

Determination of MDD/OMC – Standard compaction tests shall be carried out to determine the MDD and OMC of the material. Typically, 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 5-8.

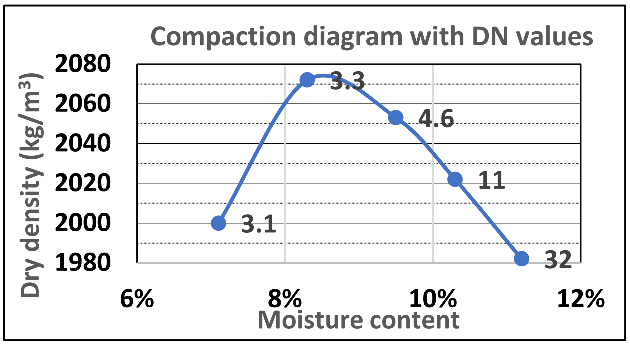

Figure 5-8: MDD curve with DN values (mm/blow) determined on each mould

Determination of material strength – The method for determination of material strength is what constitutes the greatest difference between the two design methods described in this Manual:

- For the DCP-DN method, the material strength is determined through the Laboratory DN test as described in Chapter 9 – Structural Design: Paved Roads, Section 9.2.5, and measured in terms of the DN value.
- For the DCP – CBR method, the material strength is determined through the standard CBR test and measured in terms of the CBR value (normally soaked CBR).

5.5.6 Specialised Tests

Mineralogical and Durability tests - Use of residual basic igneous rock (including basalt and dolerite) 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).
- Durability Mill index of less than 125 (SATCC, 1994).

The use of residual soils derived from basic igneous rock are potentially problematic if the limits stated above are exceeded. The risk of using the material can be minimised if consideration is given to:

- The variability of the material deposit, with good selection and control procedures in place for the operation of the pit and on site.
- The provision of good drainage conditions (many materials used for LVRs are particularly sensitive to moisture).
- The adequacy of the pavement design.
- The use of an impervious surfacing (Cape Seal, Otta Seal, Double Surface Dressing etc.).

With careful selection, these materials can normally be used for lightly trafficked paved roads in accordance with the specific pavement requirements (see *Chapter 9 – Structural Design: Paved Roads*).

Testing of natural sands - If well compacted (100% Vibrating Hammer), certain types of natural sands exhibit high load bearing capacities and can be successfully used as a pavement material for LVRs. Experience has shown that Kalahari-type sands which fulfil specific Soil Constant requirements ($5 < BLS_{0.075}/ø_{mean}<10$ for base and sub-base) and exhibit minimum soaked CBRs of 60% and 30% for base and sub-base respectively (or equivalent DN values), can be used successfully for paved LVRs *(Note: BLS = Bar Linear Shrinkage; ø = sediment size scale which is used in sedimentology and is equal to the logarithm to the base 2 of the grain diameter, i.e.* $ø = -log_2 d$ *where d = particle size in mm.*

5.6 Construction Material Requirements

5.6.1 General

The different types of road construction materials required are:

- Common embankment fill.
- Imported (selected) subgrade.
- Sub-base and base aggregate.
- Road surfacing aggregate.
- Block or Paving stone (e.g. for cobblestone pavements).
- Aggregates for structural concrete.
- Filter/drainage material.
- Special requirements (e.g. rock-fill for gabion baskets).

Some of these materials require extensive processing and will thus be costly. The road design should thus be carefully planned to minimise use of the more expensive materials.

It should also be noted that the majority of low volume road designs for which this document is relevant will be upgraded from an existing earth or gravel road, which may have been in service for many years. The strength built up in the underlying material must be capitalised on and as little additional structure as possible should be constructed. Other aspects such as shape, drainage and the repair of localised problem areas have also usually been attended to over the years.

There will, however, always be areas that require full construction or reconstruction, including any realignment to improve the geometry or avoid particular problem areas and areas that may require widening. In these cases, full pavement construction will be necessary, requiring materials for a number of applications as discussed below. It is important, however, that any sections of road that are widened have layers (and layer properties) as close to those of the existing road as possible, so that the upgraded road behaves as an integral structure.

5.6.2 Common Embankment Fill

In general, location and selection of fill materials for low volume roads poses few problems with materials requiring CBR values at their expected worst in-situ moisture and density condition of 3 to 5% corresponding to DN values of 33 mm – 48 mm/blow. Exceptions include organic soils and clays with high liquid limits and plasticity. Problems may also exist in lacustrine 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 excavation of the side drains (exception in areas of expansive soils). Borrow pits to provide fill materials should be avoided as far as possible and special consideration should be given to the undesirable impacts of winning fill in agriculturally productive areas where land expropriation costs can be high.

It is unusual to construct high fills for low volume roads (except for bridge and water crossing approaches) and in most cases the fills are limited to sufficient material to raise the pavement above the natural ground level to allow the placement of small water crossing structures (pipes and small culverts). Unless fills cross naturally weak subgrade areas (swampy or black cotton soils), it is usually not necessary to raise them much higher than about 1 metre and a low-quality material is usually adequate. In such areas with weak subgrades, it may be necessary to design fills such that they are drained (rock fill layers at their base, with or without geo-synthetic layers) but the material quality above this would not need to be any higher than for the low fills described previously.

5.6.3 Imported (Selected) Subgrade

Where subgrade soils are weak or problematic, import of higher quality selected subgrade material 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, import of stronger (CBR>15% or DN ≤ 14 mm/blow, at expected worst moisture condition) subgrade materials can provide savings with regard to the pavement thickness design, although the cross section of the pavement must always allow for effective drainage as discussed in *Chapter 8 – Drainage and Erosion Control*. Where material improvement is necessary or unavoidable, mechanical and chemical stabilisation methods can be considered.

Subgrades are conventionally classified on the basis of the laboratory soaked CBR tests on samples compacted to 93% BS heavy compaction. In their worst condition, the samples are soaked for four days or until zero swell is recorded. However, in most cases, the in-situ moisture regime under the road is likely to be significantly drier than this and the samples should probably be tested at Optimum Moisture Content (OMC), after allowing them to equilibrate in a sealed plastic bag for at least 4 days. Traditionally the subgrade strength for design is assigned to one of five strength classes reflecting the sensitivity of thickness design to subgrade strength. However, this manual will take the in-situ subgrade conditions into account directly in the pavement design.

It would normally be inappropriate to lay a pavement on very weak subgrades (in-situ CBR less than 3% or DN value of > 48 mm/blow, i.e. weaker than S2). However, for unpaved roads to be upgraded, materials of this quality would probably have been replaced or improved over the life of the unpaved road and will not generally be a problem. On new alignments, however, special treatment would be required.

The main aim of the selected subgrade layer is to provide a uniform platform on which to place the sub-base (where needed) and base course. This layer is also used to provide a suitable substrate on which to compact the sub-base and base.

5.6.4 Base and Subbase

Where possible, the in-situ gravel wearing course of the existing unpaved road should be used as the subbase for upgraded roads (or even base if the pavement structure is appropriate). However, it is often the case that the wearing course is too thin and sometimes the case that the material is of inadequate quality.

A wide range of local materials including lateritic, calcareous and quartzitic gravels, river gravels and other transported gravels, or granular residual materials resulting from weathering of rocks can be used successfully as base. Subbase and base materials must satisfy the specifications for the respective design methods as set out above, as well as the specification of maximum particle size. However, under certain circumstances, mechanical treatments 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. For this reason, the borrow pits for base and sub-base materials are usually spaced widely. In current practices, distances of about 50 km between borrow pits are not unusual, but the shorter the haul distances, the better. Main sources of sub-base and base materials are rocky hillsides, high steep hills, and riverbanks.

The minimum thickness of a deposit normally considered workable for excavation for materials for subbase and base is of the order of one metre. However, thinner horizons could also be exploited if there are no alternatives. The absolute minimum depends on material availability and the thickness of the overburden. If there is no overburden, as may be the case in arid areas, horizons as thin as 0.3 m may be economically excavated.

Under conditions of good drainage and when the water table is not near the ground surface, the field moisture content under a sealed pavement will be equal to or less than the optimum moisture content. In these cases, subbase and base materials should thus be tested in the laboratory in an unsaturated state, equivalent to that expected to prevail in the road during normal service conditions.

If the base allows water to drain into the lower layers, as may occur with unsealed shoulders and under conditions of poor surface maintenance where the base is pervious, saturation of the subbase is likely. In these circumstances the bearing capacity should be determined on samples soaked in water for a period of four days.

For the DCP-SN/CBR methods, subgrade Class S6 covers all subgrade materials having a soaked CBR greater than 30 % and which comply with the plasticity requirements for natural sub-base. In such cases, no sub-base is usually required.

5.6.5 Surfacing Aggregate

Aggregates to be used in a bituminous surfacing layer must be durable and strong, should show good adhesion with bituminous binders, and be resistant both to the polishing and abrasion action of traffic. The main qualities for surfacing aggregate are summarised in Table 5-8.

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Table 5-8: Basic requirements for surfacing aggregate

Key engineering property	Material requirement
Strength	Aggregate particles need to be resistant to any loads and abrasion imposed during construction and the design life of the pavement.
Durability	Aggregate particles need to be resistant to mineralogical change and physical breakdown due to any wetting and drying cycles and abrasion imposed during construction or pavement design life.
Skid Resistance (Surface aggregate only)	Aggregate particles must be resistant to polishing. This is usually assisted by having more than one mineral type in the rock.
Adhesiveness	Aggregate must be capable of adhesion to bitumen and sustaining that adhesion for its design life.

Adhesion failure implies a breakdown of the bonding forces between a stone aggregate and its coating of bituminous binder, leading to physical separation. Mechanical failure by fretting and subsequent ravelling of the surface is one possible, but invariable, consequence of adhesion failure. Basic rocks (e.g. dolerite and basalt) are considered to have better adhesion properties than acidic rocks (e.g. granites and quartzites). The comparatively poor performance of acid rocks may not only be related to the high silica content but to the formation of sodium, potassium and aluminium hydroxides. This is considered more likely in feldspathic minerals.

Experience has indicated, for example, that coarse granite containing large feldspar crystals is likely to experience bitumen adhesion difficulties.

Apart from the petrological nature of the material, its cleanliness or freedom from dust is also a factor. Limits of less than 1 % dust (<75 µm) are difficult to obtain by screening alone and washing of the aggregate may be required.

The resistance to abrasion is related to the petrological properties of the material: the proportion of hard minerals; the proportion and orientation of cleaved minerals; grain size; the nature of the interparticle bonding or cementation and the proportion of stable minerals resistant to weathering.

Resistance to polishing is considered a function of material fabric, texture and mineralogy. Rocks which contain a number of minerals of differing hardness and which show a degree of friability tend to give high polishing resistance. Rocks that exhibit a moderate degree of decomposition give higher Polished Stone Value (PSV9 results than fresh un-weathered rocks. There is, therefore, an inverse relationship between polishing resistance and abrasion resistance.

The specification for aggregates for different types of surfacing seals are provided in *Chapter 11 - Surfacing*.

5.6.6 Block or Paving Stone

Paving stones (or blocks or cobbles) can be produced by cutting or breaking large natural boulders. Each stone should be a strong, homogenous, isotropic rock, free from significant discontinuities such as cavities, joints, faults and bedding planes. Rocks such as fresh granite, basalt and crystalline limestone have proven to be suitable materials. Quartzite rock is not suitable, nor is any rock that polishes or develops a slippery surface or abrades under traffic.

The material infilling the spaces between the cobble stones should be a loose, dry, natural or crushed stone material with a particle size distribution equivalent to a well-graded coarse sand to fine gravel. It must be clean and free from clay coating, organic debris and other deleterious materials.

5.6.7 Clay Bricks and Cement Blocks

Burnt clay bricks and concrete blocks are potentially useful surfacing materials. Both of these can provide good riding quality and high skid resistance and are highly labour intensive in their construction. Problems due to poor construction or insufficient support can be easily maintained with only the localised areas showing distress requiring removal and resetting, after correcting the causes of the problems. It is important that the blocks/bricks have adequate strengths and are durable.

5.6.8 Aggregates for Structural Concrete

Concrete aggregate is divided into two parts: coarse aggregate and fine aggregate. The fine aggregate is normally naturally occurring sand, with particles up to about 2 mm in size. The coarse aggregate is normally stone with a range of sizes from about 5 mm to 20 mm (or sometimes larger); it may be naturally occurring gravel, or more commonly crushed or hand-broken quarry stone. In areas without hard stone resources and with an established fired clay brick industry, burnt bricks can be crushed to be used in concrete.

Aggregates must be entirely free from soil or organic materials such as grass and leaves, as well as fine particles such as silt and clay, otherwise the resulting concrete will be of poor quality. Some aggregates, particularly those from salty environments, may need to be washed to make them suitable for use.

Both the coarse and fine aggregates need to contain a range of particle sizes and are mixed together in such a way that the fine aggregates fill the space between the coarse aggregate particles. A ratio by volume of one part fine aggregate to two parts coarse aggregate is generally used. Aggregates can be crushed and screened by hand or by machine.

5.6.9 Filter/Drainage Material

Filter materials have crucial roles in assisting in the prevention or in controlling the ingress of water and in the reduction of pore water pressures within both the earthworks and the pavement. Filter materials can account for a significant proportion of the construction material costs, particularly in wetter regions where road designs need to cater for the dispersion of large volumes of water, both as external drains and as internal layers within wet-fill embankments. The general requirements for filter material are a highly permeable mix comprising a durable aggregate that is resistant to chemical alteration as shown in Table 5-9.

Table 5-9: Basic requirements for filter/drainage materials

Key engineering property	Material requirement(1)
Permeability	The fundamental filter property is primarily a function of material grading. It is generally desirable for filter aggregates to be single-sized and equi-dimensional as this aids flow distribution and facilitates packing. It is also considered better to use material with rounded to sub-rounded rather than angular particles.
Strength	Aggregate particles need to be load resistant to abrasion and any loads imposed by the road design.
Resistance to Degradation	Aggregate particles need to be resistant to breakdown due to wetting and drying and weathering during construction and for the life of the project.
Resistance to Erosion	The as-placed material must be resistant to internal and external erosion.
Chemical Stability	Aggregate should generally be inert and resistant to alteration by groundwater. Weak surface coatings such as clay, iron oxide, calcium carbonate, gypsum etc. are undesirable.
Grading	d_{15} for filter material/d_{15} for adjacent subsoil ≥ 5 with minimum of 50% retained on 2 mm sieve.(2)

(1) Actual requirements will depend on the individual situation and environment.
(2) d_{15} = 15th percentile particle size

5.6.10 Special Materials

Naturally occurring materials
It is often necessary to produce larger rock particles to fill gabion baskets, for rock fill or to provide erosion protection materials. These can be either hand-picked from suitable gravel materials or produced by breaking blasted stone from a quarry, the latter being significantly costlier. Such materials need to be hard and durable with property requirements as shown in Table 5-10.

Table 5-10: Basic requirements for rock used for fill and erosion protection

Key engineering property	Material requirement
Strength	Aggregate particles need to be load resistant to abrasion and any loads imposed by the road design.
Resistance to Degradation	Aggregate particles need to be resistant to breakdown due to wetting and drying and weathering during construction and for the life of the project.
Resistance to Erosion	The as-placed material must be resistant to internal and external erosion.
Chemical Stability	Aggregate should generally be inert and resistant to alteration by groundwater. Weak surface coatings such as clay, iron oxide, calcium carbonate, gypsum etc. are undesirable.

Commercial products
Many commercially produced products are used in road construction. These include products such as lime, cement, bituminous binders, bitumen emulsions, non-traditional stabilisers, etc. These are normally procured from registered manufacturers or vendors and must comply with national or international (e.g. ASTM, BS) standards where no national standards exist.

The use of commercial additives and stabilisers to low volume roads requires careful design. Their use is seldom cost-effective and should be avoided as far as possible.

5.7 Material Improvement and Processing

5.7.1 General

Obtaining materials that comply with the necessary strength requirements for a pavement layer can be difficult. Many of the natural gravels tend to be coarsely graded and relatively non-plastic. The use of such materials on unpaved roads results in very high roughness levels and high rates of gravel loss in service and, in the final analysis, very high life-cycle costs.

In order to achieve suitable wearing course properties a suitable particle size distribution (PSD) can be obtained by breaking down oversized material to a maximum size of 50 mm or smaller. Atterberg limits may be modified by granular/mechanical stabilisation (blending) with other materials. Chemical and mechanical measures may be used to deal with problem soils like loose Kalahari sands or black cotton soils. Material processing/ improvement measures are discussed briefly below.

5.7.2 Reducing Oversize

There are various measures for reducing oversize including the use of labour, mobile crushers, grid rollers or rock crushers. Unless the percentage of oversize material is very high, manual removing of the oversize aggregates during the processing of the pavement layers on site is normally feasible.

Other methods of reducing the oversize material will depend on the type of project and/or material to be broken down:

- Hand labour: This is quite feasible, especially on relatively small, labour-based projects where material can either be hand screened and/or broken down to various sizes and stockpiled in advance of construction.

Chapter 5 – Materials

- Mobile crushers: The crushing of borrow pit materials may be achieved with a single stage crushing unit or, in the other extreme, multi-stage crushing and screening plant.
- Grid rollers: These are manufactured as a heavy mesh drum designed to produce a high contact pressure and then to allow the smaller particles resulting from the breakdown to fall clear of the contact zone, as shown in Figure 5-9. It is, however, essential that the material is re-mixed after breaking down with a grid roller to separate broken particles before compaction commences.
- Rock crusher: The "Rockbuster" is a patented plant item which is basically a tractor-towed hammer mill, as shown in Figure 5-10. The hammer mill action of the Rockbuster will act on the material that it passes over, breaking down both large and small sizes. There is the potential to "over-crush" a material and create too many fines in the product. It may be necessary to windrow out only the larger particles in a material and process these with the Rockbuster, with the crushed material then blended back into the original product.

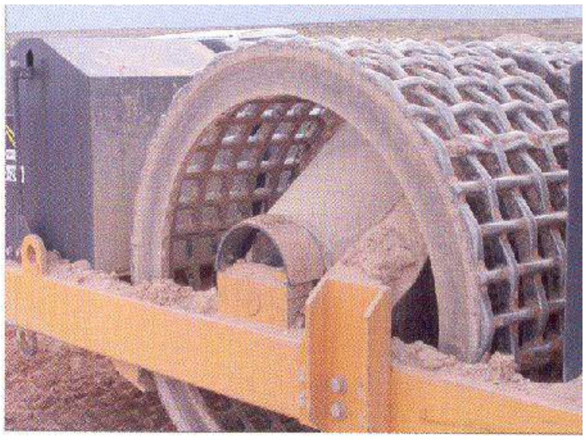

Figure 5-9: Grid roller

Figure 5-10: Rockbuster

5.7.3 Mechanical Stabilisation

Where materials with a suitable grading and/or plasticity are unavailable locally, granular mechanical stabilisation may be possible by undertaking the following:

- Mixing of materials from various parts of a deposit at the source of supply.
- Mixing of selected, imported material with in-situ materials.
- Mixing two or more selected imported natural gravels, soils and/or quarry products on-site or in a mixing plant. Such stabilisation can achieve the following:
  - Correction of grading generally associated with gap graded or high fines content gravels.
  - Correction of grading and increasing plasticity of dune or river-deposited sands which are often single sized.
  - Correction of grading and/or plasticity in crushed quarry products.

The following methodology, using a ternary diagram, as shown in Figure 5-11, has been developed for determining the optimal mix ratio for blending two or more materials to meet the required grading specification for unpaved roads (the optimum grading is shown by the shaded area) but could be applied to improve the grading of any material. The points A and B in the figure shown are an example using two typical soils, the gradings of which are summarised in Table 5-11.

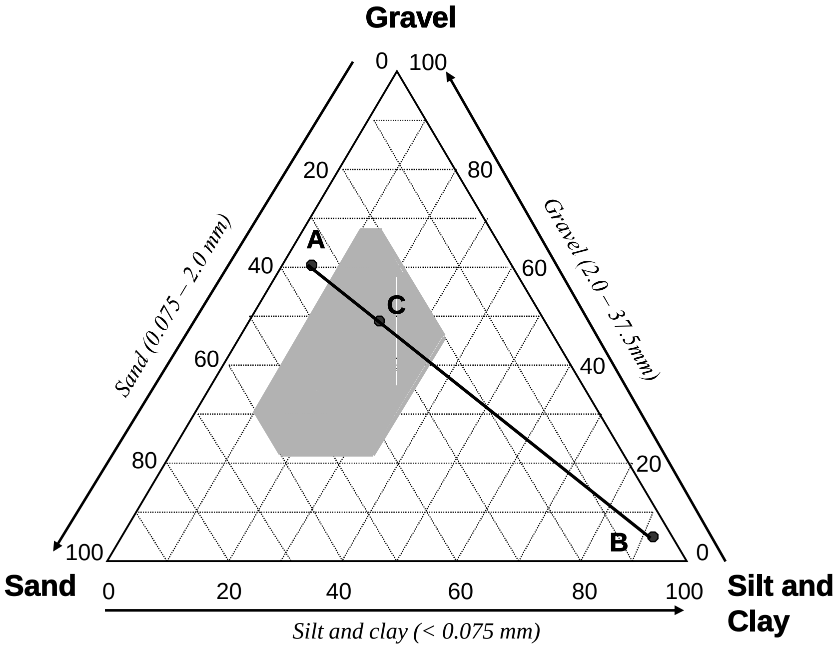

Figure 5-11: Use of ternary diagram for determining proportions during material blending

Table 5-11: Gradings of materials used for blending in Figure 5-14

Parameter	Material	
	A	B
% passing screen size (mm):		
37.5	100	100
26.5	85	100
4.75	49	97
2.0	40	96
0.425	19	94
0.075	5	92
Linear shrinkage	NP	5
Shrinkage product[1]	0	470
Grading coefficient	20	4
% silt/clay (P075)	5	92
% sand (P2 - P075)	35	4
% gravel (P37 - P2)	60	4

[1] Shrinkage product = Bar linear shrinkage x per cent passing 0.425 mm

Blending procedure:

1. Identify potential material sources that can be used to improve the available material.
2. Determine the particle size distribution of the available material and that considered for addition or blending (wet sieve analysis recalculated with 100 per cent passing the 37.5 mm sieve).
3. Determine the percentages of silt and clay (<0.075 mm), sand (0.075 - 2.0 mm) and gravel (2.0 -37.5 mm) for each source.
4. Plot the material properties on the ternary diagram as points A and B respectively (see example in Figure 5-11).
5. Connect the points. When the two points are connected, any point on the portion of the line in the shaded area indicates a feasible mixture of the two materials. The optimum mixture should be at point C in the centre of the shaded area.

6. The mix proportions are then the ratio of the line AC:BC (in this case 9.5 to 37 or 3.9, i.e. 4 loads of material A will be blended with 1 load of material B). This can be equated to truck loads and dump spacing.
7. Once the mix proportions have been established, the Atterberg Limits of the mixture should be determined to check that the shrinkage product is within the desirable range (140 – 400 (or 260 if necessary). The quantity of binder added should be adjusted until the required shrinkage product is obtained but ensuring that the mix quantities remain within the acceptable zone. If the line does not intersect the shaded area at any point, the two materials cannot be successfully blended and alternative sources will have to be located, or a third source used for blending.

5.7.4 Chemical Stabilisation

Where naturally occurring materials that meet the specified requirements cannot be located within economical haul distance from the project site, and where mechanical stabilisation methods cannot be applied to address the deficiencies, chemical stabilisation should be considered.

Stabilisation of gravel with cement or lime is more commonly applied to road bases. The suitability of materials for stabilisation with either cement or lime can be determined from Table 5-12.

Table 5-12: Guide to selection of stabilisation methods

Form of stabilisation	Soil properties					
	> 25% passing the 0.075 mm sieve			< 25% passing the 0.075 mm sieve		
	PI ≤ 10	10 <PI ≤20	PI > 20	PI ≤ 6, PP ≤ 60	PI ≤ 10	PI > 10
Granular						
Cement						
Lime						
Bitumen						
PP (Plasticity Product) = PI x % passing 75 µm						
Key:		Usually suitable		Doubtful		Usually not suitable

The most common type of cement used for stabilisation is CEM-II (or CEM-I if available) and, for lime, hydrated lime is preferred.

Black cotton soils normally have PI much in excess of 20 and, although substitution may be appropriate for small sections, these soils can also be effectively stabilised using lime to reduce the plasticity.

As an alternative to cement or lime stabilisation, bitumen emulsion stabilisation could be considered. However, this is a costly option and is hardly applicable for rural LVRs. In the rare event that this is the most cost-effective option, reference is made to *Chapter 9* for the pavement design.

5.7.5 Recycling

In urban areas, particularly in residential areas, streets which were historically paved have in many cases due to lack of adequate maintenance deteriorated to such an extent that they are now considered unpaved. Providing LVRs in these types of environments should consider the appropriate use of the existing materials, including remnants of paved surfacing, as well as stabilised or natural gravel bases, crushed stone bases, etc. Importation of new materials in the urban environment is constrained not only by sources, but also by limitations to the geometric design defined by kerbs, manholes, property accesses etc. In-situ recycling of materials should therefore be considered allowing for re-use of existing materials to provide a rehabilitated pavement within the lines and levels of the old pavement structure.

The operation of the recycling equipment should be such that material of the required grading characteristics can be created that can be compacted to the design requirements.

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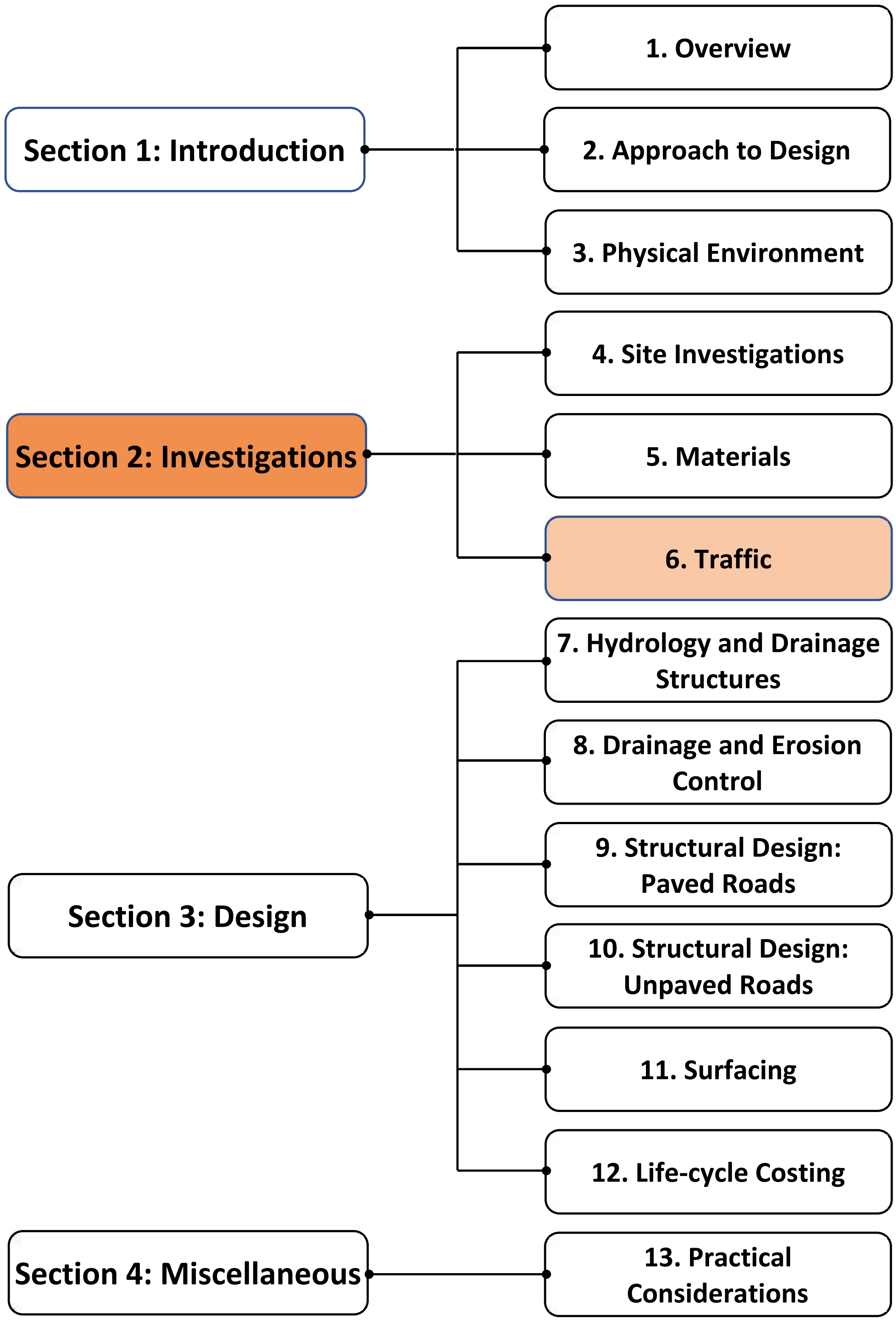

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6.1 Introduction

6.1.1 Background

Reliable data on traffic volumes and characteristics are essential for both pavement structural design and geometric design, and for assisting in the planning of road safety measures as summarised below:

- **Pavement design**: The deterioration of the pavement is influenced by both the magnitude and frequency of individual axle loads. For the structural design of LVRs, a range of Traffic Load Classes (TLC) are defined based on the traffic loading calculated in terms of cumulative equivalent standard axles carried in the specified design life. Thus, each TLC is applicable over a range of traffic levels.
- **Geometric design**: The volume and composition of traffic, both motorized and non-motorized, influence the design of the cross-section (travelled way and shoulders). The geometric design standards (refer to *Volume 2 – Part A*) cater adequately for the traffic volumes expected on LVRs and are modified based on the characteristics of the traffic using the road, such as different traffic mixes including numbers large vehicles, motor cycles, non-motorised traffic (NMTs) and pedestrians.
- **Road safety**: The volume, type and characteristics of the traffic using the road all influence the type of road safety measures required to ensure a safe road environment.

In view of the above, a reliable estimate of the existing (base line) and future traffic volumes is required to undertake the design of the road in an appropriate manner.

6.1.2 Purpose and Scope

The purpose of this chapter is to outline the procedures to be followed in determining the traffic loading over the design life of the road as a basis for designing the road pavement.

The chapter considers types of surveys that provide the inputs for determining the design traffic loading. This requires the data to be sufficiently accurate to select the correct Traffic Load Class (TLC) for structural design from the six classes appropriate to LVRs. Simplified methods of accomplishing this are also described.

6.2 Surveys

6.2.1 General

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

- Classified Traffic Surveys.
- Origin-Destination Surveys.
- Axle Load Surveys.

6.2.2 Traffic Surveys

A classified traffic count is one of the most important items of data for both geometric and pavement structural design as well as for planning purposes in terms of evaluating economic benefits derived from construction of LVRs. For these purposes, it is necessary to ascertain the volume and composition of current and future traffic in terms of motorcycles, cars, light, medium, heavy and very heavy goods vehicles, buses, and, importantly, non-motorised vehicles and pedestrians.

The most common types of surveys for counting and classifying the traffic in each class are:

- Manual Traffic Survey.
- Automatic Traffic Surveys.
- Moving Observer Methods.

Axle load surveys are also required to determine vehicle loading whilst Origin-Destination surveys are sometimes carried out to facilitate the estimation of diverted traffic.

Although the methods of traffic counting may vary, the objective of each method remains the same, essentially to obtain an estimate of the Annual Average Daily Traffic (AADT) using the road, disaggregated by vehicle type. Prediction of such traffic is notoriously imprecise, especially where the roads serve a predominantly developmental or social function and when the traffic level is low.

Usually motorised traffic volumes will decrease in the wet season to, typically, 80% of their dry season level. However, on poor quality roads this difference can be even more marked, and the wet season traffic can decrease to as much as 35% of dry season traffic levels as shown in Figure 6-1. For the purposes of this manual it can be assumed that roads have trafficability problems when wet season traffic levels fall below about 60% of dry season levels. It is also possible that dry season traffic may be lower than wet season traffic, e.g. in areas where sands tend to become loose and less traversable in the absence of ground moisture.

Thus, 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 6-2, short duration traffic counts in low traffic situations can lead to large errors in traffic estimation.

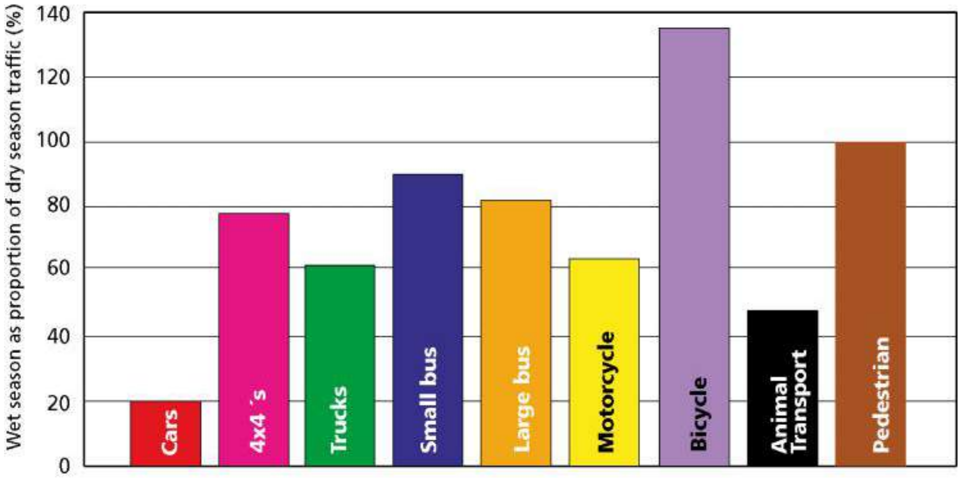

Figure 6-1: Difference in wet season and dry season traffic levels on poor quality roads

(Source: Parsley and Ellis, 2003)

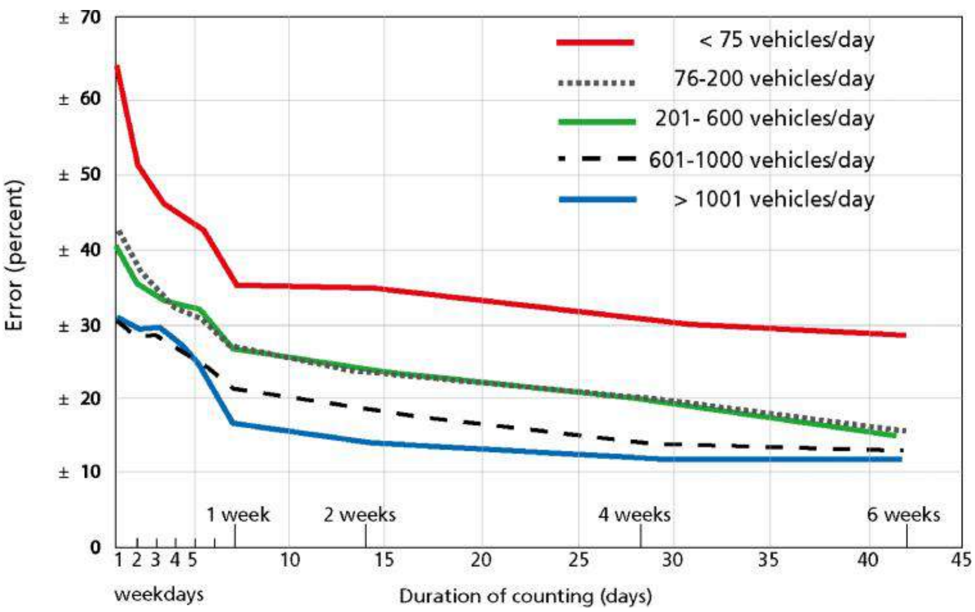

Figure 6-2: Possible errors in ADT estimates from random counts of varying duration

(Source: Howe 1972)

Reducing errors in estimating traffic for LVRs.

Errors in estimating traffic can be reduced by:

- Counting for 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 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 such as within villages or marketplaces should be avoided.

If any junctions occur along the road length, counts should also be conducted before and after the junctions.

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 season

An appropriate, weighted average adjustment will need to be made according to the season in which the traffic count was undertaken and the length of the wet and dry seasons, as illustrated in Figure 6-3.

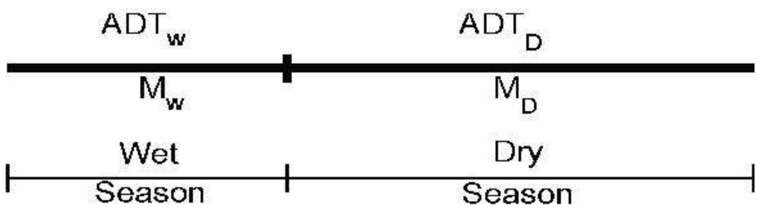

Figure 6-3: Basis for traffic count adjustment in relation to seasonal characteristics

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

Vehicle Classification

Table 6-1 shows the vehicle classification system used for compiling the results of the traffic survey described above.

Table 6-1: Vehicle classification system

Class	Type	Axles	Description	Use
A	Car	2	Passenger cars and taxis	Capacity analysis for geometric design
B	Pick-up	2		
C	4-wheel drive	2		
D	Minibus	2	≤ 28 seats	
E	Medium bus	2	28 – 40 seats	Capacity and axle load analysis for pavement design
F	Large bus/coach	2	> 40 seats	
G	Light Goods Vehicle (LGV)	2	≤ 3.0 tonnes empty weight	
H	Medium Goods Vehicle (MGV)	2 - 5	>3.0 tonnes empty weight	
I	Heavy Goods Vehicle (HGV)	> 6	>3.0 tonnes empty weight	
J	Tractor			Capacity analysis for geometric design
K	Motorcycles, motor cycle taxis			
L	Bicycles			
M	Animal carts			
N	Pedestrians			

6.2.3 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 6-5.

6.2.4 Axle Load Surveys

Axle load surveys provide critical and essential information that is required for both cost-effective pavement design as well as 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 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.

Methods of acquiring axle load data

Axle load surveys can be expensive and are unlikely to be undertaken for an individual LVR project. It is only medium or heavy vehicles that need to be evaluated (classes D, F, G and H plus trailers I, J and K) and they only contribute significantly if they are well loaded rather than nearly empty. The type of load is also important because some materials are of high volume but low density and therefore contribute little to the pavement loading. Roads that are likely to carry lorries transporting timber, quarry products, building materials and other heavy and dense goods will often be overloaded but a road serving a single village is unlikely to carry such vehicles. Thus, the axle loading of vehicles depends on the function of the road and estimating axle loading without the benefit of a representative axle load survey is not straightforward.

Full axle load surveys

In the case that the required resources in terms of funds, equipment and personnel are available and a full axle load survey is to be carried out, the established procedures applied by the Roads Authority should be followed.

Simplified axle load surveys

Assuming that an 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 or light). The number of Vehicle Equivalency Factors (VEFs = ESAs per vehicle) can be estimated based on Table 6-2 and using guidance on types of load in Table 6-3.

Table 6-2 should be modified based on the nature of traffic in the project area. Only fully loaded vehicles will make a significant contribution to pavement damage, except for vehicles carrying dense loads which may be overloaded even when partially full.

Table 6-2: Indicative average VEFs for different vehicle types

Class	Type	Axles	Average VEF per vehicle	
			1. All vehicles loaded	2. Half of the vehicles loaded
A	Car	2		
B	Pick-up	2		
C	4-wheel drive	2		
D	Minibus	2		
E	Medium bus	2	0.8	0.4
F	Large bus/coach	2	1.2	0.6
G	Light Goods Vehicle (LGV)	2	1.0	0.5
H	Medium Goods Vehicle (MGV)	2-5	1.5	0.75
I	Heavy Goods Vehicle (HGV)	> 6	3.5	1.75

The VEFs in Table 6-2 are based on data from the region and are provided for guidance only. The actual VEFs will be derived by the designer based on an axle load survey, if this can be justified based on the size of the project or data provided by the RA.

The axle load surveys would take account of the full spectrum of commercial vehicle axle loads, including overloaded axles, which would be accounted for in the average VEFs used to estimate the design traffic loading.

Table 6-3: Approximate VEF values to be used only when no data are available

Dense goods for which the average VEFs in Column 1 in Table 6.2 apply	Light goods for which the average VEFs in Column 2 in Table 6.2 apply
Quarry products and ore (e.g. gravel, sand etc.)	Household products excluding white goods
Sheet or rod metal	
Bulk liquids	
Logging	
Bulk agricultural products	
Machinery	

6.3 Determination of Design Traffic

6.3.1 General

The procedure for determining the traffic loading for pavement design purposes is summarized in Figure 6-4 and each step explained in the following text. The traffic analysis for pavement design cannot be separated from the analysis for geometric design since the geometric design requirements and ultimately selection of road class and cross-section width will influence the traffic load lane distribution. The analysis for geometric and pavement design purposes should therefore always be carried out together as illustrated in the design example in the Appendix to this chapter.

6.3.2 Procedure

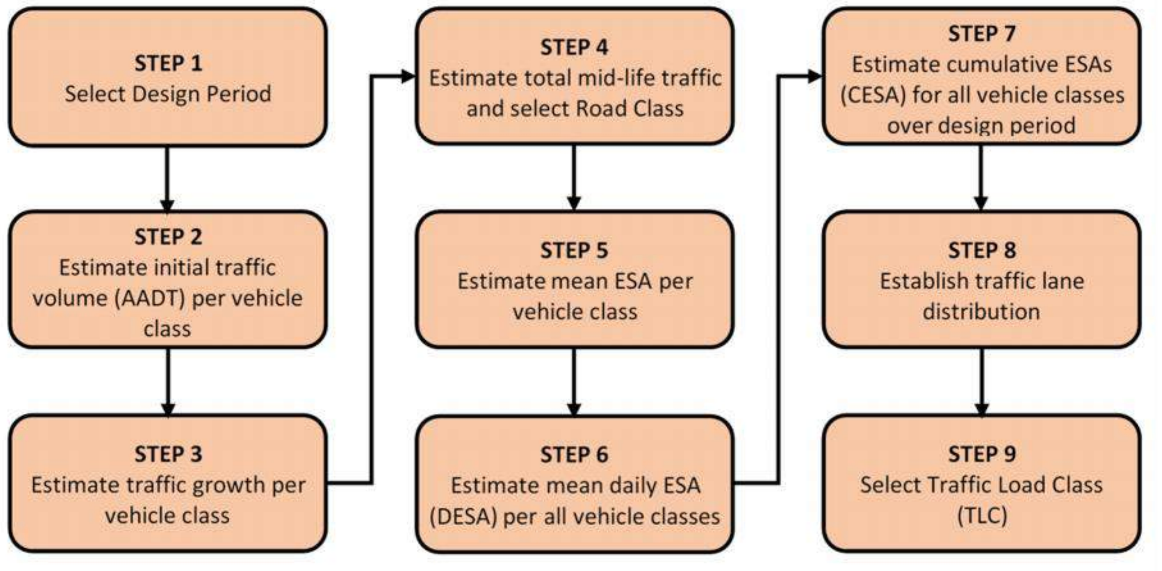

Figure 6-4: Procedure for establishing design traffic class

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. Such a level of terminal serviceability is defined in terms of surface condition as approximately 12 IRI (International Roughness Index) which is equivalent to a roughness of about 10 000 mm/km as measured with a calibrated Bump Integrator or as a Present Serviceability Rating of about 0.5. At such a level of deterioration drivers would rarely exceed a speed of 50 km/hr. The designs described in *Chapter 9 – Structural Design: Paved Roads* and *Chapter 10 – Structural Design: Unpaved Roads* are based on an acceptable level of service provided that adequate maintenance, both routine and periodic, is carried out in order to meet the design life.

Various factors that influence the choice of design period include:

- Functional classification.
- Strategic importance of the road.
- Funding considerations.
- Maintenance strategies (highly trafficked facilities will demand long periods of low maintenance activity.
- Anticipated time for future 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 6-4 provides guidance on the selection of the structural design life. 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 6-4: 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 6.2.2, the initial traffic volume for each vehicle class can be determined. For structural design purposes, it is only the commercial vehicles in classes D to K inclusive (refer to Table 6-2) that will make any significant contribution to the total number of equivalent standard axles.

In contrast, for geometric design purposes it is necessary to count non-motorised and intermediate means of transport including pedestrians, bicycles, motorcycles, tractors and trailers and, possibly, animal transport.

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 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 (refer to Figure 6-5):

- 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 judgement 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 historic precedent can sometimes assist in estimating this, otherwise a rule of thumb is that generated traffic is typically 20% of the existing traffic but it can be considerably higher.

Both diverted traffic and generated traffic occur quickly after the completion of the road.

Estimating traffic growth over the design period is very 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, are 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 the traffic growth, including the following:

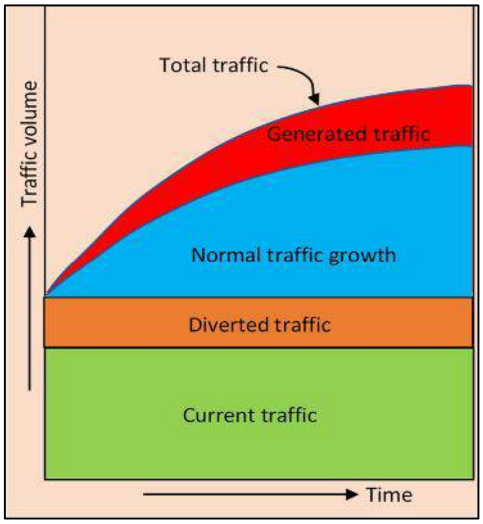

Figure 6-5: Traffic development on an improved road

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

Government predictions of economic growth: Economic growth is closely related to the growth of traffic. Economic growth rates can be obtained from government plans and government estimated growth figures. The growth rate of traffic should preferably be based on regional growth estimates because there are usually large regional differences.

It should be born 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:

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

This is illustrated in Figure 6-6 which shows the multiplier for the AADT in the first year of analysis to obtain the AADT in any other year.

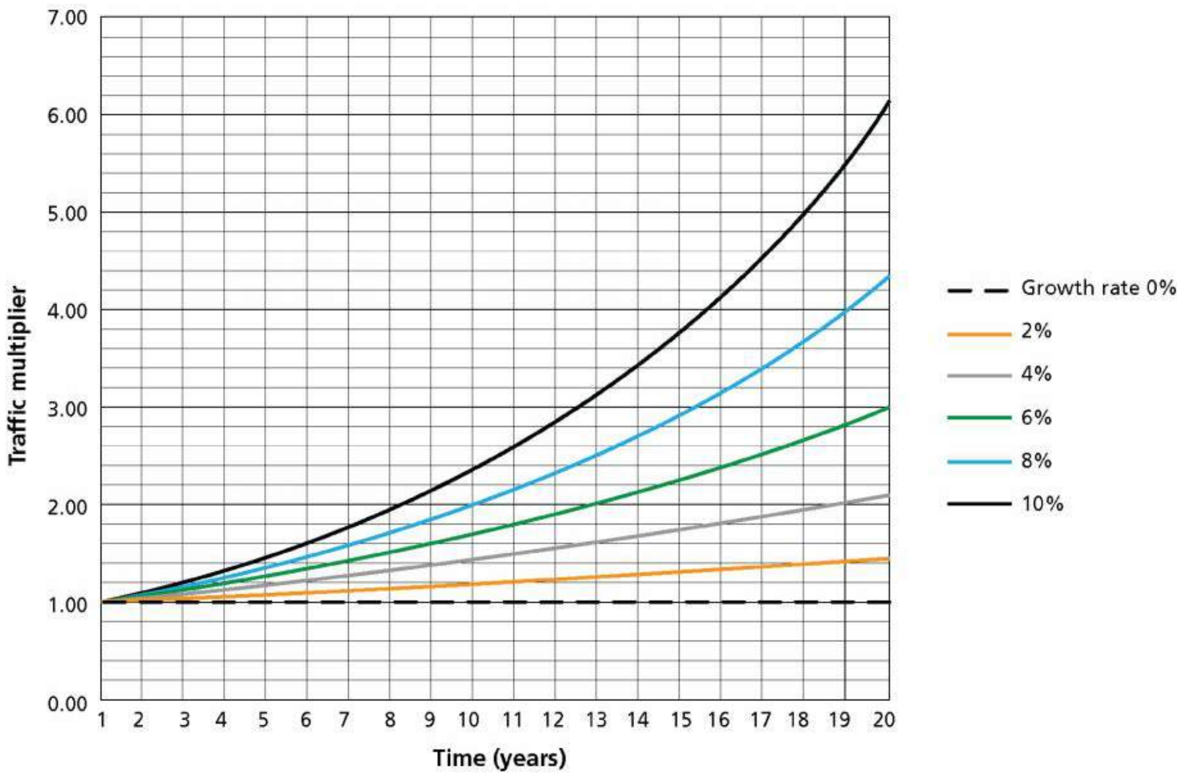

Figure 6-6: Multiplier to obtain AADT in any year for different growth rates

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

This is the last step for geometric design purposes (refer to *Volume 2a: Chapter 4 – Traffic*). The Road Class will determine the cross-section width and influence the traffic lane distribution as shown in Table 6-5.

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 vehicle class 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 vehicles in Class G (HGV) may not all have the same number of axles. The VEF is determined per vehicle class, and not distinguishing between vehicles with 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 suggest 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).

Guidance on the likely average VEF for different vehicle classes is given in Table 6-2. However, 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 2 and the VEFs derived in Step 4 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).

Figure 6-7 shows the multiplier for the CESA in the first year to calculate the CESA after any other number of years up to year 20.

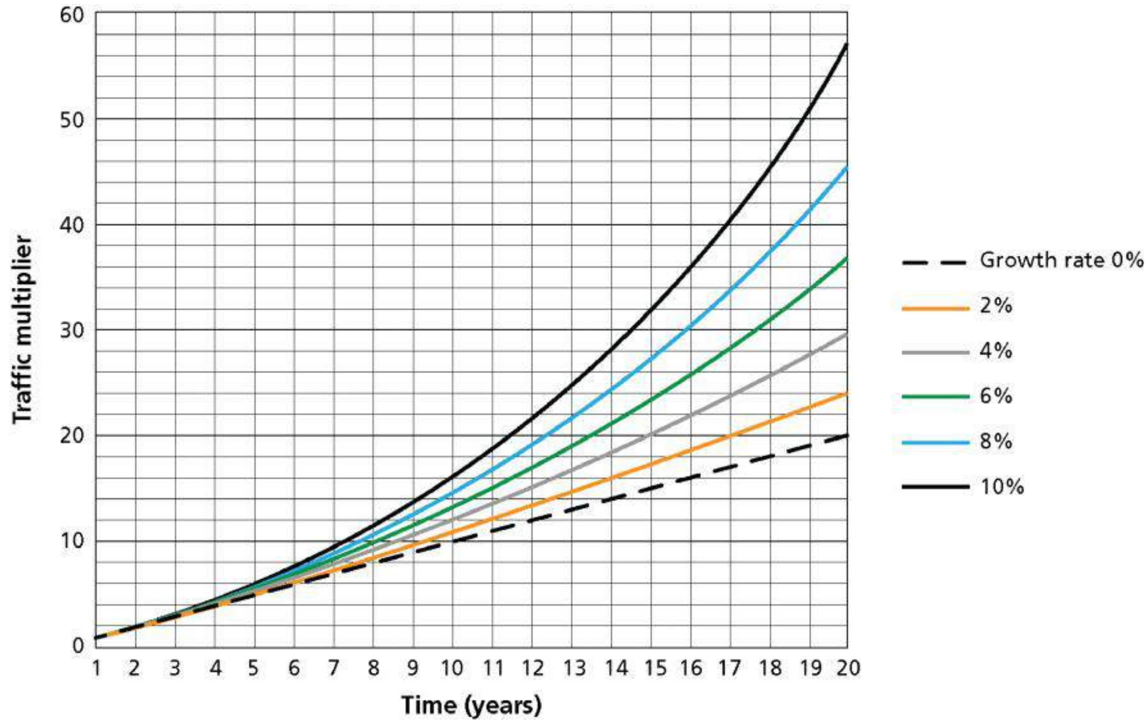

Figure 6-7: Multiplier for the first year CESA to calculate the CESA after any number of years

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 6-5.

Table 6-5: Pavement width adjustment factors for design traffic loading

Cross-section	Paved width	Corrected design traffic loading (ESA)	Explanatory notes
Single carriageway.	< 3.5 m.	Double the sum of ESAs in both directions.	The driving pattern on this cross-section is very channelized.
	Min. 3 m but less than 4.5 m.	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. 4.5 m but less than 6 m.	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 travelled way width of 6.0 m and above, the pavement shall be designed 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 Table 6-6.

Table 6-6: Traffic Load Classes for structural design

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

6.3.3 Sensitivity Analysis

For the final selection of 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, based on the current assumptions, is close to the upper boundary of the TLC and the sensitivity analysis indicates that the upper boundary may be exceeded, it may be prudent to assume the next higher TLC and assess the impact of this on the pavement design. 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.

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Appendix: Traffic analysis

Example of the calculation of AADT for geometric design purposes

The design life for the road is taken as 15 years, and it is required to determine the geometric design class.

The current traffic and corresponding growth rates are shown in Table 6-7.

Table 6-7: Example of traffic count figures and growth rates

Vehicle Classification	Current Number of vehicles	Traffic growth rate up to end of Year 5 (%)	Traffic growth rate from Year 6 to Year 15 (%)
Pedestrians	200	8	8
Bicycles	80	3	2
Motorcycles	40	8	6
Car	12	5	5
Pick-up/4-wheel drive	40	5	5
Small bus	8	5	5
Bus/coach	6	5	5
Small truck (2 axle)	16	7	5
Truck (3 or 4 axle)	10	7	5

The computation of the mid-life motorised traffic is as shown in the Table 6-8.

Table 6-8: Computation of mid-life motorised traffic

Vehicle Classification	Current Number of vehicles	Growth rate for first growth period (%)	Number of vehicles up to end of Year 5	Growth rate for second growth period (Y6 to Y8) (%)	Number of vehicles up to end of Year 8
Car	12	5	15	5	18
Pick-up/4-wd	40	5	51	5	59
Small bus	8	5	10	5	12
Bus/coach	6	5	8	5	9
Small truck (2 axle)	16	7	22	5	26
Truck (3 or 4 axle)	10	7	14	5	16
Total design AADT					**140**

The total design AADT is 140 at the end of year 8, and the geometric design class is therefore LVR3 from Table 5-1 in the *Geometric Design Manual – Part A, Chapter 5 – Cross Sections*, with a carriageway width of 4.5 - 5.5 m depending on the terrain.

The conversion of pedestrians, bicycles and motorcycles into equivalent cars is shown in Table 6-9.

Table 6-9: Example of calculation of Car Equivalents (CE)

Vehicle Classification	Current Number	PCU factors	Current Car Equivalents (CE)
Pedestrians	200	0.15	30
Bicycles	80	0.20	16
Motorcycles	40	0.25	10

The AADT at mid-life (year 8) of the design life of the road is then computed first by computing the vehicle numbers for the first growth period to year 5, then by computing the vehicle numbers for the second growth period from year 6 to year 8 inclusive.

The formula for computing the number of vehicles at end of any given year is:

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

where: $AADT_x$ is the AADT for a given vehicle class in year x

i is the growth rate for a given vehicle class,

x is the number of years from the time at which the traffic count was conducted to the year being considered.

The Car Equivalent values of the non-motorised traffic at the mid-life of the design period can now be computed as shown in Table 6-10.

Table 6-10: Computation of mid-life Car Equivalents of non-motorised traffic

Vehicle Classification	Current Number of vehicles	Growth rate for first growth period (%)	Number of vehicles up to end of Year 5	Growth rate for second growth period (Y6 to Y8) (%)	Number of vehicles up to end of Year 8
Pedestrians (PCU)	30	8	44	8	56
Bicycles (PCU)	16	3	19	2	20
Motorcycles (PCU)	10	8	15	6	17
Total					**93**

Since the sum of CEs in year 8 is not greater than 300, there is no need to adjust the carriageway or shoulder widths of the design that will be achieved by considering motorised vehicles.

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 6-11: 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 Mean ESA per vehicle class

The Vehicle equivalence factors have been determined as follows using n = 4:

Table 6-12: 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 5: 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 = 8.25
- MGV 1 x 4.0 = 4.0
- HGV = 0

Total ESA/day = 15.55 (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 = 0.45
- LGV 5.5 x 0.75 = 4.12
- MGV 1 x 2.0 = 2.0
- HGV = 0

Total ESA/day = 8.97 (direction 2)

Step 6: Cumulative ESA (CESA) for all vehicle classes over the design period

The design CESA can be computed from the following equation:

$$CESA = 365 * DESA * [(1 + r)^N – 1]/r$$

$$= 365 \times (15.55+8.97) \times [(1 + 0.045)^{15} – 1]/0.045$$

$$= 365 \times 24.5 \times [(1.045)^{15} – 1]/0.045$$

$$= 365 \times 24.5 \times [1.935- 1]/0.045$$

$$= 365 \times 24.5 \times 20.78$$

= 185,825 ESA

Step 7: Determine traffic load distribution

From Table 6-5 the traffic loading for design for a 5 m carriageway is 80% of the ESAs in both directions.

Traffic loading = 0.8 x 185,825 = 148,660

= 0.15 MESA

Step 8: Select Traffic Load Class

Traffic Load Class = TLC 0.3

Example of calculation of Vehicle Equivalent Factors (VEF)

The following axle loads were collected during an axle load survey:

Table 6-13: Example of axle load data

Vehicle Type	Vehicle Number	Direction of Travel	Mass of Front Axle (kg)	Mass of 1st Rear Axle (kg)	Mass of 2nd Rear Axle (kg)
Bus/Coach	1	East	2400	4400	
Bus/Coach	2	East	1900	3700	
Bus/Coach	3	West	2100	3800	
Bus/Coach	4	West	2000	3650	
Small Truck (2-axle)	1	East	1460	6700	
Small Truck (2-axle)	2	East	3200	6070	
Small Truck (2-axle)	3	West	3100	5400	
Small Truck (2-axle)	4	West	1600	4600	
Small Truck (2-axle)	5	East	2500	5200	
Small Truck (2-axle)	6	West	2800	4900	
Small Truck (2-axle)	7	West	1600	3100	
Small Truck (2-axle)	8	East	3150	5350	
Small Truck (2-axle)	9	West	3000	5000	
Small Truck (2-axle)	10	West	2800	5150	
Truck (3-axle)	1	West	3350	8400	8600
Truck (3-axle)	2	East	5000	8100	8000
Truck (3-axle)	3	East	4900	8000	7000
Truck (3-axle)	4	East	3400	6700	6400
Truck (3-axle)	5	West	3100	7050	6800
Truck (3-axle)	6	West	3800	8100	8300

The procedure for calculation of the average VEF for each vehicle class is as follows:

Step 1: Compute the equivalent standard axle load (EF) for each axle of each vehicle using the formula:

Equivalence factor EF = (mass of axle in kg/8160)^n,

where n = 4.0 (recommended for LVRs).

Step 2: Sum the EF of all the axles of each vehicle to obtain the vehicle equivalence factor VEF.

Step 3: Compute the average VEF for each vehicle class in each travel direction.

The results are presented in Table 6-14.

Table 6-14: Calculation of VEF

Vehicle Type	Direction of travel	Mass of Front Axle (kg)	Mass of 1st Rear Axle (kg)	Mass of 2nd Rear Axle (kg)	EFs			VEF	Avg. VEF (East)	Avg. VEF (West)
					Front	1st rear	2nd rear			
Bus/Coach	East	2400	4400		0.01	0.08		0.09	0.15	0.09
Bus/Coach	East	1900	3700		0.00	0.04		0.04		
Bus/Coach	West	2100	3800		0.00	0.05		0.05		
Bus/Coach	West	2000	3650		0.00	0.04		0.04		
2-axle truck	East	1460	6700		0.00	0.45		0.45	0.29	0.13
2-axle truck	East	3200	6070		0.02	0.30		0.32		
2-axle truck	West	3100	5400		0.02	0.19		0.21		
2-axle truck	West	1600	4600		0.00	0.10		0.10		
2-axle truck	East	2500	5200		0.01	0.16		0.17		
2-axle truck	West	2800	4900		0.01	0.13		0.14		
2-axle truck	West	1600	3100		0.00	0.02		0.02		
2-axle truck	East	3150	5350		0.02	0.18		0.20		
2-axle truck	West	3000	5000		0.02	0.14		0.16		
2-axle truck	West	2800	5150		0.01	0.16		0.17		
3-axle truck	West	3350	8400	8600	0.03	1.12	1.23	2.38	1.49	1.85
3-axle truck	East	5000	8100	8000	0.14	0.97	0.92	2.03		
3-axle truck	East	4900	8000	7000	0.13	0.92	0.54	1.59		
3-axle truck	East	3400	6700	6400	0.03	0.45	0.38	0.86		
3-axle truck	West	3100	7050	6800	0.02	0.56	0.52	1.10		
3-axle truck	West	3800	8100	8300	0.05	0.97	1.07	2.09		

In the above example, the damage exponent n = 4, which is generally recommended for LVRs, has been used for the calculation of the VEF. As shown in Table 6-1, for natural gravel pavements the range of “n” varies from 3 to 6. A value of 3 may be considered on a well-balanced pavement. The designer should therefore investigate the effect of different n-values on the calculation of VEF and ultimately on the traffic loading over the design life.

For other pavement types, which may be more applicable in peri-urban and urban environments, the recommended n-values from Table 6-15 should be used.

Table 6-15: Recommended damage exponents "n" for different pavement types

Pavement type*		Range**	Recommended
Granular / Granular		3 – 6	4
Granular / Cemented		2 – 4	3
Cemented / Granular	Pre-cracked***	4 – 10	5
	Post-cracked	3 – 6	
Cemented / Cemented	Pre-cracked***	3 – 6	4.5
	Post-cracked	2 – 5	
Hot-mix base / Cemented		2 – 5	4

Source: TRH 4 (1996)

Notes:

* Pavement type defined as "type of base over type of subbase material".

** The higher values of the range usually refers to a fatigue failure mode in the upper regions of the pavement, while the lower values more towards rutting failure mode.

*** Pre-cracked phase of a lightly cemented layer is normally less than 10% of structural life.

Section 3
Design

Low Volume Roads Manual
Volume 1 – Pavement Design

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- 3. Physical Environment

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- 4. Site Investigations
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7.1 Introduction

7.1.1 Background

Hydrology and hydraulic analysis for road drainage design may be defined as the estimation of flood (rainfall) run-off from a catchment for a specified but rare storm and the design of drainage structures of appropriate capacity. It includes the assessment of risks associated with such rare climatic events. The level of risk that is acceptable depends on the consequences of failure of the drainage structure. For trunk roads and for expensive structures such as bridges, limited risk can be tolerated, and so high safety factors and expensive drainage measures are employed. For small drainage structures that can be repaired relatively quickly and easily and for the lower classes of road, higher risks can be tolerated. The challenge for the engineer is to choose a level of protection that is commensurate with the class of road and the structures on it, hence a certain amount of engineering judgement is always required.

7.1.2 Purpose and Scope

This Chapter is concerned with hydrology and the process of determining the quantity of water that the drainage design must cope with. When risk factors have been selected and the volume of water that must be catered for has been determined, then the individual features of the drainage can be designed using hydraulic design principles.

The Chapter deals primarily with new structures such as drifts, culverts and small span bridges. However, the design principles are the same for reconstruction, rehabilitation, extension and upgrading of existing structures. The structures are ranked in order of increasing complexity as follows:

- Drifts or simple fords
- Simple culverts
- Vented drifts
- Large diameter culverts
- Small bridges

The Chapter does not cover aspects of road drainage such as drainage ditches and sub-surface drainage. These are addressed in *Chapter 8 – Drainage and Erosion Control*.

7.2 Design Storm

7.2.1 General

The first step in the determination of the flow of water that the drainage system needs to cope with depends on the severity of the design storm. The risk of a severe storm occurring is defined by the statistical concept of its likely return period. This is directly related to the probability of such a storm occurring in any one year. Thus, a very severe storm may be expected, say, once every 50 years, but a less severe storm may be expected every 10 years. This does not mean that such storms will occur on such a regular basis. A severe storm expected once every 50 years has, on average, a probability of occurring in any year of 1 in 50 based on historic rainfall data.

Rainfall data for the last 50 years or so are usually available for most of Malawi hence estimates can be made of the rainfall in quite rare storm events. In areas where the data are only available for a relatively short period of less than 10 or 20 years (i.e. insufficient for estimating storms with return periods of, say, 25 years or more) an estimate can be made from the more limited data using the adjustment factors shown in Table 7-1 based on a normalized storm return period of 10 years. Estimating the 100-year storm from, say, less than 5 years data will not be very accurate. A minimum of 10 years is recommended. From the Table, a storm with a return period of 20 years will provide 1.15 times more water than the storm with a return period of 10 years.

Table 7-1: Adjustment factors for different storm return periods

Return period (years)	1	2	5	10	12.5	20	25	50	100
Adjustment factor	0.55	0.7	0.85	1.0	1.05	1.15	1.25	1.45	1.6

7.2.2 Design Storm for Different Structures

The indicative design storm return period for different structures is shown in Table 7-2. However, there are a number of situations where the design storm could be more severe. Principal routes such as access roads to local markets or emergency routes to a nearby hospital will require higher levels of reliability and shorter periods of closure caused by high water levels hence the design storm return period chosen should be longer (i.e. more severe). The proximity and distance of an alternative route will also affect the choice of design storm (and drainage structure). If there is an alternative secure route with a short acceptable detour, this will allow the road to be closed for longer periods whereas the lack of any alternative route (or one of excessive length) will require a more conservative design. Therefore, the choice of storm design period requires careful engineering judgement and local consultations. Table 7-3 should be used when less risk can be tolerated.

Table 7-2: Indicative storm design return period (years) for different structures

Type of drainage structure	Geometric design standard			
	LVR5/4	LVR3	LVR3	LVR1
Gutters and inlets	2	2	2	1
Side ditches	10	5	5	2
Drift or vented drift (1)	10	5	5	2
Culvert diameter <2 m	15	10	10	5
Large culvert diameter >2 m	25	15	10	5
Gabion abutment bridge	25	20	15	-
Short span bridge <10 m	25	25	15	-
Masonry arch bridge	50	25	25	-
Medium span bridge (10 m-50 m)	50	50	25	-
Long span bridge >50 m	100	100	50	-

Source: Adapted from international review

Note: A drift and a vented drift are designed to be overtopped safely, hence the design is usually based on higher level of risk (shorter storm return period) than for culverts.

Table 7-3: Storm design return period (years) for severe risk situations

Type of drainage structure	Geometric design standard			
	LVR5/4	LVR3	LVR2	LVR1
Gutters and inlets	5	5	5	2
Side ditches	15	10	10	5
Drift or vented drift	15	10	10	5
Culvert diameter <2 m	25	20	20	10
Large culvert diameter >2 m	50	25	20	10
Gabion abutment bridge	50	25	20	-
Short span bridge <10 m	50	50	25	-
Masonry arch bridge	50	50	25	-
Medium span bridge (15 m-50 m)	100	100	50	-
Long span bridge >50 m	100	100	100	-

Source: Adapted from international review

If the maximum water flow cannot be reasonably estimated, it may be necessary to provide a structure that can be over-topped during periods of unpredicted water flow.

7.3 Methods of Design

7.3.1 General

Water crossing structures must be designed to have a capacity equal to or greater than the maximum water flow that is expected in the water course. Structures not designed for overtopping must have an acceptable freeboard. This maximum flow depends on the characteristics of the storm itself, namely the intensity, duration and the spatial extent of the rainfall, and the characteristic of the ground, or catchment, on which the rainfall falls.

The area of the drainage catchment (A) should be estimated from topographical maps or through the use of aerial photographs. The following methods are used for estimating maximum flow in a watercourse:

- Direct observation of the size of watercourse, erosion and debris on the banks, history and local knowledge.
- Replicating successful practice.
- The Rational Method – for estimating peak discharges for small drainage areas up to about 100 hectares (larger catchments can be considered using modified Rational Methods, for example, using the areal reduction factor shown in Equation 7-1 below.

7.3.2 The Direct Observation Method

The cross-sectional area of the watercourse and the high-water levels at the design storm level are required and the cross-sectional area of the apertures of the structure should then be designed to be equal to that of the storm design flow.

It may be possible to observe previous high-water marks from existing structures, trees or other vegetation near the watercourse. Small debris floating down the river will be caught on branches and twigs during floods and indicate the water level during a flood. The highest flood is the most likely to be visible because it will often obliterate evidence of smaller flood tide marks. The problem is that there is often no indication of how old the flood level indicators are and hence what the return periods will be. The evidence of higher floods in the past may have been removed by natural weathering. This method will therefore give an indication of a recent high flood level, but it does not guarantee to be the highest expected flood level. The information gathered by observation may be supplemented by interviews with local residents, especially the more senior members of the community.

If there are people living near the proposed crossing point it will be possible to ask them how high the water level has risen in previous floods. A number of people should be questioned as memories 'fade' over time. It may be possible to ask people individually how high the biggest flood had been over the previous years and then take an average of the results obtained. Validation may be improved if enquiries are made for each riverbank independently and for different locations along the banks that provide information that can be correlated. Alternatively, a group may be asked to collectively agree the maximum height of the floodwater. It will also be necessary to ask how often floods of the maximum size occur in order to determine the return period.

7.3.3 Replicating Successful Practice

If a high proportion of structures along a road or in a region have been in operation for a number of years without overtopping, it is reasonable to assume that the relationship between catchment area, catchment characteristics, rainfall intensity and maximum water flow used in their design is valid. The design of new structures can be based on simply the catchment area using the same relationships.

7.3.4 The Rational Method

The flow of water in a channel, Q, is calculated from equation 7-1.

$Q = 0.278 \times C \times I \times A$ (m^3/s)

Where:

C = the catchment run-off coefficient

I = the intensity of the rainfall (mm/hour) for the T_c (time of concentration of the catchment area)

A = the area of the catchment (km^2)

The Catchment Run-off Coefficient "C" is obtained from Table 7-4.

Table 7-4: Run-off coefficient

C_T (slope-topography		C_S (soils)		C_V (vegetation	
Very flat (<1%)	0.03	Sand and gravel	0.04	Forest	0.04
Undulating (1-5%)	0.08	Sandy clays	0.08	Farmland	0.11
Hilly (5-10%)	0.16	Clay and loam	0.16	Grassland	0.21
Mountainous (>10%)	0.26	Sheet rock	0.26	No vegetation	0.28
Runoff coefficient $C = C_T + C_S + C_V$					

The intensity of rainfall (I) is obtained from Intensity-Duration-Frequency (IDF) charts usually developed by the Meteorological Department. Such charts vary across the country and, where locally available, should ideally be used. However, in many situations such charts may not be available (because rain stations often measure only the rainfall in 24 hours), hence the engineer will need to rely on less accurate data.

The time taken for water to flow from the farthest extremity of the catchment to the crossing site is also required. This is called the Time of Concentration (T_c) and the duration of the storm must be set equal to this value because this will give the maximum flow rate "Q".

T_c = Distance from farthest extremity (m) / Velocity of flow (m/s)

The velocity of flow depends on the catchment characteristics and slope of the watercourse. It is estimated from Figure 7-1.

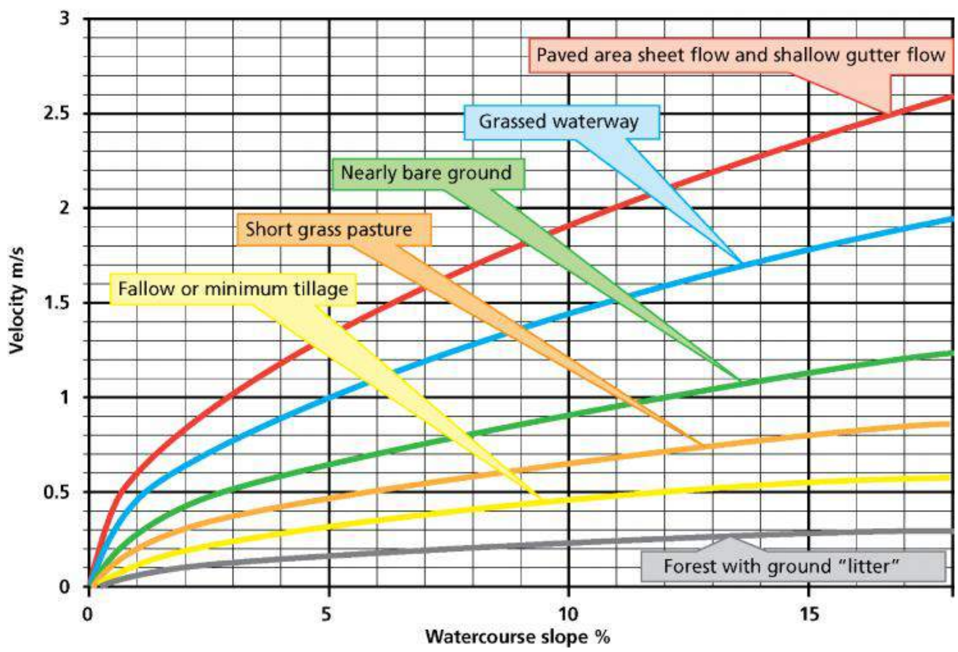


Figure 7-1: Velocity of flow for varying surface cover

The storm design return period is taken from Table 7-2 or Table 7-3.

In the Rational Method it is assumed that the intensity of the rainfall is the same over the entire catchment area. The consequence of applying the method to large catchments greater than 80 hectares is an over-estimate of the flow and therefore a conservative design.

A simple modification can be made to take into account the spatial variation of rainfall intensity across a larger catchment. The effective area of the catchment is reduced by multiplying by the areal reduction factor (ARF) given by the following equation:

ARF = 1 - 0.04 x t – 1/3 x A x 0.5

Where:

t = storm duration in hours

A = catchment area in km^2

This modification allows the catchment size limit to be increased considerably. Secondly, in rural areas where the catchment is often relatively simple (simple in terms of the complexity of the ground cover) the accuracy of the method is increased. However, the run-off coefficient also depends on the existing moisture conditions in the soil and on the storm intensity. When the ground conditions are wet or the storm intensity being used for design is high, the effective value of the runoff coefficient will increase considerably, compensating to some extent for the reduction in run off caused by a larger catchment area.

7.4 Flow Velocity

7.4.1 General

It is also important to determine the velocity of the water flow during peak flows because this affects the amount of scour that can be expected around the structure and hence the protective measures that may be required. The velocity can be measured in two ways.

7.4.2 Direct Observation in Flood Conditions

An object which floats, such as a small stick, should be thrown into the flow upstream of the potential structure. The time it takes to float downstream a known distance (e.g. about 100m) should be measured. The velocity can then be calculated by dividing the distance the floating object has travelled by the time taken. This exercise should be repeated at least 3 times, but preferably 5 times, to get an accurate result. Tests where the floating object is caught on weed or other debris in the water should be discarded. The opportunities for making such observations during flood conditions are obviously very limited.

7.4.3 Manning's Equation

Design volumes of run-off in side drains and other channels can be estimated using the Rational Method. The cross-sectional area of the drain must be sufficient to accommodate the expected flow of water "Q", where:

Q = A x V

The flow velocity is calculated from the Manning equation:

$V = 1/n \times R^{2/3} \times S^{1/2}$

Where:

V = cross sectional average velocity in m/s
A = cross-sectional area of water (m^2)
R = hydraulic depth (area for the stream flow divided by the wetted perimeter) (Figure 7-2).
S = hydraulic gradient (slope of the drain or watercourse)
n = roughness coefficient (Table 7-5)
Q = discharge volume flow rate (m^3/sec)

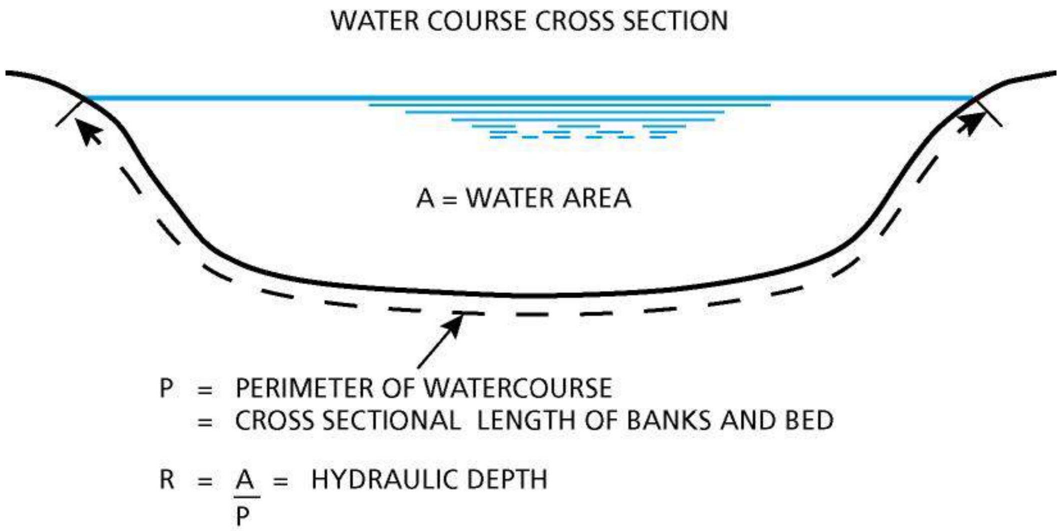


Figure 7-2: Definition of hydraulic depth in Manning's Formula

Table 7-5: Roughness coefficient (n) for drains

Material in the drain	Roughness coefficient
Sand, loam, fine gravel, volcanic ash	0.022
Stiff clay	0.020
Coarse gravel	0.025
Conglomerate, hard shale, soft rock	0.040
Hard rock	0.040
Masonry	0.025
Concrete	0.017

7.5 Water Crossings and Associated Structures

7.5.1 General

The policy of the road or local government authorities is invariably to provide as much all-weather access to as many people as possible, hence priorities need to be set. When selecting which roads and which potential structures to address and the type of structure to be constructed, the spot improvement and staged construction approach should be used, and a cost benefit analysis carried out. A typical provision rate for culverts in rolling terrain is about 2 or 3 per km. In severe terrain or in flat, floodable areas, the frequency will be higher. The cost of providing adequate cross drainage is therefore an important component of road costs.

7.5.2 Type of Structure

The greatest potential cost savings for water crossing options is in the choice of structure type. This Chapter considers different water crossing options, from drifts to small bridges with spans of <10m, explaining the characteristics of each, the conditions suitable for their use and the advantages and disadvantages associated with each structure.

The design of each structure to cope with the design storm water flow depends on the type of structure and its location and is described in the relevant Sections below. In general, for any structure that might constrict the water flow, e.g. culverts, the size of the aperture through which the water flows should be sufficiently large to prevent water from backing up on the upstream side. This is particularly important in locations where the water flow is relatively fast and therefore able to cause serious erosion and scour if not adequately controlled.

At the most basic level, a simple "drift" can be created in a stable sandy bed of an occasional watercourse by burying stones of 15 cm – 30 cm size just below the surface and covering them with sand. This substantially improves the bearing capacity for vehicles.

While structure types are presented individually it should be remembered that, for alluvial plains, a combination of a number of structures, perhaps constructed under a staged construction approach, might be the most cost-effective means of spanning the watercourse.

7.5.3 Location of Structures

The following questions should be asked:

- Is the crossing site on a curve in the water course? If so, the water course may move outwards and damage the structure. If the road crosses the water course at an angle of 90°, construction will be cheaper and there will be a reduced risk of erosion and damage.
- Are there signs of erosion or sedimentation? If so, it is probable that these will get worse and damage or block the structure.
- Has the water course strong bedding material? If so, there is less risk of erosion and undercutting.
- Does the water course flood the surrounding land? If so, an embankment will also be needed across the area.
- What are the properties of the materials on the approaches to the water course?
- Is the water course in a deep valley? If so, the approaches can be very steep and at risk of erosion and failure.

After considering these questions, the engineer may decide that it is cheaper to realign the road to a more suitable site.

7.5.4 General Considerations

Traffic and vehicle loading should also be considered when choosing the type of structure. Some types of structure are particularly prone to damage caused by grossly overloaded vehicles at some locations. For example, the durability of corrugated steel or concrete pipe culverts on unpaved roads will be greatly affected when overloaded because loss of fill over the culvert pipe will occur with eventual direct loading for which the culvert pipe was not designed and cannot withstand.

If the loading capacity is limited in any way, signage should be provided to clearly state the loading capacity of the structure. Local road network managers and administrators should also be made aware of any load limitations and the likely consequences of these being exceeded.

If it is not possible, with the resources available, to construct a crossing that will withstand the largest vehicle that could travel down the road, it will be necessary to install a robust non-removable barrier each side of the structure to prevent overloaded vehicles from crossing.

When the structure is being designed, the size (dimension) of the vehicle should also be taken into consideration to ensure that it can safely cross the structure without damage to the vehicle or structure. The width of a structure substantially influences the initial construction cost. For bridges the cost is roughly proportional to deck area and for culverts, roughly proportional to the length of the barrel.

Widening most structures in the future is often an expensive process hence, with potentially high traffic growth rates, a vital decision is required concerning whether the structure is to be designed for one or two-way traffic flow. It is probable that two-way traffic for bridges will only be justifiable for the highest category of LVR although local conditions may override this. The secondary decision is with respect to the safe width for the predominant traffic type. These decisions become more important with the increasing size of the proposed structure.

It should be noted that a culvert or other drainage structure is required at all low points of the road alignment. The cost of their provision is usually significant in the overall cost of a low volume road, particularly for unpaved roads. The frequent occurrence of culvert headwalls and width narrowing, and the difficulty for drivers to see them in advance, particularly when travelling at night, raises important safety issues. The provision of minimum two-lane width culverts can therefore be justified in most cases (except for the most severely constrained projects). Culvert headwalls should be set back behind the carriageway and shoulder, and clearly marked or have guideposts at each end of the culvert to prevent vehicles driving into the inlets, outfalls or ditches when passing on-coming traffic. These requirements may be relaxed to provide only clear carriageway width in slow speed mountainous alignments.

For larger structures restricting access to one lane is justified. For single-lane motor vehicle traffic the clear carriageway width (between kerbs, parapets, guardrails or marker posts) should be a minimum of 3.75 m. This width should allow easy single way traffic but clearly restrict two vehicles from passing on the structure at the same time. To accommodate motorcycles as well as a vehicle a minimum of 4.5 m is required. In view of the rapid increase in motorcycle traffic that is occurring it is usually prudent to use 4.5 m.

Where justifiable, full two-lane motor traffic provision should allow a minimum of 6.5 m between kerbs provided that vehicles are restricted to slow speed passage.

Where physical restrictions are necessary to prevent passage of heavy good vehicles these will need to limit free passage to about 2.3 m. This requires a clear indication that the roadway narrows (advance warning signs), as well as very clear signage at the road intersections on each side of the road section in question, that there are size limitations on the stretch of road.

7.5.5 Design for Climate Resilience

Climate change will affect roads and highways in many ways. The accepted characteristics, amongst others, are higher temperatures, higher rainfall, more intense storms and more frequent storms. This will lead to the need to cope with generally more water, more frequent floods, and faster and more destructive water velocities. Thus, much of the historic data on which hydrological analysis and hydraulic design relies, may lead to an under estimation of design floods and high-water levels. Until new flood models are developed and verified, one of the simplest and important actions that can be taken is to design drainage structures based on estimates of storm characteristics with currently higher return periods as in Table 7-3 for severe risk situations. This is essentially increasing the safety factor. In addition, there are various other strategies that will help to increase climate resilience. In general, these comprise:

- Identifying the most vulnerable areas and essentially increasing the ‘safety factor’ inherent in their design.
- Ensuring that the drainage systems are well maintained and functioning correctly.
- In critical areas or high priority roads where the consequences of failure and closure are more severe, local realignment, if appropriate, may be required, but this will usually only be considered as part of an emergency repair, rehabilitation or upgrading project after storm damage has occurred.

Increasing the safety factor includes:

- using drifts and vented drifts that can be safely overtopped instead of culverts that can often become blocked by debris;
- adding additional protection to culverts that might be blocked by debris;
- better surface drainage so that water is dispersed off the road more frequently;
- reducing water concentration by means of additional cross drains and mitre drains to lower the volume of water that each one needs to deal with (ref. *Chapter 8 – Drainage and Erosion Control*).

Erosion is a serious problem in many areas and adverse climate change and deforestation will make matters worse. Erosion is discussed in *Chapter 8 - Drainage and Erosion Control*. There are also likely to be more severe geotechnical problems (e.g. slope stability) caused by climate change and these are dealt with in *Chapter 4 – Site Investigations*.

Ensuring that the drainage system is working correctly is essentially a maintenance issue although there will be examples of poorly designed culverts with improper alignment or grade relative to the channels and ditch lines that will need to be repaired or replaced, usually after failures have occurred.

7.6 Low-Level Water Crossings

7.6.1 General

A low-level water crossing is simply one that is designed to be over-topped. The most common is a drift which is constructed from stones or concrete. The simplest is a ford or splash which consists of unbound hand packed stone. Drifts and fords are often dry and cater for relatively low water flows. For higher water flows of longer duration, a vented drift is more appropriate. This is essentially a drift constructed on top of a series of culverts thereby allowing considerable water to flow before being over-topped only during severe storms. For such structures that are designed to be overtopped, the culverts should be designed in the normal way using the nomographs shown in Section 7.7.4. The ability to be over-topped with little risk of failure is a relatively inexpensive way to reduce risks, especially for wide river crossings subject to unpredictable flash floods.

7.6.2 Drifts

A drift consists of a flat slab and two inclined approach ramps over which water and vehicles can pass, thus a drift carries water over the road. Drifts are the cheapest form of watercourse crossing. They are also referred to as Irish bridges, fords or splashes. The terms describe essentially the same structure, but it is generally accepted that a ford or splash is constructed from the existing riverbed whereas a drift is a ford or splash with an improved running surface constructed from imported materials.

Drifts are suitable for shallow water courses with a gentle gradient and at sites where raising the road over a culvert would require the transport of large quantities of earth.

There are two types of drift:

Relief drifts: These relieve side drains of water where the road is on sloping ground and water cannot be removed from the uphill side drain by mitre drains. It is an alternative to a relief culvert.

Small watercourse (or stream) drifts: Where stream flows are very low with normal water depth of less than 200 mm) drifts may be used to allow the stream to cross the road, as illustrated in Figure 7-3.

7.6.3 Key Features of Drifts

The key features of drifts are:

- Stream drifts are structures which provide a firm place to cross a river or stream. Relief drifts transfer water across a road without erosion of the road surface. Water flows permanently or intermittently over a drift. Therefore, vehicles are required to drive through the water in times of flow.
- Drifts are particularly useful in areas that are normally dry with occasional heavy rain causing short periods of floodwater flow.
- Drifts provide a cost-effective method for crossing wide rivers which are dry for the majority of the year or have very slow or low permanent flows.
- Drifts are also easier to maintain than culverts and will also act as traffic calming measures.

- Drifts are particularly suited to areas where material is difficult to excavate, thus making culverts difficult to construct.
- Drifts are also particularly suited in flat areas where culverts cannot be buried because of lack of gradient.
- The drift approaches must extend above the maximum design flood level flow to prevent erosion of the road material.
- If necessary, guideposts must be provided on the downstream side of the drift and be visible above the water when it is safe for vehicles to cross the drift.
- Buried cut-off walls are required upstream and downstream of the drift to prevent under cutting by water flow or seepage.
- The approach road level will normally mean that approach ramps are required. Approach ramps should be provided to the drift in the bottom of the watercourse with a maximum gradient of 10% (7% for roads with large numbers of heavy trucks).
- Drifts should not be located near or at a bend in the river.
- Some form of protection is usually required downstream of a drift to prevent erosion.

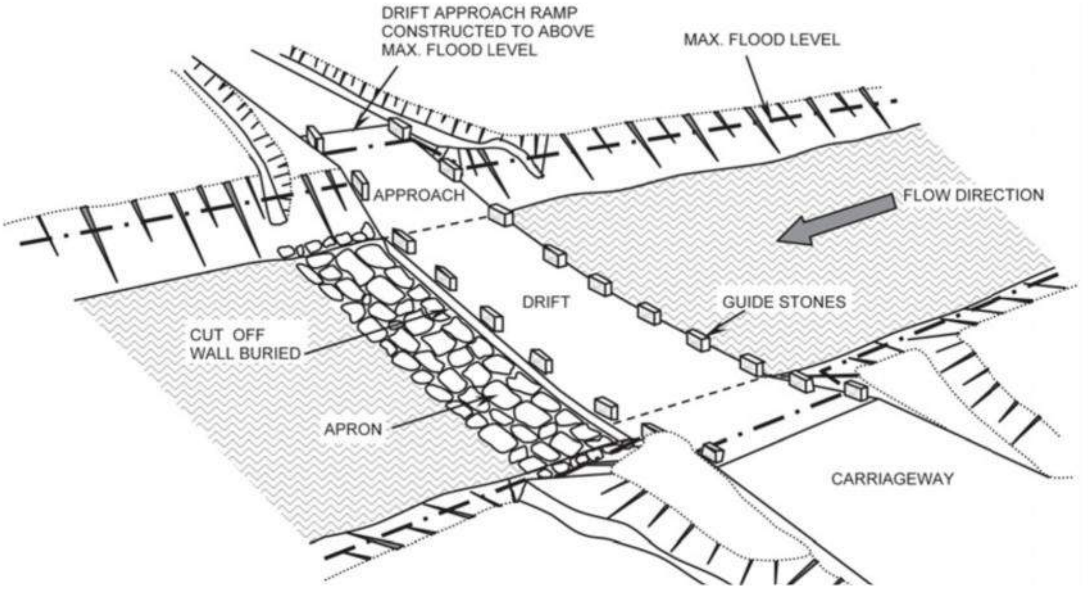


Figure 7-3: Key features of a stream drift

Posts are fixed to indicate the water level and edges of the drift when water is flowing over it. It is also possible to construct guide blocks along the sides of the drift to help pedestrians to pass when water is flowing. Stones are placed in the water course (apron) to prevent erosion downstream and upstream of the drift.

Drifts should be constructed with a shape as close as possible to the shape of the existing water course. The slab should be at the same level as the bed of the water course and road cross fall should not be more than 2%. If the riverbed gradient is more than 2%, a stepped structure (cascade) should be used at the outlet. The extent of the drift is normally calculated using the Observational Method. Drifts should cover the entire width of the water course when water is flowing. It is also possible to estimate the width and depth of the drift using the Rational Method. The ramps should extend at least one metre beyond the required high-water point. The cost of a drift is normally estimated per metre length.

The slab and the ramps should be durable and non-erodible. They can be made from hand placed stone or concrete. When concrete is used, it should be reinforced with steel mesh (6 mm bars laid in a 150 mm - 200 mm grid). Construction joints should be provided so that each slab is no more than 5 metres long.

Vehicles that pass over a drift can spread water on the approach roads. This can make the road surface slippery or suffer from erosion, especially if the approach road is steeper than 5% and if the water flows over the drift for more than two days after rain. In this case, the road should be gravelled or provided with an improved surface for 50 metres in each direction.

The following criteria should be considered when designing drifts:

- The level of the drift should be as close as possible to the existing riverbed level.
- The normal depth of water should be a maximum of 200 mm.
- Approach ramps should have a maximum gradient of 10% (7% for roads with large numbers of heavy trucks).

7.6.4 Advantages and Disadvantages of Drifts

The advantages and disadvantages of drifts are summarized in Table 7-6.

Table 7-6: Advantages and disadvantages of drifts

Advantages	Disadvantages
• Low cost: at the most basic level - can be constructed and easily maintained entirely with local labour and materials. • Volume of excavated material in most cases is small. • Drifts do not block with silt or other debris carried by floodwater. • They can accommodate much larger flows than culverts. • Easier to repair than culverts. • Water flows over a wide area, resulting in less water concentration and erosion downstream than piped culverts.	• The crossing can be impassable to traffic during flood periods. • Foot passage can be inconvenient or hazardous when water is flowing. • Drifts require vehicles to slow down when crossing. This could be considered an advantage because of the traffic 'calming' effect.

7.6.5 Splash

A splash is a type of low-cost drift consisting of a shallow channel protected against erosion passing across a track. Splashes are recommended for low water volumes. The surface of the channel is protected by a material which is low cost and non-erodible, such as a layer of flat stones. Most low-cost surfaces are porous and should not be used if the water flows for more than three hours after rain.

When the water course is flowing, all the water should pass between the edges of the splash to prevent damage to the surface of the track. Normally the maximum length (in the direction of the road) of a splash is 5 metres.

Splashes can also be used to reduce the flow in a side drain, in the same manner as a relief culvert.

7.7 Culverts

7.7.1 General

Culverts are usually constructed in narrow well-defined water courses, but they can also have many apertures in order to cross wide and shallow water courses. Culverts perform two basic functions.

Relief culverts

These are placed at low points in the road alignment (where there is no definable stream) or along long downhill gradients, but the topography of the ground requires a significant amount of cross drainage which cannot be accommodated by side drains, as illustrated in Figure 7-4. A relief culvert should be located at the point where the high volume of water starts to cause erosion or the drain to overtop. Relief culverts should be used only when solutions such as regular drain clearing to maintain and ensure maximum flow and use of a mitre drain are not possible.

Stream culverts
These allow a watercourse to pass under the roadway.

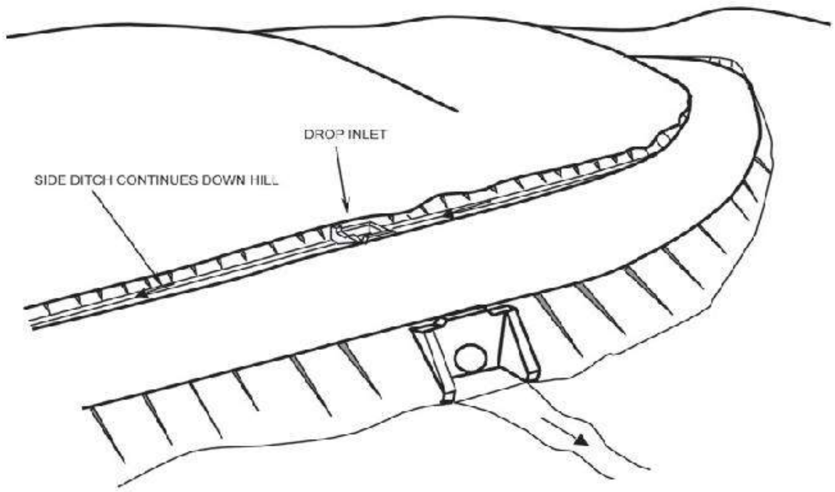


Figure 7-4: Key features of a relief culvert

Culverts can be pipe, box, slab or arch type, round, elliptical and square.

7.7.2 Key Features of Culverts

The key features of culverts are:

- Culverts are the most commonly used drainage structures on low volume roads. They can vary in number from about two per kilometre in dry and gently rolling terrain up to six or more for hilly or mountainous terrain with high rainfall. In flat areas with high rainfall, the frequency may also be increased to allow water to cross the road alignment in manageable quantities.
- In addition to well-defined water crossing points, culverts should normally be located at low points or dips in the road alignment.
- Relief culverts may be required at intermediate points where a side drain carries water for more than about 200 m without a mitre drain or other type of outlet.
- Headwalls are required at the inlet and outlet to direct the water in and out of the culvert and prevent the road embankment from eroding (sliding) into the watercourse. Wing walls at the ends of the headwall may also be used to direct the water flow and retain the material of the embankment or inner ditch slope.
- Aprons with buried cut off walls are also required at the inlet and outlet to prevent water seepage, scouring and undercutting.
- Culvert alignment should follow the watercourse both horizontally and vertically where possible.
- The gradient of the culvert invert should be between 2% and 5%. Shallower gradients could result in silting whereas steeper gradients result in scour at the outlet because of high water velocity.
- Culvert invert levels should be approximately in line with the water flow in the streambed, otherwise drop inlet and/or long outfall excavations may be required.
- Cross culverts smaller than 750 mm in diameter should be avoided, as they are very difficult to maintain (clean). A culvert of 900 mm is preferred from a maintenance perspective, but extra cover is required which may result in humps on the road alignment.

- Where foundation material is poor, culverts should be placed on a good foundation material or raft foundations to prevent settlement and damage. On very soft ground, it may be necessary to consider concrete, steel or timber piles to provide adequate foundations. This will require specialist design expertise not covered by this Manual.
- It is necessary to protect the watercourse from erosion downstream from the structure.
- Culverts can exist in pairs or in groups to enable larger stream flows to be accommodated using standard unit designs.
- When silt supply is high, culverts may need to be installed at higher gradient or extra maintenance may be required.

7.7.3 Advantages and Disadvantages of Culverts

The advantages and disadvantages of culverts are summarized in Table 7-7.

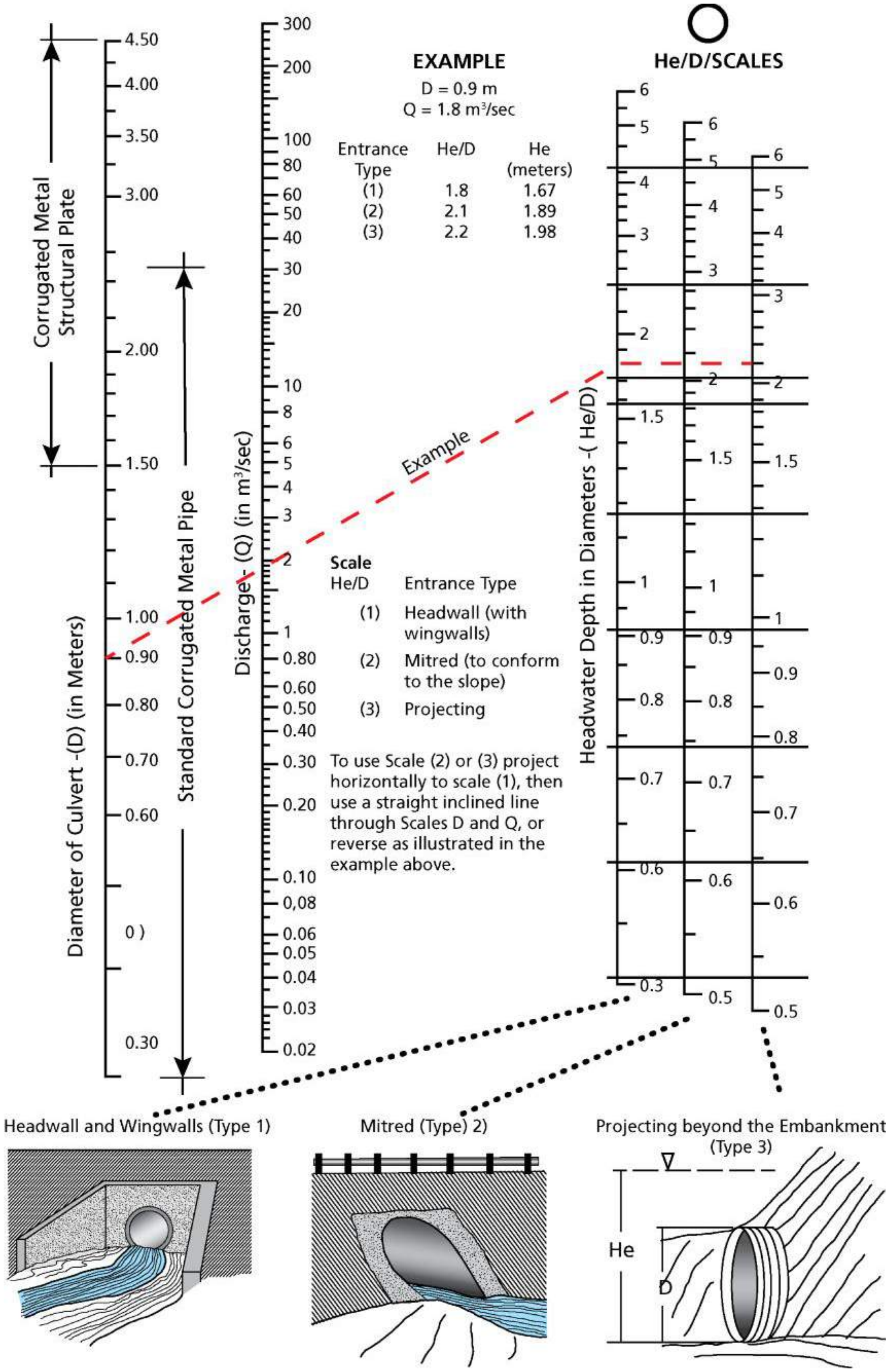
Table 7-7: Advantages and disadvantages of culverts

Advantages	Disadvantages
• Culverts provide a relatively cheap and efficient way of transferring water across a road. • They can be constructed and maintained with local labour and materials. • Culverts allow vehicle and foot passage at all times. • Culverts do not require traffic to slow down when they are crossed but humps above culverts can also be used for traffic calming if drivers are provided with appropriate warning signs. • Culverts allow water to cross the road at various angles to the road direction for a relatively small increase in costs.	• Regular maintenance is often required to prevent the culvert silting up, or to remove debris blockage. • Culverts act as a channel, forcing water flow to be concentrated, so there is a greater potential for downstream erosion compared with drifts. • Culverts are not suited to occasional high-volume flows and cannot cope well with sheet flow.

7.7.4 Design of Culverts

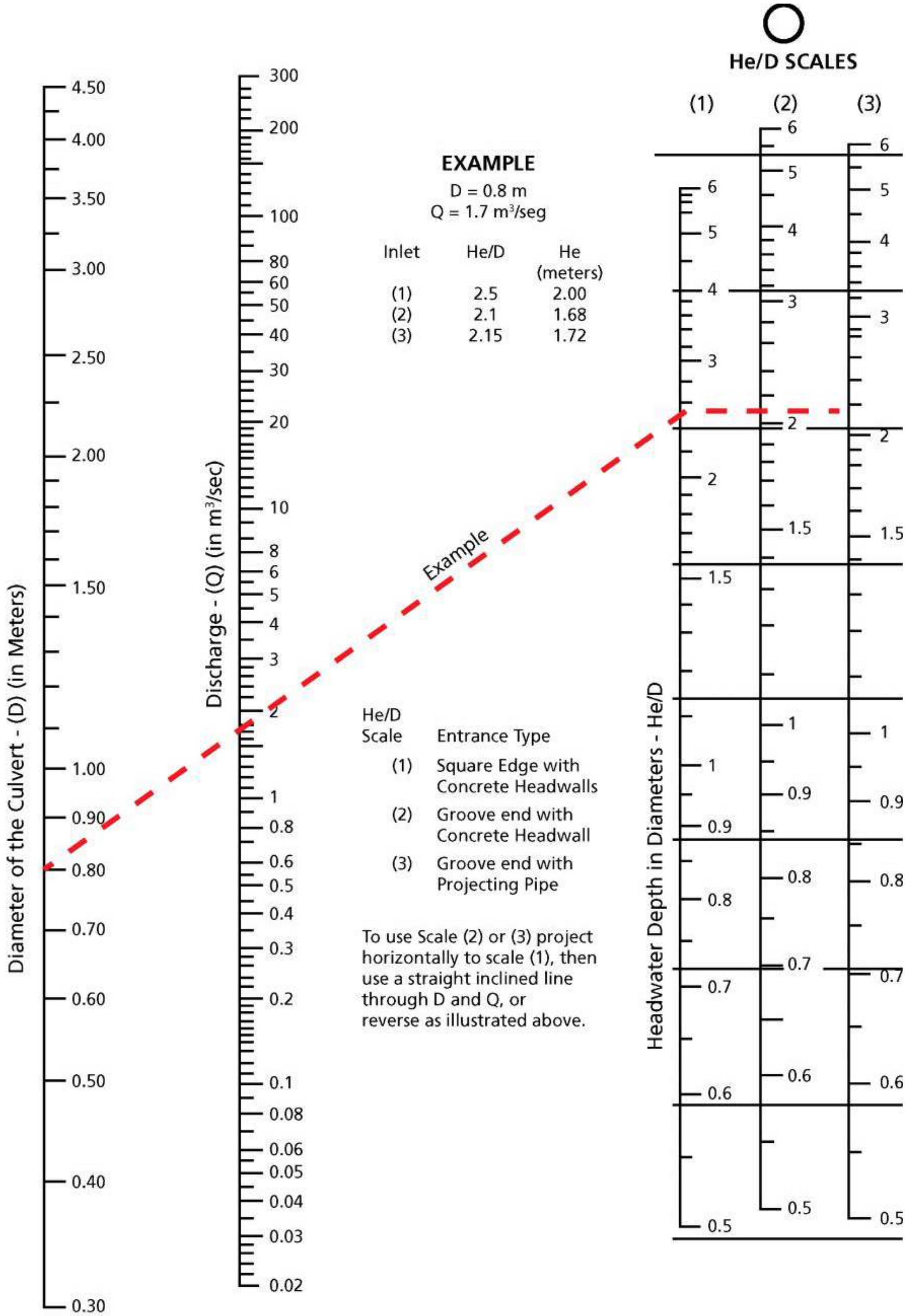
The water flow through the culvert should be estimated by any one of the methods presented in this chapter. The required size of a culvert opening is estimated using the nomographs given in Figure 7-5 for corrugated metal pipes, Figure 7-6 for concrete pipes and Figure 7-7 for concrete box culverts. These figures apply to culverts with inlet control where there is no restriction to the downstream flow of the water.

The nomographs are used by identifying the value for the flow of water generated by the design storm on the middle scale and drawing a line from that point across to the left-hand scale of the three scales on the right labelled H/D. These scales are for the three types of inlet shown on the nomographs. H/D is the ratio of the maximum head of water to the diameter of the culvert opening that is required to discharge the design flow through the culvert. In general, risks are reduced if the maximum flow does not cause the culvert to run at maximum capacity except for the design storm. Finally, the line is extended to the left to intersect the line labelled 'Diameter of Culvert (D) in metres. The culvert size obtained from the nomograph will probably not be one of the standard sizes that are available hence the next higher available size should be chosen or the nearest available size if the difference is small (<10 % in diameter).



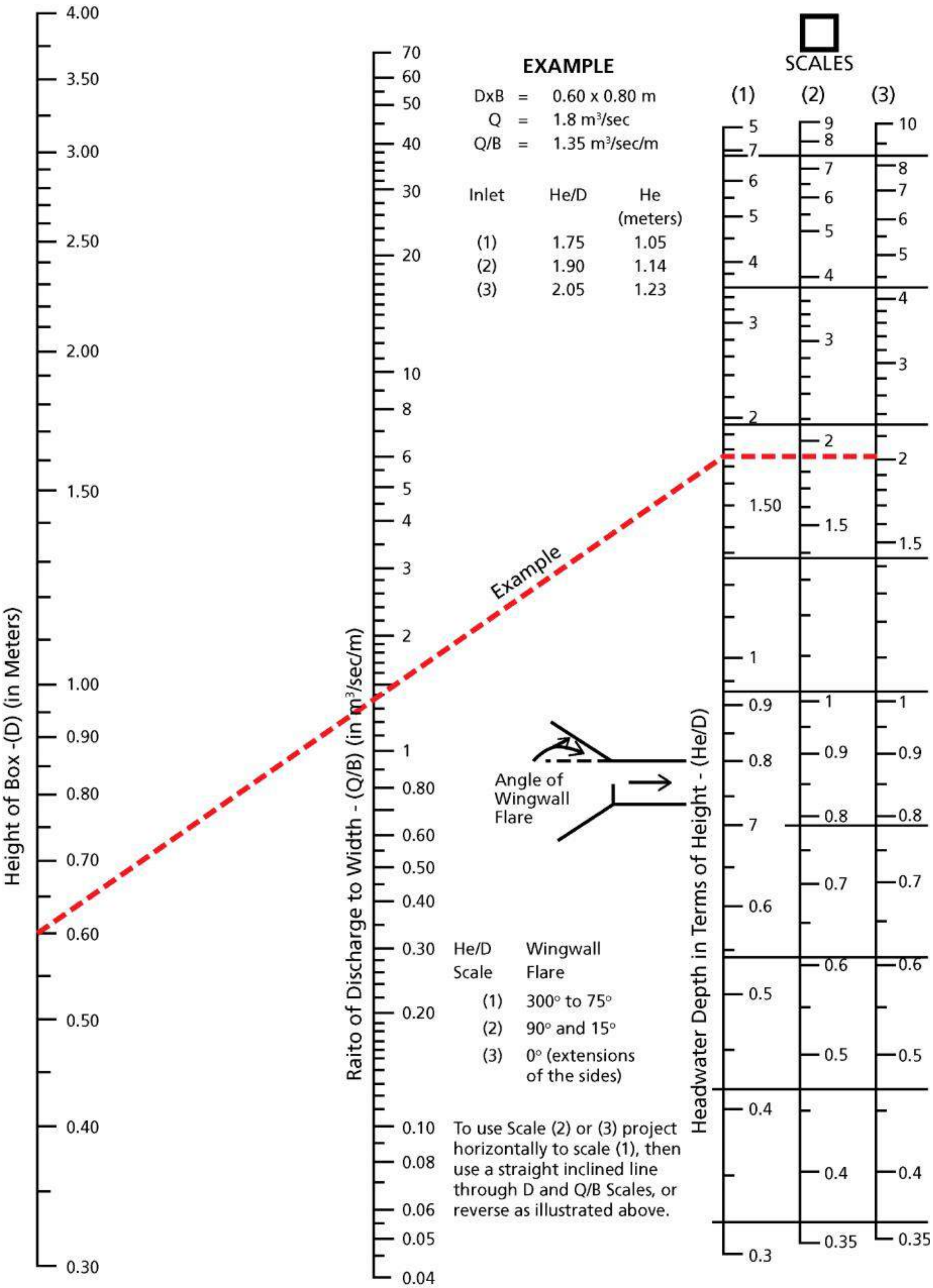
Source: FHWA (2012).

Figure 7-5: Headwater depth and capacity for corrugated metal pipe culverts with inlet control.



Source: FHWA (2012).

Figure 7-6: Headwater depth and capacity for concrete pipe culverts with inlet control



Source: FHWA (2012)

Figure 7-7: Headwater depth and capacity for concrete box culverts with inlet control

In flat terrain, where there is a high risk of silting, as shown in Figure 7-8, a factor of safety of 2 (in terms of water flow capacity) should be allowed in the design of the culvert. To minimise the effect of potential silting and deposition of debris in the culvert, the slope/fall should be 3%-5%.

7.7.5 Construction of Culverts

Culverts can be constructed in various ways, depending on the materials and the available skills. Inlet and outlet structures are normally constructed with wet stone masonry. Options for the barrel of the culvert include:

- Walls constructed in wet masonry with a slab of reinforced concrete. This type can have a height up to 1.5 m and a width up to 2 m.
- Arch constructed in wet masonry, using a variety of temporary supports.
- Concrete pipes constructed alongside the site using a collapsible mould or bought from local suppliers if the quality is acceptable.
- A metal arch or pipe.

In areas where it is difficult to obtain stones or rocks, culverts can also be constructed using sacks filled with a mixture of sand and cement.

Culverts should be designed to discharge the peak flow of water for the chosen design period, overtopping only during exceptional rains with a return period in excess of the chosen design period. Therefore, the most important aspect of culvert design is the open area through which the water flows. The required open area is calculated using the methods described in Section 7.3 above and the nomographs of Section 7.7.4.

The minimum recommended size of a culvert opening is a tube of diameter 750 mm. This diameter allows a worker to enter the tube during maintenance to remove obstructions. A greater open area can be obtained by using larger tubes, more tubes or different types of aperture. Larger apertures are more efficient in material usage but can require a deeper channel or more fill to carry the road over the culvert. The use of large diameter tubes can require special equipment to transport and lift them into place.

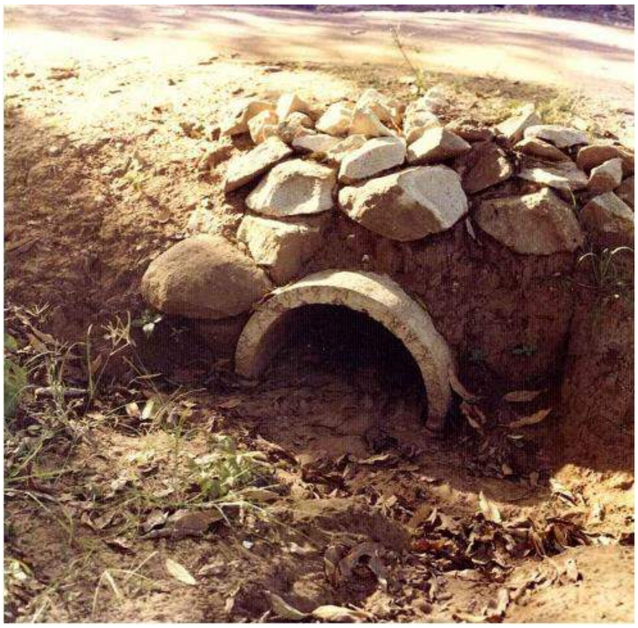


Figure 7-8: Siltation problems at culvert outlet

The invert of the culvert at the outlet must be at the same level as the bed of the outlet channel. However, it is possible to lower the invert of the culvert and the bed of the outlet channel by 300 mm if the latter is no more than 20 m long.

If the construction of the culvert requires the road to be raised locally in order to provide sufficient cover above the top of the culvert, humps in the road will be created. To avoid this, a drift could be used rather than a culvert, but this will be more expensive. Alternatively, a wider stone masonry culvert could be used to provide the same capacity as a 750 mm culvert. In general, if two solutions are equally valid and acceptable from an engineering and safety point of view, then the least expensive option should be chosen.

When culverts are located on earth roads, it is recommended that a layer of gravel or other improved surface is placed over the culvert and for 20 m on either side of the approaches. Backfill around the culverts must be with approved material, e.g. gravel or soil-cement, and properly compacted.

7.8 Vented Drifts and Causeways

7.8.1 General

A vented drift is a combination of a culvert and a drift. They are suitable for carrying roads across water courses which have a perennial (permanent) water flow for most of the year and which have large flows for less than three days after heavy rains.

7.8.2 Key Features of Vented Drifts and Causeways

A typical example of a vented drift is shown in Figure 7-9 (schematic) and 7-10 (photo). The key features are:

- These structures are designed to pass the normal dry weather flow of the river through pipes below the road. Occasional larger floods pass through the pipes and over the road, which may make the road impassable for short periods of time.
- Vented causeways are the same concept as vented drifts but are longer with more pipes to cross wider watercourse beds.
- The level of the road on the vented drift should be high enough to prevent overtopping except at times of peak flows.
- There should be sufficient pipes to accommodate standard flows. The location of pipes in the drift will depend on the flow characteristics of the river.
- A vented drift should be built across the whole width of the water- course.
- A vented drift requires approach ramps which must be surfaced with a non-erodible material and extend above the maximum flood level.
- Watercourse bank protection will be required to prevent erosion and eventually damage to the entire structure.
- The approach ramps should not have a steeper grade than 10 % (7 % where there is significant heavy vehicle traffic).
- The upstream and downstream faces of a vented drift require buried cut off walls (preferably down to rock) to prevent water undercutting or seeping under the structure.
- An apron downstream of the pipes and an area of overtopping is required to prevent scour by the water flowing out of the culvert pipes or over the structure.
- There is also a requirement to protect the watercourse from erosion downstream from the structure. There will be considerable turbulence immediately downstream of the structure in flood conditions.
- The road surface vertical alignment of a vented drift should be a slight sag curve to ensure that, at the start and end of overtopping, water flows across the centre of the vented drift and not along it.
- There should be guide stones on each side of the structure to mark the edge of the carriageway and indicate when the water is too deep for vehicles to cross safely.

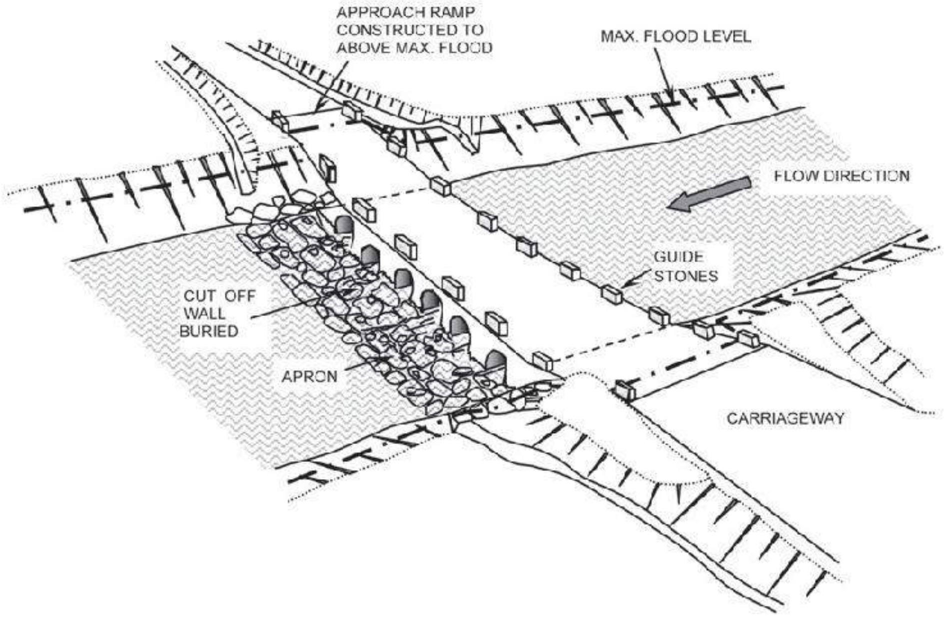


Figure 7-9: A typical vented drift / causeway (schematic)

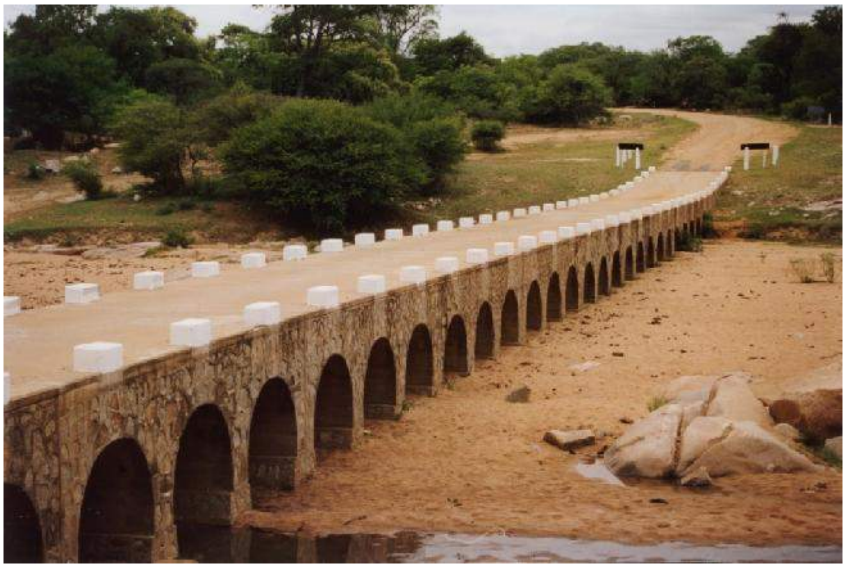


Figure 7-10: A typical vented drift / causeway

7.8.3 Advantages and Disadvantages of Vented Drifts and Causeways

The advantages and disadvantages of vented fords and causeways are summarized in Table 7-8.

Table 7-8: Advantages and disadvantages of vented drifts/causeways

Advantages	Disadvantages
• Vented drifts can allow a large amount of water to pass without overtopping. • They are cheaper to construct and maintain than bridges. • Construction of vented fords is fairly straightforward compared with bridges. • Vented fords are well suited to cope with short high-volume flows. • Can be constructed and maintained primarily with local labour and materials. • Good at coping with sheet flow.	• Vented drifts can be closed for short periods during periods of flooding and high flow. • Floating debris can lodge against the upstream side of the structure and block pipes. • Foot passage can be inconvenient or hazardous when water is flowing.

7.8.4 Design of Vented Drifts/Causeways

The apertures must allow normal flows hence the size of the apertures should be obtained by observing the flow under normal conditions (but not during the dry season). The apertures plus the area above the apertures to a depth of 200 mm must allow the water to flow during heavy rains. They can be designed using the Observational Method or the Rational Method. Vented drifts generally have higher capacity and construction costs than drifts or culverts.

The invert of the apertures should be at the same level as the outlet channel. Each ramp should extend, at a gradient of no more than 10%, to at least one metre above the highest water level observed during heavy rain. Gravel or an improved surface should also be placed on the road for 50 metres in each direction.

So that vehicles can pass safely when water is flowing over the slab and the slab itself is not fully visible, posts are fixed to indicate the water level and the edges of the structure. It is also possible to construct blocks along the sides of the vented drift to help pedestrians to pass when water is flowing.

7.9 Submersible Bridge

7.9.1 General

A submersible bridge is a form of vented drift with large apertures for the normal water flow. During heavy rains the water can also pass over the deck and the two approach ramps that are constructed over the apertures.

Submersible bridges allow roads to cross water courses which have large flows for most of the year and very large flows lasting for up to a week during and after heavy rains.

Posts are fixed to indicate the width of the structure and the level of the water so that vehicles can pass when water is flowing over the deck.

7.9.2 Features of Submersible Bridges

Submersible bridges are often designed using the Observational Method. The apertures and the area above the apertures up to a depth of 200 mm must be sufficient to allow the water to flow during heavy rains.

Each approach ramp should extend at least 2 m beyond the highest water level observed during heavy rains. The level of the deck should be sufficiently high to allow the normal flow of water to pass under the bridge.

It is preferable that all the pillars are seated on a rock foundation. If the bridge is fixed on a rock foundation with steel dowels, it can be up to 2.5 m high. If the bridge is not fixed with steel dowels, it can be up to 1.5 m high.

If it is not possible to provide sufficient open area with a submersible bridge of 1.5 m or 2.5 m height, it is necessary to construct a high-level bridge.

The structure should be inspected every year to check that maintenance and repairs are being carried out as required.

7.10 Masonry Arch Culverts

7.10.1 General

For high water flows a masonry arch culvert provides a greater capacity than a pipe culvert and is sometimes a more appropriate option than a small bridge. A masonry arch culvert is illustrated in Figure 7-11. Their capacity is designed in a similar way to small bridges. For guidance on such structures refer to TRL Overseas Road Note 9.

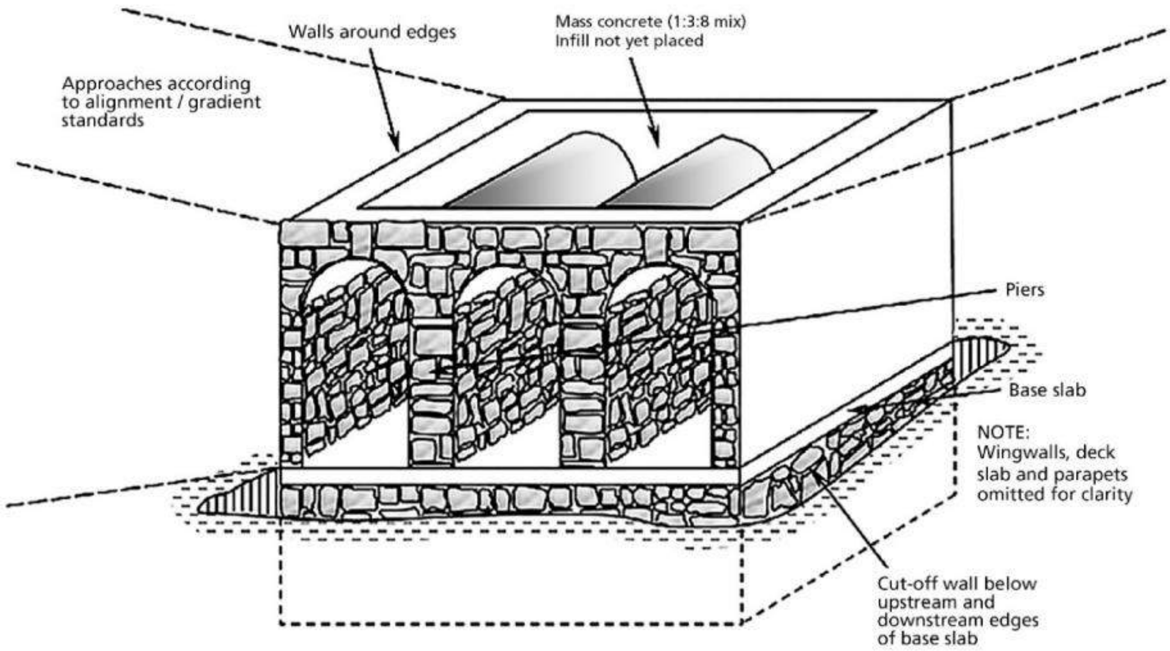


Figure 7-11: Key features of a masonry arch culvert

7.10.2 Key Features of Masonry Arch Culverts

The key features of large arch culverts are:

- Formwork is required to construct the openings. This formwork can be made from wood, stones or metal sheeting and either incorporated into the structure or removed once construction is complete.
- Although these structures are not generally designed to be overtopped, they can be designed and constructed to cope with an occasional overtopping flood flow.
- The road alignment needs to be a minimum of 2m above the bottom of the watercourse.
- Approach embankments are required at each end of the structure.
- Large arch culverts require solid foundations with a buried cut-off wall on both upstream and downstream sides to prevent water seepage erosion and scouring.
- These structures require large amounts of internal fill material during construction.
- Guide stones or kerbs should be placed at the edge of the carriageway to increase vehicle safety.
- If the crossing is to be used by pedestrians, consideration should be given to installing guard rails and central refuges for long crossings where pedestrians can move off the roadway to allow traffic to pass.
- Water from the roadside drains should be carefully channelled into the watercourse away from the structure to prevent erosion of the bank or scour of the culvert structure.

7.10.3 Advantages and Disadvantages of Masonry Arch Culverts

The advantages and disadvantages of large arch culverts are summarized in Table 7-9.

Table 7-9: Advantages and disadvantages of large arch culverts

Advantages	Disadvantages
• Large arch culverts are usually easier and cheaper to construct than bridges. • They can accommodate flows significantly higher than smaller culverts and vented fords. • Can be constructed and maintained with local labour and materials, without the need for craneage. • They may easily be designed and constructed for occasional overtopping. • They generally require less maintenance than conventional bridges.	• The water opening in large arch culverts is smaller than for a bridge of the same size, which reduces the potential flow rate past the structure at peak flows. • Large arch culverts can require a significant amount of internal fill material.

An alternative to a large or multi-barrel culvert is a reinforced concrete box culvert. This Manual does not cover this type of structure. For guidance on such structures refer to TRL Overseas Road Note 9.

7.11 Embanked Crossings

7.11.1 General

In completely flat terrain where a water course floods during the rains or with generally poor drainage, the road will usually be on an embankment. Such an embanked crossing must have one or more cross-drainage structures, hence, the design information for the cross-drainage structures is as described in previous sections. Under these circumstances the flow can be relatively slow provided that enough culverts are available, but insufficient culverts can lead to rapid flow along the side of the embankment and consequent scouring (see Section 7.11.2 below). The simplest method of estimating the required culvert openings is by asking the local people how long the water usually takes to dissipate from peak flood condition after the rain. Calculating the likely volume and required number and size of culverts necessary to prevent the flow velocity exceeding the velocities shown in Table 7-14 is then relatively straight forward.

A combination of drainage structures can also be used on the same embankment (e.g. small bridge plus culverts/causeway).

7.11.2 Construction

Embankments can normally be constructed with soils that are found near to the road. The soil that is used should form a strong and stable layer when compacted. Topsoil and loose sand should be avoided.

The road surface on the embankment must remain dry all year round. Therefore, the height of the embankment depends on the water level in the area. The top of the embankment should be at least 500 mm above the highest water level. The soil in the embankment should be spread in 100 mm - 150 mm thick layers and well compacted. The surface should be formed to give a camber. An improved surface should be provided, e.g. good gravel with a surface seal, to better support the weight of the vehicles and to protect the surface of the road against erosion. Grass should be planted or allowed to grow on the sides of the embankment to protect the sides against erosion. Rip rap may also be used for this purpose.

An embanked crossing normally needs structures to allow the water to flow. The required size of the apertures can be calculated using the Observational or Rational method. Structures to provide the necessary cross-flow openings should be constructed along the embankment at intervals of no more than 50 m. The deepest part of the crossing should have the largest structure.

Some embanked crossings, for example, across an area where a water course floods over the land, are constructed with a short length of the embankment at a lower level. The short lengths of embankment act as a drift and can prevent large pieces of debris from blocking the openings in the embankment when the water course is flooding. The part of the lower embankment should be protected against erosion when the water flows over.

Over time, erosion and other factors can make the embankment narrow, requiring vehicles to pass dangerously close to the edge (i.e. within 500 mm). The embankment must be widened in the following manner:

- Decide if the embankment needs to be widened on one or both sides.
- Wait until the site is as dry as possible.
- Remove vegetation and loose soil from the side of the embankment to 1 metre beyond the base.
- Cut steps in the embankment slope. Each step should have a horizontal surface at least 500 mm wide and should be cut into stable, well compacted soil.
- Place and compact suitable material in layers 150 mm thick to the required width of the embankment.
- Shape the slope and protect it with suitable vegetation.
- Reconstruct the road to its original width, with an improved surface such as gravel or Geo-Cells.

7.12 Bridges

7.12.1 General

These are generally the costliest drainage structures. This Section covers arch and simply supported bridge types. The Manual does not cover large or multiple span bridges, which may be simply supported or continuous over piers. For such structures and bridges with spans more than 10 metres, refer to the Overseas Road Note 9 (TRL, 1992).

The key features of a simply supported bridge deck are illustrated in Figure 7-12.

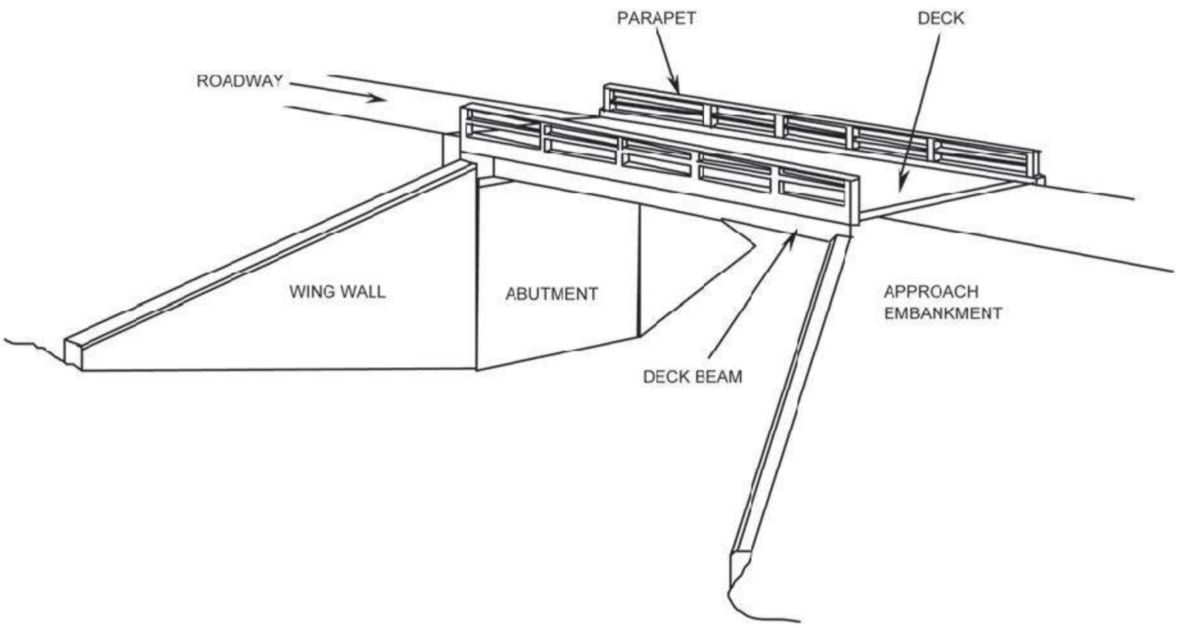


Figure 7-12: Key features of a simply supported bridge deck

Chapter 7 – Hydrology and Drainage Structures

7.12.2 Key Features of Small Bridges (< 10 m span)

Key features are:

- The arch is the simplest form of bridge.
- There are several different elements to a simply supported deck bridge. These are a superstructure (comprising deck, parapets, guide stones and other road furniture) and substructure (comprising abutments, wing walls, foundations, piers and cut off walls).
- Bridges are generally the most expensive type of road structure, requiring specialist engineering advice and technically approved designs.
- Bridges can be single span or multi span, with several openings for water flow and intermediate piers to support the superstructure.
- The main structure is always above flood level, so the road will always be passable.
- Abutments support the superstructure and retain the soil of the approach embankments.
- Wing walls are needed to provide support and protect the road embankment from erosion.
- Embankments must be carefully compacted behind the abutment to prevent soil settlement which would result in a drop off between the bridge deck and the road surface at the end of the bridge.
- Weep holes are needed in the abutment to allow water to drain out from the embankment and avoid a build-up of ground water pressure behind the abutment.
- Bridges should not significantly affect the flow of water (i.e. the openings must be large enough to prevent water backing up and flooding or over topping the bridge).
- The shape of the abutments and piers will affect the volume of flow through the structure and the amount of scouring.
- Bridges require carefully designed foundations to ensure that the supports do not settle or become eroded by the water flow. On softer ground this may require piled foundations which are not covered in this Manual.
- Water from the roadside drains should be channelled into the watercourse to prevent erosion of the bank or scour of the abutment structure.
- Guide stones or kerbs or guard rails should be placed at the edge of the carriageway to increase vehicle safety.
- If the crossing is to be used by pedestrians, proper protected footways should be designed on both sides of the carriageway.
- Reinforced concrete parapets are preferred rather than steel guard rails. They should be flared away at the ends and ramped for safety reasons. Warning or guard posts should be provided on the bridge approaches because vehicles need to slow down for safety.

7.12.3 Advantages and Disadvantages of Small Bridges (Spans <10 m).

The advantages and disadvantages of small bridges are summarised in Table 7-10.

Table 7-10: Advantages and disadvantages of small bridges

Advantages	Disadvantages
• The road is always passable because the structure should not be overtopped. • Simple arch bridges can be constructed primarily with local labour skills and local materials without the need for craneage. However, simply supported spans are more complex.	• Bridges are normally significantly more expensive than other road structures. • They are more complex than other structures and will require specialist engineering support for design and construction. • Additional height and earthworks in approach embankments. • Bridges may require heavy duty lifting cranes for the deck components. • Although all structures should be inspected for defects, bridges require regular detailed checks. • Bridges are likely to fail if flood flow predictions are incorrect and they are over topped. • A small amount of scour and erosion can often result in major damage to structures.

7.12.4 High-level Bridge

A high-level bridge allows a road to cross a wide or a deep watercourse without the water passing over the top of the deck. The design of high-level bridges is beyond the scope of this Manual.

7.13 Structure Selection

7.13.1 General

The objective in selecting a structure for a water crossing is to choose the most appropriate design for each location. This selection should be based on the factors outlined in Figure 7-13.

For small watercourses and relief structures the choice of structure will, in general, be between a culvert and drift and, for larger watercourses, between a vented ford and a large diameter culvert, or possibly a bridge. The choice of structure will be determined by all the factors discussed above, but particularly by the predicted maximum water flow, its seasonal variations and the duration of road closures that can be tolerated. It should also be noted that the Figure only highlights the key issues and should only be used as a guide when determining the most appropriate structure.

The flow rate is dependent on the rainfall in the catchment area and the runoff conditions as explained in Section 7.2. The design of structures is primarily based on the peak flow rate, but it is often necessary to know the normal flow rate for two reasons.

- For the design of drifts and vented drifts it is necessary to ensure that vehicles can cross the drift during the normal flow or that, in the case of a vented drift, the water passes through the pipes and the vented ford is not overtopped.
- To check that there will be no long-term damage to the structure due to erosion. The short period of peak flows may not damage erodible parts of the structure, but it is necessary to ensure that parts of the structure permanently in contact with the water flow are not damaged.

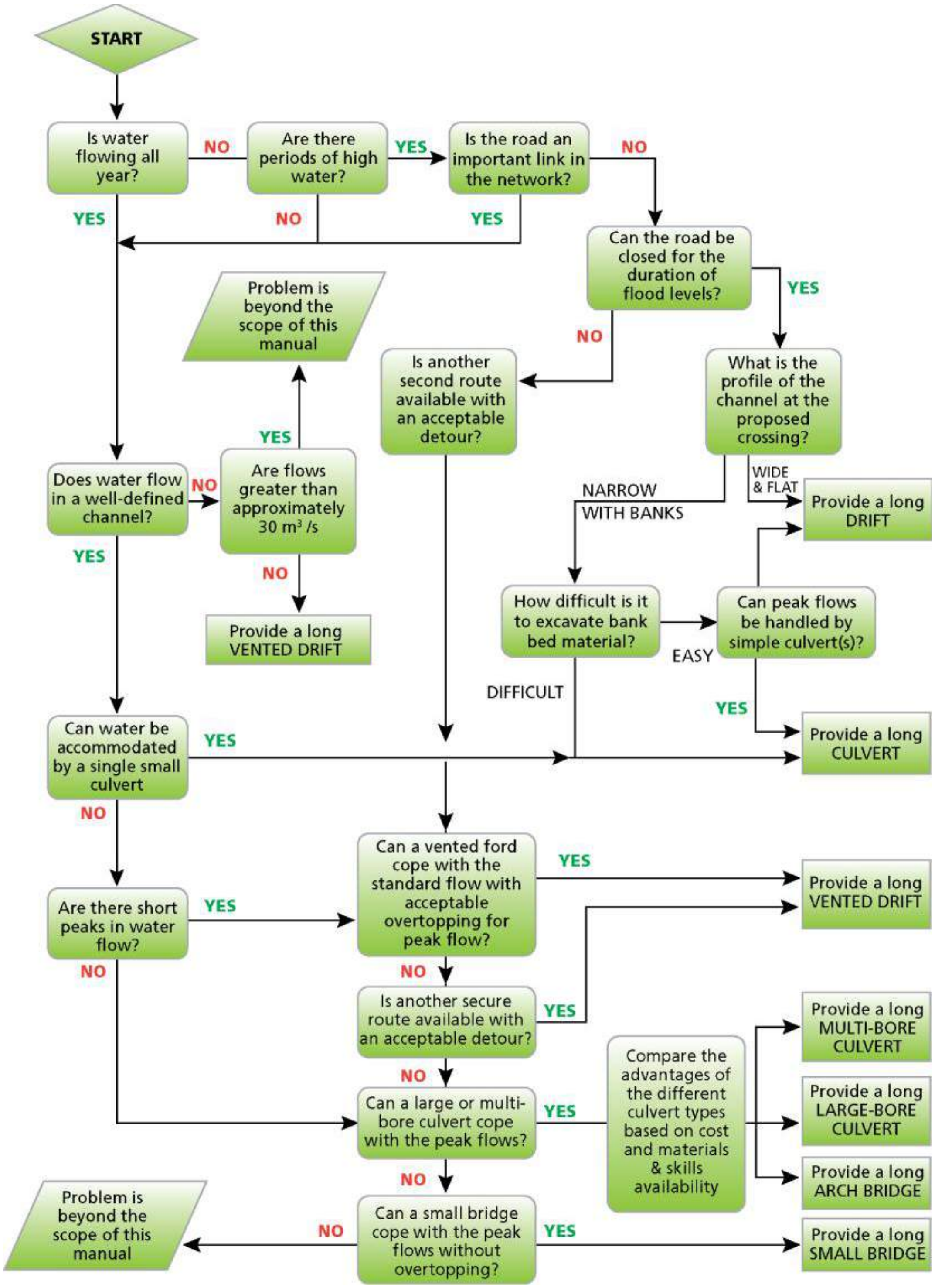


Figure 7-13: Recommended cross drainage structures

7.13.2 Closure Periods for Seasonal Flow

An investigation into the variation in seasonal water flows is required if the proposed structure will be overtopped. It is necessary to determine the proportion of the year that higher flows will be experienced to estimate the number of days the structure may not be passable. It may be necessary to raise the running surface of the structure, such as a vented ford, to ensure that the structure is only overtopped during particularly rainy months. Unless detailed rainfall data is available for the area it is likely that the only suitable methods for collecting seasonal flood levels and flows will be tapping the knowledge of the local population.

In the absence of any local information and data, suggested upper and lower bounds for closure times are shown in Table 7-11.

Table 7-11: Suggested closure times

Criteria	Drift most favourable	Drift least favourable
ADT	< 5 vpd	>200 vpd
Average annual flooding	< twice per year	More than 10 times per year
Average duration of traffic interruption per occurrence	< 24 hours	>3 days
Extra travel time for detour	< 1 hour	>2 hours / no detour

A combination of structures may often be the most cost-effective solution. Wide perennial flood plains may be best crossed by vented fords with long approach embankments with relief culverts along their length. Similarly bridge lengths could be shortened in combination with relief culverts if erosion potential at the crossing point is found to be minimal due to flat terrain and stable material.

This Manual provides an overview of each individual structure type, but consideration should always be given at initial design and cost estimation stage as to whether a combination of structures will be more cost effective for watercourse crossings.

When the problem is beyond the scope of this Manual, specialist bridge engineering skills are required.

7.14 Scour Control

7.14.1 General

Erosion is a frequent problem that must be addressed during drainage design. The majority of structural failures of drainage structures occur during flood periods and over 50% of these failures can be attributed to scour.

Erosion/scour is also closely linked with the geotechnical problems of slope stability that are discussed in *Chapter 4 – Site Investigations*, and bioengineering methods of stabilising slopes and preventing erosion described in *Chapter 8 – Drainage and Erosion Control*.

Thus, erosion and scour are complex subjects that often requires specialist advice. This Chapter deals with the most common issues likely to be encountered on LVRs.

7.14.2 Characteristics of Scour

There are basically three types of scour or erosion. The first two are caused by the existence of the drainage structure itself in concentrating the flow of water and/or increasing its velocity. There are two aspects:

- Erosion/scour around the structure itself that threatens its integrity and its continued existence.
- Erosion/scour that occurs because of the structure but upstream and especially downstream away from it.

The third type is essentially natural scour or erosion that occurs within all natural water channels irrespective of the existence of man-made drainage structures. This will alter the hydraulic environment over time and needs to be considered in the design of the road.

The amount of scour is dependent on the speed of the water flow, how particle loaded the water is, and the erodibility of the material that the water comes into contact with. If the flow is not parallel to the constriction more scour will occur on one side than the other. Water is accelerated around abutments, piers and other obstructions, creating vortices with high velocities at abrupt edges on the obstruction, increasing the scour depth, often dramatically.

Trapped debris can also restrict the flow of water and cause an increase in water velocity. It is important that structures are designed to minimise the chances of debris being trapped and to ensure that inspections and maintenance are carried out after flood periods to remove any lodged debris.

Finally, if the water is already carrying a large amount of material eroded from further upstream, a greater amount of scour will occur at the structure.

It is difficult to predict the level of scour that may be experienced for a particular design. Existing methods require detailed knowledge that is not readily available, and they are not very accurate. Thus, engineering judgement is required. For guidance on design of small bridges, refer to Overseas Road Note 9 (TRL, 1992).

7.14.3 Designing to Resist Scour

This Manual proposes a number of 'rules' for designing to resist scour. It must be stressed that these rules are not infallible and local knowledge should also be taken into account when designing a structure.

Constrictions

The amount of scour experienced at a structure is proportional to the restriction in the normal water flow. Hence, as a general principle, wherever possible, any constrictions to water flow should be minimised.

Cut-off walls

Cut-off walls, also called curtain walls, should be provided at the edge of a structure to prevent water eroding the material adjacent to the structure. The location of cut-off walls for the various structures is shown in Table 7-12.

Table 7-12: Cut-off wall locations

Structure	Locations
Drift	Upstream and downstream sides of drift slab.
Culvert	Edges of inlet and outlet apron.
Vented drift/ford	Upstream and downstream sides of main structure and approach ramps.
Large diameter culvert	Upstream and downstream sides of approach ramps. The foundations of the main structure should be built at a greater depth than standard cut-off walls below the possible scour depth.

The absence of cut-off walls at the inlet of the structure could easily allow water to seep under the apron causing settlement and eventually collapse of the structure. At the downstream end of the structure the flowing water could erode the material next to the apron, eventually eroding under the apron and causing it to collapse.

The minimum depth of the cut-off walls depends on the ground conditions. Where a rock layer is close to the ground surface the cut-off walls should be built down to this level and firmly keyed into the rock using dowels. In other situations, the depth of the cut-off wall should be greater than the expected depth of scour. This is best estimated from local experience under similar conditions. The depth is measured from the lowest point in the bed of the watercourse at the crossing point. If no information is available Table 7-13 provides guidance. For larger structures advice should be sought from an experienced engineer.

Table 7-13: Foundation and cut off wall depths

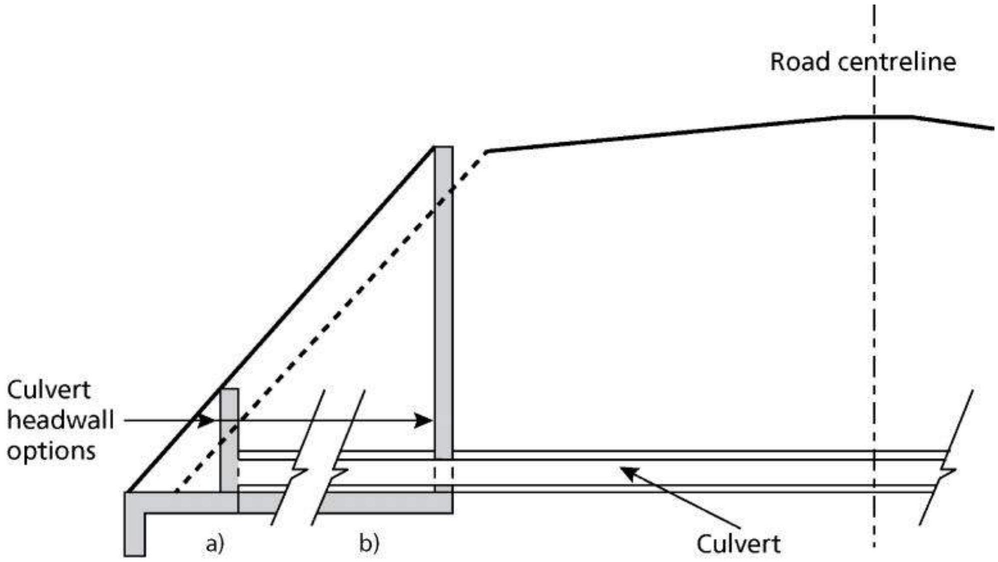
Structure	Cut off wall depth (m)	Comments
Drift	1.5	
Relief culvert	1.0	
Water course culvert	1.5	Headwalls and wingwalls
Vented drift	2.0	

Use of piers
If piers are necessary, they should be aligned exactly in the direction of water flow.

Culvert headwalls and wingwalls
Headwalls and wingwalls are required at each end of a culvert and serve several different purposes:

- They direct the water in or out of the culvert.
- They retain the soil around the culvert openings.
- They prevent erosion near the culvert and seepage around the pipe, which causes settlement.

The headwall can be positioned at different places in the road verge or embankment as shown in Figure 7.14.



a) Longer culvert, low headwall cost
b) Shorter culvert, high headwall cost

Figure 7-14: Possible culvert headwall positions

The closer the headwall is placed to the road on an embankment the larger and more expensive it will be. The most economical solution for headwall design will be to make it as small as possible. Although a small headwall will require a longer culvert, the overall structure cost will normally be smaller. If, due to special circumstances at a proposed culvert site, a large headwall with wingwalls is required it should be designed as a bridge wingwall (with a soil retaining function).

Where a road is not on an embankment the size of the headwall will be small regardless of position. In this case the position of the headwalls will be determined by the road width and any requirements of national standards. The headwalls should be positioned at least 1 m beyond the edge of the carriageway to prevent a restriction in the road and reduce the possibility of vehicle collisions.

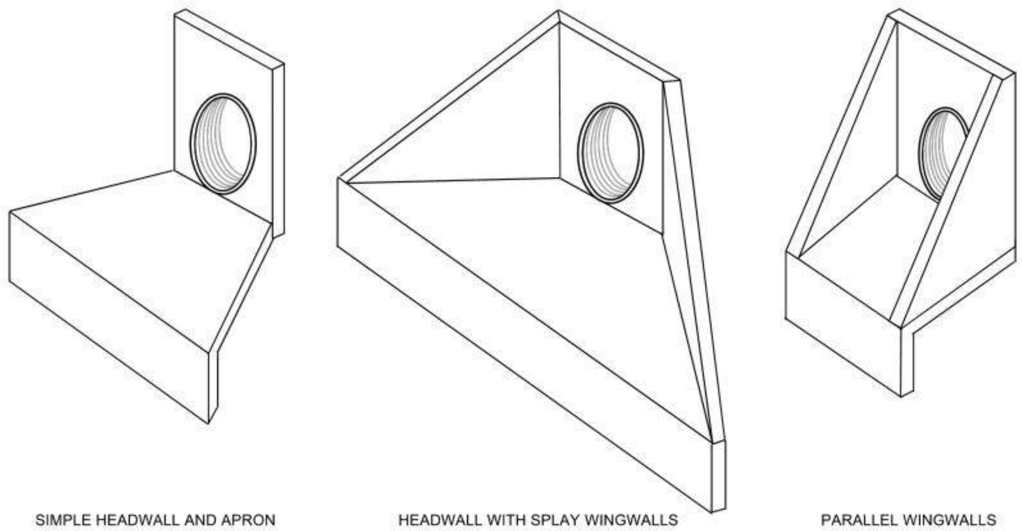
Headwalls should project just above the road surface (+/- 150 mm) and be painted white so that they are visible to drivers. Marker posts may also be used to warn drivers of the existence of a headwall which may be a potential road safety hazard. There are a number of different layout options for culvert headwalls which are shown in Figure 7-15.

Headwall with drop inlet
This arrangement should be used when the road is on a steep side slope to reduce the invert slope of the culvert.

Headwall with L-inlet

This arrangement should be used where the road is on a gradient and water is to be transferred from the carriageway side drain on the high side of the road.

Headwall and adjacent works must be designed so that the culverts can be de-silted manually under maintenance arrangements. This can be difficult with a drop inlet and silt trap arrangement.



This arrangement can be used for simple low flow relief culverts. It is the cheapest option but prone to erosion for larger flows.	This arrangement should be used for larger flows instead of the simple arrangement. The wingwalls increase the protection from erosion.	Where the road is on an embankment it is essential that wingwalls be provided to prevent the risk of water seepage and subsidence of the embankment. This arrangement is likely to be the most economical.

Figure 7-15: Headwall and wingwall arrangements

Vertical positioning of culverts

The vertical positioning of culverts requires particular attention. The consideration of the natural vertical alignment of the watercourse must take precedence over the vertical alignment of the road. Neglect of this factor has led to many culverts being installed incorrectly, leading to excessive silting, erosion and in many cases failure. The most appropriate culvert type will depend on the outfall gradient.

Flat outfall (less than 5%): This culvert type should be used in flat areas and for watercourses with shallow gradients. In these cases, the road should be built up over the culvert with ramps 20 m - 50 m long or to comply with national road vertical alignment standards. A culvert will silt up if it is positioned too low.

Intermediate outfall (approx. 5 to 10%): This arrangement requires the culvert to be excavated slightly into the existing ground, although the invert of the culvert at the inlet should be at the same level as the bed of the watercourse. The outlet of the culvert will be below the existing ground level and will require an outfall ditch to be dug with a gradient of 2% - 4%. The road will still have to be built up with ramps or alignment adjustment over the culvert to provide the minimum required cover.

Steep outfall (more than 10%): The culvert can be installed without building up the road level. The culvert should be buried to provide adequate cover over the pipe. A drop inlet will be required at the entrance and a short outfall ditch at the exit. On steeply sloping ground careful attention should be given to preventing erosion downstream of the culvert.

Pipes transferring large water volumes

One of the most important design rules when constructing a road water structure is to disrupt the flow as little as possible. Unfortunately, this is not possible for a culvert that is transferring water from a side drain. The water must make an abrupt right-angle change in direction to enter the culvert. For large flows there will therefore be a large amount of turbulence in the water and the potential for scour. The following key features should be used in the inlet design for large flows:

- Rounded wingwalls to 'guide' water into the pipe.
- Sloping wingwall on the inside radius.
- Lined channel sides and base which extend 5 m up the channel.
- Cut-off wall provided at the edge of the inlet.
- The box culvert option should be considered because this will cause less restriction and turbulence.

Aprons

An apron is required at the inlet and outlet of culverts and downstream of drifts and vented fords to prevent erosion. As the water flows out of or off a structure it will tend to erode the watercourse downstream, causing undercutting of the structure. Aprons should be constructed from a material, which is less susceptible to erosion than the natural material in the streambed.

Drift aprons: Where the discharge velocity across the drift is less than 1.2 m/s, which may be experienced for relief drifts, a coarse gravel layer (10 mm) will provide sufficient protection downstream of the drift. For discharge velocities greater than 1.2 m/s more substantial protection will be required utilizing larger stones. The width of the apron should be at least half the width of the drift and extend across the watercourse for the whole length of the drift.

Culvert aprons: Aprons should be provided at both the inlet and outlet of culverts. They should extend the full width between the headwall and any wingwalls. If the culvert does not have wingwalls the apron should be twice the width of the culvert pipe diameter. The apron should also extend a minimum of 1.5 times the culvert diameter beyond the end of the pipe. Cut-off walls should also be provided at the edge of all apron slabs. The choice of apron construction is likely to depend on the type of material used for construction of the culvert. It may be constructed from gabion baskets, cemented masonry or concrete.

Vented ford aprons: The apron for vented fords should extend the whole length of the structure including downstream of the approach ramps to the maximum design level flood. The other design requirements for vented ford aprons are the same as culvert aprons.

7.15 Downstream Protection

7.15.1 General

Whenever watercourses are channelled through pipes, such as in culverts and vented drifts, or through narrow openings in bridges, severe erosion can be caused to land and property downstream of the structure. If agricultural land or buildings are close to the proposed structure, careful consideration must be given to erosion protection. Undersized structures can also cause water to back up causing flooding upstream and possible property damage.

7.15.2 Remedial Measures

The use of aprons downstream of a structure should prevent erosion and undercutting of the structure itself. However, in small, constrained channels severe erosion may still occur after the apron, particularly where the watercourse is on a gradient. It is therefore often necessary to provide additional protection to the watercourse, to reduce the velocity of the water and prevent erosion.

For slow flowing water it is unlikely that any protection will be needed, but for faster flowing water the maximum allowable velocity will depend on the bed material and the amount of silt or other material already being carried in the water.

Erosion can occur in any channel regardless of the presence of any structure. It is therefore not possible to state how far downstream of a structure channel protection should extend. However, the following issues should be taken into account:

- The general erodibility of the bed, which will be based on the type of material and gradient.
- The likelihood of damage to the structure if erosion occurs downstream.
- The potential effects of erosion on downstream areas (e.g. buildings or farming land).

The maximum water flow velocities that can be tolerated without channel protection related to the type of bed material are shown in Table 7-14.

Table 7-14: Maximum water velocities (m/s)

Soil type	Clear water	Water carrying fine silt	Water carrying sand and fine gravel
Fine sand/coarse silt	0.40	0.70	0.40
Fine sand	0.45	0.75	0.45
Sandy soil	0.50	0.70	0.60
Silty soil/sandy clay	0.60	0.90	0.60
Alluvial silts (non-colloidal)	0.60	1.05	0.60
Alluvial silts (colloidal)	1.15	1.50	0.90
Stiff clay	1.15	1.5	0.90
Firm soil/coarse sand	0.75	1.05	0.70
Volcanic ash	0.75	1.05	0.60
Firm soil, silt and gravel	1.00	1.50	1.50
Gravel (5 mm)	1.10	1.20	1.20
Gravel (1 mm)	1.20	1.50	1.50
Coarse gravel (25 mm)	1.50	1.90	2.00
Graded silt to gravel	1.10	1.60	1.60
Graded sand and gravel	1.20	1.50	
Cobbles (50 mm)	2.00	2.40	2.40
Cobbles (100 mm)	3.00	3.5	3.50
Lined with established grass on good soil	1.70	2.40	1.70
Lined with bunched grasses (exposed soil between plants)	1.10	1.10	1.10
Grass with exposed soil	1.0	1.80	
Shales	1.85	1.85	1.50
Rock	Negligible scour at all velocities		

7.15.3 Bank Elevation and Bed Material of the Watercourse

The resistance of the watercourse banks and bed to erosion will dictate the type of foundation bank protection and hence structure that can be built. For material which is easily erodible it will be necessary to have deep foundations and possibly extensive bed and bank protection or structures which are not susceptible to damage. The steepness of the banks and difficulty in excavating soil material will also determine the most convenient approach roads.

A major factor affecting the cost of building a structure is the amount of material which needs to be imported to or exported from the site. Constructing structures that that will not be overtopped may often require large approach ramps/ embankments as illustrated in Figure 7-16.

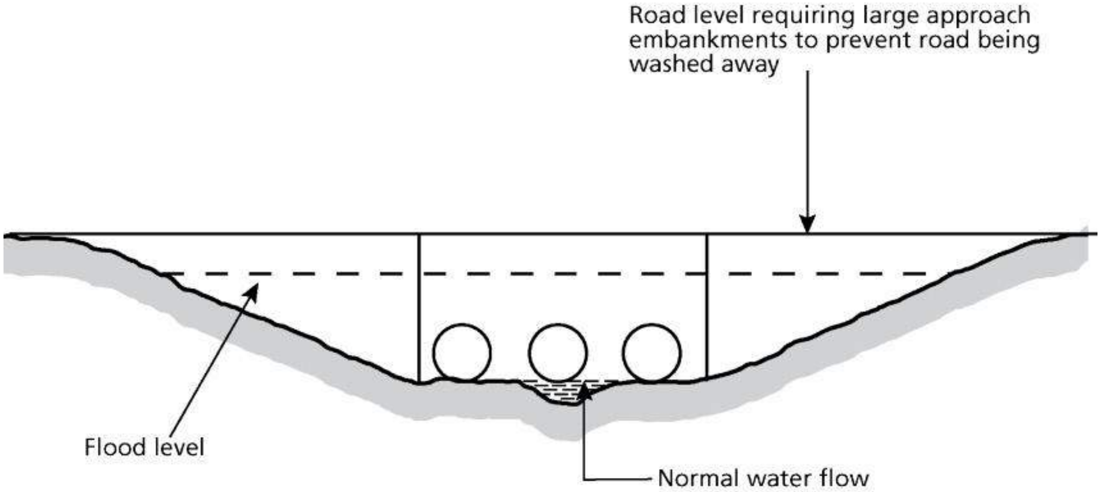


Figure 7-16: Large embankments required to prevent road flooding

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Low Volume Roads Manual
Volume 1 – Pavement Design

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8.1 Introduction

8.1.1 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 contained 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 drainage and moisture control for achieving satisfactory pavement life.

8.1.2 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 erosion caused by run-off from the road prism.

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 and erosion control in the vicinity of the road.

The chapter does not consider the hydrological aspects of drainage, such as the sizing of catchment areas, determination of run-off volumes and sizing of water crossings and drains to accommodate the flow. These aspects are addressed in *Chapter 7 – Hydrology and Drainage Structures*.

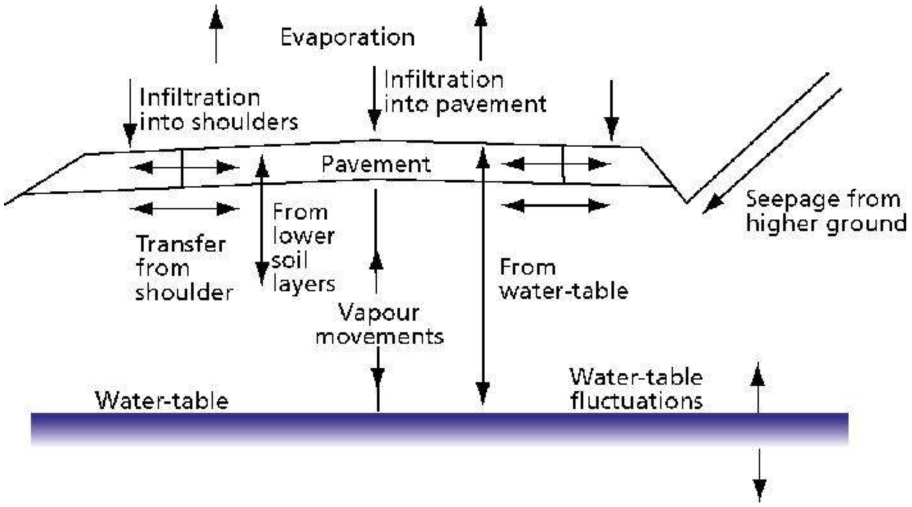
8.2 Sources of Moisture in a Pavement

8.2.1 General

The various causes of water movement into and out of a pavement are listed in Table 8-1 and illustrated in Figure 8-1. The Table highlights those aspects that should be addressed when designing an effective drainage system.

Table 8-1: 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 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 collection of water from vapour phase onto the underside of an impermeable surface.
Through the pavement surface	Through cracks under pumping action through the intact surfacing.
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.



Source: ARRB (2000)

Figure 8-1: Moisture movements in road pavements

8.2.2 Components of Drainage

Drainage is divided into external and internal drainage. External drainage is concerned with the control of water that is outside the road structure. However, since it is impossible to guarantee that water will not enter road structures during their service lives, it is important to ensure that water is able to drain out from within the pavement itself. Thus, 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 than can be adopted to avoid trapping water within the pavement.

8.3 External Drainage

8.3.1 General

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 by means of drainage channels.
- Allowing water to cross the road effectively from one side to the other.

8.3.2 Road Surfacing

The pavement surfacing of either a sealed or unsealed road constitutes an essential part of the drainage of a road. This surfacing, together with the cross slope on the carriageway ensures that rainwater does not enter the foundation of the road but is led to the side of the road.

Unpaved roads: The use of a natural gravel wearing course requires that the material is well protected from surface water. This can be achieved by reducing the permeability of the surfacing by ensuring that:

- The material is reasonably well graded with an appropriate plasticity for binding the material together (refer to *Chapter 10 – Structural Design: Unpaved Roads*).
- The soil is well compacted to at least 95% of BS heavy compaction.
- An appropriate camber of 4% - 6% is used to shed rainwater effectively.

Paved roads: 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 used for this purpose, some being more impermeable than others. These are addressed in *Chapter 11 – Surfacing.*

8.3.3 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 8-2.

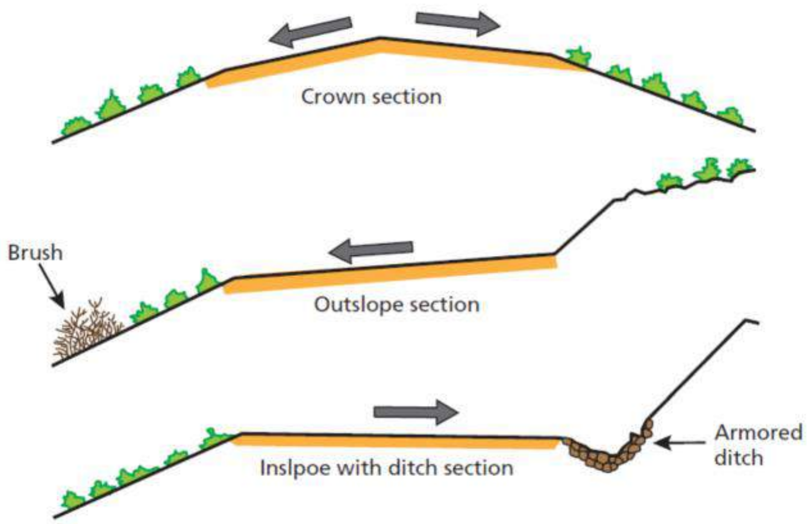


Figure 8-2: Three types of crossfall

Carriageway camber/crossfall slope: The design of the crossfall is often a compromise between the need for reasonably steep crossfalls for drainage and relatively flat crossfalls that are good for driver comfort and safety. The ideal crossfall depends on the pavement surfacing. The carriageway crossfall on straight road sections for various types of pavement surfaces is shown in Table 8-2.

Table 8-2: Typical carriageway crossfall values on straight road sections

Type of Pavement Surface	Crossfall (%)
Portland cement concrete	2.0 – 3.0
Asphalt concrete	2.5 – 3.5
Bituminous surface treatment	3.0 – 4.0
Gravel/waterbound macadam	4.0 – 6.0
Earth[1]	5.0 – 7.0

Note 1: For sandy roads, a 2%–3% camber would be more suitable to avoid wash out of the finer fraction.

On paved roads a camber of 3.5% is recommended. Although steeper than many traditional specifications, it does not cause problems for drivers in a low speed environment. 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 and 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 the shoulders. However, it is not usually possible to increase the crossfall from the value used for the running surface to a greater value for the 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 is given in Table 8-2.

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 (see Section 8.4). This can be achieved by constructing the top of the fill or lower subgrade with a crossfall of 4%-5% (TRL, 1993). 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 8-3, may be more difficult to achieve in practice than constructing all layers with the same crossfall.

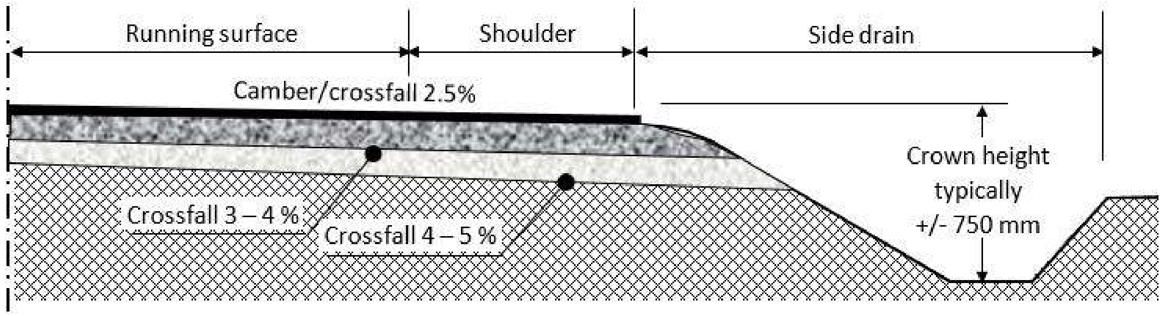


Figure 8-3: Illustrative drainage arrangements

Figure 8-4: Infiltration of water through a permeable surfacing

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 8-4, 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.

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 subbase level. These channels should be back-filled with 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.

8.3.4 Crown Height

Paved roads

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 centre line) 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 8-5). The recommended minimum crown height of 0.75 m applies to unlined drains 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 8-3. The capacity of the drain should meet the requirements for the design storm return period (*Chapter 7 – Hydrology and Drainage Structures*).

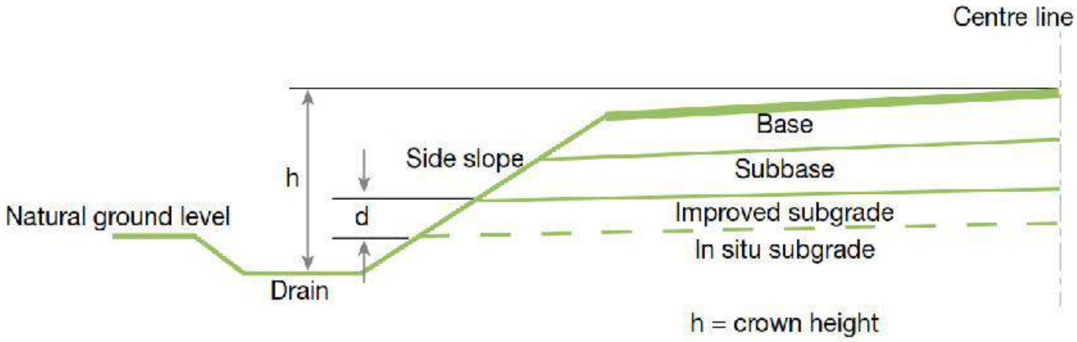


Figure 8-5: Crown height for paved road in relation to depth of drainage ditch

Table 8-3: 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, it is also 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 (distance d_{min} as indicated in Figure 8-5) to minimise 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 corresponding loss of strength.

Unpaved roads

Engineered earth roads: Engineered earth roads should be raised adequately to allow side drains to be constructed and water to be removed from the road structure. The crown of the road should be at least 350 mm above the base of the drain, irrespective of climatic zone, as the soils in engineered roads are generally more moisture sensitive than those in a well-compacted gravel road (see Section 10.2.3).

Gravel roads: The minimum crown height is dependent on the climate and road design class as shown in Table 8-4. Unless the existing road is well below existing ground level, this can usually be achieved by proper shaping of the roadbed to ensure adequate road levels, coupled with cutting table drains to an appropriate depth below existing ground level. Where necessary, additional fill will have to be imported or obtained from shallow cuttings to achieve the required h_{min}.

Table 8-4: Minimum height, h_{min}, between road crown and drain invert level

Road Class	Climate	
	Wet (N<4)	Dry (N>4)
	h_{min} (m)	h_{min} (m)
LVR5	0.55	0.45
LVR4	0.50	0.40
LVR3	0.45	0.35
LVR2	0.40	0.30
LVR1	0.35	0.25

Because of the critical importance of observing the minimum crown height and minimum height of the bottom of the sub-base above existing ground level along the entire length of the road, the measurement of this parameter should form an important part of the drainage assessment carried out during site investigations. This is to avoid any existing drainage problems associated with depressed pavement construction, often observed on gravel roads that have evolved over time with no strict adherence to observing minimum crown heights as illustrated in Figure 8-6.

Roads with sunken profiles

There may be situations where a road has a sunken profile for more than 200 m with no possibility of discharging water, as illustrated in Figure 8-6.

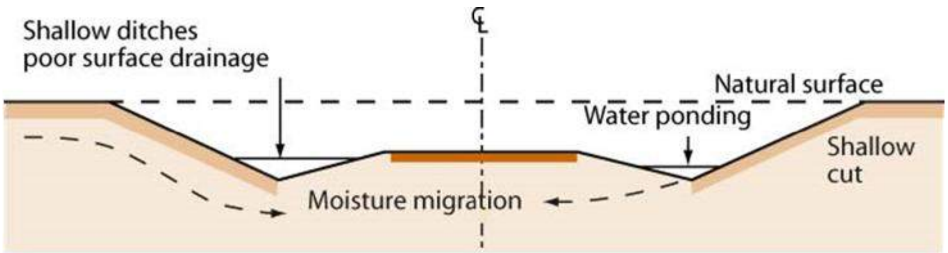


Figure 8-6: Potential drainage problems associated with sunken road profiles

The options for dealing with a sunken profile are presented in Table 8-5.

Table 8-5: Options for dealing with a sunken profile

Solution	Flat terrain (gradient < 2 %)	Rolling/mountainous terrain (gradient ≥ 2 %)
Raise the road formation to satisfy the h_{min} requirement and facilitate discharge of water through mitre drains.	Primary option.	May not be required if water can be discharged satisfactorily through mitre drains at regular intervals.
Lined drains	Not applicable.	Only required if the amount and velocity of the water in the side drains are likely to cause erosion.
Catch-water drains	Should be considered preferably in combination with a raised formation.	May be required to prevent high volume of water in side drains
Soak-away ponds	Should be considered preferably in combination with a raised formation.	Normally not required

Details of the basic option presented in Table 8-5 are discussed below.

Lined drains: If the side ditches are unable to discharge every 200 m, the build-up of water becomes significant and erosion is likely to occur, particularly on steeper gradients. Where this is likely, ditch lining with concrete should be considered. Widening the side drain to accommodate the high water flow may also be required.

Catch-water drains: In some locations the volume of water in the side drains may be relieved by constructing additional drains parallel to the road and several metres outside of the side drains. These should be 1m wide and excavated to a level just below the side drains. Water should be channelled from the side drain to the parallel drain by constructing mitre drains between them every 20 m. Note, however, that this option creates considerable additional maintenance requirements because deep parallel drains are difficult to de-silt. The solution is only viable in open terrain with no space restrictions.

Soak-away ponds: If the soil adjacent to the road is free draining, mitre drains can be constructed to soak-away ponds. These may be constructed of approximate dimensions 5 m x 5 m x 1 m deep every 50 m along both sides of the road. This capacity would be sufficient to hold the water falling on the road from a storm of up to 100 mm of rain. Note, however, that this option may not be applicable in extremely wet regions where water does not soak or evaporate rapidly enough.

8.3.5 Drains

Side drains

Side drains serve two main functions namely to collect and remove surface water from the immediate vicinity of the road and, where needed, to prevent any sub-surface water from adversely affecting the road pavement structure. It is essential to install a system of side drains that discharges water frequently to avoid high flow concentrations that will inevitably lead to erosion.

Side drains can be constructed in three forms: V-shaped, rectangular or trapezoidal (Figure 8-7). The choice depends on the type of technology (use of graders, labour-based technology), the required hydraulic capacity, arrangements for maintenance, space restrictions, traffic safety and any requirements relating to the height between the crown of the pavement and the drain invert. For safety reasons a wide trapezoidal and shallower drain for a given flow capacity is preferable to a deeper "V" or rectangular drain.

V-shaped drains: Although relatively easily constructed by a motor or towed grader, they should be discouraged because of their potential to scour.

Rectangular shaped drains: These require little space but need to be lined with rock, brick or stone masonry or concrete to maintain their shape.

Trapezoidal drain: These can be constructed and maintained easily by hand and improve traffic safety.

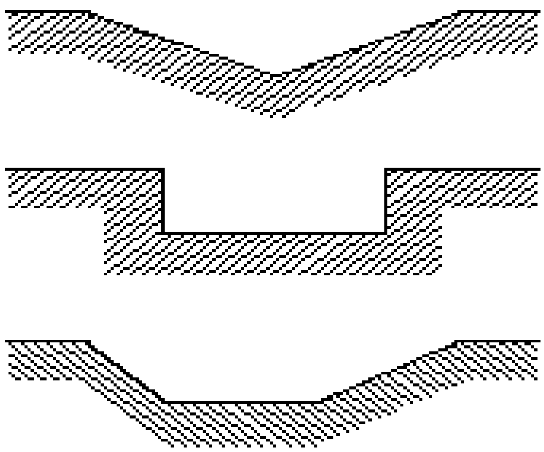


Figure 8-7: Typical types of side drains

The minimum recommended width of the side drain is 500 mm and the minimum recommended longitudinal gradient is 0.5%. Slackening of the side drain gradient in the lower reaches over significant lengths of drain should be avoided in order to prevent siltation.

Side drains are normally located beyond the shoulder breakpoint and parallel to the centre line of the road. While usually employed in cuts, they may also be used to run water along the toe of a fill to a point where the water can conveniently be diverted, either away from the road prism or through it by means of a culvert. When used in conjunction with fills, side drains should be located as close to the edge of the reserve boundary as is practicable to ensure that erosion of the toe of the fill does not occur.

The following recommendations are made regarding desirable slopes for side drains:

- To avoid ponding and siltation, the minimum slope should be in the range 0.4% to 0.5%.
- To avoid erosion drains steeper than 3% may need scour protection, depending on the erodibility of the soil and the vegetative cover. The distance between scour checks depends on the road gradient and the erosion potential of the soils. Table 8-6 shows recommended values for normal soils. The spacing should be reduced for highly erodible soils.

Table 8-6: Spacing between scour checks

Road gradient (%)	Scour check interval (m)
3	Usually not required
4	17
5	13
6	10
7	8
8	7
9	6
10	5
12	4

Access across side drains for pedestrians, animals and vehicles needs to be considered. Community representatives should be consulted with regard to locations, especially for established routes and in the villages or towns. The methods that could be used are:

- Widening the drain, taking its alignment slightly away from the road and hardening the invert and sides of the drain.
- Beam/slab covers or small culverts.

The arrangement must be maintainable and not risk blockage of the side drain. Failure to accommodate these needs will usually result in ad hoc arrangements that compromise the function of the side drain causing blockage of the water flow.

Mitre drains

These drains are constructed at an angle to the centre line of the road. They are intended to remove water from a side drain and to discharge it beyond the road reserve boundary. The amount of water in the drain should ideally be dispersed and its speed correspondingly reduced before discharge. Speed can be reduced not only by reducing the volume (more frequent spacing of mitre drains), and hence the depth of flow, but also by positioning the mitre drain so that its toe is virtually parallel to the natural contours. The downstream face of a mitre drain is usually protected by stone pitching, since the volume and speed of flow of water which it deflects may cause scour and ultimately lead to breaching of the mitre drain.

In order to ensure that water flows out of the side drain into the mitre drain, a 'block-off' is required as shown in Figure 8-8.

It is essential that the mitre drain is able to discharge all the water from the side drain. If the slope of the mitre drain is insufficient, the mitre drain needs to be made wide enough to ensure this.

The angle between the mitre drain and the side drain should not be greater than 45 degrees. An angle of 30 degrees is ideal. If it is necessary to take water off at an angle greater than 45 degrees, it should be done in two or more bends so that each bend is not greater than 45 degrees as shown in Figure 8-8.

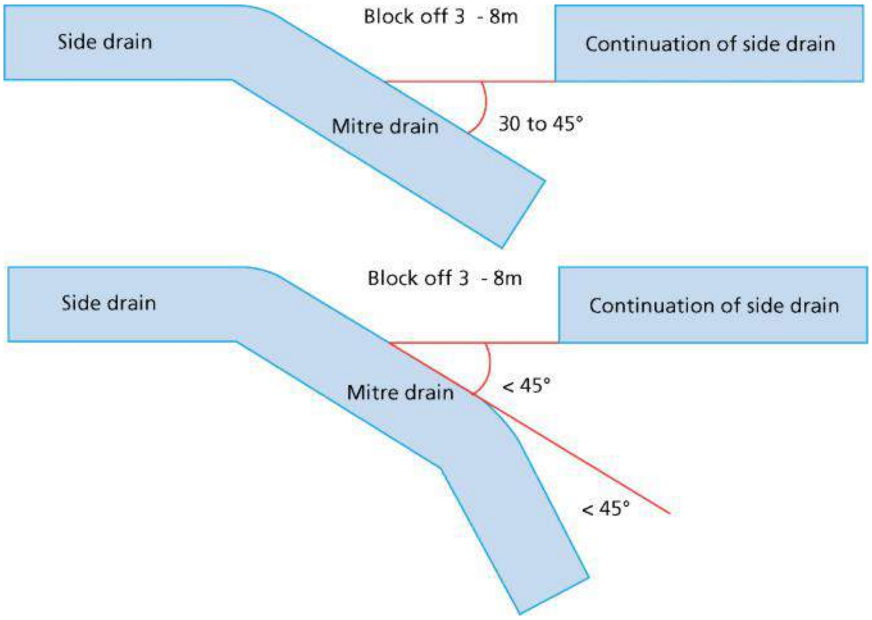


Figure 8-8: Schematic layout of mitre drains

The desirable slope of mitre drains is 2%. The gradient should not exceed 5% otherwise there may be erosion in the drain or on the land where the water is discharged. The drain should lead gradually across the land, getting increasingly shallower. Stones may need to be laid at the end of the drain to help prevent erosion.

In flat terrain, a small gradient of 1% or even 0.5% may be necessary to discharge water, or to avoid very long drains. These low gradients should only be used when absolutely necessary. The slope should be continuous with no high or low spots. For flat sections of the road, mitre drains are required at frequent intervals to minimise silting. In mountainous terrain, it may be necessary to accept steeper gradients. In such cases, appropriate soil erosion measures should be considered.

As indicated in Table 8-7, the maximum spacing of mitre drains is dependent on the road gradient. However, depending on engineering judgement mitre drains could be required more frequently than this and values as low as one every 20 m may be required to avoid damage to adjacent land, especially where it is cultivated.

Table 8-7: Maximum spacing of mitre drains

Road Gradient (%)	Maximum mitre drain interval (m)
12	40
10	80
8	120(1)
6	150(1)
4	200(1)
2	80(2)
<2	50

Source: Adapted from Robinson and Thagesen (1996).
Notes: 1. A maximum of 100 m is preferred but not essential.
2. At low gradients silting becomes a problem.

When land restrictions do not permit the inclusion of mitre drains at the intended intervals, widening and lining of the drains may be required to cope with the increased volume of water and to protect against erosion in the side drains.

Catch-water (Interceptor, Cut-off) drains

These drains are constructed to prevent water flowing into vulnerable locations (e.g. down cut faces) by 'catching', 'intercepting' or 'cutting off' or the water flow and diverting it to a safe point of discharge, usually a natural watercourse, as illustrated in Figure 8-9.

Catch-water drains above cut faces should have a gradient of 2% on their full length and should be at least 3 – 5 m from the cut face. If steeper gradients in the drain are unavoidable then scour checks should be installed or the drain should be lined. The drain should also be lined where seepage will weaken the cut slope. Alternatively, the drain should be replaced by a vegetated earth bund. Depending on topography, catch-water drains should be relative wide and shallow, as illustrated typically in Figure 8-9, with sides back-sloped at about 2:1 (vertical:horizontal) or less.

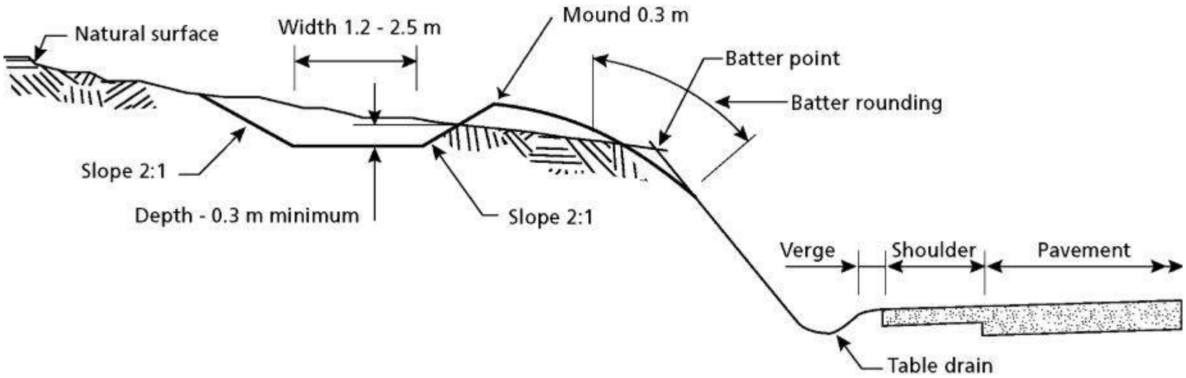


Figure 8-9 : Catch-water drain

Chutes

Chutes are structures intended to convey a concentration of water down a slope that, without such protection, would be subject to scour, as shown in Figure 8-10. Since flow velocities are very high, stilling basins are required to prevent downstream erosion. The entrance of the chute needs to be designed to ensure that water is deflected from the side drain into the chute, particularly where the road is on a steep grade. On embankments it may be necessary to lead water to the top of chutes using kerbing.

Figure 8-10: Open chute

It is important that chutes be adequately spaced to remove excess water from the shoulders of the road. Furthermore, the dimensions of the chutes and stilling basins should be such that these drainage elements do not represent an excessive risk to errant vehicles. Generally, they should be as shallow as is compatible with their function and depths in excess of 150 mm should be viewed with caution.

Because of the suggested shallow depth, particular attention must be paid to the design and construction of chutes to ensure that the highly energised stream is not deflected out of the chute. This is a serious erosion hazard which can be obviated by replacing the chute with a pipe.

8.4 Internal Drainage

8.4.1 General

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.

8.4.2 Avoiding Permeability Inversion

A permeability inversion exists when the permeability of the pavement and subgrade layers decreases with depth. Under 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 wheeltrack 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. However, it is unlikely that for LVRs there will be any luxury in the choice of materials. 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.

8.4.3 Sub-surface Drainage

Seepage may occur where the road is in cut and may result in groundwater entering the sub-base or subgrade layers as illustrated in Figure 8-11 and Figure 8-12. 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 as shown in Figure 8-13. 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.

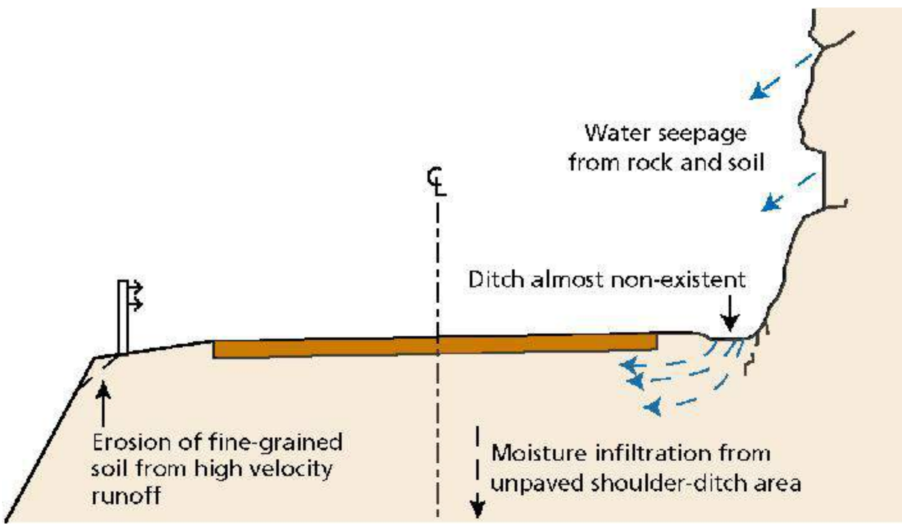


Figure 8-11: Inadequate side drains

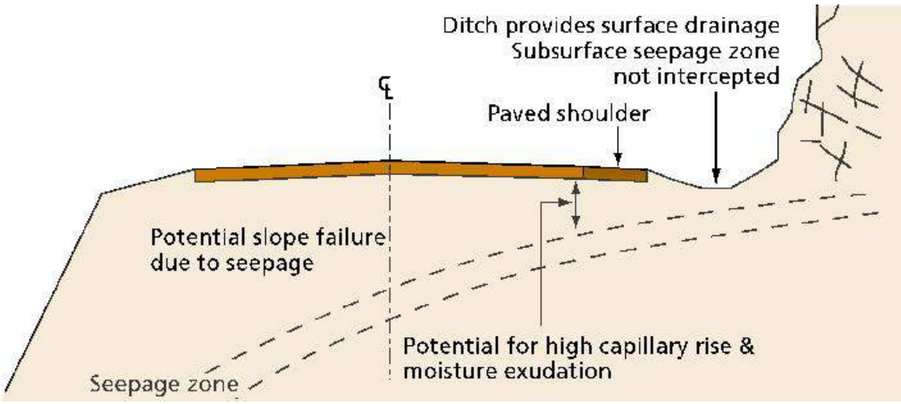


Figure 8-12: Inadequate side drains and subsurface drainage

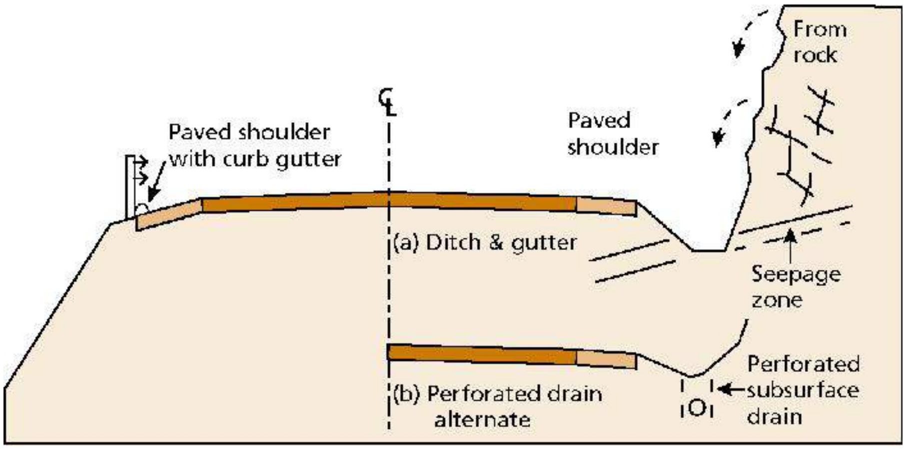


Figure 8-13: Proper interception of surface runoff and subsurface seepage

8.5 Types of Erosion

8.5.1 General

Parts of the drainage system and surrounding terrain are subject to erosion if the quantity and velocity of the flowing water is above critical values. Engineering measures must be taken to prevent serious erosion both close to the road itself but also for some distance away from the road where the drainage system has concentrated the water flow.

Erosion of soil is caused by water travelling on the surface of the soil at a velocity that produces stresses in the soil that exceed the soils cohesion and therefore its resistance to movement. This, in turn, is influenced by a wide range of factors including:

- Physical factors. These include soil type, geology, topography, land cover and climate, particularly rainfall.
- Road location. The location of the road in relation to slope, stream channels, and sensitive soils has a direct effect on erosion and the amount of sediment that needs to be controlled.
- Road standards and construction. Designed road width, steepness of cut banks or road fills, methods of construction, and drainage installations will directly affect the area of disturbance and potential for failure following road construction.

8.5.2 Erosion Problems

Erosion problems may be categorized based on the mitigation measures required in the three distinct areas of the road (Figure 8-14) namely:

- The upper catchment (the area draining towards the road).
- The road reserve.
- The lower catchment area (the area on the down-stream side of the road reserve

Souce: Erikson and Kidanu (2010)

Figure 8-14: Erosion areas–upper catchment, road reserve and lower catchment

Upper catchment area

Washaways or erosion in the road reserve are generally caused by runoff from the upper catchment area, often causing damage to side slopes, drains and drainage structures. The main factors contributing to these problems include:

- High runoff rates caused by naturally low water retention capacity in the catchment areas due to poor vegetation cover and/or impervious soils and/or hilly terrain.
- Excessive water runoff as a result of poor land management practices resulting in low water retention capacity in cultivated fields and grazing land and from built up areas (roofs, foot paths and cattle tracks, etc.).
- Runoff from cultivated land, grazing areas, footpaths, cattle tracks, and so on, adjacent to a road. This carries sediment and transports it towards the road resulting in siltation of the road drainage structures.
- Lack of appropriate erosion control measures.

Road reserve

Typical soil erosion problems in the road reserve include:

- Scouring/gullying in side drains.
- Scouring in culvert inlets and outlets.
- Gullying on culvert outlets.
- Scouring of bridges' wing-walls and abutments.
- Siltation of culverts and drains.
- Slope failures on embankments.

Lower catchment area

The lower catchment refers to the area below the road reserve that receives water from the upper catchment and the road itself. This is where the most serious soil erosion problems usually occur (due to the nature of road alignments acting as barriers to natural surface run off and concentrating water flows) with serious damage to land and other properties. Typical soil erosion problems in the lower catchment areas include:

- Gullying of culvert outfalls.
- Gullying of mitre drain outfalls.
- Flooding and silt deposition causing damage to crops and property.

8.6 Erosion Control Measures

8.6.1 General

The first step is to identify erosion prone areas such as high rainfall areas, hilly areas with unstable slopes, deforested areas and areas with easily erodible soil types. This should be followed by an assessment of the road reserve and the upper and lower catchment areas for likely or potential erosion problems due to runoff from these areas.

8.6.2 Gully Erosion

Gully erosion occurs when runoff water accumulates and rapidly flows in narrow channels during or immediately after heavy rains removing soil to a considerable depth. Typical examples of gully activity are illustrated in Figure 8-15.

Source: Erikson and Kidanu (2010)

Figure 8-15: Example of V-shaped gully activity (left) and U-shaped stabilizing gully (right).

A starting point for preventing gully erosion is to anticipate this type of erosion by providing appropriate structures and improving land use in the upper and lower catchment areas. The potential erosion problems can be determined from land use patterns, visual inspection and through community informants. The working principle should be 'arrest it before it is established and control it when it exists'.

There are two basic principles involved for preventing gully control erosion. These are:

- Reduce the runoff volumes of water entering the gulley.
- Reduce the erosive power (speed) of the water flow by installing scour checks and/or check dams (drop structures) which create steps that dissipate the energy of the water flow allowing vegetation to establish itself and stabilize the gully.

Typical measures for the control of gullies are shown in Table 8-8.

Table 8-8: Typical gully control measures

Type of gulley	Suggested control measures
V-Shaped gullies	Install check dams and plant grass in trapped sediments.
U-shaped gullies	No work is needed in most U-shaped gullies since they do not manifest active erosion and usually stabilize on their own. If necessary, prevention of side wall erosion on bends through stone lining and/or establishing suitable vegetation along the foot of the bank is required. Grass like "Vetiver" in gullies proved to be the fastest and cheapest way of stabilising erosion damages.
Head of gulley	Dam construction downstream of the fall. Sloping and protecting the channel. Construction of drop structures (steps that dissipate energy of the water flow).

8.6.3 Protection of Drains

Unlined drains

Critical length: This is defined as the maximum length of an unlined ditch in which water velocities do not give rise to erosion. The maximum velocity of water can be calculated from the slope, shape and dimensions of the ditch, volume of water and from the roughness coefficient of the material (refer to *Chapter 7 – Hydrology and Drainage Structures, Section 7.4*). The recommended maximum permissible velocities for different types of material to prevent scour in un-lined drains are given in Table 7-14. Knowing the maximum permissible velocity for each type of material, the maximum length of unlined ditch in this material can then be determined.

Where flow velocities exceed those shown in Table 7-14 and are likely to cause scouring, the velocity in the drains must be reduced. Methods include:

- Scour checks.
- Grassed waterways.
- Drain lining.

Scour checks: Act as small dams and, when naturally silted up on the upstream side, effectively reduce the gradient of the drain on that side, and therefore the velocity of the water. Scour checks are usually constructed with natural dry-packed stone, stone masonry, concrete or with wooden stakes (e.g. bamboo) in combination with dry-packed stones.

The level of the scour check must be a minimum of 200 mm below the edge of the carriageway in order to avoid the water being diverted out of the side drains.

Typical designs for scour checks are shown in Figures 8-16 and 8-17.

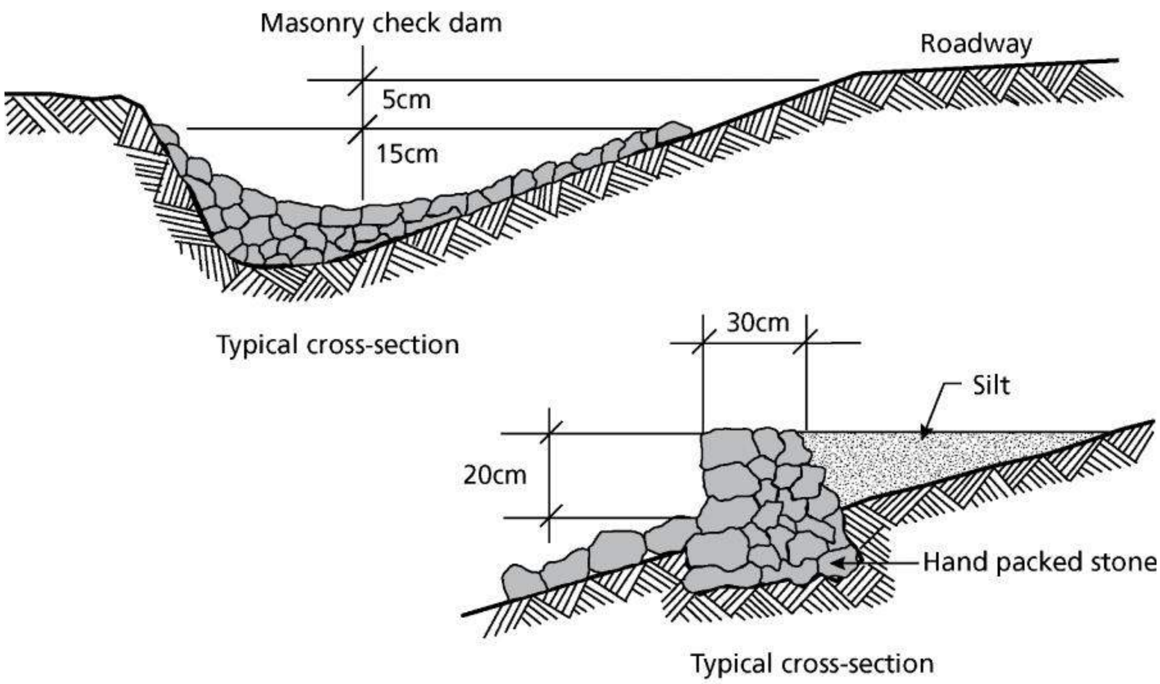


Figure 8-16: Scour check made from hand-packed stone

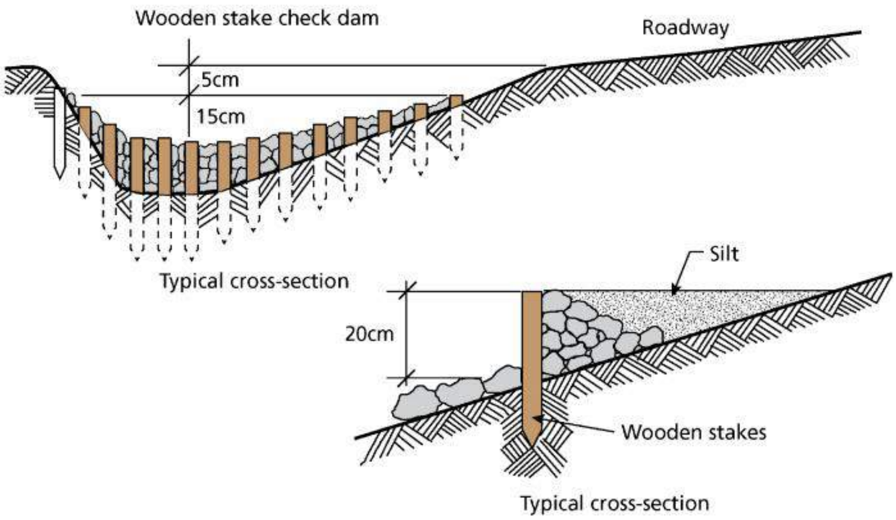


Figure 8-17: Scour check made from wooden stakes

The distance between scour checks depends on the road gradient and the erosion potential of the soils. Table 8-5 in Section 8.3.5 shows recommended values. In areas that evidently have erodible soils, it is recommended that the drains are lined at gradients above 3%. The preferred shape of lined side drains is trapezoidal.

Lined drains

Particularly in on steep gradients or where the distance between mitre drains is too long, it may be necessary to consider lining the side drains to avoid severe erosion. The drain lining should connect to the edge of the surfacing as shown in Figure 8-19. Drain lining can be made from mass concrete, concrete blocks, stone masonry or brick masonry. Rock, if available in the vicinity of the road, is the preferred (lowest cost) option and can be laid as dry or wet masonry. The size of the stone should be a minimum of 200 mm to avoid the rock being washed away by water. The masonry work needs to be well laid to ensure that water does not enter underneath the lining allowing it to become unstable, undercut and eventually wash away.

Figure 8-18: Poor – Gap between surfacing and drain lining

Figure 8-19: Good – Drain lining connected to edge of surfacing

8.6.4 Protection of Outfalls

In principle, unless culverts and mitre drains discharge directly into a natural water course, onto a non-erodible area, or into water harvesting structures, there is need to construct artificial waterways to conduct the runoff safely to valley bottoms where it can join a stream or river. These waterways are usually aligned straight down a slope (perpendicular to the contours). Where there is a natural depression or small valley that is well-stabilised with vegetation (natural waterway) road drainage water can be directly discharged but if there is no such natural waterway, an artificial waterway must be installed. It may be necessary to grass the waterway as illustrated in Figure 8-20.

On relatively steep slopes, it may be necessary to stabilise the waterway with check dams which are similar in principle to scour checks, as shown in Figure 8-20.

Source: Erikson and Kidanu (2010)

Figure 8-20: Grassed waterways with and without checkdams

8.6.5 Protection of Slopes

It is generally not appropriate to rely on the eventual re-establishment of natural vegetation to protect the side slopes of roads, particularly where they are steep and located in high rainfall areas. In such situations, the use of appropriate bio-engineering solutions is recommended.

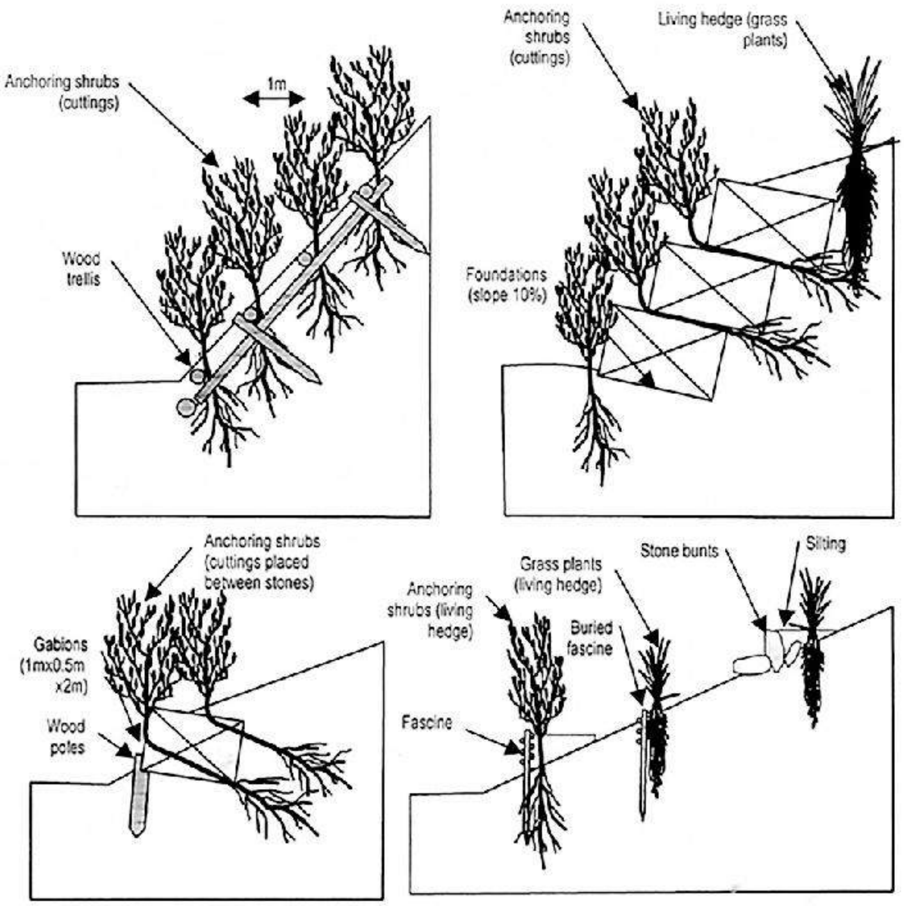
Bioengineering can be broadly defined as the use of vegetation, either alone or in conjunction with engineering structures and non-living plant material, to reduce erosion and shallow-seated instability on slopes. In bio-engineering applications there is an element of slope stabilisation as well as slope protection in which the principal advantages are:

- Vegetation cover protects the soil against rain splash and erosion and prevents the movement of soil particles down slope under the action of gravity.
- Vegetation increases the soil infiltration capacity, helping to reduce the volume of runoff.
- Plant roots bind the soil and can increase resistance to failure, especially in the case of loose, disturbed soils and fills.
- Plants transpire considerable quantities of water, reducing soil moisture and increasing soil suction.
- The root cylinder of trees holds up the slope above through buttressing and arching.
- Tap roots or near vertical roots penetrate into the firmer stratum below and pin down the overlying materials.
- Surface run-off is slowed by stems and grass leaves.

Examples of bio-engineering solutions typically employed include the following:

- The use of Vetiver grass for stabilising terraces and gullies.
- The use of trees, shrubs and other grasses to stabilise slopes, protect embankments, and provide live check structures in drains.

Figure 8-21 illustrates bio-engineering measures aimed at controlling erosion on moderate slopes.



Source: Lebo and Schelling (2001)

Figure 8-21: Bio-engineered slope protection measures

Key factors in plant selection

The main factors are:

- The plant must be of the right type to undertake the bio-engineering technique that is required. The possible categories include:
  - A grass that forms large clumps.
  - A shrub or small tree that can be grown from woody cuttings.
  - A shrub or small tree that can grow from seed in rocky sites.
  - A tree that can be grown from a potted seedling.
- The plant must be capable of growing in the location of the site (i.e. water requirements and slope angle). There is no single species or technique that can resolve all slope protection problems.
- It is always advisable to use local species which do not invade and harm the indigenous environment, and which have been shown to be capable of protecting the slopes from sliding in the past.
- Large trees are suitable on slopes of less than 3H:2V or in the bottom 2 m of slopes steeper than 3H:2V. Maintaining a line of large trees at the base of a slope can help to buttress the slope and reduce undercutting by streams.
- Grasses that form dense clumps generally provide robust slope protection in areas where rainfall is intense. They are usually best for erosion control, although most grasses cannot grow under the shade of a tree canopy.

- Shrubs (i.e. woody plants with multiple stems) can often grow from cuttings taken from their branches. Plants propagated by this method tend to produce a mass of fine, strong roots. These are often better for soil reinforcement than the natural rooting systems developed from a seedling of the same plant.
- In most cases the establishment of full vegetation cover on unconsolidated fill slopes may take one to two rainy seasons. Likewise, the establishment of full vegetation on undisturbed cut slopes in residual soils and colluvial deposits may need 3 to 5 rainy periods. Less stony and more permeable soils have faster plant growth rates, and drier locations have slower rates. Plants do not establish easily on slopes steeper than 1V:1H.
- Plant roots cannot be expected to contribute to soil reinforcement below a depth of 500 mm.
- Plants cannot be expected to reduce soil moisture significantly at critical periods of intense and prolonged rainfall.
- Grazing by domestic animals can destroy plants if it occurs before they are properly grown. Once established, plants are flexible and robust. They can recover from significant levels of damage (e.g. flooding and debris deposition).

Site preparation: Before bio-engineering treatments are applied, the site must be properly prepared. The surface should be clean and firm, with no loose debris. It must be trimmed to a smooth profile, with no vertical or overhanging areas. The object of trimming is to create a semi-stable slope with an even surface to form a suitable foundation for subsequent works.

The soil and debris slopes must be trimmed to the final desired profile, with a slope angle of between 30° and 45°. (In certain cases, the angle will be steeper, but this should be carefully reviewed in each case). Excessively steep sections of slope must be trimmed off, whether at the top or bottom. In particular, slopes with an over-steep lower section should be avoided since a small failure at the toe can destabilise the whole slope above.

All small protrusions and unstable large rocks must be removed. Indentations that make the surrounding material unstable must be eradicated by trimming back the whole slope around them. If removing indentations would cause an unacceptably large amount of work, they should be excavated carefully, and a buttress wall built. All debris must be removed from the slope surface and toe and taken to an approved tipping site. If there is no toe wall, the entire finished slope must consist of undisturbed material.

Recommended techniques: Table 8-9 indicates the different types of bio-engineering techniques recommended for various kinds of slopes and soil materials for both cut and fill situations.

Table 8-9: Recommended general bioengineering procedures

Site characteristics	Recommended techniques
Cut-Slopes	
Cut slopes in soil, very highly weathered rock or residual soil, at any grade up to 1H:2V.	Grass planting in lines, using slip cuttings. Only likely to be successful in wet areas where slope is > 1H:1V.
Cut slopes in colluvial debris, at any grade up to 1H:1V (steeper than this would need a retaining structure).	
Trimmed landslide head scarps in soil, at any grade up to 1H:2V.	
Roadside lower edge or shoulder in soil or mixed debris.	
Cut slope in mixed soil and rock or highly weathered rock, at any grade up to about 1H:4V.	Direct seeding of shrubs and trees in crevices.
Trimmed landslide head scarps in mixed soil and rock or highly weathered rock, at any grade up to about 1H:4V.	

Fill Slopes	
Fill slopes and backfill above walls without a water seepage or drainage problem; these should first be re-graded to be no steeper than 3H:2V.	Brush layers (live cuttings of plants laid into shallow trenches with the tops protruding) using woody cuttings from shrubs or trees.
Debris slopes underlain by rock structure, so that the slope grade remains between 1H:1V and 4H:7V.	Palisades (the placing of woody cuttings in a line across a slope to form a barrier) from shrubs or trees.
Other debris-covered slopes where cleaning is not practical, at grades between 3H:2V and 1H:1V.	Brush layers using woody cuttings from shrubs or trees.
Fill slopes and backfill above walls showing evidence of regular water seepage or poor drainage; these should first be re-graded to be no steeper than about 3H:2V.	Fascines (bundles of branches laid along shallow trenches and buried completely) using woody cuttings from shrubs or trees, configured to contribute to slope drainage.
Large and less stable fill slopes more than 10m from the road edge (grade not necessarily important, but likely eventually to settle naturally at about 3H:2V).	Truncheon cuttings (big woody cuttings from trees).
The base of fill and debris slopes.	Large bamboo planting; or tree planting using seedlings from a nursery.

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9.1 Introduction

9.1.1 Background

The objective of pavement design is to produce an economic, well balanced pavement structure, in terms of material types and thicknesses, that can withstand the expected traffic loading over a specified period (the chosen design life of the pavement), without deteriorating below a pre-determined level of service. To achieve this goal, sufficient knowledge of the subgrade strength, pavement materials, traffic loading, local environment factors (particularly climate and drainage) and their interactions is required to be able to predict reasonably the performance of any pavement configuration. In addition, there should be a clear view as to the level of performance and pavement condition that is considered appropriate in the circumstances for which the pavement structure is being designed.

Pavement design for LVRs presents a particular challenge to designers. This is largely because, until relatively recently, such roads were not specifically catered for and the step from a gravel road to a paved road one was a large one. However, considerable research has been carried out internationally that has led to the development of simplified pavement design methods that enable unpaved roads to be upgraded economically to a paved standard by making optimal use of local materials that do not meet the standard specifications found in most design manuals. It is these design methods that are described in this chapter.

One of the biggest changes in the design process discussed in this chapter is that it is based on extending design procedures for unpaved roads upwards for low volume paved roads. The more conventional practice of trying to reduce conventional paved road designs for low volume roads has been found to have numerous pitfalls. Probably the most significant of these is the use of the normal assumptions of isotropic, elastic, uniform material concepts, that are less appropriate for natural gravels and local materials used in LVRs.

9.1.2 Approach to Design

The general approach to the design of LVRs differs in several respects from that for HVRs. For example, conventional pavement designs are generally directed at relatively high levels of service requiring numerous layers of selected materials. However, significant reductions in pavement costs for LVRs can be achieved by reducing the number of pavement layers and/or thickness, by using local materials and by using lower-cost, more appropriate surfacing options.

An important aspect of the design of HVRs is the minimization of pavement deflections. However, many of the lighter LVR pavement structures can tolerate relatively higher deflections (more than 1.0 mm). This is not necessarily a problem, but the choice of surfacing would certainly be influenced, with more flexible types of bituminous seal being necessary.

Ultimately, the challenge of good pavement design for LVRs is to provide a pavement that is appropriate to the road environment in which it operates and fulfils 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.

9.1.3 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 level of service as cost-effectively as possible. The materials comprising these pavement layers must possess the following attributes if the pavement is to perform satisfactorily within the dictates of the prevailing road environment:

- Sufficient stiffness (load-spreading ability) which is achieved essentially through inter-particle friction and shear strength (as measured with the DCP), which depend on the presence of horizontal confining stresses.
- Sufficient bearing capacity which is the ability to withstand repeated cycles of vertical stress without excessive deformation.

Figure 9-1 illustrates conceptually the way in which a pavement functions under loading.

In essence, the wheel load, W, is transmitted to the pavement surface through the tire. The pavement then spreads the wheel load to the subgrade so that the maximum pressure on the subgrade is reduced sufficiently to avoid overstressing it to an unacceptable level. This can be achieved by proper selection of pavement materials of appropriate thickness and quality.

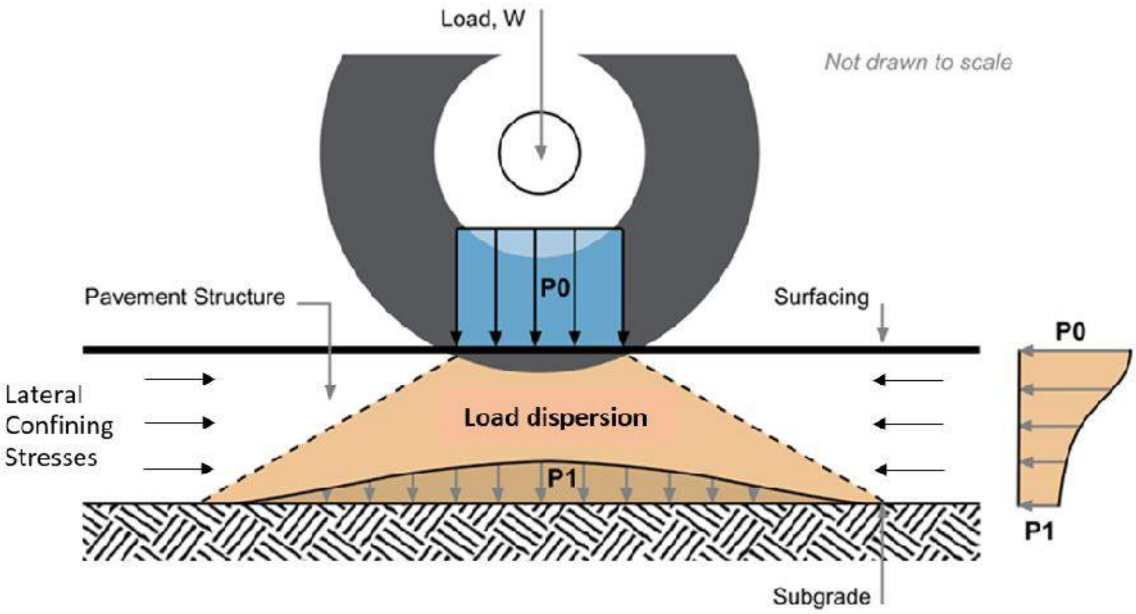


Figure 9-1: Dispersion of surface load through a granular pavement structure

9.1.4 Purpose and Scope

The purpose of this chapter is to provide details of the manner of determining the structural requirements of Low Volume Sealed Road (LVSR) pavements in terms of the required layer thicknesses and material quality for different traffic categories.

The chapter covers three methods of design for rural roads, two of which are essentially 'catalogue' methods, which are the most common methods of design for LVSRs. For each design method, guidance has been provided on developing appropriate pavement structures based on experimental and empirical evidence for a range of subgrade strengths and traffic loading classes.

The most common surfacing of LVSRs is a thin, flexible bituminous layer designed to produce a durable and waterproof seal, as discussed in *Chapter 11 - Surfacing*. Such seals do not add any significant structural strength to the pavement and thus do not affect the pavement design.

Section 9.2 also deals with the design of LVSRs with surfacings that provide a structural component such as concrete and discrete element surfacings etc. The structural design for such roads is therefore, different from the design of those with thin, flexible surfacings.

Section 9.3 provides guidance on the design of urban and peri-urban roads and streets based on a number of catalogues.

9.2 Design of Rural LVSRs

9.2.1 General

The approach to the design of rural LVSRs 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 designs that will meet with the multiple social, economic and environmental requirements of Malawi in a sustainable manner. For example, a recognition that pavement distress is generally attributable more to the effects of the natural environment than to the traffic loading. As described below, the different manners of dealing with this, and other, issues are addressed in the three design methods presented in this chapter.

9.2.2 Design Methods

Three methods of pavement design are addressed in this chapter as follows:

- The DCP-DN method
- The DCP-SN method
- The DCP-CBR method

As illustrated in Figure 9-2, the above methods are, in principle, suitable for the design of entirely new roads where none existed before (greenfield projects) or for upgrading existing roads to a higher standard (brownfield projects).

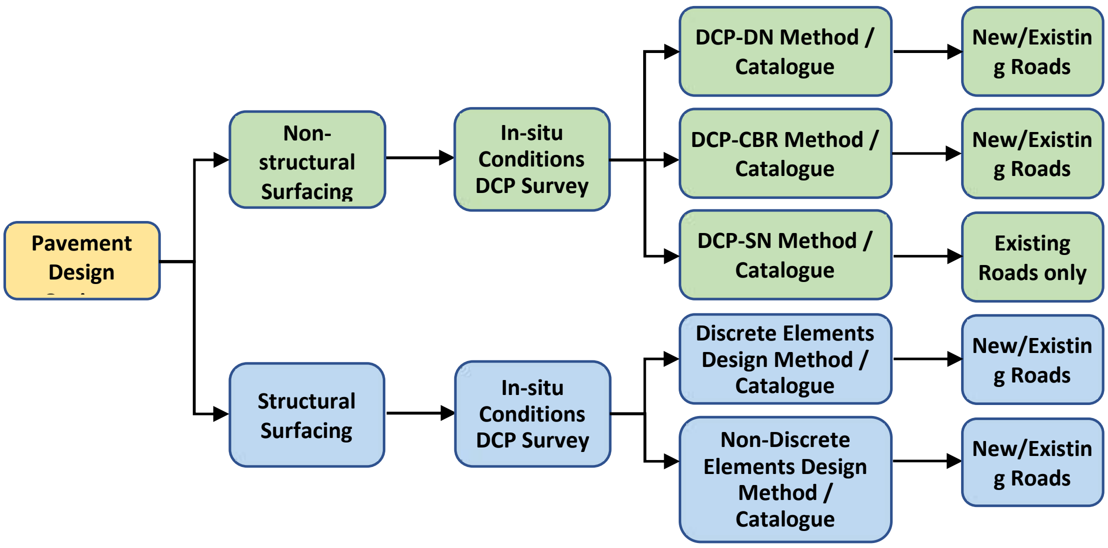


Figure 9-2: Pavement design options available

The design procedure and requirements for each of the 3 design methods mentioned above are summarized below.

9.2.3 Design Procedure

As with all empirical methods of pavement design, the four main requirements of the design procedure are as follows:

- Assessment of design traffic loading.
- Assessment of subgrade strength.
- Selection of pavement materials.
- Determination of pavement layer requirements (thickness and/or strength).

Apart from the determination of traffic loading, which is generally quite straight forward, as presented in *Chapter 6 – Traffic*, the other aspects of the design procedure vary between the three LVR methods mentioned above and, as a result, must be fully understood in order to produce credible designs. For this reason, a brief description of the key input requirements for the three design methods is presented below, whilst the detailed procedures for the design are addressed in Sections 9.2.5 to 9.2.7.

9.2.4 Design of Roads with Non-structural Surfacings

General

A primary objective of the pavement design of LVSR is to make optimum use of natural materials close to, or in the vicinity of the road, in order to reduce costs. The design must, at the same time, be responsive to the local environment. Since natural gravels are often quite moisture sensitive, it is imperative that the designer gets intimately familiar with the behaviour of the materials, for instance under the influence of moisture. As described below, the different design methods handle the issue of material classification and specification differently.

Design input requirements

To achieve an optimum pavement design, the design engineer must first determine the strength of the subgrade or the existing road structure and determine the strengths and layer thicknesses required for the new pavement structure based on the appropriate design catalogue or the Structural Number requirements and the associated specifications. A comparison of the existing situation with the required structure then provides the engineer with the information required to design the additions and modifications that are necessary.

In all of the pavement design methods described in this Manual, the DCP is used for characterizing the strength of the existing in-situ materials through a DCP survey along the alignment. The procedure for carrying out a DCP Survey is described in *Chapter 4 – Site Investigations*. However, the manner of determining the subgrade strength differs between the design methods as described below.

Choice of design method

The designer can choose any of the three design methods described in this Manual in order to compare the resulting designs and associated costs before a final decision is made on the preferred option.

The primary differences between the three methods are summarised in Table 10-1, and the various issues are discussed in detail later in the chapter.

Table 9-1: Comparison of DCP-DN and DCP-SN/CBR methods

Property	DCP-DN method	DCP-SN/CBR method
Strength	Use DCP-DN (in mm/blow) directly to assess in-situ conditions. No modifications required.	Use DCP to assess in-situ conditions. Requires conversion of DN to CBR. CBR converted to soaked values and layer strength coefficients for SN.
Uniform Sections	CUSUM[1] based on actual DN for each layer and DSN_{450} or DSN_{800} values of each point.	CUSUM based on SN or SNC of each individual point or any of the parameters obtained from the DCP test, e.g. subgrade strength, subgrade thickness, subbase strength etc.
Layers	Default 150 mm layers with weighted average layer strength. Layer thicknesses may be varied.	Variable layer thicknesses with average strength. Analyses for multiple layers (bases, subbases and subgrade(s).
Design	Subgrade strength assessed at anticipated long-term moisture condition. Variable strength for base/subbase depending on traffic load class.	Requires minimum soaked CBR of 45% for base. For upgrading requirements, the lower 10th, lower 25th or 50th percentile (depending on traffic) of the Subgrade CBR within a uniform section is used to determine the Subgrade Class.

The details of each design method are described in the following sections. It should be noted that the design catalogues for all methods assume adequate drainage in terms of a minimum crown height above the drain invert (h_{min}) as discussed in *Chapter 8 – Drainage and Erosion Control*. If this is not the case, it will be necessary to raise the formation by constructing an additional fill layer(s).

[1] CUSUM (Cumulative Sum) analysis is a technique used to detect changes in a data set. See page 9-9.

9.2.5 DCP-DN Design Method

General

This method is based entirely on the use of the DCP device that provides a close approximation of the shear strength of a 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 optimised 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 optimising the utilisation of the in-situ material strength as much as possible. This is achieved by:

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

To make optimum use if the in situ materials in the pavement structure, they 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.

Layer-strength Diagram: Each DCP test provides a profile in the depth of the pavement which gives an indication of the in-situ properties of the materials in all the pavement layers down to a depth of penetration of 800 mm, as illustrated in Figure 9-3.

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.

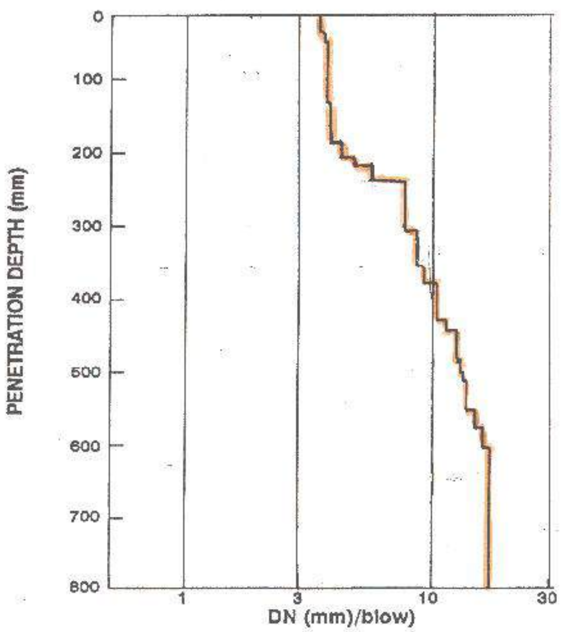


Figure 9-3: Typical profile of DN values with depth

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.

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.

Assessment of subgrade/pavement layer strength: Understanding what influences the performance of a LVRR 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 stabilization, thereby creating a new 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 9-4 shows a typical output of the materials assessment process.

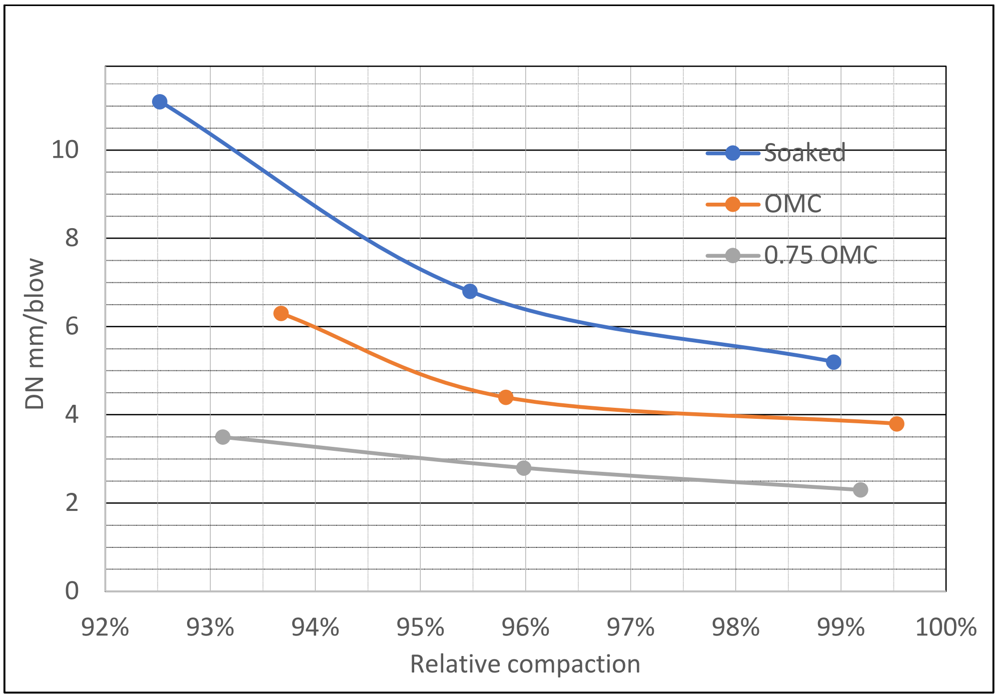


Figure 9-4: DN/density/moisture relationship

Figure 9-4 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:

a. 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

b. 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:

c. an acceptable DN value (based on the design assumptions) represents a composite measure of the key interacting variables that affect material strength, and

d. 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 9-4, the following essential design considerations will ensure optimum use of the material:

) Achieving the highest practicable level of density (so-called "compaction to refusal") by employing the heaviest rollers available) should be specified. This will result in a stronger material with a lower voids content and a reduced permeability, thereby enhancing the overall properties and performance of the material.

a) Adopting appropriate measures to keep the subgrade and pavement layer materials as dry as possible in service. This can be achieved by provision of adequate drainage, both external and internal, as discussed in *Chapter 89 – Drainage and Erosion Control.*

The information presented in Figure 9-4 provides the designer with the required information to ascertain under which moisture and density conditions the material will satisfy the design DN requirement. The laboratory testing program required to produce such a figure entails a fairly comprehensive testing program, as presented in Appendix 1. However, in practice, it will not be necessary to carry out the laboratory DN testing program over the full range of moisture contents. A less comprehensive testing program will generally provide the required information for design purposes, as described below in Step 5 of the *DCP-DN Design Procedure.*

Design procedure

The DCP-DN design procedure is shown step-by-step in Figure 9-5 below and explained below.

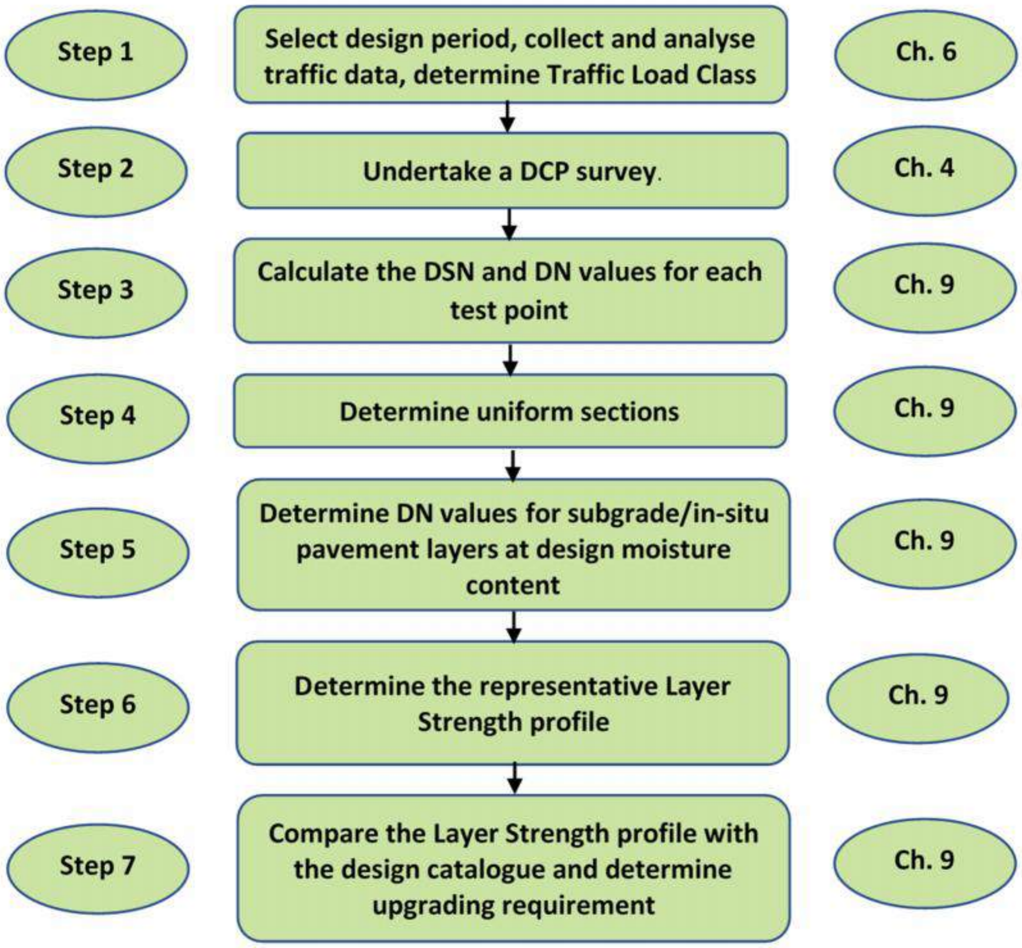


Figure 9-5: DCP-DN design procedure

Step 1: Determine the Traffic Load Class as described in *Chapter 6 – Traffic.*

Step 2: Carry out a DCP survey as described in *Chapter 4 – Site Investigations*.

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 in the AfCAP LVR DCP program, the calculation of the following useful parameters is done automatically:

- The weighted average of the DN value of each 150 mm layer down to a depth of 800 mm. This is the standard configuration of the AfCAP LVR DCP software, but the layer thicknesses can be varied, if required.
- The number of blows DN_{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} provide 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 somewhat 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 strength of the subgrade from 450 to 800 mm depth and is most often used together with the DN of the top three layers for determining uniform sections.

Step 4: Determine uniform sections.

The technique for determining uniform sections using a spreadsheet is as follows:

- Transfer the DN and DSN values for all the test points generated in Step 3 to a spreadsheet.
- Summarize the data, as shown in Table 9-2.
- Undertake a Cumulative Sum (CUSUM) analysis by calculating the 'Cusum' for the DN of three top layers as well as DSN_{450} or DSN_{800}, using the formula:

$$CUSUM_{DN} = \sum_{1}^{n}(DN_i - DN_{avg})$$

where n = number of test points.

- Plot the 'cusum' values in a graph, as shown in Figure 9-6.
- The start and end of the uniform sections are then selected as the best approximation of the points where the different curves change direction.

Table 9-2: Excel spreadsheet used for the CUSUM analysis

Cusum Analysis Unpaved Example																
Test			DSN800			0-150 mm			151-300 mm			301-450 mm			451-800	601-800
no	Chainage	Position	DSN	DSN-Avg	Cusum	DN	DN-Avg	Cusum	DN	DN-Avg	Cusum	DN	DN-Avg	Cusum	DN	DN
1	0.000	RHS	179	88.30	88.30	0.68	-18.72	-18.72	2.59	-18.41	-18.41	2.03	-16.78	-16.78	1.23	
2	0.500	RHS	357	266.30	354.60	0.68	-18.72	-37.43	0.58	-20.42	-38.83	0.92	-17.89	-34.68	0.45	
11	1.000	RHS	188	97.30	451.90	4.01	-15.39	-52.82	4.20	-16.80	-55.62	4.54	-14.27	-48.95	6.38	
3	1.500	RHS	160	69.30	521.20	0.56	-18.84	-71.65	4.06	-16.94	-72.56	2.21	-16.60	-65.55	1.06	
4	2.000	RHS	150	59.30	580.50	3.85	-15.55	-87.20	9.10	-11.90	-84.46	22.11	3.30	-62.25	4.80	
115	2.500	LHS	134	43.30	623.80	4.55	-14.85	-102.05	4.10	-16.90	-101.36	8.50	-10.31	-72.57	18.70	
5	3.000	LHS	43	-47.70	576.10	25.95	6.55	-95.49	21.52	0.52	-100.83	22.94	4.13	-68.44	16.07	
7	3.500	RHS	34	-56.70	519.40	54.00	34.60	-60.89	46.80	25.80	-75.03	20.90	2.09	-66.35	17.20	
8	4.000	LHS	52	-38.70	480.70	15.54	-3.86	-64.74	40.00	19.00	-56.03	18.90	0.09	-66.26	12.30	
9	4.500	LHS	33	-57.70	423.00	29.10	9.70	-55.04	21.20	0.20	-55.83	22.10	3.29	-62.98	33.50	
111	5.000	LHS	90	-0.70	422.30	6.40	-13.00	-68.04	8.80	-12.20	-68.02	7.70	-11.11	-74.09	15.60	
14	5.500	LHS	79	-11.70	410.60	5.20	-14.20	-82.23	11.30	-9.70	-77.72	14.10	-4.71	-78.80	21.90	
114	6.000	RHS	83	-7.70	402.90	7.60	-11.80	-94.03	8.50	-12.50	-90.22	16.80	-2.01	-80.81	11.95	
10	6.500	LHS	54	-36.70	366.20	12.30	-7.10	-101.12	11.80	-9.20	-99.42	16.40	-2.41	-83.23	21.20	
12	7.000	LHS	48	-42.70	323.50	31.60	12.20	-88.92	18.10	-2.90	-102.31	12.00	-6.81	-90.04	18.30	
13	7.500	RHS	37	-53.70	269.80	22.30	2.90	-86.02	15.70	-5.30	-107.61	17.80	-1.01	-91.05	29.30	
15	8.000	RHS	36	-54.70	215.10	53.40	34.00	-52.01	46.40	25.40	-82.21	22.20	3.39	-87.66	15.30	
117	8.500	RHS	21	-69.70	145.40	27.70	8.30	-43.71	44.20	23.20	-59.01	47.00	28.19	-59.48	44.40	
116	9.000	LHS	15	-75.70	69.70	45.00	25.60	-18.10	43.00	22.00	-37.00	45.00	26.19	-33.29	75.00	
112	9.500	RHS	21	-69.70	0.00	37.50	18.10	0.00	58.00	37.00	0.00	52.10	33.29	0.00	41.30	
			90.70			19.40			21.00			18.81			20.30	

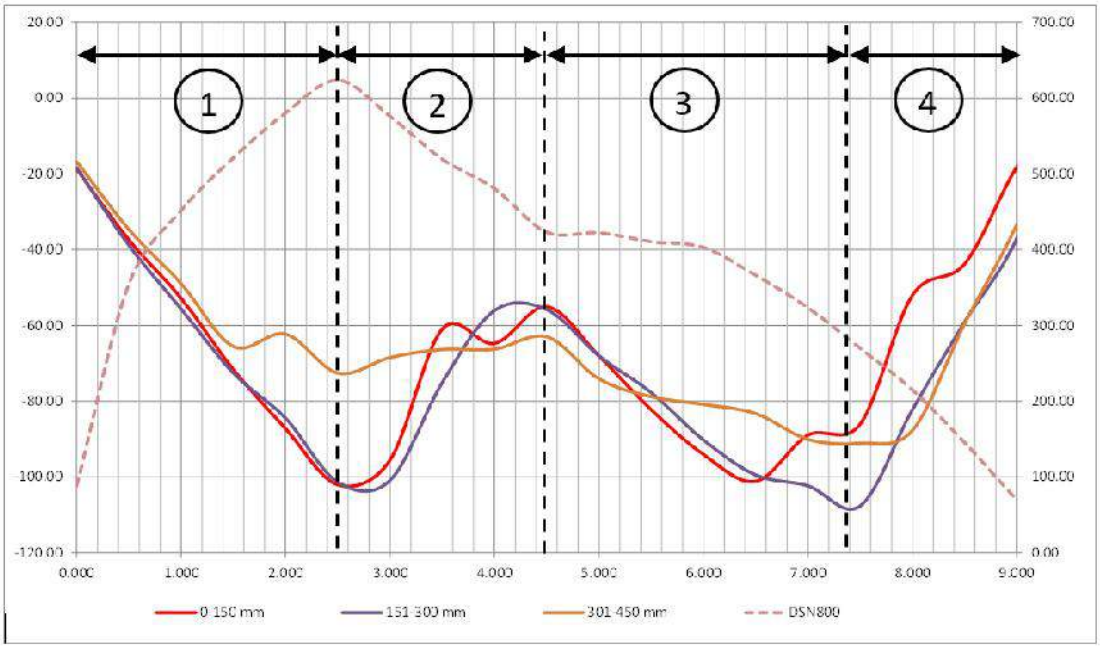


Figure 9-6: Plot of the CUSUM analysis for determination of uniform sections

The DCP results normally exhibit a fairly wide spread of DN values for an entire 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 limiting the risk associated with 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 the uniform section, which may give rise to over- or under-design, “outliers” should be eliminated before determining the uniform sections.

Figure 9-7 and 9-8 illustrates the variability of the DCP results, and the extreme and average values, respectively.

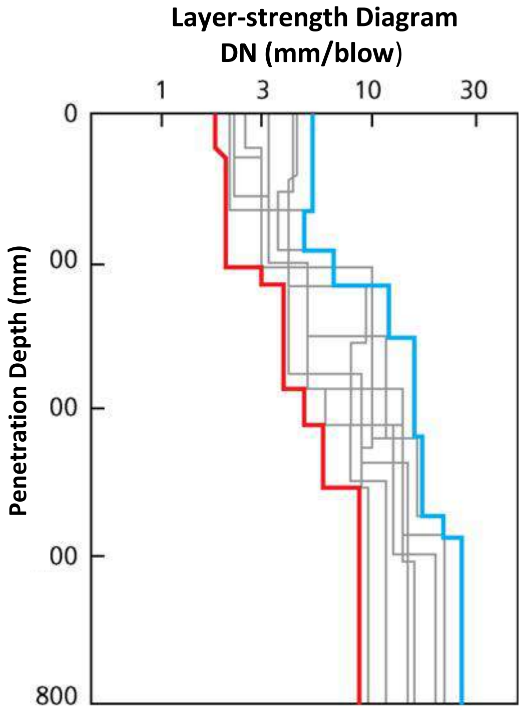


Figure 9-7: Collective DCP strength profile for a uniform section

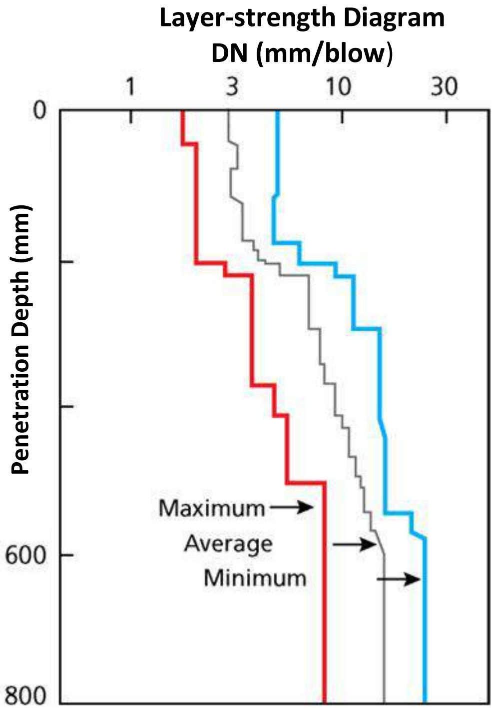


Figure 9-8: Average & extreme DCP strength profiles for a uniform section

The determination of uniform sections, as explained and illustrated above, can also be done from within the AfCAP LVR-DCP software.

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

- Three bulk samples shall be collected from each uniform section, as described in *Chapter 4 – Site Investigations*, and thoroughly mixed 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 Appendix 1.
- The determination of the strength (DN value) of the subgrade, and in-situ pavement layer(s), if any, is based on the anticipated in-service long-term equilibrium moisture (EMC) of these layers, as well as the field densities likely to be achieved during construction. The choice of EMC is based on engineering judgment, as influenced by a knowledge of the micro-climate and the likely in-service moisture and related drainage 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, as required, provide a solid foundation for 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 *Chapter 9 – Drainage and Erosion Control*, Table 9-3); 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 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" (*see Chapter 13 – Practical Considerations: Section 13.6 - Compaction)* 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 9-3:

Table 9-3: 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 will be similar to Figure 9-4, except that it will show only the two curves related to the moisture contents at which the material was tested.

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

Figure 9-9 shows a typical output from the AfCAP LVR DCP program for the analysis of a uniform section at the time of the DCP survey. The in-situ layer strength profile (solid red line) is compared to the required catalogue strength profile (dashed blue line). This comparison allows the adequacy of the various pavement layers, in depth, to be assessed for carrying the expected future traffic loading. Where the red line lies to the right of the dashed blue line, the layer strength is inadequate. The figure shows that the two upper layers did not have sufficient strength for TLC 0.1 at the time of the DCP survey.

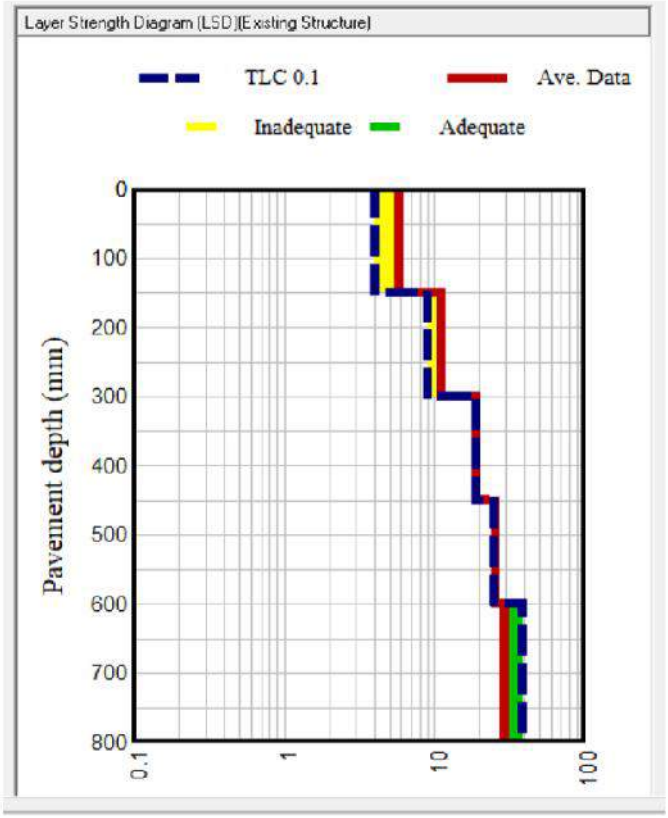


Figure 9-9: Typical output from the analysis of a uniform section

The required strength profile indicated in Figure 9-9 is derived from the DCP-DN structural design catalogue shown in Table 9-4 below. This catalogue prescribes the pavement layer thicknesses and strengths in 150 mm increments to a depth of 800 mm, i.e., the required strength profile. The layer strengths are varied in relation to traffic loading and increase gradually as the design traffic loading increases.

Table 9-4: DCP-DN Design Catalogue for different traffic load classes (TLCs)

Traffic Class MESA	TLC 0.01 0.003-0.01	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 ≤ 6.2	DN ≤ 4.4	DN ≤ 3.2	DN ≤ 2.5	DN ≤ 2.3
150-300 mm Sub-base ≥ 95% Mod. AASHTO	DN ≤ 17	DN ≤ 13	DN ≤ 9	DN ≤ 6.6	DN ≤ 5.2	DN ≤ 4.7
300-450 mm Subgrade ≥ 95% Mod. AASHTO	DN ≤ 29	DN ≤ 21	DN ≤ 15	DN ≤ 11	DN ≤ 8.7	DN ≤ 7.8
450-600 mm In situ material	DN ≤ 44	DN ≤ 32	DN ≤ 23	DN ≤ 17	DN ≤ 13	DN ≤ 12
600-800 mm In situ material	DN ≤ 65	DN ≤ 48	DN ≤ 34	DN ≤ 25	DN ≤ 19	DN ≤ 17
DSN_{800}	≥ 39	≥ 52	≥ 73	≥ 100	≥ 128	≥ 143

Source: Adapted from Kleyn and van Zyl (1988)

The following steps in the design process are best illustrated by presenting the Layer Strength Diagram (LSD) of the pavement in a tabular format. The design process can be illustrated with two hypothetical scenarios shown below.

Scenario 1

The DCP survey was carried out in the wet season. Samples from the three top layers showed that the two top layers were in a soaked condition with a moisture content at 1.2 OMC, whereas the third layer had a moisture content approximately at OMC as shown in Table 9-5. The pink colour indicates the layers with insufficient strength and green colour indicates layers with sufficient strength compared to the requirement of the TLC.

Table 9-5: In-situ layer strength profile (mm/blow/layer) in Scenario 1

Pavement Layer (mm)	Required DN value for TLC 0.1	Section 4 (km 4+400 - 8+380)		
		In-situ		
		DN	MC	Density
0-150	<= 4.4	5.8	1.2 OMC	?
150-300	<= 9	11	1.2 OMC	?
300-450	<= 15	14	≈ OMC	?
450-600	<= 23	23		
600-800	<= 34	29		

The design process is based on consideration of the following:

1. An assumption is made on the long-term moisture content in the pavement, all factors being taken into consideration as described above (climatic zone, micro-climate, drainage conditions, etc.). In this case, it is assumed that all the pavement layers will equilibrate in service at the OMC, which is used as the design moisture content.

2. The density of the in-situ layers at the time of the survey is not known. However, this is not important as the laboratory DN value at the minimum density to be achieved in the field can be obtained from the laboratory DN testing program, to ascertain whether or not the DN value at the specified density and the assumed EMC can be achieved.

3. The strength of the third layer is already satisfactory (DN value ≤ 19 mm) at the in-situ density. This layer is, therefore, accepted as it is with no further modification. The same is true for the bottom two layers, but these are of lesser importance on LVSRs because the traffic-induced stresses below 450 mm depth are insignificant.

4. The DN value at OMC and the minimum required relative compaction of 98% and 95% respectively for the two top layers, as shown in Table 9-6, are established from the laboratory DN test.

Table 9-6: Representative layer strength profile in Scenario 1

Pavement Layer (mm)	Required DN value for TLC 0.1	Section 4 (km 4+400 - 8+380)		
		In-situ		
		DN	MC	Density
0-150	<= 4.4	3.9	OMC	≥ 98%
150-300	<= 9	8.7	OMC	≥ 95%
300-450	<= 15	14	≈ OMC	?
450-600	<= 23	23		
600-800	<= 34	29		

5. Table 9-5 shows that the representative layer strength profile for this uniform section satisfies the design requirement for the Traffic Load Class TLC 0.1. The in-situ pavement can thus be used and will only need reshaping and recompacting (to refusal) before surfacing.

Scenario 2

The DCP survey was carried out in the dry season, and the moisture content of all layers was approximately at OMC, as shown in Table 9-7. As for Scenario 1, it is assumed that all the pavement layers will equilibrate in service at OMC, which is used as the design moisture content. Adjustment of the DN values for moisture content is therefore not required.

Table 9-7: In-situ layer strength profile (mm/blow/layer) in Scenario 2

Pavement Layer (mm)	Required DN value for TLC 0.1	Section 4 (km 4+400 - 8+380)		
		In-situ		
		DN	MC	Density
0-150	<= 4.4	5.8	≈ OMC	?
150-300	<= 9	11	≈ OMC	?
300-450	<= 15	14	≈ OMC	?
450-600	<= 23	23		
600-800	<= 34	29		

In this case, the designer has three options for how to satisfy the design requirements:

1. Rip and recompact the two top layers.
2. Rip and recompact Layer 2 and mechanically stabilise the Layer 1.
3. Import a new base layer with a Laboratory DN value ≤ 4 mm/blow, as shown in Table 9-8.

Table 9-8: Representative layer strength profile in Scenario 2

Pavement Layer (mm)	Required DN value for TLC 0.1	Section 4 (km 4+400 - 8+380)		
		In-situ		
		DN	MC	Density
0-150	<= 4.4	<= 4	≈ OMC	?
150-300	<= 9	5.8	≈ OMC	?
300-450	<= 15	11	≈ OMC	?
450-600	<= 23	14		
600-800	<= 34	23		

Step 7: Compare the representative LSD for each uniform section with the required LSD as per the DCP-DN Design catalogue shown in Table 9-4, and determine the upgrading requirements.

The design catalogue is based on the anticipated long-term, in-service moisture condition, as explained in Step 5 above. If there is a risk of prolonged moisture ingress into the road pavement, then the pavement design should be based on the soaked condition.

By comparing the representative LSD for the section in the two scenarios with the requirement as per the design catalogue, the upgrading requirements are as shown in Table 9-9.

Table 9-9: Upgrading requirements in the two different scenarios

Pavement Layer (mm)	Required DN value for TLC 0.1	Scenario 1	Scenario 2
		Section 4	Section 4
		4.400 to 8.380 km	4.400 to 8.380 km
0-150	<= 4.4	3.9	≤ 4.0
150-300	<= 9	8.7	5.8
300-450	<= 15	19	11
450-600	<= 23	25	19
600-800	<= 34	29	25

From the analyses described above, the options for upgrading to meet the requirements of the DCP-DN Design Catalogue are as follows:

Scenario 1: No additional pavement layers are required.

Scenario 2: One additional pavement layer with a DN ≤ 4.0 at OMC and 98% SANS – GR30 compaction is required. The properties for the imported layer have been established through the Laboratory DN test on representative samples from the borrow pit as discussed below. By constructing the new base on top, all the other layers move down one position in the pavement structure, and their strength requirement reduces. The in-situ base layer will then satisfy the requirement for the subbase in the upgraded structure.

In general, the following options for upgrading the pavement should be considered:

Option 1: If the in-situ strength profile of the existing gravel road, as determined by the laboratory DN testing described above, complies with the required strength profile indicated by the DCP-DN catalogue for the particular traffic class, the road would need to be only re-shaped, compacted and surfaced (assuming that the existing road is adequately above natural ground level to permit the necessary drainage requirements). In case of a sunken profile, the subgrade has to be raised to achieve adequate drainage by filling in layers complying with the DN requirements of the DCP-DN design catalogue.

Option 2: If the in-situ strength profile of the existing gravel road does not comply with the required strength profile indicated by the DCP catalogue for the particular traffic class, then the upper pavement layer(s) would need to be:

- **Reworked** – if only the density is inadequate and the required DN value can be obtained at the specified construction density and anticipated in-service moisture content.
- **Overlaid** – if the material quality (DN value at the specified construction density and anticipated in-service moisture content) is inadequate, then appropriate quality material will need to be imported to serve as the new upper pavement layer(s).
- **Mechanically stabilized** – as above, but new, better quality material is blended with the existing material to improve the overall quality of the layer.
- **Augmented** – if the material quality (DN value) is adequate, but the layer thickness is inadequate, then imported material of appropriate quality will need to be imported to make up the required thickness prior to compaction.

If none of the above options produces the required quality of material, recourse may be made to more expensive options, such as soil stabilisation. However, the design and construction requirements of stabilised layers is outside the scope of this Guideline which focuses on the use of natural, untreated, materials.

Assessment of borrow pit materials

As indicated in *Chapter 5 – Materials, Section 6.5: Materials Sampling and Testing,* a full range of tests of borrow pit materials, as discussed below in the section on laboratory testing, should be carried out at the minimum testing frequency as follows:

- Base: Every 5,000 m^3
- Subbase: Every 10,000 m^3

There is a need to fully understand how the imported pavement materials will perform under a specific design scenario. The process to assess the performance and the risk associated with the design assumptions is essentially the same as for the assessment of the subgrade and in-situ pavement layer materials. The recommended laboratory testing program is as presented in Table 9-3.

Material tests

General

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 judgement 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 under *Design concepts* above, 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 Plastic Modulus 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.

Laboratory testing

The laboratory testing program is part of a wider materials investigation program to provide all of the information needed to determine the characteristics, potential use and volumes available of construction materials, as discussed in *Chapter 6 – Materials*.

The program will comprise the following tests:

- Grading and Atterberg limits
- Compaction
- Laboratory DN
- Durability

Grading and Atterberg limits: Grading envelopes and Atterberg limits (-PI and PM) are not specified for the DCP-DN method, for the reasons discussed above in the section on *Assessment of subgrade/pavement layer strength.* 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 material strength in service.

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.

Maximum aggregate size: Oversize aggregates will prevent proper compaction of the pavement layers and should be broken down as described in *Chapter 6 – Materials* or removed by hand if the aggregates cannot be sufficiently crushed by the working and compaction of the layers.

The result of the sieve analysis will establish whether the maximum aggregate size exceeds the limits for use in the respective pavement layers.

Compaction: The standard compaction test is required to calculate the relative compaction of the samples. For details, see *Chapter 6 - Materials.*

Laboratory DN test: This test is described in Appendix 1.

Durability: This test is seldom required for LVRR. For details, see *Chapter 6 – Materials, Section 6.5.6 – Specialized Tests.*

Material specifications

In summary, the three materials 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: ≤ 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".

9.2.6 The DCP-SN Method

General

As explained in Section 9.2.1, this method is mostly applicable for the upgrading of existing roads having one or more structural layers.

To make optimum use of the existing layers, the method makes use of the structural number concept (AASHTO, 1993) which is based on the in-situ CBRs derived from the DN values as explained below. The difference between the structural number of the existing road and that required for the upgraded road, which is obtained from the catalogue of structures, defines the additional requirements for upgrading.

The DCP survey provides the thicknesses and in situ strengths of the layers of the existing road along the entire alignment. By converting the DN values to CBR values, a diagram of CBR versus depth is obtained. The equation used to do this is based on the TRL correlation of DN to CBR, which is:

$$\mathrm{Log}_{10}\,\mathrm{CBR} = 2.48 - 1.057\ \mathrm{Log}_{10}\,\mathrm{DN}$$

The analysis of the DCP data using the UK DCP program provides the overall strength of the pavement at each test point based on the structural number approach.

Structural Number (SN)

The structural number is essentially a measure of the total thickness of the road pavement weighted according to the 'strength' of each layer and calculated as follows:

$$SN = 0.0394 \sum m_i.a_i.h_i$$

Where: SN = structural number of the pavement,

a_i = strength coefficient of the i-th layer,

h_i = thickness of the i-th layer, in millimetres,

m_i = 'drainage' coefficients that modify the layer strength coefficients of unbound materials if drainage is poor and/or climate is favourable or severe.

The summation is over the number of pavement layers, n.

The individual layer strength coefficients are determined from the normal tests that are used to define the strength of the material in question, e.g. CBR for granular materials, UCS for cemented materials etc. Table 9-10 shows typical values.

The drainage coefficients are effectively calibration factors for the moisture regime experienced by the road and are therefore related to both climate and drainage. Values range from 0.7 for extremely poor conditions up to 1.3 for very good conditions, but the usual working range is 0.9 to 1.1. In wet areas, a value of m_i of 0.9 will provide a suitable safety factor. However, for a well-designed road, the effects of its moisture regime or climate on the strength of the road are primarily manifest in the strength of the subgrade, and so a value of 1.0 should be used for the pavement layers for relatively 'normal' conditions and a value of 1.1 for very dry conditions.

Modified Structural Number (SNC)

The effect of different subgrades can also be included in the structural number approach. The subgrade contribution is defined as follows:

$$SNC = SN + 3.51 (\log10\ CBRs) – 0.85 (\log10\ CBRs)2 – 1.43$$

where:

SNC = Modified structural number of the pavement

CBRs = In-situ CBR of the subgrade

The modified structural number (SNC) has been used extensively over the past decades and forms the basis for defining pavement strength in many pavement performance models. It should be used to identify the overall strength of each DCP test point in the old road if the subgrade is particularly variable.

Table 9-10: Pavement layer strength coefficients

Layer Type	Condition	Coefficient
Surface treatment		$a_i = 0.2$
Granular unbound base layers	Default	$a_i = (29.14\ CBR - 0.1977\ CBR^2 + 0.00045\ CBR^3)\ 10^{-4}$
	CBR > 100%	$a_i = 0.145$
	CBR = 100%	$a_i = 0.14$
	CBR = 80%	
	With a stabilised layer underneath	$a_i = 0.135$
	With an unbound granular layer underneath	$a_i = 0.13$
	CBR = 65%	$a_i = 0.12$
	CBR = 55%	$a_i = 0.107$
	CBR = 45%	$a_i = 0.1$
Bitumen treated gravels and sands	Marshall stability = 2.5 MN	$a_i = 0.135$
	Marshall stability = 5.0 MN	$a_i = 0.185$
	Marshall stability = 7.5 MN	$a_i = 0.23$
Cemented	Equation	$a_i = 0.075 + 0.039\ (UCS) – 0.00088(UCS)^2$
	CB 1 (UCS = 3.0 – 6.0 MPa)	$a_i = 0.18$
	CB 2 (UCS = 1.5 – 3.0 MPa)	$a_i = 0.13$
Granular unbound sub-bases	Equation	$a_j = -0.075 + 0.184(\log_{10} CBR) – 0.0444(\log_{10} CBR)^2$
	CBR = 40%	$a_i = 0.11$
	CBR = 30%	$a_i = 0.1$
	CBR = 20%	$a_i = 0.09$
	CBR = 15%	$a_i = 0.08$
	CBR = 10%	$a_i = 0.065$
Cemented	(UCS = 0.7 – 1.5 MPa)	$a_i = 0.1$

Note: Unconfined Compressive Strength (UCS) is stated in MPa at 14 days.

Uniform (or relatively homogenous) sections are determined at this stage using the 'CUSUM' method applied to the DCP data, usually the SN or modified structural number (SNC) values, and the required designs can be determined based on the appropriate percentiles.

As for the DCP-DN method, the sectioning in uniform sections limits the variability within the sections and thus the risk associated with design decisions based on an assessment of average strength of the in-situ pavement within each section. By excluding very weak point from the CUSUM analysis, the risk is further reduced. Such points must then be assessed separately.

Design procedure

The Structural Number design procedure is shown step-by-step in Figure 9-10 and explained in the following text.

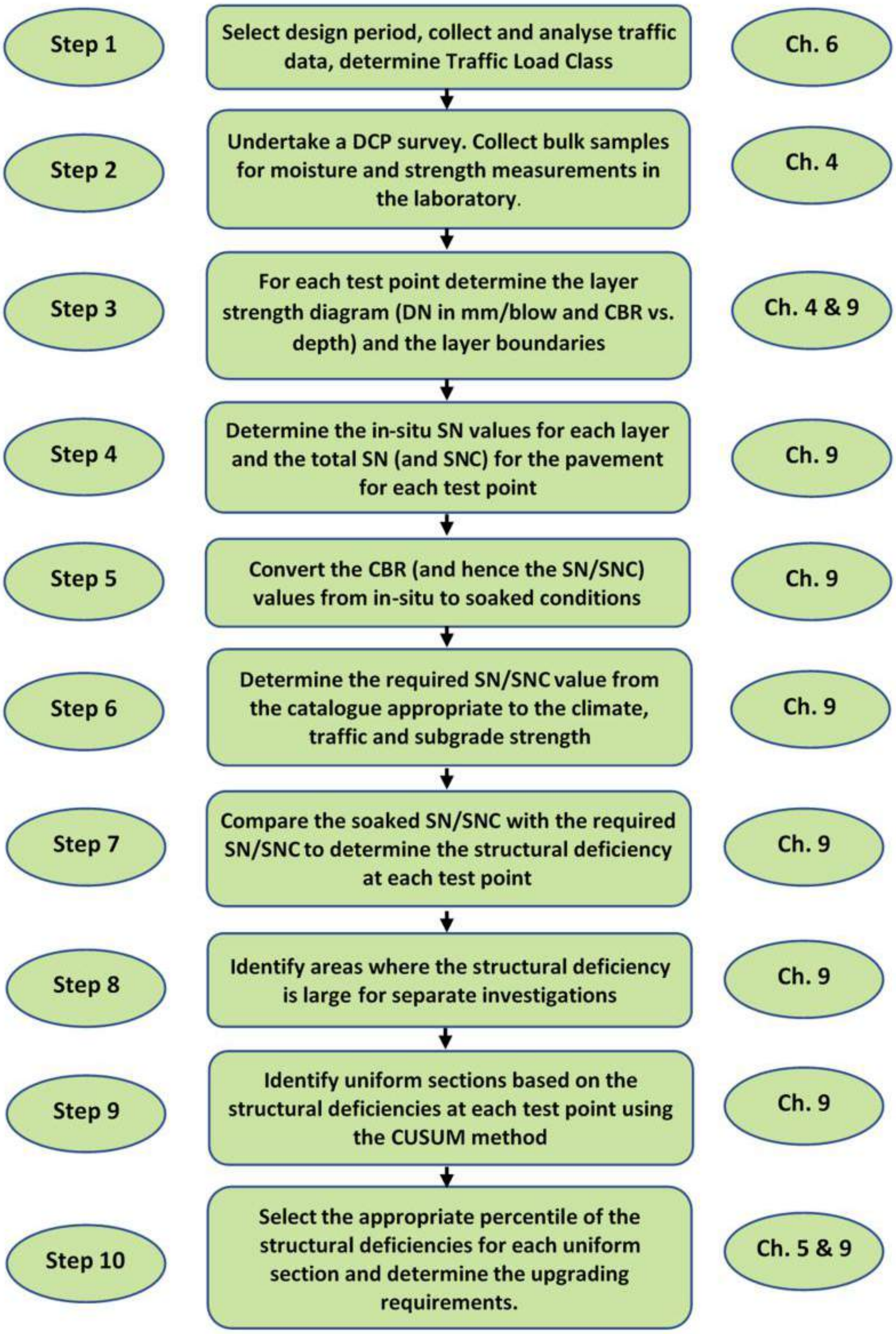


Figure 9-10: The DCP-SN design procedure

Step 1: Determine the Traffic Load Class as described in *Chapter 6 – Traffic.*

Step 2: Carry out a DCP survey as described in *Chapter 4 – Site Investigations*.

Step 3: For each test point determine the layer strength diagram (DN in mm/blow and CBR vs. depth) and the layer boundaries. A typical output from the UK DCP program is shown in Figure 9-11.

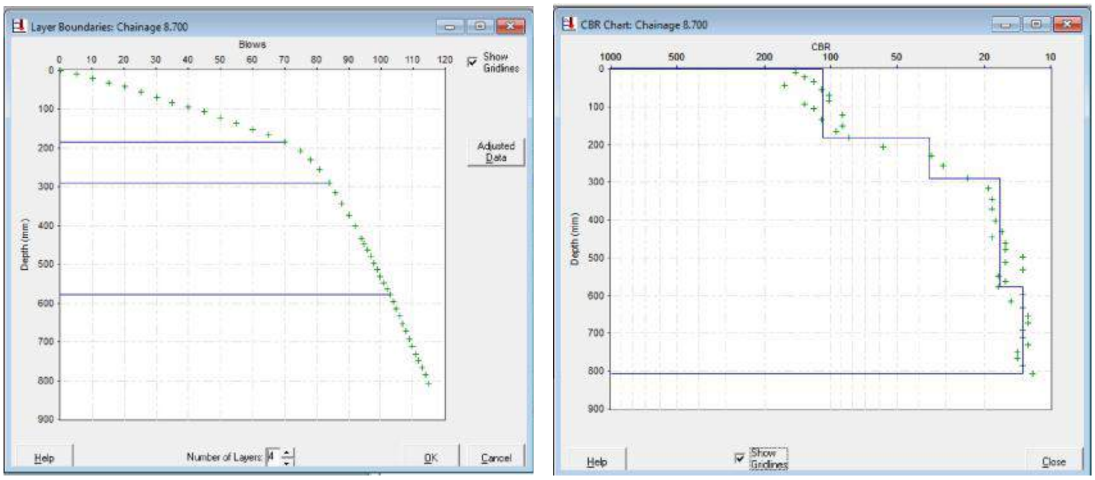


Figure 9-11: Typical Layer-strength Diagrams computed in the UK DCP programme

Step 4: Determine the in-situ SN values for each layer and the total SN (and SNC) for the pavement for each test point.

To determine the total in-situ SN and SNC, the layers must first be defined as base, subbase or subgrade as shown in Figure 9-12.

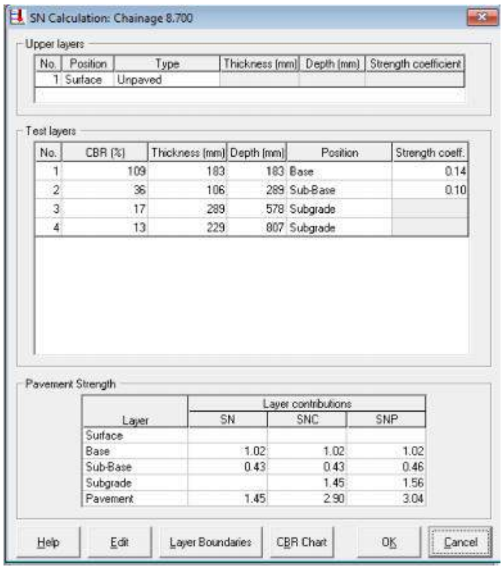


Figure 9-12: Calculation of SN / SNC in the UK DCP Programme

Step 5: Convert the CBR (and hence the SN/SNC) values from in-situ to soaked conditions

To convert from the in-situ values to the soaked values requires a measurement of the in-situ moisture condition, expressed as the ratio of in-situ moisture content divided by the optimum moisture content, and the use of Table 9-11. The in-situ moisture condition is obtained from the samples collected for laboratory analysis after determination of uniform sections. A minimum of three samples per uniform section is recommended. It is often more useful to obtain the samples once the DCP survey has been analysed and the most appropriate sampling points can be identified to ensure that maximum benefit is obtained from the sampling and testing. However, the delay between the in-situ testing and sampling must be minimal (less than 14 days).

The relationship between soaked and in situ strength (CBR) depends on the characteristics of the materials as presented in Table 9-11, which is based on extensive research.

Table 9-11: Relationship between in-situ DCP-CBR and soaked CBR

Soaked CBR	Approximate in-situ DCP-CBR					
	Subgrade		Wearing course			
	Wet	Dry	Very dry	Dry	Moderate	Damp
80			318	228	164	117
45			244	175	126	90
25	59	65	186	134	96	69
15	45	50	147	106	76	54
10	38	43				
7	33	37				
3	20	24				
Note: Moisture contents are expressed as ratios of in-situ to optimum moisture content: Very dry = 0.25, Dry = 0.5, Moderate = 0.75, Damp = 1.0						

Step 6: Determine the required SN/SNC values at each test point from the catalogue appropriate to the climate, traffic and subgrade strength.

Table 9-12: Structural Numbers for Pavement Design Chart 1 (Table 9-14: Wet areas)

Subgrade Class (CBR)	TLC 0.01	TLC 0.1	TLC 0.3	TLC 0.5	TLC 1.0
	< 0.01	0.01 – 0.1	0.1 – 0.3	0.3 – 0.5	0.5 – 1.0
S1 (<3%)	Special subgrade treatment required				
S2 (3-4%)	1.05	1.7	1.95	2.05	2.30
S3 (5-7%)	0.95	1.45	1.70	1.85	2.1
S4 (8-14%)	0.8	1.25	1.55	1.65	1.85
S5 (15-29%)	0.7	1.0	1.25	1.35	1.5
S6 (>30%)	0.6	0.7	0.85	0.95	1.0

Note: These values exclude a contribution from the surfacing

Table 9-13: Structural Numbers for Pavement Design Chart 2 (Table 9-15: Mod/Dry areas)

Sub-grade Class (CBR)	TLC 0.01	TLC 0.1	TLC 0.3	TLC 0.5	TLC 1.0
	< 0.01	0.01-0.1	0.1-0.3	0.3-0.5	0.5-1.0
S1 (<3%)	Special subgrade treatment required				
S2 (3-4%)	1.05	1.55	1.80	2.0	2.15
S3 (5-7%)	0.9	1.35	1.55	1.70	1.95
S4 (8-14%)	0.7	1.05	1.35	1.45	1.6
S5 (15-29%)	0.6	0.85	1.05	1.1	1.3
S6 (>30%)	0.6	0.7	.75	0.85	0.9

Note: these values exclude a contribution from the surfacing

Table 9-14: Modified Structural Numbers for Chart 1 (Table 9-14: Wet areas)

Subgrade Class (CBR)	TLC 0.01	TLC 0.1	TLC 0.3	TLC 0.5	TLC 1.0
S2 (3-4%)	1.1	1.75	2.0	2.1	2.35
S3 (5-7%)	1.55	2.05	2.35	2.45	2.7
S4 (8-14%)	1.85	2.25	2.6	2.7	2.9
S5 (15-29%)	2.2	2.55	2.75	2.9	3.05
S6 (>30%)	2.5	2.6	2.75	2.85	2.9

Table 9-15: Modified Structural Numbers for Chart 2 (Table 9-15: Mod/Dry areas)

Subgrade Class (CBR)	TLC 0.01	TLC 0.1	TLC 0.3	TLC 0.5	TLC 1.0
S2 (3-4%)	1.1	1.6	1.85	2.05	2.2
S3 (5-7%)	1.5	1.95	2.15	2.35	2.55
S4 (8-14%)	1.75	2.1	2.4	2.5	2.65
S5 (15-29%)	2.1	2.35	2.55	2.65	2.8
S6 (>30%)	2.5	2.6	2.65	2.75	2.8

Step 7: Compare the SN/SNC with the required SN/SNC from the relevant catalogueue to determine the structural deficiency at each test point.

Step 8: Identify areas (a) where the structural deficiency is large and (b) areas where layers are very weak and unlikely to meet the specifications for the layer that they will become in the upgraded design. As for the DCP-DN method, these points must be investigated separately to identify the likely cause and possible solutions to the problem.

Step 9: Identify uniform sections based on the structural deficiencies (ΔSN or ΔSNC) at each test point using the CUSUM method as illustrated in Table 9-2 and Figure 9-6.

Step 10: Select the appropriate percentile of the structural deficiencies for each uniform section and design the upgrading requirements in terms of additional layers and/or layer processing.

Tables 9-12 to 9-15 show the target values of SN and SNC for different subgrade conditions and for different traffic levels calculated from the design charts for roads with a thin bituminous surfacing. The difference between the required structural number and the existing structural number is the deficiency that needs to be corrected.

For each uniform section, the percentiles in Table 9-16 of the ΔSN or ΔSNC for determination of the strengthening requirements should be used.

Table 9-16: Percentile of ΔSN or ΔSNC to be used for design

Traffic Load Class	Percentile for design
TLC 0.01 and TLC 0.1	Median
TLC 0.3	Upper 75th percentile
TLC 0.5 and TLC 1.0	Upper 90th percentile

However, when the strengthening requirements are large, it may be more cost-effective to carry out some reconstruction and, conversely if they are small, maintenance may be all that is required. Table 9-17 provides a guide to the possible treatments.

Table 9-17: Structural deficiency criteria

Structural deficiency based on appropriate percentiles	Action	Notes
0.2 or negative	Maintain with a surface treatment (e.g. a surface dressing).	A thin granular overlay can be used to correct other road defects.
0.2 – 1.2	New granular layer. The existing layers must be checked for quality (sub-base or base course). The minimum thickness of new base course should be 50mm.	Some localised remedial works can be expected. A surface treatment is required.
1.2 – 1.8	The existing base course is likely to be only of sub-base quality and should be checked. Additional sub-base and a new base course are required.	Some localised remedial works will be needed. A surface treatment is required.
> 1.8	The existing layers are likely to be less than sub-base quality, hence a new sub-base and base course are required. Chemically stabilising existing material should be considered.	Localised remedial treatment and a surface treatment are required.

9.2.7 The DCP-CBR method

General

For a new road, the method is somewhat simpler because there are no existing structural pavement layers. In this case, only the subgrade properties are required. The structural designs are then obtained directly from the catalogue of structures.

The approach behind the DCP-CBR design method is similar to that of the DCP-DN method, i.e. to achieve a balanced pavement design whilst optimising the use of the in-situ material strength as far as possible.

The subgrade is classified using the standard soaked CBR test to provide a strength index. It is not expected that the subgrades will become soaked in service except in exceptional circumstances. The design catalogues, therefore, show different thickness designs based on climate and drainage conditions for the same indexed subgrade class. A standard soaked CBR test is also used to evaluate the strength of the imported pavement materials.

For roads with non-structural bituminous surfacings, two climatic zones are defined, and two design catalogues (charts) are used as shown in Tables 9-13 and 9-14.

The use of each chart also depends on the drainage and sealing provisions and the available materials as described below.

Design procedure

The DCP-CBR design procedure is shown in Figure 9-13 and explained below. The first three steps are identical to the DCP-SN method.

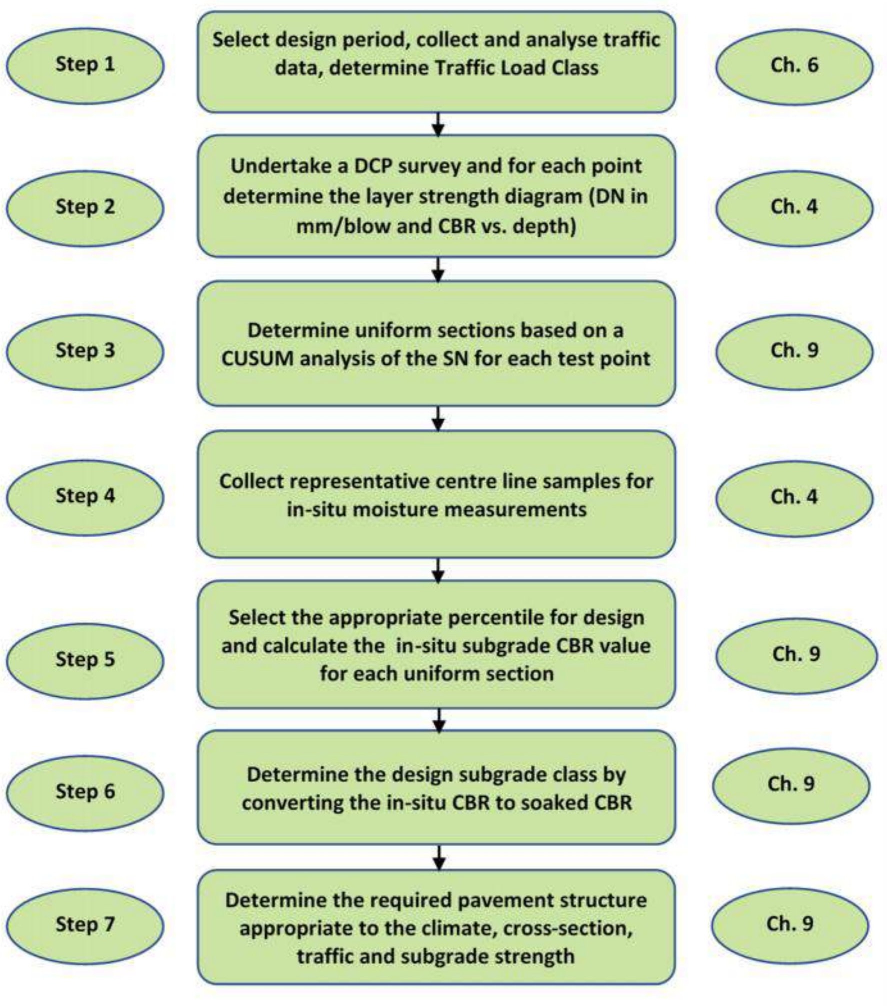


Figure 9-13: The DCP-CBR design procedure

Step 1: Determine the Traffic Load Class as described in *Chapter 6 – Traffic.*

Step 2: Carry out a DCP survey as described in *Chapter 4 – Site Investigations*.

Step 3: For each test point determine the layer strength diagram (DN in mm/blow and CBR vs. depth) and the layer boundaries.

Step 4: Determine uniform sections based on a CUSUM analysis of the SN for each test point as illustrated in Table 9-2 and Figure 9-6.

Step 5: Select the appropriate percentile for design and calculate the corresponding in-situ subgrade CBR value for each uniform section.

The appropriate percentile must be selected from Table 9-18.

Table 9-18: Percentile of subgrade in-situ CBR

Traffic Load Class	Percentile for design
TLC 0.01 and TLC 0.1	Median
TLC 0.3	Lower 25th percentile
TLC 0.5 and TLC 1.0	Lower 10th percentile

Step 6: Determine the design subgrade class by converting the in-situ CBR to Soaked CBR.

As for the DCP-SN method, Table 9-11 is used for the conversion. Preferably, samples should be taken from each DCP uniform section and tested in the laboratory to determine its true soaked CBR value.

Step 7: Determine the required pavement structure appropriate to the climate, cross-section, traffic and subgrade strength.

The selection of the appropriate design chart is shown in Figure 9-14 and explained below.

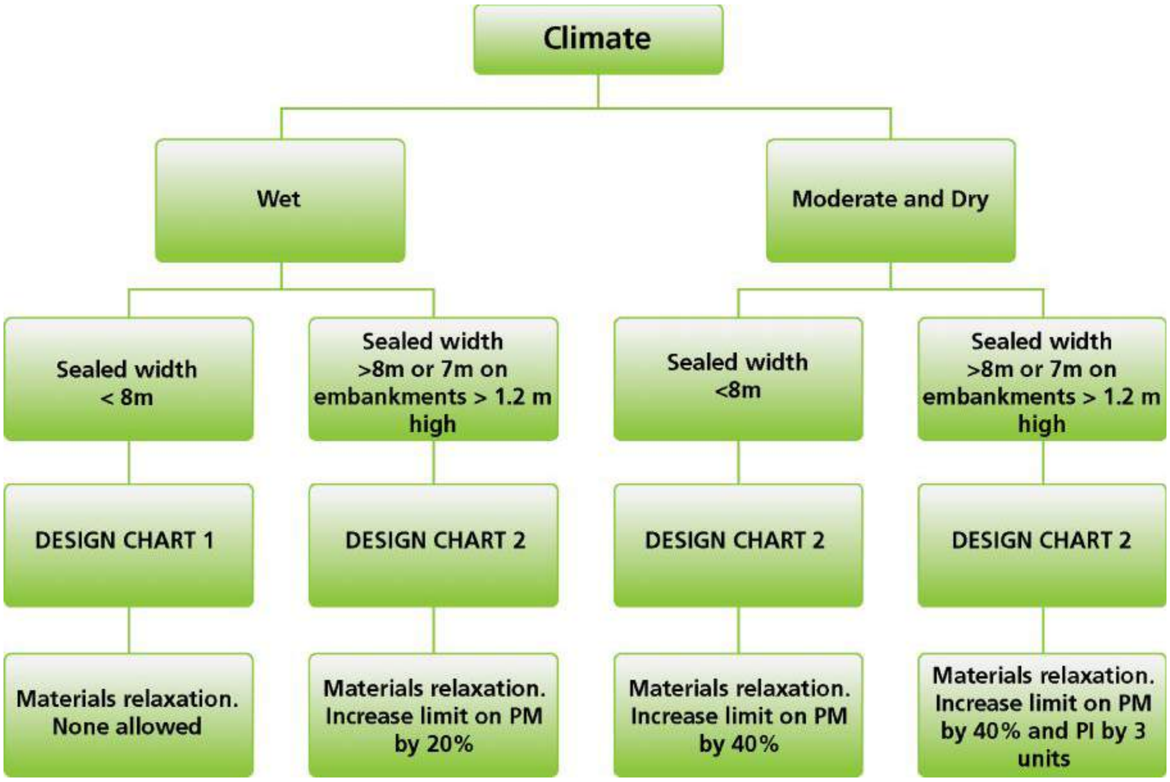


Figure 9-14: DCP-CBR pavement design flow chart

Wet climatic zone: In the wet climatic zone, the following situations and solutions apply:

Where the total sealed surface width is less than 8 m, Pavement Design Chart 1 (Table 9-19) should be used. No adjustments to the base material requirements are required.

a) Where the total sealed surface width is more than 8 m, Pavement Design Chart 2 (Table 9-20) should be used. The limit on the plasticity modulus of the base may be increased by 20% (see Table 9-23).

b) Where the total sealed surface is less than 8 m, but the pavement is on an embankment in excess of 1.2 metres in height, Pavement Design Chart 2 (Table 9-20) should be used. The limit on the plasticity modulus of the base course may be increased by 20% (see Table 9-23).

If the design engineer deems that other risk factors (e.g. poor maintenance and/or construction quality) are high, then Pavement Design Chart 1 should be used.

Moderate and dry climatic zone: In a moderate or dry climatic zone Pavement Design Chart 2 (Table 9-22) should be used.

a) Where the total sealed surface width is less than 8 m, the limit on the plasticity modulus of the road base may be increased by 40%. (see Table 9-23).

b) Where the total sealed surface width is more than 8 m, or the pavement is on an embankment in excess of 1.2 m in height, the plasticity modulus of the road base may be increased by up to 40% and the plasticity index by 3 units. (see Table 9-23).

Table 9-19: Pavement design Chart 1 (wet areas)

Sub-grade CBR	TLC 0.01 < 0.01	TLC 0.1 0.01 – 0.1	TLC 0.3 0.1 – 0.3	TLC 0.5 0.3 – 0.5	TLC 1.0 0.5 – 1.0
S1 (<3%)	Special subgrade treatment required				
S2 (3-4%)	150 G45 150 G15	150 G65 125 G30 150 G15	150 G80 150 G30 175 G15	175 G80 150 G30 175 G15	200 G80 175 G30 175 G15
S3 (5-7%)	125 G45 150 G15	150 G65 100 G30 100 G15	150 G65 150 G30 125 G15	175 G65 150 G30 125 G15	200 G80 150 G30 150 G15
S4 (8-14%)	200 G45	150 G65 125 G30	150 G65 200 G30	175 G65 200 G30	200 G80 200 G30
S5 (15-29%)	175 G45	125 G65 100 G30	150 G65 125 G30	150 G65 150 G30	175 G80 150 G30
S6 (>30%)	150 G45	150 G65	175 G65	200 G65	200 G80

Table 9-20: Pavement design Chart 2 (moderate and dry areas)

Sub-grade CBR	TLC 0.01 < 0.01	TLC 0.1 0.01-0.1	TLC 0.3 0.1-0.3	TLC 0.5 0.3-0.5	TLC 1.0 0.5-1.0
S1 (<3%)			Special subgrade treatment required		
S2 (3-4%)	150 G45 150 G15	150 G65 125 G30 100 G15	150 G65 150 G30 150 G15	175 G80 150 G30 150 G15	175 G80 175 G30 175 G15
S3 (5-7%)	125 G45 125 G15	150 G55 175 G30	175 G65 175 G30	175 G80 200 G30	175 G80 250 G30
S4 (8-14%)	175 G45	150 G55 100 G30	150 G55 150 G30	175 G65 150 G30	175 G80 175 G30
S5 (15-29%)	150 G45	200 G55	125 G55 125 G30	125 G65 125 G30	150 G80 125 G30
S6 (>30%)	150 G45	175 G45	175 G55	175 G65	175 G80

Once the quality of the available materials and haul distances are known, the design charts can be used to review the most economical designs.

When the project is located close to the boundary between the two climatic zones, the wetter value should be used to reduce risks. When the design is close to the borderline between two traffic design classes, and in the absence of more reliable data, the next highest design class should be used.

The design charts do not cater for weak subgrades (CBR < 3%) and other problem soils, which will need specialist input and design, typically requiring imported better-quality selected subgrade materials.

Material Specifications for the DCP-SN/CBR Design Method

General: Natural gravel materials for this design method are classified, as shown in Table 9-21.

It is important that the BS 1377 standards are followed. Only if no BS method for a specific test exists should an ASTM or AASHTO method be followed. There are often significant differences in results obtained by the different test methods. The biggest differences in the test results are obtained for the Atterberg limits and the compaction characteristics/strength tests – other test methods have relatively insignificant differences in the results.

Table 9-21: Pavement material and nominal specifications for the DCP-CBR design method

Code	Material	Abbreviated Specifications
NG80	Natural gravel	Min. CBR: 80 % @ 98/100 % BS heavy compaction and 4 days soaking Max. Swell: 0.2 % Max. Size and grading: Max size 37.5mm, grading as specified. PI: Dependent on material type, traffic and subgrade strength or as specified.
NG65	Natural gravel	Min. CBR: 65 % @ 98/100 % BS heavy compaction and 4 days soaking Max. Swell: 0.2 % Max. Size and grading: Max size 37.5mm, grading as specified PI: Dependent on material type, traffic and subgrade strength or as specified.
NG55	Natural gravel	Min. CBR: 55 % @ 98/100% BS heavy compaction and 4 days soaking Max. Swell: 0.2 % Max. Size and grading: Max size 37.5mm, grading as specified PI: Dependent on material type, traffic and subgrade strength or as specified.
NG45	Natural gravel	Min. CBR: 45 % @ 98/100 % BS heavy compaction and 4 days soaking Max. Swell: 0.2 % Max. Size and grading: Max size 37.5mm, grading as specified PI: Dependent on material type, traffic and subgrade strength or as specified.
NG30	Natural gravel	Min. CBR: 30% @ 95/97 % BS heavy compaction & highest anticipated moisture content Max. Swell: 1.0 % @ 100 % BS heavy compaction Max. Size and grading: Max size 63mm or 2/3 layer thickness PI: Dependent on material type, traffic and subgrade strength or as specified.
NG25	Natural gravel	Min. CBR: 25% @ 95/97% BS heavy compaction & highest anticipated moisture content Max. Swell: 1.0 % @ 100% BS heavy compaction Max. Size and grading: Max size 63mm or 2/3 layer thickness. PI: Dependent on material type, traffic and subgrade strength or as specified.
NG15	Gravel/soil	Min. CBR: 15% @ 93/95% BS heavy compaction & highest anticipated moisture content Max. Swell: 1.5 % @ 100% BS heavy compaction Max. Size: 2/3 of layer thickness PI: Dependent on material type, traffic and subgrade strength or as specified.
NG7	Gravel/soil	Min. CBR: 7 % @ 93/95 % BS heavy compaction & highest anticipated moisture content Max. Swell: 1.5 % @ 100% BS heavy compaction Max. Size: 2/3 layer thickness PI: Dependent on material type, traffic and subgrade strength or as specified.
NG3	Gravel/soil	Min. CBR: 3 % @ 93/95 % BS heavy compaction & highest anticipated moisture content Max. Swell: N/A Max. Size: 2/3 layer thickness

Source: Modified from Gourley and Greening, 1999.

Note: Two alternative minimum levels of compaction are specified. Where the higher densities can be attained in the field (from field measurements on similar materials or other established information), they should be specified by the Engineer.

Unlike for the DCP-DN method, materials for this design method must simultaneously satisfy the specifications for grading (grading envelope and GM) and Atterberg limits (PI and PM) as given in Table 9-22 and Table 9-23, which again is dependent on the Subgrade Class defined in Table 9-24.

Table 9-22: Particle size specification for natural gravel road bases

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

Source: Gourley and Greening, 1999.

The strength and plasticity specifications vary depending on the traffic level and subgrade class, as outlined in Table 9-23. The soaked CBR test is used to specify the minimum road base material strength.

Table 9-23: Plasticity specifications for natural gravel road base materials

Subgrade Class(1)	Property	Limit of design traffic class				
		0.01 M	0.1 M	0.3 M	0.5 M	1 M
S2	PI PM Grading	<12 <400 B	<9 <150 B	<6 <120 A	<6 <90 A	<6 <90 A
S3	PI PM Grading	<15 <550 C(2)	<12 <250 B	<9 <180 B	<6 <90 A	<6 <90 A
S4	PI PM Grading	Note (3) <800 D(4)	<12 <320 B	<12 <300 B	<9 <200 B	<9 <90 A
S5	PI PM Grading	Note (2) n/s D(4)	<15 <400 B	<12 <350 B	<12 <250 B	<9 <150 A
S6	PI PM Grading	Note (2) n/s D(4)	<15 < 550 C(2)	<15 < 500 B	<12 < 300 B	<9 < 180 A

Notes:

(1) S2 to S6 are the subgrade classes defined by their CBR values (@100 % BS light compaction).

(2) Grading 'C' is not permitted in wet environments or climates; grading 'B' is the minimum requirement.

(3) Maximum PI = 8 x GM.

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

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

PI = Plasticity index, PM = Plasticity modulus = PI x $P_{0.425}$, n/s = not specified.

Table 9-24: Subgrade class definitions

Subgrade Class	Design CBR (%)	Notes
S2	3 - 4	May be used in fills not exceeding 2 m in height
S3	5 - 8	May be used in all fills
S4	9 - 14	May be used in all fills
S5	15 -29	May be used in all fills and as selected fill layer: the selected fill is usually compacted to 95% heavy compaction.
S6	≥ 30	May be used in all fills and as sub-base if the upper 150 mm or the sub-base layer is fully compacted to 95% heavy compaction.

Source: Gourley and Greening, 1999.

A maximum plasticity index of 6 % is specified for higher traffic classes and also on weaker subgrades. For designs in dry environments, the plasticity modulus for each traffic and subgrade class can be increased depending on the crown height and whether unsealed or sealed shoulders are used.

Specifications for lateritic gravel pavement materials: The requirements for the selection and use of lateritic gravels for bases, as shown in Table 9-25, are slightly different to those given for other natural gravels, as shown above. A maximum PI of 9 % has been specified for some of the higher traffic levels (0.3 – 0.5 MESA) and weak subgrades (S2). For design traffic levels greater than 0.3 MESA, a requirement is set that the liquid limit should be less than 30 %. Below this traffic level, this requirement is relaxed to a liquid limit of less than 35%. Where sealed shoulders over one-metre wide are specified in the design, the maximum plasticity modulus may be increased by 40 %. A minimum field compacted dry density of 2,000 kg/m³ is required for these materials.

Table 9-25: Specification for lateritic gravel base course materials

Subgrade Class	Property	Limit of design traffic class (MESA)				
		<TLC 0.01	TLC 0.1	TLC 0.3	TLC 0.5	TLC 1.0
S2	PI PM GE	<15 <400 B	<12 <150 B	<9 <150 A	<9 <120 A	<6 <90 A
S3	PI PM GE	<18 <550 B	<15 <250 B	<12 <180 B	<9 <120 A	<6 <90 A
S4	PI PM GE	<20(1) ƒ800 GM 1.6-2.6	<15 <320 B	<15 <300 B	<9 <200 B	<9 <90 A
S5	PI PM GE	<25(1) n/s GM 1.6-2.6	<18 <400 B	<15 <350 B	<12 <250 B	<9 <150 B
S6	PI PM GE	<25(1) n/s GM 1.6-2.6	<20 <550 B	<18 <400 B	<15 <300 B	<12 <180 A

Notes:
(1) PI maximum = 8 x GM.
n/s = not specified.
Unsealed shoulders are assumed.
PI = Plasticity Index.
PM = Plasticity Modulus.
GE = Grading Envelope.
GM = Grading Modulus.

Source: Gourley and Greening, 1999.

Specifications for subbase materials:

- **Strength requirements**: A minimum CBR of 30% is required at the highest anticipated moisture content when compacted to the specified field density, usually a minimum of 95% (preferably 97% where practicable) AASHTO T180 compaction.

  Under conditions of good drainage and when the water table is not near the ground surface, the field moisture content under a sealed pavement will be equal to or less than the optimum moisture content in the AASHTO T180 compaction test. In such conditions, the subbase material should be tested in the laboratory in an unsaturated state.

  If the base allows water to drain into the lower layers, as may occur with unsealed shoulders and under conditions of poor surface maintenance where the base is pervious, saturation of the subbase is likely. In these circumstances, the bearing capacity should be determined on samples soaked in water for a period of four days. The test should be conducted on samples prepared at the density and moisture content likely to be achieved in the field.

- **Particle size distribution and plasticity requirements**: In order to achieve the required bearing capacity, and for uniform support to be provided to the upper pavement, limits on soil plasticity and particle size distribution may be required. Materials which meet the recommendations of Table 9-26 and Table 9-27 will usually be found to have the adequate bearing capacity.

Table 9-26: Typical particle size distribution for subbases

Sieve size (mm)	Percent by mass passing
50	100
37.5	80-100
20	60-100
5	30-100
1.18	17-75
0.3	9-50
0.075	5-25

Table 9-27: Plasticity requirements for granular subbases

Climate	Annual rainfall mm	Weinert N value	Liquid Limit	Plasticity Index	Linear Shrinkage
Wet	> 1000	N < 2	< 35	< 6	< 3
Moderate	500 - 1000	2 < N < 4	< 45	< 12	< 6
Dry	< 500	N > 4	< 55	< 20	< 10

9.2.8 Design of Pavements with Non-discrete Surfacings

General

Structural surfaces may have a place for use on LVRs. The initial cost is usually a constraining factor but the whole life costs may sometimes make these options favourable. The most common use is for semi-urban areas where marketing and trading take place and where vehicle movements are unpredictable and on sections that are very steep or otherwise difficult from an engineering point of view.

Un-reinforced concrete (URC)

The un-reinforced cement concrete option for LVRs involves casting slabs 4.0 m to 5.0 m in length between formwork with load transfer dowels between them. The thickness of the concrete depends on the traffic and subgrade support, as shown in Table 9-28. In some cases, where continuity of traffic demands it, the slabs should be constructed one lane at a time hence the width of each slab will be half the carriageway width.

Table 9-28: Thickness design for un-reinforced concrete (URC) pavement (mm)

Subgrade Class (CBR)	TLC 01	TLC 0.1	TLC 0.3	TLC 0.5	TLC 1.0
	< 01	0.01-0.1	0.1-0.3	0.3-0.5	0.5-1.0
S2 (3-4%)	160 URC	170 URC	175 URC	180 URC	190 URC
	150 G30	150 G30	150 G30	150 G30	150 G30
S3 (5-7%)	150 URC	160 URC	165 URC	170 URC	180 URC
	125 G30	125 G30	125 G30	125 G30	125 G30
S4 (8-14%)	150 URC	150 URC	160 URC	170 URC	180 URC
	100 G30	100 G30	100 G30	100 G30	100 G30
S5 (15-29%)	150 URC	150 URC	160 URC	170 URC	180 URC
	100 G30	100 G30	100 G30	100 G30	100 G30
S6 (>30%)	150 URC	150 URC	160 URC	170 URC	180 URC

Notes: 1. Cube strength = 30 MPa at 28 days. 2. On subgrades > 30%, the material should be scarified and re-compacted to ensure the depth of material of in situ CBR >30% is in agreement with the recommendations.

Concrete strips

Concrete strips are currently not commonly used in Malawi, but they are a viable solution where traffic volumes are very low (< about 30 vpd) or simply to enable vehicles to traverse short sections with difficult terrain and ground conditions as shown in Figure 9-15. Should the traffic volume exceed 30 vpd, consideration can be given to using twin strips. The pavement thickness under discrete elements given in Table 9-32 is used for the design. It is important to ensure adequate support under the strips to prevent cracking and movement under load, especially in conditions of high moisture.

Figure 9-15: Concrete strips to provide access up a steep, sandy section

The strips must be constructed of C20 class concrete (minimum requirement). If heavy trucks are expected, mesh wire reinforcement shall be used and placed at 1/3 depth from the surface. The concrete strips shall be 0.5 m wide, 1.5 m to 3.0 m (max) in length and 0.2 m in thickness. The distance from centre to centre shall be 1.5 m.

9.2.9 Design of Roads with Discrete Element Surfacings

General

Discrete element surfaces for LVRs do not usually provide much structural strength in terms of load spreading because the interlock between the elements is poor. However, such surfacings are very useful for areas of marketing and trading and on the approach to and in junctions. Some have the advantage that they can be uplifted and replaced if damage to the surfaces themselves occurs or if there is a need to repair the underlying layers because of soil movement and deformation.

Hand-packed Stone (HPS)

HPS paving 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. A degree of interlock is achieved and has been assumed in the designs shown in Table 9-29. The structures also require a capping layer when the subgrade is weak, and a conventional sub-base of NG30 material or stronger is required. A capping layer also provides a smooth, stable platform to work on.

The HPS is normally bedded on a thin layer of sand (SBL). An edge restraint or kerb constructed, for example, of large or mortared stones improves durability and lateral stability.

HPS pavements can be quite rough and uncomfortable for the drivers and cause rapid wear of the vehicle suspension. A regulating layer on top of the HPS can be used to provide a smooth(er) riding surface, but this will, of course, also add to the cost of the pavement.

Table 9-29: Pavement designs for Hand Packed Stone (HPS) pavement (mm)

Subgrade Class (CBR)	TLC 01	TLC 0.1	TLC 0.3	TLC 0.5	TLC 1.0
	< 01	0.01-0.1	0.1-0.3	0.3-0.5	0.5-1.0
S2 (3-4%)	150 HPS	200 HPS	200 HPS	250 HPS	NA
	50 SBL	50 SBL	50 SBL	50 SBL	
	175 G30	125 G30	150 G30	150 G30	
		150 G15	200 G15	200 G15	
S3 (5-7%)	150 HPS	200 HPS	200 HPS	250 HPS	NA
	50 SBL	50 SBL	50 SBL	50 SBL	
	125 G30	200 G30	150 G30	150 G30	
			150 G15	150 G15	
S4 (8-14%)	150 HPS	200 HPS	200 HPS	250 HPS	NA
	50 SBL	50 SBL	50 SBL	50 SBL	
	100 G30	150 G30	200 G30	200 G30	
S5 (15-29%)	150 HPS	200 HPS	200 HPS	250 HPS	NA
	50 SBL	50 SBL	50 SBL	50 SBL	
	Note	*Note*	*Note*	*Note*	
S6 (>30%)	150 HPS	200 HPS	200 HPS	250 HPS	NA
	50 SBL	50 SBL	50 SBL	50 SBL	
	Note	*Note*	*Note*	*Note*	

Notes: The capping layer of NG15 material and the sub-base layer of NG30 material can be reduced in thickness if stronger material is available.

The capping layer can be NG10 provided it is laid 7% thicker.

On subgrades > 15%, the material should be scarified and re-compacted to ensure the depth of material of in situ CBR >15%.

Pave or Stone Setts

Stone sett surfacing or Pavé consists of a layer of roughly cubic (100 mm) stone setts laid on a bed of sand or fine aggregate within the mortared stone or concrete edge restraints. The 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 with a roller. Suitable structural designs are shown in Table-9-32.

Clay Bricks

Fired Clay Bricks are the product of firing moulded blocks of silty clay. The road surfacing consists of a layer of edge-on engineering quality bricks within mortar bedded and jointed edge restraints, or kerbs, on each side of the pavement. The thickness designs are as shown in Table 9-32 for TLC 0.01 and TLC 0.1. Fired clay brick surfacings are not suitable for traffic classes above TLC 0.1.

Cobble Stones or Dressed Stone pavements

Cobble or Dressed Stone surfacings are similar to Pave and consist of a layer of roughly rectangular dressed stones laid on a bed of sand or fine aggregate within the mortared stone or concrete edge restraints. The 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 roller. Cobble stones are generally 150 mm thick and dressed stones generally 150 mm to 200 mm thick. These options are suited for homogeneous rock types that have inherent orthogonal stress patterns (such as granite) that allow for an easy break of the fresh rock into the required shapes by labour-based means.

The thickness designs are given in Table 9-30.

Table 9-30: Pavement designs for various discrete element surfacings (DES) (mm)

Subgrade Class (CBR)	TLC 01 < 0.01	TLC 0.1 0.01-0.1	TLC 0.3 0.1-0.3	TLC 0.5 0.3-0.5	TLC 1.0 0.5-1.0
S2 (3-4%)	DES	DES	DES	DES	DES
	25 SBL	25 SBL	25 SBL	25 SBL	25 SBL
	100 G65	125 G65	150 G80	150 G80	150 G80
	100 G30	150 G30	150 G30	175 G30	200 G30
	100 G15	150 G15	175 G15	200 G15	200 G15
S3 (5-7%)	DES	DES	DES	DES	DES
	25 SBL	25 SBL	25 SBL	25 SBL	25 SBL
	125 G65	150 G65	125 G80	150 G80	150 G80
	100 G30	175 G30	125 G30	150 G30	175 G30
			150 G15	150 G15	175 G15
S4 (8-14%)	DES	DES	DES	DES	DES
	25 SBL	25 SBL	25 SBL	25 SBL	25 SBL
	150 G65	150 G65	150 G80	150 G80	175 G80
		100 G30	150 G30	200 G30	225 G30
S5 (15-29%)	DES	DES	DES	DES	DES
	25 SBL	25 SBL	25 SBL	25 SBL	25 SBL
	125 G65	100 G65	125 G80	150 G80	150 G80
		125 G30	125 G30	125 G30	150 G30
S6 (>30%)	DES	DES	DES	DES	DES
	25 SBL	25 SBL	25 SBL	25 SBL	25 SBL
	125 G65	150 G65	150 G80	150 G80	150 G80
	Note	*Note*	*Note*	*Note*	*Note*

Notes: The capping layer of NG15 material and the sub-base layer of NG30 material can be reduced in thickness if stronger material is available.
The capping layer can be NG10 provided it is laid 7% thicker.

Mortared options

In some circumstances (e.g. on slopes in high rainfall areas and subgrades susceptible to volumetric change) it may be advantageous to use mortared options for the discrete element surfacings. This can be done with Hand-packed Stone, Stone Setts (or Pavé), Cobblestone 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. Empirical evidence indicates that inter-block cracking may occur. For this reason, the option is currently only recommended for the lightest traffic load classes up to TLC 0.1.

Precast concrete blocks

Precast concrete blocks are usually constructed on a cement-stabilised subbase. A 25 mm - 50 mm sand blanket should be placed on top of the cement-stabilised subbase to provide a cushion and a drainage layer. The blocks shall be made of concrete of 28-day cube strength of 20MPa with thickness depending on the expected traffic as per the manufacturer's specifications, e.g. 60 mm for light traffic, 80 -120 mm for heavy/very heavy traffic. For increased bearing capacity, interlocking blocks are recommended.

9.3 Design of Urban and Peri-urban Roads and Streets

9.3.1 General

The methods of pavement design covered in the previous sections can also be applied in urban and peri-urban environments. However, since urban and peri-urban roads and streets are often relatively short and the pavement design is constrained by the geometry (building lines, levels), services, adjacent land use and drainage design, the subdivision in uniform sections with potentially different pavement structures to achieve an Environmentally Optimised Design, has little relevance. For this reason, catalogue designs, as shown in the following sections, will in most cases be appropriate for urban and peri-urban roads and streets.

9.3.2 Design Considerations

The structural design of pavement in urban and peri-urban environments cannot be executed in isolation. The following parameters should be considered before deciding on the structural design of the street pavement:

- Area layout planning.
- Road category and function.
- Drainage design.
- Pattern of the daily traffic in urban and peri-urban environment, with a pronounced peak hour traffic.
- High level of access compared to mobility.
- At grade intersections and junctions, checkpoints and other features such as parking, walkways and loading areas.
- Appropriate surfacing on different areas of the street and adjacent public spaces.
- Ability to perform maintenance.
- Community preferences.

It is important to attend to area layout planning and drainage design before the structural design of the street is addressed, as this will provide better definition to the role of the street in the larger development and affect the final structural design of the pavement. For example, streets that cross contours at an angle, or even perpendicularly, pose the most drainage problems. In such cases, the risk of erosion may require that a street be paved or provided with erosion protection.

The layout planning, mobility, road category and function, are addressed in the *Geometric Design Manual: Part B*. Previous chapters in this part of the Manual deal with traffic (*Chapter 6 - Traffic*) and drainage (*Chapter 8 – Drainage and Erosion Control*).

Due to the constraints for undertaking large scale road works in an urban environment, as well as the disruption to traffic the works would inevitably cause, the typical design life of an urban surfaced street generally ranges between 10 years and 30 years with the recommended value of 20 years.

9.3.3 Determination of Design Subgrade Class

The determination of the subgrade strength is vital for the choice of pavement design options. The design parameter for the subgrade is the soaked CBR at a representative density, as shown in Table 9-31.

For structural design purposes, when a material is classified according to the CBR, it is implied that not more than 10% of the measured values for such a material will fall below the classification value. A proper preliminary soil survey should be conducted.

Class	Subgrade CBR
SG1	>15
SG2	7 to 15
SG3	3 to 7
SG4	< 3*
* Special treatment required	

Table 9-31: Subgrade classes

It is current practice to use soaked CBR values, but using them in dry regions may be over-conservative. However, it should be borne in mind that streets serve as part of the urban drainage system and cannot be raised to achieve improved drainage. The CBR value of a material could be increased if the in-situ conditions are expected to be unsoaked, e.g. in dry regions. An estimate of the CBR at the expected moisture conditions, i.e. at the optimum moisture content (OMC) or, say, 75% of OMC, can be determined in the laboratory by refraining from soaking the samples before CBR testing or even drying back to the required moisture content.

These subgrade CBR classes are used to determine the support for the pavement structure (also known as "Foundation"). The designed foundation structure is also represented as part of the pavement design; this is illustrated in the different catalogues presented in this section.

9.3.4 Pavement Design Options

Several options are available for urban and peri-urban pavement design. In addition to a consideration of the factors mentioned above, the final design must be based on a careful life-cycle cost analysis (see *Chapter 12 – Life Cycle Costing*), and also take into account other factors such as:

- Environment (Topography, climate and In-situ subgrade condition)
- Availability of materials (natural gravels, cement, bitumen, paving blocks etc.)
- Skills and experience of local contractors
- Availability of (specialised) plant and equipment
- Suitability for labour-based methods of construction, the use of which may be highly relevant in urban areas

The following sections will discuss the most relevant pavement design options, viz:

a) Granular pavements
b) Cement stabilised pavements
c) Bitumen stabilised pavements
d) Asphalt hot-mix pavements
e) Concrete pavements
f) Paving blocks or bricks
g) Cast in-situ paving blocks

The suitability of the above pavement types is shown in Table 9-32.

Table 9-32: Suitability of pavement types

Pavement type		Road/Street Category and Pavement Class						Reasons for not recommending
		Distributors				Access		
Base	Subbase	District	Tertiary/Local			Collectors/Streets		
		Class 3	Class 4			Class 5		
		Traffic load class[1]						
		ES 3	ES 3	ES 1	ES 0.3	ES 0.3	ES 0.1	
Granular[2]	Granular							
	Cemented							
Cemented	Granular							1
	Cemented							2
Bitumen stabilised	Granular							
	Cemented							2
Asphalt hot-mix	Granular							2
	Cemented							2
Concrete	Granular							3
	Cemented							2
Paving blocks	Granular							
	Cemented							
Cast in-situ blocks								
	Legend:	Recommended			Not recommended			

1. Traffic load class: ES 3 = 1.0-3.0; ES 1 = 0.3-1.0; ES 0.3 = 0.1-0.3; ES 0.1 = 0.03-0.1 MESA
2. Includes crusher run, crushed stone, etc.
Reasons for not recommending: 1) Cracking, pumping and rocking of blocks. 2) Expensive. 3) Extra thickness required to prevent fatigue cracking.

a) Granular base pavement

This type of pavement comprises of a thin bituminous surfacing, a base of untreated gravel or crushed stone, a granular or cemented subbase and a subgrade of various soils or gravels. The mode of distress in a pavement with an untreated subbase is usually deformation, arising from shear or densification in the untreated materials. The deformation may manifest itself as rutting or as longitudinal roughness eventually leading to cracking. In pavements with cemented sub-bases, the subbase improves the load-carrying capacity of the pavement, but at some stage, the subbase will crack. The cracking may propagate until the layer eventually exhibits properties similar to those of a natural granular material. Under heavier traffic, this might give rise to failure.

Pavement design catalogues for granular base pavements are shown in Table 9-35 and Table 9-36.

b) Cemented stabilised base pavement

As concrete, cemented materials are elastic, possess tensile strength and may crack under repeated flexure. These materials also crack because of shrinkage and drying. In a cemented stabilised pavement, most of the traffic stress is absorbed by the cemented layer and little by the subgrade. It is likely that some block cracking will appear at an early stage due to drying and shrinkage and thermal stresses in the cemented layers. Traffic-induced cracking will cause the blocks to break up into smaller pieces. These cracks propagate through the surface. The ingress of water through the surface cracks may cause the blocks to rock under traffic, resulting in the pumping of fines from the lower layers. Rutting or roughness will generally be low up to this stage but is likely to accelerate as the extent of the cracking increases.

Cement stabilised bases are expensive as well, and hence should be low on the order of alternatives to be considered.

The pavement design catalogue for cemented stabilised bases for low volume roads is shown in Table 9-37.

c) Bitumen stabilised base pavement

Bitumen stabilised materials (BSM) are pavement materials that are treated with either bitumen emulsion or foamed bitumen. The materials treated are normally granular materials, previously cement-treated materials or reclaimed asphalt (RA) layers. Where an existing pavement is recycled, old seals or asphalt surfacing is usually mixed with the underlying layer and treated to form a new base or subbase layer.

Road pavements constructed with bitumen stabilised materials (BSMs) using either bitumen emulsion or foamed bitumen are environmentally sustainable and cost-effective; and, when good construction techniques are used, these pavements perform very well. BSMs are suited to both construction of new pavements and to pavement rehabilitation using in- situ recyclers and/or conventional construction equipment. BSMs are also ideally suited to labour-intensive construction.

The main features and behaviour of BSMs are:

- BSMs exhibit a significant increase in cohesion in comparison to the parent material. The friction angle of the treated material is typically similar to the untreated material.
- BSMs acquire flexural strength as a result of the combined effect of the visco-elastic properties of the dispersed bitumen droplets. Since the individual bitumen droplets are not linked, and the coarser aggregate particles remain uncoated, BSMs retain the granular characteristics of the parent material. It is, therefore, stress dependant and is not prone to cracking when subjected to tensile stresses.
- BSMs perform well when cohesive strength is optimised through proper mix design (to determine the optimal bitumen and active filler contents), whilst retaining enough flexibility so that friction resistance is still activated under load.

- Since the bitumen is dispersed only amongst the finer aggregate particles, the fines are encapsulated and immobilised. This improves the moisture sensitivity and durability of the treated materials. Provided sufficient bitumen is applied, the tendency for the BSM to pump under loading in saturated conditions is also significantly reduced because the fines are bound.

The behaviour and stiffness of BSMs vary significantly depending on the quantities of bitumen and active filler used. In particular, when excessive cement is used, the materials behave more like cement-treated materials and the benefit of adding bitumen is questionable. For this reason, cement contents that exceed 1% are not recommended, and the ratio of added bitumen to added active filler should always exceed one.

Unlike a hot mixed asphalt, BSMs are not overly sensitive materials. Small variations in both the amount of bitumen added and untreated material properties will not significantly change the strength achieved through treatment. This allows the inevitable variability in the recycled material to be tolerated.

Layers of BSM-emulsion may be subjected to traffic within a few hours (after the bitumen emulsion in the upper portion of the layer breaks). After compaction, layers of BSM-foam have sufficient strength to be trafficked immediately with little detrimental effect. Traffic disruption and time delays are minimised by working in half widths and opening to traffic soon after completion. The construction and maintenance of detours are, therefore avoided.

BSMs are classified into three classes, depending on the quality of the parent material and the design traffic. The three classes are:

- BSM1: This material has a high shear strength and is typically used as a base layer for design traffic of more than 6 MESA. For this class of material, the source material is typically a well-graded crushed stone or reclaimed asphalt (RA).
- BSM2: This material has moderately high shear strength and is typically used as a base layer for design traffic applications of less than 6 MESA. For this class of material, the source material would typically be graded natural gravel or RA.
- BSM3: This material typically consists of soil-gravel and/or sand, stabilised with higher bitumen contents. As a base layer, the material is only suitable for design traffic applications of less than 1 MESA.

Indicator tests for classification of BSMs, and a catalogue design for BSM pavements, are provided in Table 9-41 and Table 9-42 in Section 9.3.8.

d) Hot-mix bitumen base pavement

These pavements have a bituminous layer more than 80 mm thick. Bituminous materials are visco-elastic and under repeated stresses, they may either crack, or deform, or both. In hot-mix bitumen-base pavements both deformation and fatigue cracking are possible.

Two types of subbases are recommended, namely either an untreated granular subbase or a weakly stabilised cemented subbase. Rutting may originate in either the bituminous or the untreated layers, or in both. If the subbase is cemented there is a probability that shrinkage or thermal cracking will reflect through the base to the surfacing, especially if the bituminous layer is less than 150 mm thick or if the subbase is excessively stabilised.

Because of the cost, hot-mix bitumen pavements are not often used in low volume urban and peri-urban roads. However, hot-mix bitumen layer does not require standing time after layer placement, thus minimising construction delays and can be used in the case of fast-tracking construction such as construction under traffic.

e) Concrete pavement

The high modulus of elasticity and rigidity of concrete compared with other road-making materials provide a concrete pavement with a reasonable degree of flexural or beam strength. This property leads to a wide distribution of applied wheel loads, thereby limiting the pressures applied to the subgrade at relatively shallow depths. The concrete layer alone, therefore, provides the major portion of the load-carrying capacity of a concrete pavement.

A cemented subbase is desirable for providing a uniform foundation and limiting the risk of pumping of the subbase and subgrade fines.

The different main types of concrete pavements are:

- Jointed unreinforced (Plain) Concrete pavement (JPCP)
- Jointed Reinforced Concrete Pavement (JRCP)
- Continuously Reinforced Concrete Pavement (CRCP)
- Prestressed Concrete Pavement (PCP) and
- Fibre Reinforced Concrete pavement (FRCP)

Concrete slab thickness can be determined by a computer program. If this is not available, an indication of slab thickness can be taken from Table 9-33. The table presents a suggested range of slab thicknesses at LVR intersections as a function of road classes.

Table 9-33: Concrete slab thickness in LVR intersections

Street Class	Description	Two-way Average Daily Traffic (ADT)	Two-way Average Daily Truck Traffic (ADTT)	Typical range of slab thickness (mm)
Light residential	Short streets in residential areas, often not through streets	< 200	2 -4	100
Residential	Through streets in residential areas that occasionally carry a heavy vehicle (truck or bus)	200 - 100	10 - 50	150

Joints in a concrete layer are necessary, primarily to control the location of cracks and prevent random cracking that occurs from natural actions in the concrete pavement. Joints accommodate the expansion and contraction of concrete slabs caused by temperature fluctuations and account for stresses that develop from slab curling and warping. During the construction, joints also divide the pavement into suitable placement areas or elements for the contractor.

Details of concrete joint spacing and design are beyond the scope of this Manual and must be looked up in relevant literature, some of which are listed in the bibliography.

f) Paving blocks and bricks

In urban and peri-urban areas, block paving is associated with streets, walkways, parking areas, driveways, pedestrian areas and loading areas (waste removal area).

Design catalogues for paving blocks pavements for low volume roads in different environmental conditions and for no-vehicle associated purposes (walkway, slope protection and utility such as swimming pool surrounds) are presented in Table 9-38 and Table 9-39 in Section 9.3.8.

g) Cast in-situ block pavement

This pavement uses the so-called Geocells. Geocells are made of strands of durable material, like a plastic, which when stretched horizontally on a prepared subbase or selected layer creates pockets into which soil is poured to form a pavement layer. Due to the confinement of the surrounded geo-cell material, the bearing strength is dramatically improved. Geo-cells have originally been developed to stabilise sands and cohesionless material, but have developed into erosion protection mats, in situ moulds for concrete blocks and other labour-intensive applications. The cell dimensions can vary in height and size but generally resemble a standard paving block size.

Geo-cells have been used with great success as in situ paving blocks when filled with concrete. Such applications have been used in hard standing areas on quay walls and container stacking areas with success. Geo-cell concrete blocks are relatively expensive as the cell material and its setting up is often perceived as an extra cost versus normal concrete blocks. It is, however, ideal to be used in cohesionless sand and arid regions to form a strong structural work platform. The addition of treated material (cemented or bituminous) in the cells results in additional cost compared to filling them with just soil or sand, but it improves the design life of this compound structure.

Table 9-40 in Section 9.3.8 presents the pavement design catalogue for cast in-situ block pavement.

9.3.5 Other Pavement Options

Roller compacted concrete

Roller Compacted Concrete (RCC), also known as Rollcrete, may provide a viable pavement alternative in urban and peri-urban environments. Rollcrete is a dry-mix concrete produced from a continuously graded crushed stone product using high fly ash content cement.

Rollcrete is normally placed by means of graders or pavers and compacted with vibratory compactors. Its consistency is stiff enough to remain stable under vibratory rollers yet wet enough to permit adequate mixing and distribution of paste without segregation.

The major difference between RCC mixtures and conventional concrete mixtures is that RCC has a higher percentage of fine aggregates, which allows for tight packing, low void content and consolidation. Final compaction is generally achieved within one hour of mixing.

Unlike conventional concrete pavements, RCC pavements are constructed without forms, dowels or reinforcing steel. Joint sawing is not required, but when sawing is specified, transverse joints are spaced further apart than with conventional concrete pavements. The low water-cement ratio of RCC results in less shrinkage crack development than ordinary PCC mixes. The compaction of RCC is often based on the principles of soil compaction to produce a lean concrete, in which the optimum water content of the concrete is the parameter that produces the maximum dry density of the mixture.

For more detailed information on the design and construction of RCC, relevant documentation and manuals should be consulted.

Unpaved streets

Many urban and peri-urban roads and streets will remain unpaved for the foreseeable future. Efforts should, therefore, be made to minimise the problems with the roads in urban environments such as dust pollution and erosion. This can be achieved by:

- Better selection of materials
- Mechanical stabilization using two different materials to achieve a better particle size distribution and to increase or reduce the plasticity
- Applying a chemical dust palliative.

Further information on the use of dust palliatives is presented in *Chapter 10 – Structural Design: Unpaved Roads* in this Manual.

9.3.6 Appropriate Surfacings

Urban and peri-urban roads and streets carry traffic and may also carry stormwater run-off where space is limited. Thus, high-quality surfacing is required.

In selecting an appropriate surfacing, the following aspect should be taken into consideration.

- Local experience and/or preference.
- Availability of materials.
- Road category.
- Design traffic class.
- Environment (e.g. moisture, temperature and ultraviolet radiation).
- Pavement type.
- Turning movements, intersections, braking movements.
- Gradients.

Bituminous surfacings

Different types of flexible, bituminous surfacing for low volume roads are presented in *Chapter 11 - Surfacing*. The suitability of these surfacings in urban and peri-urban environments is shown in Table 9-34. The choice of surfacing should be based on consideration of its cost-effectiveness.

Table 9-34: Suitability of flexible surfacings in urban environments

Recommended initial bituminous surfacing based on frequency of turning actions in different urban environments[1]										
Surfacing type	**A**	**B**	**C**	**D**	**E**	**F**	**G**	**H**	**I**	**J**
Residential – light traffic										
Urban – including heavy vehicles										
Legend		Recommended								
		Only thick slurries (> 10 mm)								
Surfacing type		Not recommended								
A	Sand Seal									
B	Coarse Slurry seal									
C	Single Surface Dressing									
D	10 mm Single Surface Dressing + Coarse sand cover seal									
E	14 mm Single Surface Dressing + Coarse sand cover seal									
F	14 mm Cape Seal + one layer of slurry									
G	14/7 mm Double Surface Dressing									
H	20/10 mm Double Surface Dressing									
I	20 mm Cape Seal + two layers of slurry									
J	Asphalt									

1. Excludes intersections and roundabouts where non-bituminous surfacings would be more appropriate

Structural surfacings

Thicker structural surfacings may, in many cases, provide a better solution in urban environments, both from a durability and an aesthetical point of view. These surfacings are discussed above and in Section 9.2.10.

9.3.7 Intersections and Checkpoints

An intersection of two or more roads can be described as a physical area or as a functional area, as shown in Figure 9-16. The physical area is known as the junction, and the functional area includes the approaches to the junction.

Vehicles stopping and turning can stress the pavement surface severely. This often occurs at the approaches to intersection junctions and at police checkpoints. The pavement within the junction of an intersection may also receive nearly twice as much traffic as the pavement on the approaching roadways. The damage caused to the intersection is more than at almost any other pavement location. Vehicle braking and turning movements generate additional stress in the pavement, especially on the surfacing, often resulting in early severe rutting, aggregate loss, shoving and/or delamination/debonding.

A stiffer or durable surfacing is recommended in this case. The entire functional area and beyond should be considered for the surfacing solution of the intersection.

Non-structural, flexible surfacings that may be applied, are shown in Table 9-34. However, in most cases, non-discrete and discrete surfacings will provide a better, more durable solution at intersections and checkpoints.

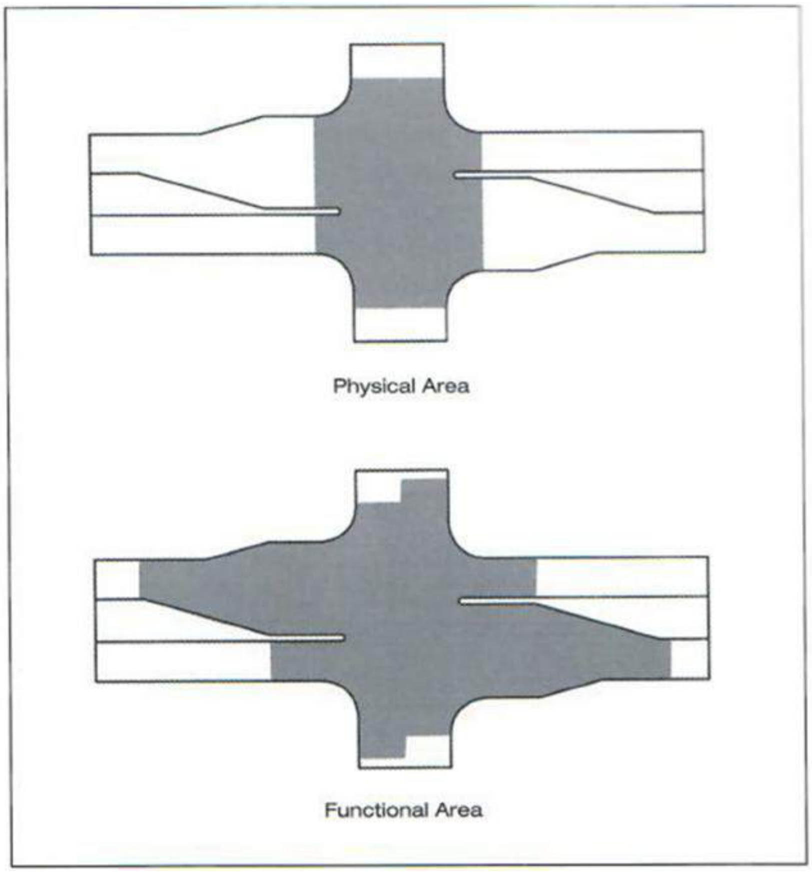


Figure 9-16: Physical and functional area of an intersection

9.3.8 Urban and Peri-urban Pavement Design Catalogues

Table 9-35: LVR Pavement Design Catalogue in Moderate and Dry regions (granular bases)

	Low Volume Granular Bases (Moderate or Dry Region) (Red book, 2005)						
	Design Traffic Class MESA / Lane over structural design Period						
Road Category	**0.001 - 0.003 MESA**	**0.003 - 0.01 MESA**	**0.01 - 0.03 MESA**	**0.03 - 0.1 MESA**	**0.1 - 0.3 MESA**	**0.3 - 1.0 MESA**	**Foundation**
District and local distributors, minor arterials and collectors, industrial roads, goods-area and bus routes						S 125 NG 80 150 C4 S 150 NG 80 150 NG 45	150 NG 15 150 NG 7 NG 3
Residential access collectors, car parks and lightly trafficked bus routes				S 100 NG 45 125 C4 S 125 NG 80 125 NG 25	S 125 NG 45 125 C4 S 125 NG 80 150 NG 25	S 125 NG 80 125 C4 S 125 NG 80 150 NG 45	
Local access roads, loops, -ways, -courts, -strips and culs de sac	S1 100 NG 45 100 NG 15	S1 100 NG 45 125 NG 15	S1 100 NG 80 125 NG 15	S1 100 NG 80 125 NG 25 S1 100 NG 45 100 C4	S 125 NG 80 125 NG 25 S 100 NG 45 125 C4	S 125 NG 80 150 NG 25 S 125 NG 80 150 C4	150 NG 7 NG 3

S: Double surface treatment (seal or combinations of seal and slurry)

S1: Single seal

C4: Cemented stabilised layer with unconfined compressive strength (UCS) between 750 kPa and 1500 kPa, and minimum indirect tensile strength (ITS) of 200 kPa

NG *x*: Natural gravel classification according to CBR where "***x***" is the minimum required CBR

MESA: Million Equivalent Standard Axles

Most likely combinations of road category and design traffic class

Table 9-36: LVR Pavement Design Catalogue in Wet regions (granular bases)

Road Category	Low Volume Granular Bases (Wet Region) (Red book, 2005) – Design Traffic Class MESA / Lane over structural design Period: 0.001 - 0.003 MESA	0.003 - 0.01 MESA	0.01 - 0.03 MESA	0.03 - 0.1 MESA	0.1 - 0.3 MESA	0.3 - 1.0 MESA	Foundation
District and local distributors, minor arterials and collectors, industrial roads, goods-area and bus routes						S 150 CR 80 150 C4 S 150 CR 80 200 NG 45	150 NG 15 150 NG 7 NG 3
Residential access collectors, car parks and lightly trafficked bus routes				S 100 NG 45 125 C4 S 125 NG 80 125 NG 25	S 125 NG 45 125 C4 S 150 NG 80 150 NG 25	S 125 CR 80 150 C4 S 150 CR 80 150 NG 45	
Local access roads, loops, -ways, -courts, -strips and culs de sac	S1 100 NG 45 100 NG 15	S1 100 NG 45 125 NG 15	S1 100 NG 80 125 NG 15	S1 100 NG 80 125 NG 25 S1 100 NG 45 100 C4	S 125 NG 80 125 NG 25 S 100 NG 45 125 C4	S 150 NG 80 150 NG 25 S 125 NG 80 150 C4	150 NG 7 NG 3

S: Double surface treatment (seal or combinations of seal and slurry)

S1: Single seal

C4: Cemented stabilised layer with unconfined compressive strength (UCS) between 750 kPa and 1500 kPa, and minimum indirect tensile strength (ITS) of 200 kPa

NG *x* : Natural gravel classification according to CBR where "***x***" is the minimum required CBR

CR *x* :Crushed rock classification according to CBR where "***x***" is the minimum required CBR

MESA: Million Equivalent Standard Axles

Most likely combinations of road category and design traffic class

Table 9-37: LVR Pavement Design Catalogue for Cemented Bases

Road Category	Low Volume Cemented Bases (Red book, 2005) — Design Traffic Class MESA / Lane over structural design Period: 0.001 - 0.003 MESA	0.003 - 0.01 MESA	0.01 - 0.03 MESA	0.03 - 0.1 MESA	0.1 - 0.3 MESA	0.3 - 1.0 MESA	Foundation
District and local distributors, minor arterials and collectors, industrial roads, goods-area and bus routes						S 125 C3 150 C4	150 NG 15 150 NG 7 NG 3
Residential access collectors, car parks and lightly trafficked bus routes				S1 100 C4 100 NG 25	S 200 C3 S 125 C3 125 NG 25	S 125 C3 125 C4	
Local access roads, loops, -ways, -courts, -strips and culs de sac	S1 100 C4 100 NG 10	S1 100 C4 125 NG 10	S 125 C4 125 NG 15	S 150 C4 150 NG 15	S 125 C4 125 NG 25	S 125 C4 150 NG 25	150 NG 7 NG 3

S: Double surface treatment (seal or combinations of seal and slurry)

S1: Single seal

C3: Cemented stabilised layer with unconfined compressive strength (UCS) between 1500 kPa and 3000 kPa, and minimum indirect tensile strength (ITS) of 250 kPa

C4: Cemented stabilised layer with unconfined compressive strength (UCS) between 750 kPa and 1500 kPa, and minimum indirect tensile strength (ITS) of 200 kPa

NG *x* : Natural gravel classification according to CBR where "***x***" is the minimum required CBR

MESA: Million Equivalent Standard Axles

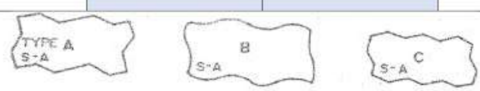
Most likely combinations of road category and design traffic class

Table 9-38: LVR Pavement Design Catalogue for Paving Blocks (Moderate and Dry Regions)

Low Volume Paving Blocks (Moderate and Dry Regions) (Red book, 2005)

Road Category	Design Traffic Class MESA / Lane over structural design Period						
	0.001 - 0.003 MESA	0.003 - 0.01 MESA	0.01 - 0.03 MESA	0.03 - 0.1 MESA	0.1 - 0.3 MESA	0.3 - 1.0 MESA	Foundation
District and local distributors, minor arterials and collectors, industrial roads, goods-area and bus routes						60 S-A S-B or S-C 20 SND 100-150 NG 45 60 S-A S-B or S-C 20 SND 125 C4	
Residential access collectors, car parks and lightly trafficked bus routes				60 S-A S-B or S-C 20 SND 100-125 NG45		60 S-A S-B or S-C 20 SND 100-150 NG45 60 S-A S-B or S-C 20 SND 125 C4	
Local access roads, loops, -ways, -courts, -strips and culs de sac			60 S-A S-B or S-C 20 SND				150 NG 7 NG 3

S-A, S-B and **S-C** : standard paving blocks shapes



SND: Bedding sand (as defined by Draft UTG2)

C4: Cemented stabilised layer with unconfined compressive strength (UCS) between 750 kPa and 1500 kPa, and minimum indirect tensile strength (ITS) of 200 kPa

NG *x* : Natural gravel classification according to CBR where "***x***" is the minimum required CBR

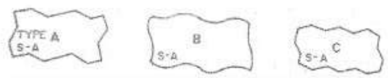
MESA: Million Equivalent Standard Axles

[shaded box] Most likely combinations of road category and design traffic class

Table 9-39: LVR Pavement Design Catalogue for Paving Blocks (Wet Regions)

Low Volume Paving Blocks (Wet Regions) (Red book, 2005)							
	Design Traffic Class MESA / Lane over structural design Period						
Road Category	0.001 - 0.003 MESA	0.003 - 0.01 MESA	0.01 - 0.03 MESA	0.03 - 0.1 MESA	0.1 - 0.3 MESA	0.3 - 1.0 MESA	Foundation
District and local distributors, minor arterials and collectors, industrial roads, goods-area and bus routes						60 S-A S-B or S-C 20 SND 150 NG 45 60 S-A S-B or S-C 20 SND 150 C4	
Residential access collectors, car parks and lightly trafficked bus routes				60 S-A S-B or S-C 20 SND 100-125 NG45 60 S-A S-B or S-C 20 SND 100 C4		60 S-A S-B or S-C 20 SND 100-150 NG45 60 S-A S-B or S-C 20 SND 100-150 C4	
Local access roads, loops, -ways, -courts, -strips and culs de sac			60 S-A S-B or S-C 20 SND				150 NG 7 NG 3

S-A, **S-B** and **S-C** : standard paving blocks shapes



SND: Bedding sand (as defined by Draft UTG2)

C4: Cemented stabilised layer with unconfined compressive strength (UCS) between 750 kPa and 1500 kPa, and minimum indirect tensile strength (ITS) of 200 kPa

NG *x*: Natural gravel classification according to CBR where "***x***" is the minimum required CBR

MESA: Million Equivalent Standard Axles

Most likely combinations of road category and design traffic class

Table 9-40: LVR Pavement Design catalogue for Cast In-situ Block Pavements

Pavement Code	Pavement Structure
A	75/1: 1 Cell Slab 150 G5 or 150 C4 In-Situ*
B	75/1: 1 Cell Slab 150 G5 or 125 C4 In-Situ*
C	75/1: 1 Cell Slab 125 G5 or 100 C4 In-Situ*
D	75/2: 1 Cell Slab In-Situ*
E	50/2: 1 Cell Slab In-Situ*

* Note requirements for *in-situ* preperation

	Design traffic during structural life (EBO)		
Route and environment Classification	$<0,2x10^6$	$0,2-0,8x10^6$	$0,8-3,0x10^6$
Minor arterial, wet Climate, 3 - 7 CBR*		A	A
Minor arterial, wet Climate, CBR > 7		A	A
Minor arterial, dry Climate, 3 - 7 CBR		B	B
Minor arterial, dry Climate, CBR > 7		C	A
Access street, wet Climate, 3 - 7 CBR	D	C	B
Access street, wet Climate, CBR > 7	E	C	B
Access street, dry Climate, 3 - 7 CBR	D	C	B
Access street, dry Climate, CBR > 7	E	C	B

* Special precautions have to be taken if the design CBR < 3

Note: 75/1: 1 Cell Slab = 75 mm thick slab made with 1:1 sand: cement ration concrete in Geocells.

Table 9-41: Indicators and tests for classification of BSMs

Test or Indicator	Material[1]	Design Equivalent Material Class			Not suitable for treatment	CF
		BSM1	BSM2	BSM3		
Soaked CBR (%)	CS (98%)	> 80	25 to 80	10 to 25	< 10	0.4
	NG (95%)		> 25	10 to 25	< 10	
P0.075 (%) (Bitumen emulsion)	CS	4 to 15			> 15	0.35
	NG		5 to 25	25 to 40	> 40	
	GS		5 to 20	15 to 30	> 30	
	SSSC			0 to 20	> 20	
P0.075 (%) (Foamed bitumen)	CS	2 to 15			> 15	
	NG		11 to 25	23 to 40	> 40	
	GS		0 to 20	13 to 30	> 30	
	SSSC			0 to 20	> 20	
Relative density	All	> 0.98	0.95 to 0.98	0.93 to 0.95	< 0.93	0.1
DCP Pen (mm/blow)	All	< 3.7	3.7 to 9.1	9.1 to 19.0	> 19.0	0.1
FWD Backcalculated Stiffness (MPa)	All	> 300	150 to 300	70 to 150	< 70	0.1
Plasticity Index	CS	< 10	> 10			0.25
	NG	<6	6 to 12	> 12		
	GS		> 11	11 to 15	< 15	
	SSSC			< 15	> 14	
Relative moisture (%)	CS	< 90	> 90			0.1
	NG	< 70	70 to 100	< 80		
	GS		> 100	80 to 100	< 100	
	SSSC			> 100	> 100	
Grading modulus	NG	2.0 to 3.0	1.2 to 2.7	0.15 to 1.2	0.15	0.2
	GS		1.2 to 2.5	0.75 to 2.7	< 0.75	
Cohesion (kPa)	All	> 250	100 to 250	50 to 100	< 50	0.45
Friction Angle (°)	All	> 40	30 to 40	< 30		0.4
Tangent Modulus (MPa)	All	> 150	50 to 150	< 50		0.1
ITS (kPa)	Dry, 100 mm[2]	> 225	175 to 225	125 to 175	< 125	0.1
	Equilib, 150 mm	> 175	135 to 175	95 to 135	< 95	0.2
ITS (wet) kPa)	100 mm	> 100	75 to 100	50 to 75	< 50	0.1
UCS (kPa)	All	1 200 to 3 500	700 to 1 200	450 to 700	< 450	0.1
Retained cohesion after MIST (%)	All	> 75	60 to 75	50 to 60	< 50	0.45
Rating	All	0.5 to 1.5	1.5 to 2.5	2.5 to 3.5	3.5 to 4.5	N/A

Notes:

1. CS = Crushed stone, NG = natural gravel, GS = gravel soil, SSSC = sand, silty sand, silt, clay; 98%, 95%, 93%, 90% are Relative densities (percentage of maximum dry density)
2. Diameter of specimen
3. CF = Certainty factor: is a factor (in the certainty theory) that assigned to each test result available for a material, the certainty that this material falls into each of the four material classes (BSM1, BSM2, BSM3 and unsuitable for treatment with BSMs). CF represents the subjective confidence in the ability of a test to serve as an accurate indicator for material strength and stiffness in the pavement layer. The value of CF ranges from 0 to 1, with a value of 1 indicating absolute confidence in a test or indicator (highly unlikely)

Table 9-42: LVR BSM Pavement Design Catalogue

	Pavement Class and Design Bearing Capacity		Foundation (CBR)
Road Category	ES0.3 ≤ 300 000	ES1 300 000 to 1 000 000	
B (95% Reliability)		S 125 BSM2	> 15
		S 125 BSM2 150 NG25	7 to 15
		S 125 BSM2 150 NG25 150 NG15 150 NG7 40 AC 300 BSM2	3 to 7
C (80 % Reliability)	S 100 BSM3 125 NG25 S 125 BSM3	S 125 BSM3 150 G6 S 150 BSM3	> 15
	S 125 BSM3 125 NG25 125 NG15 S 300 BSM3	S 125 BSM3 150 NG45 150 NG15 40 AC 200 BSM3	7 to 15
	S 100 BSM3 125 NG25 125 NG15 125 NG7 40 AC 200 BSM3	S 150 BSM3 150 NG25 150 NG15 150 NG7 40 AC 200 BSM3	3 to 7
D (50% Reliability)	S 100 BSM3	S 100 BSM3	> 15
	S 100 BSM3 125 NG25 S 250 BSM3	S 100 BSM3 125 NG25 30 AC 150 BSM3	7 to 15
	S 125 BSM3 125 NG25 S 300 BSM3	S 125 BSM3 150 NG25 30 AC 175 BSM3	3 to 7

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Appendix 1: The Laboratory DN Test

General

The laboratory DN test is the most important test in the DCP-DN design method, and it is crucially important that it is carried out 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, as explained in Section 10.3.4.

Preparation of test samples

The samples must be prepared in accordance with the SANS 3001 – GR30, as described below:

1. Procedure 1 – Scalping Method (Modified from Clause 5.2.1.2) which applies to materials that have 30% or more (by weight) retained on the 20 mm sieve, 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.
2. Procedure “ - Crushing Method (Clause 5.2.1.3) which applies to materials that have 30% or less (by weight) retained on the 20 mm sieve, 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 A1-1.

Table A1-1: Matrix for a full laboratory DN test program

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 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 10-10 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 readings.

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 AfCAP 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 10-10. 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.

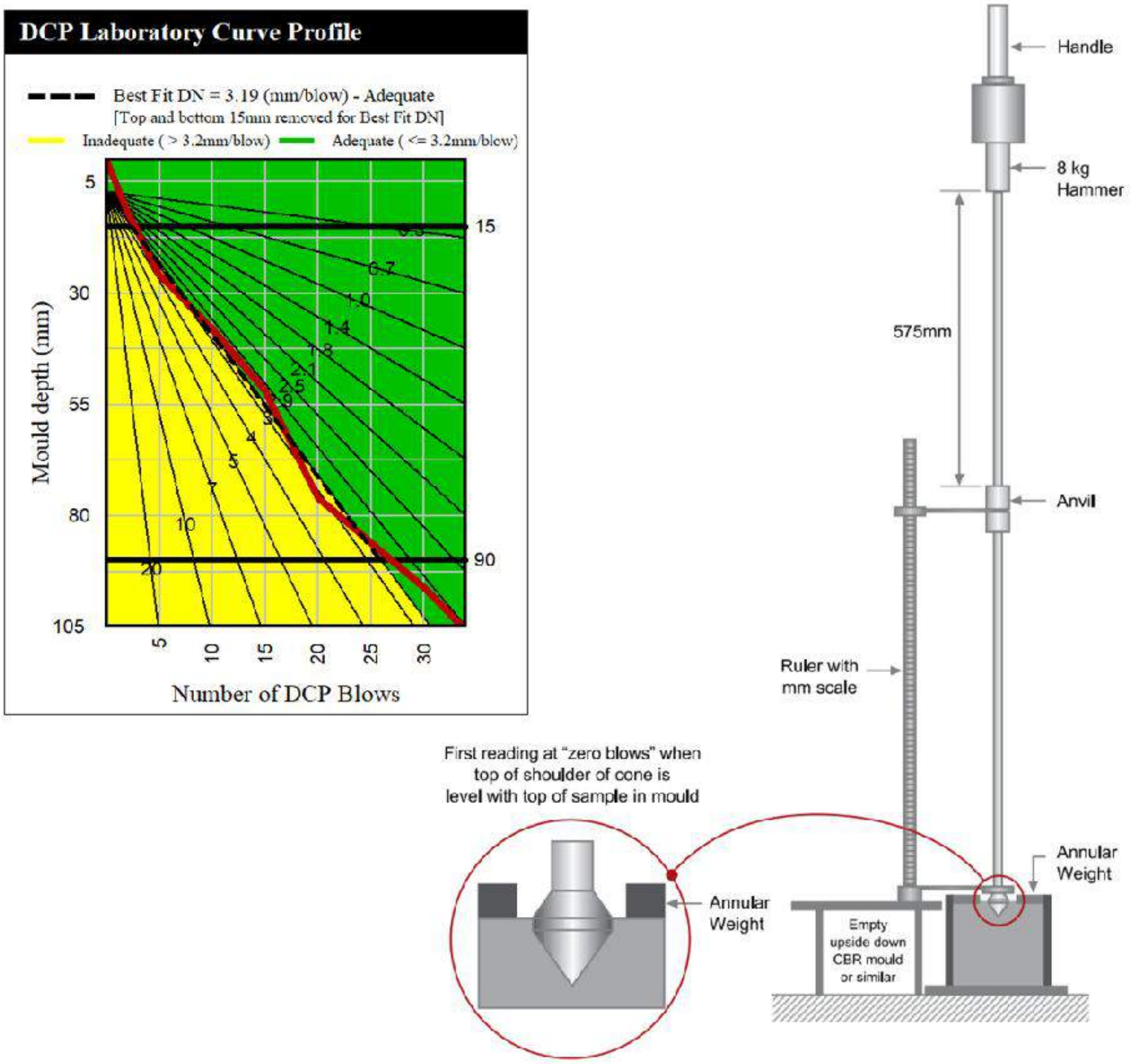


Figure A1-1: Set-up and typical output from the laboratory DN test

Note that the densities of each specimen for the same compactive effort and moisture content will never be exactly the same as illustrated in Table A1-2 and Figure 10-11. 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 densities of each specimen can be calculated with the required level of accuracy.

Table A1-2 shows a summary of a typical laboratory DN test, as described above. Plot the "best fit" DN values against the actual densities (average values of three specimens) in a diagram, as shown in Figure A1-2.

Table A1-2: Summary of typical laboratory DN test results

Compactive effort	DN mm/blow		
	Soaked	OMC	0.75 OMC
Light	11.20	6.40	3.60
Intermediate	6.90	4.50	2.90
Heavy	5.30	3.90	2.40

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

MDD 2340.000 g/cm^3				
Soaked	Relative compaction	92.5 %	95.5 %	98.9 %
	DN mm/blow	11.20	6.90	5.30
OMC	Relative compaction	93.7 %	95.8 %	99.5 %
	DN mm/blow	6.40	4.50	3.90
0.75 OMC	Relative compaction	93.1 %	96.0 %	99.2 %
	DN mm/blow	3.60	2.90	2.40

Figure A1-2 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.

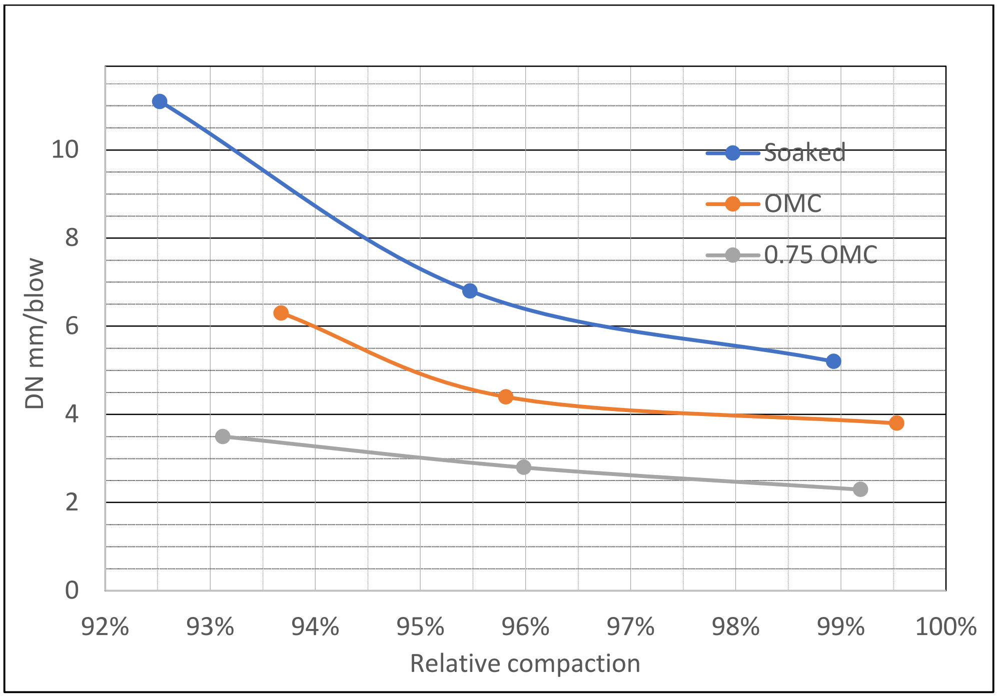


Figure A1-2: DN/density/moisture relationship

Figure A1-2 illustrates two critical factors that crucially affect the long-term performance of the road:

b) The need to specify the highest level of density practicable (so-called “compaction to refusal”) 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.

c) 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 8 – Drainage and Erosion Control.

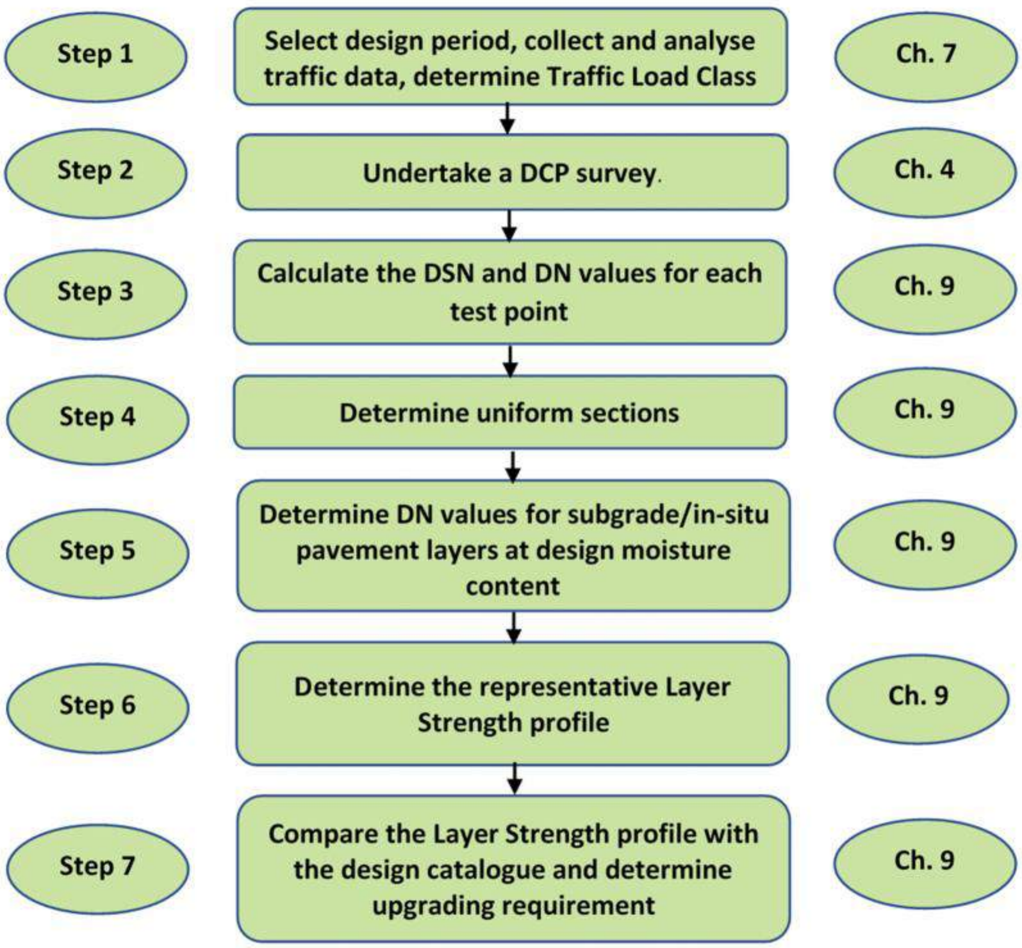
Appendix 2: Design Examples

Design example DCP-DN method

Project details

- A new paved road is to be built on the alignment of an existing gravel road
- Road length 8.6 km, sealed width 6.50 m
- Climatic area: Moderate (annual rainfall 500 – 1000 mm)

Design procedure



Steps 1: Design period & Traffic Load Class

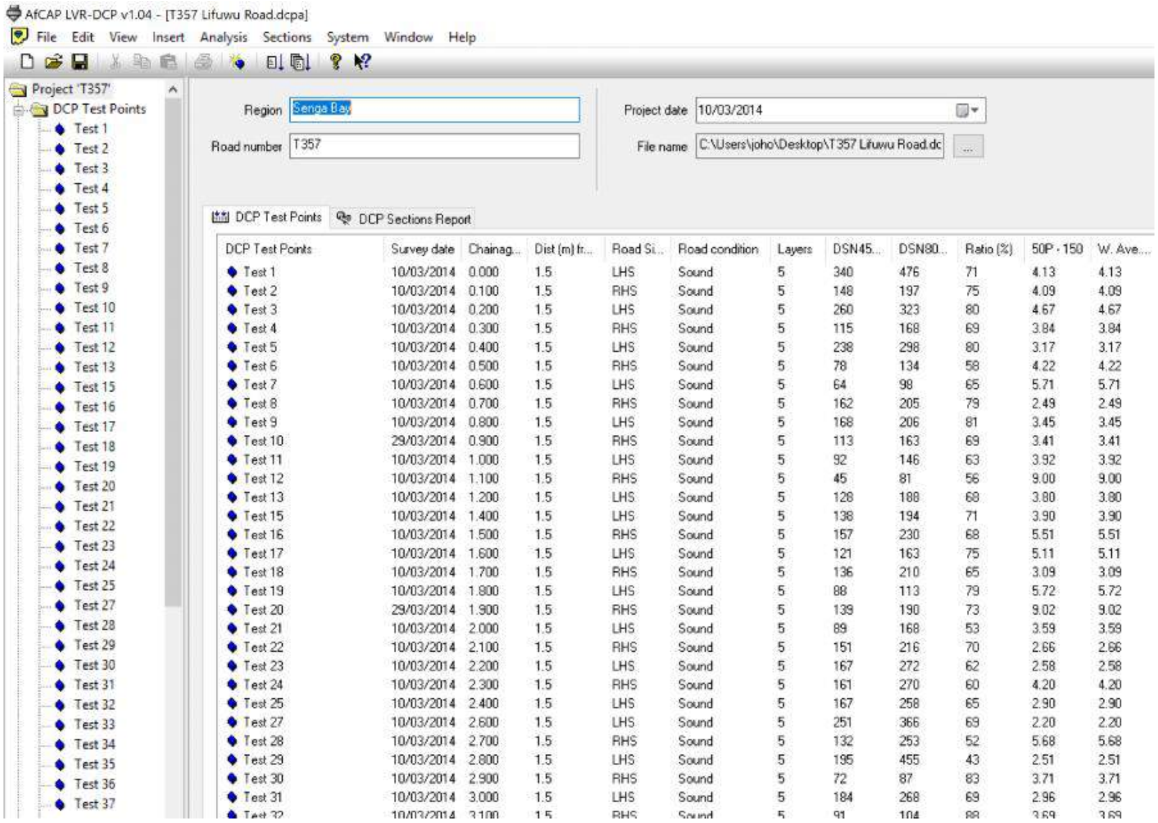
- Design period: 15 years
- Traffic class: TLC 0.3 (see *Chapter 7 - Traffic*)

Step 2: DCP survey

- A DCP survey was carried out in the intermediate season, i.e., between the wet and dry season, at 100 m intervals. In all, 87 DCP tests were carried out.

Step 3: Calculate DSN and DN values

The DSN and DN for all layers were automatically calculated by the AfCAP LVR DCP software with a typical output, as shown in Figure A2-1.


AfCAP LVR-DCP v1.04 - [T357 Lifuwu Road.dcpa]
File Edit View Insert Analysis Sections System Window Help
Project 'T357'
DCP Test Points
Test 1
Test 2
Test 3
Test 4
Test 5
Test 6
Test 7
Test 8
Test 9
Test 10
Test 11
Test 12
Test 13
Test 15
Test 16
Test 17
Test 18
Test 19
Test 20
Test 21
Test 22
Test 23
Test 24
Test 25
Test 27
Test 28
Test 29
Test 30
Test 31
Test 32
Test 33
Test 34
Test 35
Test 36
Test 37

Region: Senga Bay
Road number: T357
Project date: 10/03/2014
File name: C:\Users\joho\Desktop\T357 Lifuwu Road.dc

DCP Test Points | DCP Sections Report

DCP Test Points	Survey date	Chainag...	Dist (m) fr...	Road Si...	Road condition	Layers	DSN45...	DSN80...	Ratio (%)	50P - 150	W. Ave....
Test 1	10/03/2014	0.000	1.5	LHS	Sound	5	340	476	71	4.13	4.13
Test 2	10/03/2014	0.100	1.5	RHS	Sound	5	148	197	75	4.09	4.09
Test 3	10/03/2014	0.200	1.5	LHS	Sound	5	260	323	80	4.67	4.67
Test 4	10/03/2014	0.300	1.5	RHS	Sound	5	115	168	69	3.84	3.84
Test 5	10/03/2014	0.400	1.5	LHS	Sound	5	238	298	80	3.17	3.17
Test 6	10/03/2014	0.500	1.5	RHS	Sound	5	78	134	58	4.22	4.22
Test 7	10/03/2014	0.600	1.5	LHS	Sound	5	64	98	65	5.71	5.71
Test 8	10/03/2014	0.700	1.5	RHS	Sound	5	162	205	79	2.49	2.49
Test 9	10/03/2014	0.800	1.5	LHS	Sound	5	168	206	81	3.45	3.45
Test 10	29/03/2014	0.900	1.5	RHS	Sound	5	113	163	69	3.41	3.41
Test 11	10/03/2014	1.000	1.5	LHS	Sound	5	92	146	63	3.92	3.92
Test 12	10/03/2014	1.100	1.5	RHS	Sound	5	45	81	56	9.00	9.00
Test 13	10/03/2014	1.200	1.5	LHS	Sound	5	128	188	68	3.80	3.80
Test 15	10/03/2014	1.400	1.5	LHS	Sound	5	138	194	71	3.90	3.90
Test 16	10/03/2014	1.500	1.5	RHS	Sound	5	157	230	68	5.51	5.51
Test 17	10/03/2014	1.600	1.5	LHS	Sound	5	121	163	75	5.11	5.11
Test 18	10/03/2014	1.700	1.5	RHS	Sound	5	136	210	65	3.09	3.09
Test 19	10/03/2014	1.800	1.5	LHS	Sound	5	88	113	79	5.72	5.72
Test 20	29/03/2014	1.900	1.5	RHS	Sound	5	139	190	73	9.02	9.02
Test 21	10/03/2014	2.000	1.5	LHS	Sound	5	89	168	53	3.59	3.59
Test 22	10/03/2014	2.100	1.5	RHS	Sound	5	151	216	70	2.66	2.66
Test 23	10/03/2014	2.200	1.5	LHS	Sound	5	167	272	62	2.58	2.58
Test 24	10/03/2014	2.300	1.5	RHS	Sound	5	161	270	60	4.20	4.20
Test 25	10/03/2014	2.400	1.5	LHS	Sound	5	167	258	65	2.90	2.90
Test 27	10/03/2014	2.600	1.5	LHS	Sound	5	251	366	69	2.20	2.20
Test 28	10/03/2014	2.700	1.5	RHS	Sound	5	132	253	52	5.68	5.68
Test 29	10/03/2014	2.800	1.5	LHS	Sound	5	195	455	43	2.51	2.51
Test 30	10/03/2014	2.900	1.5	RHS	Sound	5	72	87	83	3.71	3.71
Test 31	10/03/2014	3.000	1.5	LHS	Sound	5	184	268	69	2.96	2.96
Test 32	10/03/2014	3.100	1.5	RHS	Sound	5	91	104	88	3.69	3.69

Figure A2-1: Output from calculation of all DSNs and DNs

Step 4: Determine uniform sections

- The output from the calculations was exported to Excel as a basis for the determination of uniform sections by a CUSUM analysis.
- Three uniform sections were identified as shown in Figure A2-2:
  - o Section 1: km 0+000 to 2+000
  - o Section 2: km 2+000 to 7+000
  - o Section 3: km 7+000 to 8+600

Figure A2-2: Determination of uniform sections

Step 5: Determine the DN values at the anticipated field density and the design moisture content (long-term equilibrium moisture content (EMC)) from Laboratory DN tests on three bulk samples from each uniform section.

- From the bulk samples collected within the uniform sections, it was determined that the in-situ moisture condition of the pavement was approximately at the anticipated long-term in-service moisture condition, i.e., at approximately OMC. Moisture adjustments of the DN values were therefore not required.

Step 6: Determine representative (average) Layer Strength profiles for the uniform sections

- After eliminating “outliers”, the weighted average DN for all layers per uniform section was determined using the AfCAP LVR DCP program. The representative of the Layer Strength Diagrams for the section, as shown in Figure A2-3.

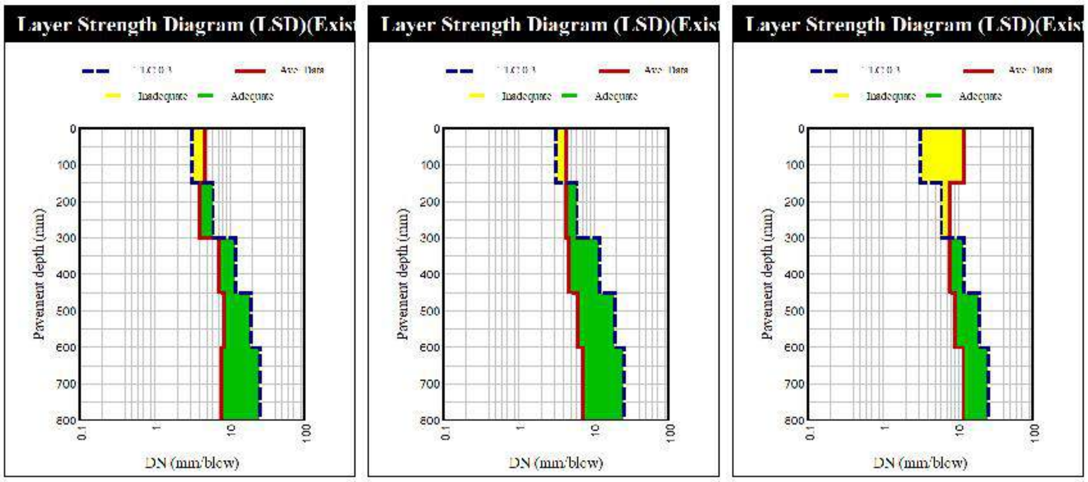


Figure A2-3: Representative LSD for the uniform section 1, 2 and 3 respectively

- The yellow color indicates layers that are too weak compared to the requirements of the design catalogue for TLC 0.3. In sections 1 and 2, the top 150 mm layer is too weak, in section 3, the two top 150 mm layers are too weak.

Step 7: Compare the Layer Strength profile with the DCP-DN design catalogue and determine upgrading requirements.

- For the design, it is easier to show the LSDs in a tabular format, as shown in Table A2-1 below.
- By comparing the in-situ DN for each layer directly with the DN requirement for the TLC, the weak layers are identified and indicated with pink color.

Table A2-1: LSD for uniform sections in tabular format before upgrading

Pavement Layer (mm)	Required DN value for TLC 0.3	Section no.		
		1	2	3
		0.000 to 2.000 km	2.000 to 7.000 km	7.000 to 8.600 km
0-150	<= 3.2	4.6	4.3	11
150-300	<= 6.6	4.0	4.3	7.7
300-450	<= 11	7.0	4.7	7.7
450-600	<= 17	8.5	6.1	9.2
600-800	<= 25	7.7	7.2	12

- The upgrading requirements are shown in Table A2-2. By importing a new base layer with DN ≤ 3.2 in all sections and, in addition, a new subbase layer with DN ≤ 6.0 in section 3, the position of the in-situ layers is shifted downwards in the pavement structure such that all pavement layers in the upgraded pavement satisfy the requirement of the design catalogue for TLC 0.3

Table A2-2: LSD for uniform sections in tabular format after upgrading

Pavement Layer (mm)	Required DN value for TLC 0.3	Section no.		
		1	2	3
		0.000 to 2.000 km	2.000 to 7.000 km	7.000 to 8.600 km
0-150	<= 3.2	≤3.2	≤3.2	≤3.2
150-300	<= 6.6	4.6	4.3	≤6.6
300-450	<= 11	4.0	4.3	11
450-600	<= 17	7.0	4.7	7.7
600-800	<= 25	8.5	6.1	7.7

Design example DCP-SN method

Project details

- The project, and thus the DCP data set, is identical to the one used for the DCP-DN design example.

Design procedure

Step	Activity	Chapter
Step 1	Select design period, collect and analyse traffic data, determine Traffic Load Class	Ch. 6
Step 2	Undertake a DCP survey. Collect bulk samples for moisture and strength measurements in the laboratory.	Ch. 4
Step 3	For each test point determine the layer strength diagram (DN in mm/blow and CBR vs. depth) and the layer boundaries	Ch. 4 & 9
Step 4	Determine the in-situ SN values for each layer and the total SN (and SNC) for the pavement for each test point	Ch. 9
Step 5	Convert the CBR (and hence the SN/SNC) values from in-situ to soaked conditions	Ch. 9
Step 6	Determine the required SN/SNC value from the catalogue appropriate to the climate, traffic and subgrade strength	Ch. 9
Step 7	Compare the soaked SN/SNC with the required SN/SNC to determine the structural deficiency at each test point	Ch. 9
Step 8	Identify areas where the structural deficiency is large for separate investigations	Ch. 9
Step 9	Identify uniform sections based on the structural deficiencies at each test point using the CUSUM method	Ch. 9
Step 10	Select the appropriate percentile of the structural deficiencies for each uniform section and determine the upgrading requirements.	Ch. 5 & 9

Steps 1: Design period & Traffic Load Class

- Design period: 15 years
- Traffic class: TLC 0.3 (see Chapter 6)

Step 2: DCP survey and collection of bulk samples

- A DCP survey was carried out in the intermediate season, i.e. between the wet and dry season, at 100 m intervals. In all 87 DCP tests were carried out.
- Bulk samples were collected from the alignment for determination of subgrade properties (moisture and strength). See *Chapter 4 – Site Investigations.*

Step 3: Determine Layer Strength Diagram and Layer boundaries

- The DCP data was analysed with the UK DCP programme. A typical layer strength diagram for a DCP test point with DN and CBR vs depth is shown in Figure A2-4.

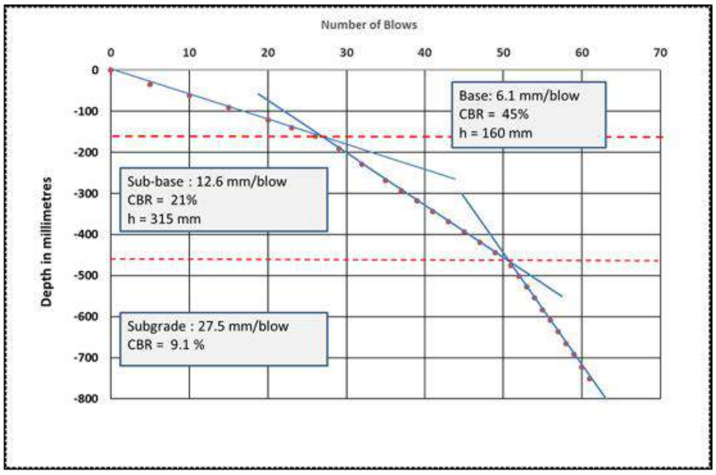


Figure A2-4: Typical DCP test result

Step 4: Determine the in-situ SN values for each layer and the total SN (and SNC) for the pavement for each test point.

- In the UK DCP program, the user must define each layer as roadbase, sub-base, or subgrade. The program then calculates the contribution of each layer to the overall structural number. The strength coefficients, as shown in Table A2-3, are calculated automatically.

Table A2-3: Example of CBRs, strength coefficients and SNs at a DCP test point

Layer No		CBR %	Thickness (mm)	Depth (mm)	Position	Strength Coefficient	SN
1		45	160	160	Roadbase	0.10	0.63
2		21	315	475	Sub-base	0.09	1.12
		9.1	-		Subgrade		
Total			475				1.75

Step 5: Adjust the in-situ CBR and SN/SNC values to soaked values

- The SN values in Table A2-3 are the values obtained at the in-situ conditions. For evaluation and design purposes, the SN of each layer in the soaked state is required. The user must estimate the soaked CBR values from the in-situ moisture contents measured in laboratory tests of the samples. This is done using Table 9-11. This conversion cannot be exact because the relationships shown in the figure depend on various material properties such as PI, hence a high level of precision is not possible. In this example, the in-situ conditions are not extreme (in terms of wet or dry). An average in situ moisture content of OMC was obtained and used with Table 9-11 to convert the CBRs to soaked conditions.

- The strength of the sub-grade must also be adjusted to give an estimate of the soaked value. However, it is only necessary to identify the subgrade class. For low values of CBR, if the in-situ moisture regime is at OMC, the soaked value is typically half to one-third of the in-situ value. If the moisture regime is dry (0.75*OMC), then the soaked value is one-third to one-quarter of the in-situ value.
- Using the revised CBR values, the revised SN is calculated for each layer and then summed to give the total value as shown in Table A2-4.

Table A2-4: Example of revised CBRs and SNs corrected for moisture at a DCP test point

Layer No	CBR (%)	Thickness (mm)	Position	Revised CBR (%)	Revised strength coefficient	Revised SN
1	45	160	Roadbase	20	0.09[1]	0.57
2	21	315	Subbase	5	0.03	0.37
	9.1	-	Subgrade			
Total		475				0.94

Note 1 If strengthening is required this layer will become sub-base

Step 6: Determine the required SN or SNC for the pavement

Having determined the subgrade strength and the existing SN for each of the DCP test points, design Chart 2 (Table 9-13 or Table 9-15) is used to determine the SN or SNC required for the new road to carry the design traffic on the design subgrade.

Step 7: Determine the structural deficiency at each test point

- The difference between the required SN and the existing SN (ΔSN) is the key parameter on which the upgrading design is based. The calculations are carried out for every test point remembering that the subgrade may not be the same for all the DCP test points.
- The choice of using SN or SNC depends on the variability of the subgrade. SNC should be used if the subgrade is variable. Calculations can be done using both SN and SNC and the more conservative results used in the design.

Step 8: Identify weak areas

- Areas with large structural deficiencies were identified for further investigations to identify the cause and possible solutions to the problem.

Step 9: Identify uniform sections

- A CUSUM analysis of the ΔSNP for all test points for the T357 Road (same as for the DCP-DN design example) gives the following diagram:

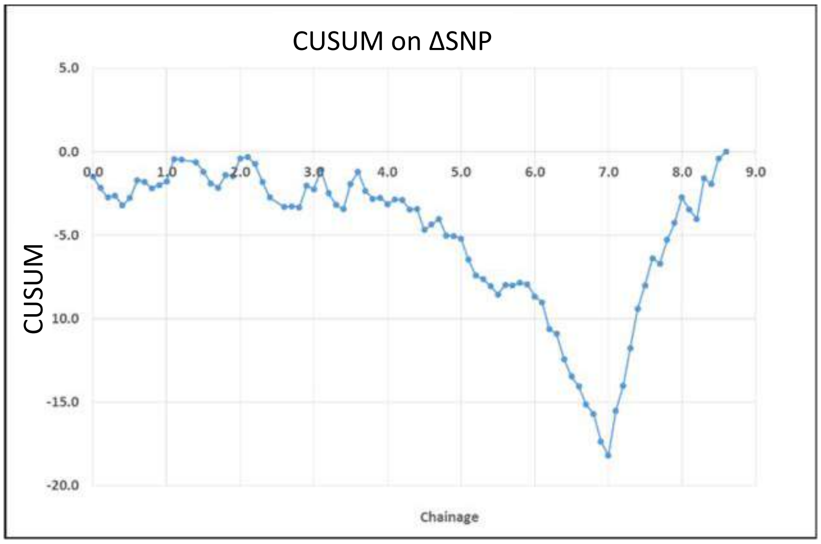


Figure A2-5: CUSUM analysis of SNP

- Changes in the slope of the trend line identify relatively homogenous or uniform sections. There are 3 sections that are distinct, namely:
  - Section 1: Chainage 0.000 to 3.600 where the pavement is generally thick but weak and not very uniform (little variation in fluctuating CUSUM).
  - Section 2: Chainage 3.700 to 7.000 where the road is stronger and more uniform (steeply decreasing CUSUM).
  - Section 3: Chainage 7.100 to 8.600 where the road is much weaker (increasing CUSUM).

Discussion

- **Section 1**

  In general, the main problem in Section 1 is that out of 35 test points there are 21 with distinct roadbase problems, identified from a visual inspection of the individual DCP test results and/or the tables summarising the properties of each individual test that are computed automatically in the program. The problems are of one or more of three types:

  1) Loose upper layer.
  2) Weak middle layer.
  3) Overall weakness in the road base.

  A total of 14 of the test points require no treatment at all except surface smoothing and compacting but these points are distributed fairly evenly along the section with never more than two being adjacent to each other. It is not feasible to change the treatment at short intervals, even with labour-based construction, because it is probably impossible to identify the boundaries. The variations along the road are frequent and probably largely random hence the sections of similar characteristics may be very short.

  The treatment for Type 1 is simply to test the top (roadbase) layer to see if it can be made strong enough by some sort of processing, for example, merely compaction or possibly blending.

  The treatment of Type 2 is not so straightforward because it could be an internal drainage problem, but the weak layer is normally underneath a strong layer and is therefore quite deep. The true nature of the problem needs to be determined before any major processing is considered but it may simply require an additional surface layer to “push” the weak layer to a lower (sub-base) level.

The treatment of Type 3 is the same as for Type 1 but the chances of getting a strong enough layer by blending and/or compaction are probably less.

In Section 1 there are just 6 DCP results that indicate that additional material may be needed simply because the existing SN is too low (in contrast to much of the Section where the existing material is deficient in quality but where the overall SN is high because of thickness). These are at chainages 0.6, 1.1, 2.0, 2.9, 3.1 and 3.5 and correspond to the lowest subgrade strengths. The CUSUM graph shows that these are identified by the large jump in the y-coordinate that occurs between the chainages mentioned and the preceding chainage. They appear to be isolated points so they may be associated with local drainage issues. Five of them are also associated with weak bases so could possibly be corrected by reprocessing the existing material. The 6th (at chainage 2.9) may require an additional layer of base material (100mm) because the existing base is only 100mm thick and the SN is marginal. A subsidiary investigation is needed to determine the extent of these six problems.

Figure shows the thicknesses required at each test point. The chainages with no vertical bar do not need additional strengthening but, based on practical issues, the entire section, after the six weak sections have been re-examined, will require the same treatment.

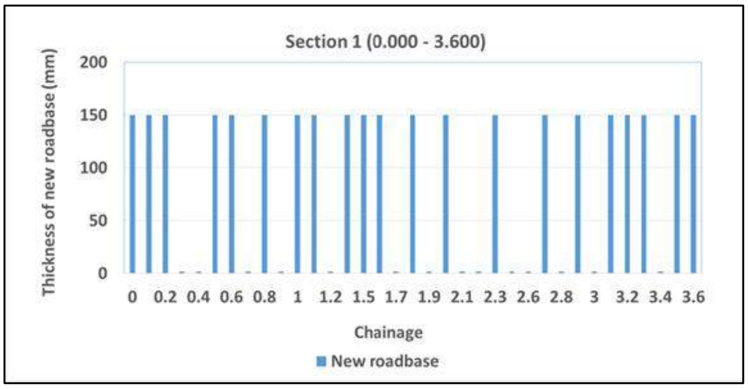


Figure A2-6: Section 1: Potential requirements for additional base

Section 2

This section is much more uniform than Section 1, but 18 of the 34 test points show the same roadbase problems, as shown in Figure . There are 16 chainages that require no treatment but some of them (but not all) tend to occur slightly more often adjacent to one another in contrast to those in Section 1. For example, chainages 4.0 to 4.2; 4.5 to 4.7; 6.8 to 7.0 but these sub-sections are also probably too short to warrant different treatment to that of the rest of the Section.

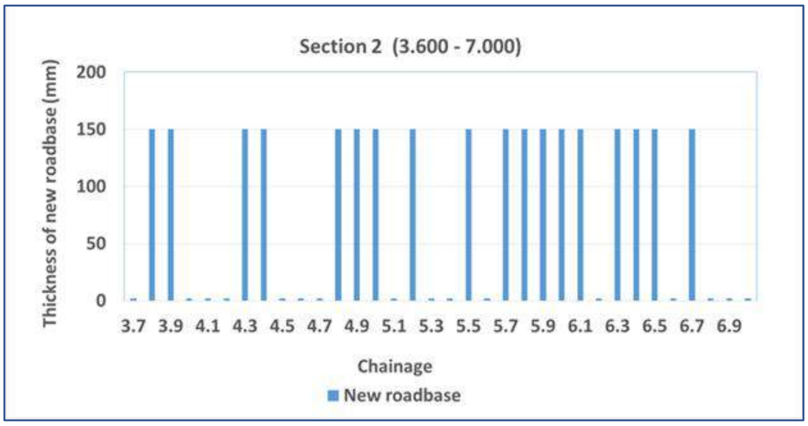


Figure A2-7: Section 2: Potential requirements for additional base

- **Section 3**

  The whole of Section 3 (16 DCP test points) also shows base deficiencies including a complete absence of any base or subbase at many of the test points, as illustrated in Figure . The subgrade is quite strong hence a relatively thin pavement is required. Most of the Section is founded on an S4 subgrade or stronger and, except for three chainages (7.1, 7.4 and 7.8) already has a sufficiently high SN. The reason why additional material is needed is that the uppermost layer (be it base, subbase or simply the top of a strong subgrade) are too weak for their position in the pavement. An additional 150 mm layer of suitable material on all the test points is required for a TLC 0.3 mesas design and, on the 8 test points with no sub-base, an addition sub-base layer is required. However, it is not usually feasible to change designs frequently hence the whole section requires two additional layers of pavement. The weakest test point is at chainage 7.1 and corresponds to the weakest subgrade and could therefore also be a drainage problem that needs investigating separately before a design for this area can be chosen.

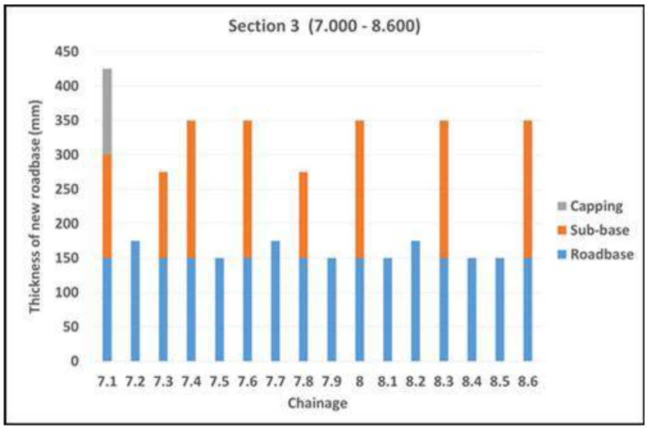


Figure A2-8: Section 3 Potential requirements for additional base and subbase

Step 10: Design for each uniform section

- For each uniform section there is a range of values of ΔSNs. The next step is to choose the appropriate percentile of those ranges for design. The percentile depends on the reliability required; the recommended values are:
  - Median for TLC 0.01 and TLC 0.1
  - Upper 75th percentile for TLC 0.3
  - Upper 90th percentile for TLC 0.5 and TLC 1.0
- Thus, for TLC 0.3 a 75th percentile is recommended but for this example there are only two different designs for two sections and only three for Section 3 hence the choice of percentile requires no calculation. A summary is shown in Table A2-5.

Table A2-5: Results of the analysis for road class TLC 0.3

Section	1	2	3
Chainage	0.000 – 3.600	3.600 – 7.000	7.000 – 8.600
Material to be added	150 mm of CBR 65%$^{(1)}$	150 mm of CBR 65%$^{(1)}$	150mm of CBR 65% and 200mm of CBR 30%

Note 1: Additional base is needed only if the existing base cannot be brought up to specified characteristics by other means, e.g. blending.

Note 2: Chainages with possible drainage problems (see text) require further investigation.

Design example DCP-CBR method

Project details

- An un-improved earth track, 5.6 km long, shall be upgraded to paved standard.
- The sealed width of the upgraded road will be 6.50 m.
- Climatic area: Moderate (annual rainfall 500 – 1000 mm)

Design procedure

Step 1	Select design period, collect and analyse traffic data, determine Traffic Load Class	Ch. 6
Step 2	Undertake a DCP survey and for each point determine the layer strength diagram (DN in mm/blow and CBR vs. depth)	Ch. 4
Step 3	Determine uniform sections based on a CUSUM analysis of the SN for each test point	Ch. 9
Step 4	Collect representative centre line samples for in-situ moisture measurements	Ch. 4
Step 5	Select the appropriate percentile for design and calculate the in-situ subgrade CBR value for each uniform section	Ch. 9
Step 6	Determine the design subgrade class by converting the in-situ CBR to soaked CBR	Ch. 9
Step 7	Determine the required pavement structure appropriate to the climate, cross-section, traffic and subgrade strength	Ch. 9

Step 1: Design period and Traffic Load Class.

- Design period 15 years
- The traffic loading has been estimated to 86,000 ESA (TLC 0.1)

Step 2: DCP survey and collection of bulk samples from the alignment

- A DCP survey was carried out at 55 test point at 100 m intervals.
- Bulk samples were collected from the alignment for determination of subgrade properties (moisture and strength). See *Chapter 4 – Site Investigations.*

Step 3: Determine the Layer Strength Diagram and layer boundaries

- The DCP data was analysed with the UK DCP programme to determine the Layer Strength profile and layer boundaries as illustrated in Figure A2-9.

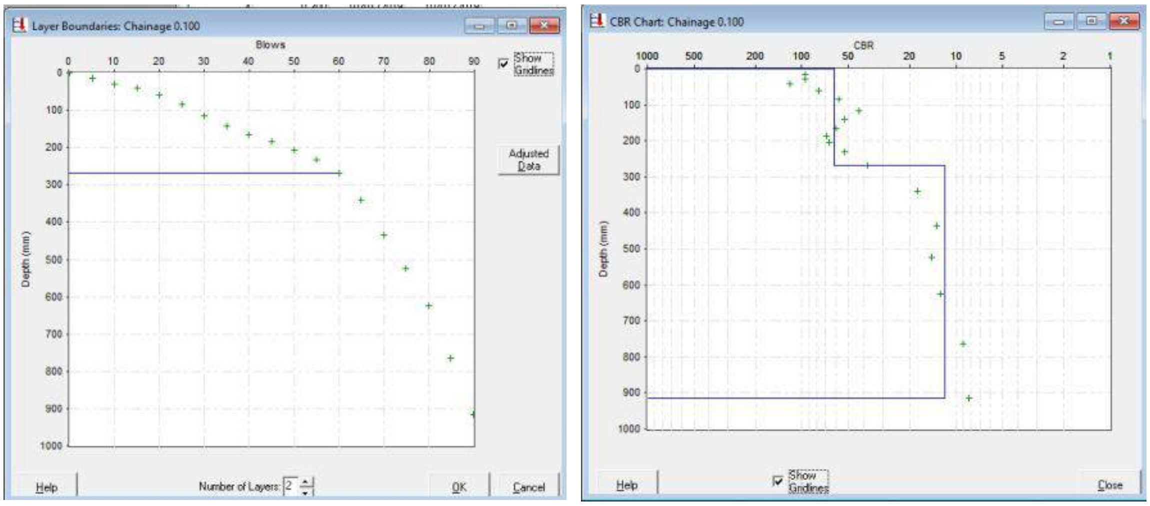


Figure A2-9: Typical Layer Strength Diagram and CBR Chart

Step 4: Determine uniform sections

- Eight uniform sections were determined on the basis of a CUSUM analysis of the Modified Structural Number (SNP) as shown below.

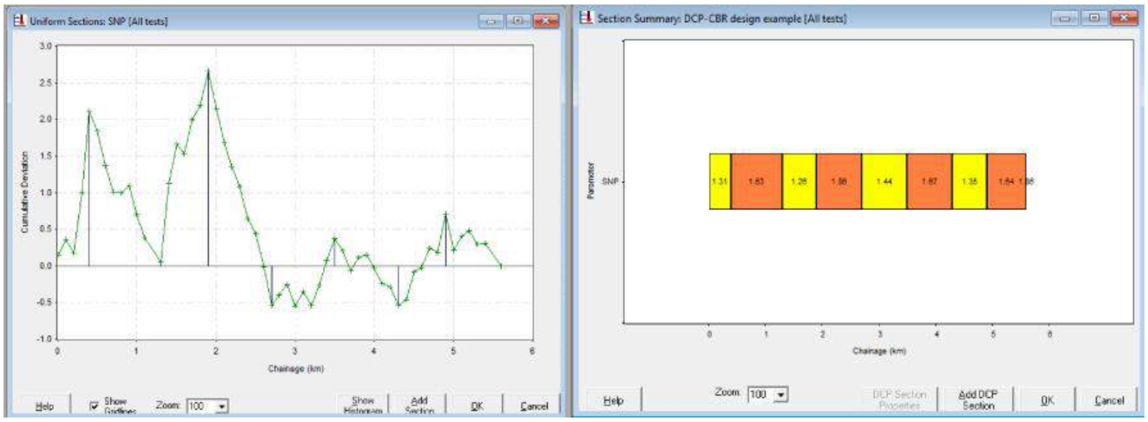


Figure A2-10: Uniform sections determined on the basis of SNP

Step 5: Select appropriate percentile and determine in-situ subgrade CBR

- The programme calculated the properties for each uniform section. For TLC 0.1 the Mean in-situ CBR was used in accordance with the criteria below:
    - Median for TLC 0.01 and TLC 0.1
    - Lower 25th percentile for TLC 0.3
    - Lower 10th percentile for TLC 0.5 and TLC 1.0

Step 6: Determine the design subgrade class

- The design subgrade class was determined by converting the Mean in-situ CBR to Soaked CBR as illustrated in Table .
- While bulk samples can be taken in with the DCP survey (Step 2), more samples may have to be taken now that the uniform sections have been identified.

- The in-situ moisture content was determined at approximately 0.75 OMC and Soaked CBR values were found to correspond well to the values found from the analysis below.

Step 7: Determine the required pavement structure

- The pavement design for each uniform section was then selected from the appropriate Design Chart as shown in Table A2-6.

Table A2-6: Pavement design of uniform sections

Pavement Design - Moderate area - Road width 6.50 m - Design Chart 2 (Table 9-20)								
Section no	1	2	3	4	5	6	7	8
Start km	0+010	0+400	1+300	1+900	2+700	3+500	4+300	4+900
End km	0+400	1+300	1+900	2+700	3+500	4+300	4+900	5+600
In-situ CBR %	11	28	10	35	19	20	13	21
Soaked CBR %	4	9	3	10	6	7	4	7
Subgrade Class	S2 (3-4%)	S4 (8-14%)	S2 (3-4%)	S4 (8-14%)	S3 (5-7%)	S3 (5-7%)	S2 (3-4%)	S3 (5-7%)
Base	150 G65	150 G55	150 G65	150 G55	150 G55	150 G55	150 G65	150 G55
Subbase	125 G30	100 G30	125 G30	125 G30	175 G30	175 G30	125 G30	175 G30
Selected	100 G15		100 G15				100 G15	

- From a practical point of view, it is not desirable to vary the design too frequently. Depending on the local situation with regard to material availability, topography, drainage conditions etc., a decision must therefore be made on the most rational and cost-effective way of upgrading this road.

Low Volume Roads Manual
Volume 1 – Pavement Design

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- **Section 2: Investigations**
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  - 6. Traffic
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  - 8. Drainage and Erosion Control
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Contents

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10.1 Introduction

10.1.1 Background

More than 90 % of the road network in Malawi 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 LVSR standard, a large network of important unpaved earth and gravel roads will remain for the foreseeable future. It is thus necessary 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, cobble stones or similar.

In their simplest forms, unpaved roads consist of tracks or earth roads over which goods or persons are moved directly on the in-situ material surface. This may, in some cases, be ripped, shaped and compacted (engineered) but generally, the only compaction is that applied by vehicles moving over it (un-engineered).

There comes a point with these "roads" when passability is excessively affected by the weather and vehicles can no longer traverse the road during inclement weather. This problem is best solved by applying a selected material with specific properties over the in-situ material to ensure all-weather passability and the roads then become "gravel" roads. Despite this, the roads may occasionally become impassable as a result of flooding of parts of the road, in which case vehicles cannot pass because of deep water and not necessarily for any reason attributed to the road surface.

Unpaved roads will typically carry a maximum of about 200 vehicles to 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.

A schematic hierarchy of low volume roads is shown in Figure 10-1.

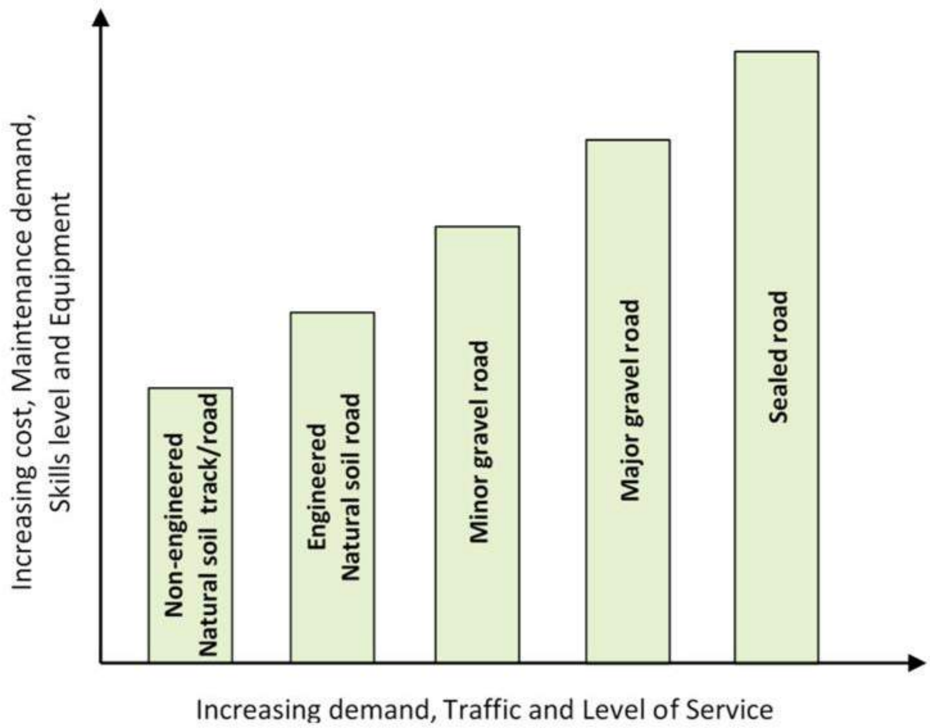


Figure 10-1: Schematic hierarchy of low volume roads

10.1.2 Purpose and Scope

The purpose of this chapter is to provide a framework for the design of unpaved roads in an economical and sustainable manner such that the appropriate levels of quality are produced.

The chapter covers the design of all levels of unpaved roads from earth roads making use of the in-situ soil to engineered and treated gravel roads. Material selection and thickness design are treated in detail.

10.2 Earth Roads

10.2.1 General

Earth roads may comprise non-engineered roads on which traffic travels directly on the in- situ material or engineered roads on which some attempt is made to:

- improve the shape of the road in terms of a typical cross section;
- introduce side-drains; and
- usually, apply some compaction to the material forming the road.

The wearing course material is generally obtained from excavation of the side-drains.

10.2.2 Non-engineered roads

Non-engineered roads usually start as one or two tracks in which the grass and surface vegetation is worn away to expose the in-situ material, or in some cases, the vegetation may be intentionally removed. With time and traffic, these tend to wear down and depressions develop in the natural ground surface. These become areas that collect precipitation or surface run-off and form conduits moving the water, which leads to softening of the material, erosion and ultimately deepening of the channels. At this stage, the tracks no longer afford viable routes for traffic and new tracks are formed adjacent to the existing ones, ultimately resulting in a wide “canal”.

The life and effectiveness of earth roads depend on the nature of the in-situ material. Often the upper part has humus and clay, which results in some sort of binding of the material that adds to the effect of any roots. Once this upper layer wears away, the track will usually deteriorate rapidly.

In some instances, the in-situ material may have properties equivalent to those required for conventional wearing course gravels, in which case they may perform reasonably well for a limited period. It should be remembered that these materials are generally not compacted and rely solely on traffic compaction to increase their density, which is accompanied by settlement and some material loss.

Only once the earth road starts deteriorating in riding quality, it may be graded and given some shape, but the overall structure is usually below natural ground level and the associated drainage problems are not addressed. At this stage, the road needs to be improved.

10.2.3 Engineered earth roads

Engineered or improved earth roads differ from the un-engineered earth roads described above in that the shape of the road structure is improved. The materials used are the same as the earth road, i.e. the in-situ or local material, but additional material that is excavated from the side of the road to form side drains (at least 150 mm below natural ground level) and this material is added to the road to increase its height and provide a better-drained road structure, as shown in Figure 10-2. The material must be shaped to assist with water runoff and compacted to improve its strength, decrease its permeability and reduce maintenance requirements.

The following principles apply for good design:

- The crown height of the earth road should be at least 500 mm above the drain invert.
- Where the topography allows, and the drain material is suitable, wide, shallow longitudinal drains are preferred. They minimise erosion and will not block as easily as narrow V-drains, which should be avoided. Where the space permits, the side drains should be 1.0 m wide at invert level with side slopes of 1:3 (V:H). Drains grass over in time, binding the soil surface and further slowing down the speed of water, both of which act to prevent or reduce erosion.

- The surface of earth roads should be graded and compacted to provide a durable and level running surface for traffic and, depending on the geometry and width of the road, the surface should have a minimum camber of 4% - 6% to ensure that water runs off the surface and into the side drains.
- Areas where there are specific problems (usually due to water or to the poor condition of the subgrade) may be treated in isolation by localised replacement of subgrade, gravelling, installation of culverts, raising the roadway, or by installing other drainage measures. This is the basis of a "spot improvement" approach.
- Water should be drained away from the carriageway side drains by excavating mitre drains to divert the flow into open space. The spacing of the mitre drains should be as indicated in *Chapter 8 – Drainage and Erosion Control: Table 8-6.* The width at drain invert should be 0.4 m – 0.6 m with a widened mouth at the end to spread the water and prevent scouring.

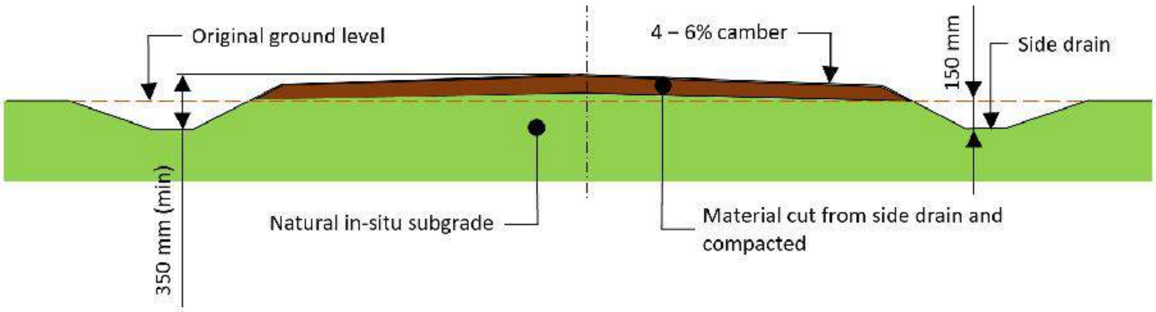


Figure 10-2: Cross section of typical improved earth road

The performance of earth roads is constrained by the quality of the in-situ materials, which in many cases, is inadequate to provide good all-weather surfaces that require minimal maintenance. Knowledge of the past performance of local materials may, however, allow the use of these even though they do not comply with the required properties for good wearing course gravels. In general, no specific material requirements are applicable to earth roads, but if the local materials comply with the requirements for gravel roads, good performance can be expected.

It is possible to estimate the likely performance of improved earth roads based on an assessment of the traffic carrying capacity of the soils under varying environmental conditions from a knowledge of the bearing capacity (CBR) of the soil, the equivalent single wheel load of the vehicles and the tyre pressures, as shown in Figure 10-3. If the strength of the earth road material is known (in terms of its in-situ CBR), the nomograph permits predictions of the expected number of vehicles that will cause a rut depth of 75 mm.

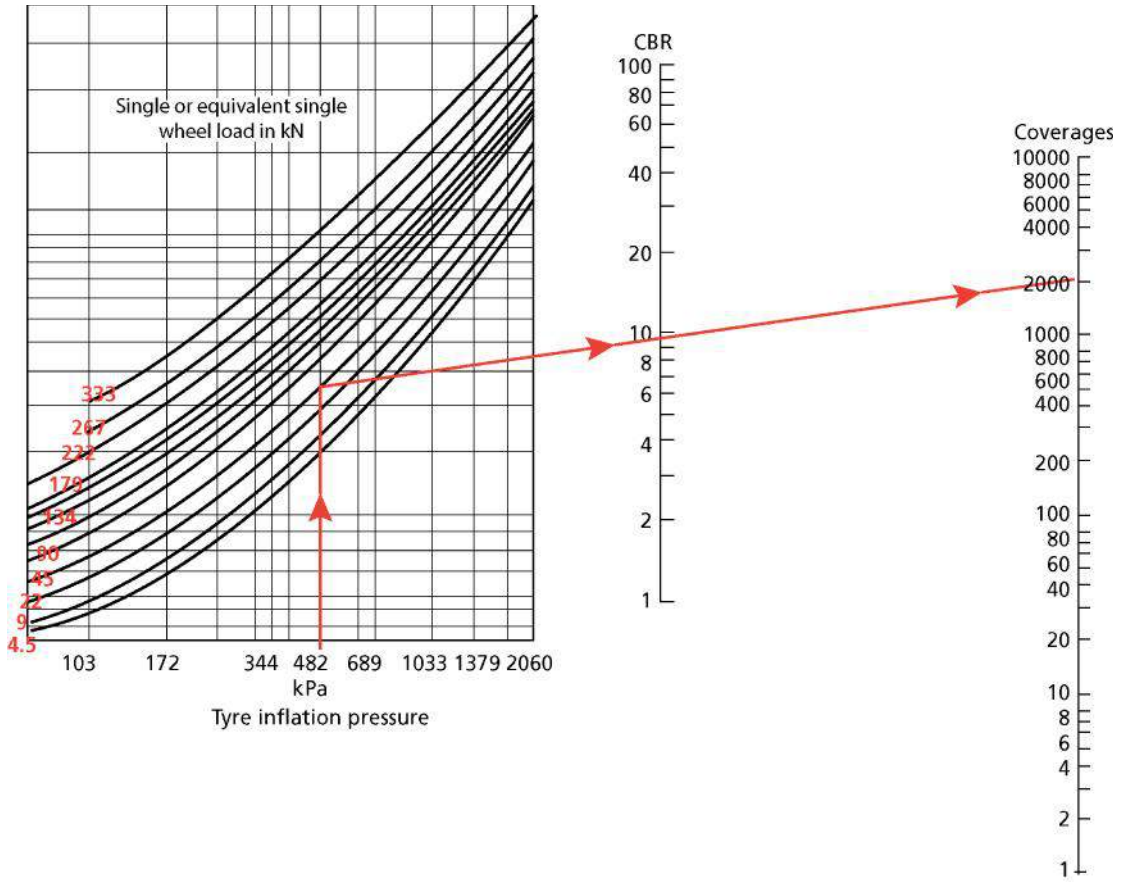


Figure 10-3: Carrying capacity of engineered earth roads

As illustrated in Figure 10-3, an engineered earth road with an in-situ CBR of 10% can be expected to provide approximately 2 000 coverages of vehicles with a single wheel load of 45 kN and a tyre inflation pressure of 482 kPa before serious deformation is likely to occur. Since the wheel loads will not be concentrated on exactly the same path but will wander slightly across the width of a road, one complete coverage is equivalent to the passage of 2.7 vehicles. Thus, 2 000 coverages are equivalent to 5 400 vehicles with the characteristics indicated above.

For a single lane road, the wheel loads will be restricted to narrower channels, and therefore the coverages will be different. For example, for a narrow single lane road, the number of vehicles that the earth road can accommodate before failure decreases to approximately 1350 vehicles. For a route carrying 50 vpd and assuming 15% of them are relatively heavy (4.54 tonne wheels), this translates into a need to maintain, re-grade or reshape the surface about every 4 months to 6 months. For soils with higher CBR, this will be longer. It is important for both designers and road managers to appreciate that engineered earth roads have a low initial cost but that they require an ongoing commitment to regularly reshape the surface to keep it in a serviceable condition.

Culverts should be installed along the line of water flow to the route where there is a need to transfer water from one side of the road to the other, for example, where the road crosses a watercourse. In flat areas, smaller diameter parallel culverts may be preferable to single large culverts, in order to ensure discharge is at ground level. However, culvert pipes smaller than 600 mm in diameter are not recommended as they are difficult to clean out of silt and debris. The inlet and outlet of the culvert must be protected against erosion.

At some point (usually dictated by the number of vehicles increasing to a certain level and depending on the material quality) the maintenance requirements for earth roads reach a stage when it becomes uneconomical or excessively difficult. At this stage, a decision must be made to construct a traditional gravel road in which materials from a selected borrow pit are used for the wearing course.

10.3 Gravel Roads

10.3.1 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. This, however, is not a function of the gravel road design and is addressed in *Chapter 7 - Hydrology and Drainage Structures*.

10.3.2 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.

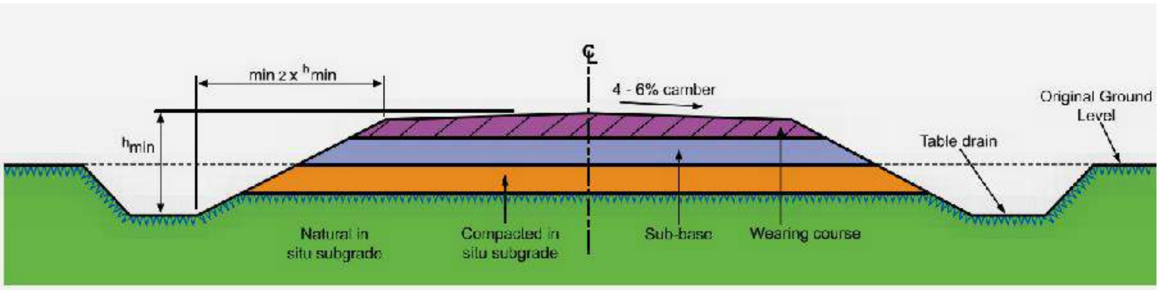


Figure 10-4: Typical gravel road cross section in flat terrain

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 10-1.

Table 10-1: Minimum height h_{min} of road crown above drain invert

Road Class	Climate factor (Weinert N value)	
	Wet (N<4)	Dry (N>4)
	h_{min} (mm)	
LVR5	550	450
LVR4	500	400
LVR3	450	350
LVR2	400	300
LVR1	350	250

10.3.3 DCP-DN Method

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 10-2 summarises the required properties of suitable wearing course gravels.

Table 10-2: 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

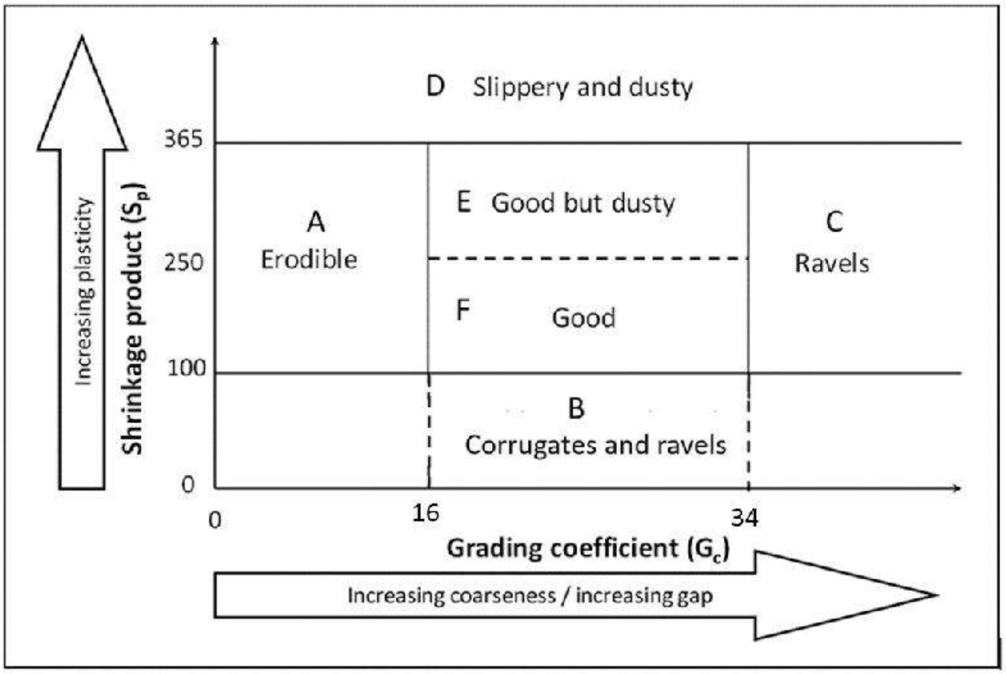
Source: Paige-Green P (1989)

Note: 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. No correlation currently exists with the 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 10-5, where:

Shrinkage Product (SP) = Linear Shrinkage$_{0.425}$ x $P_{0.425}$

Grading Coefficient (GC) = ($P_{26.5}$ - $P_{2.0}$) x $P_{4.75}$/100



Source: Department of Transport, South Africa (1990)

Figure 10-5: Chart showing performance of unpaved road materials

In the chart presented in Figure 10-5, 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 (13.5 mm/blow) which initially may appear to be 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. 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.

Figure 10-6 can be used to identify potential problems that could affect the road should the materials not fall into Zone E. 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 10-2. 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 10-7 overleaf shows examples of gravel wearing course performance and 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 10-6: Examples of gravel wearing course performance

Pavement Design

General: The mechanism of deterioration of the gravel wearing course 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 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 10-1) 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 them with an adequate thickness of stable material – heave and collapse are seldom significant problems on unpaved roads, as they are smoothed out during routine maintenance.
- Should re-graveling operations be delayed until the gravel has completely worn away (which is a regular occurrence in many countries), a “buffer” layer of reasonable quality material should be placed to avoid vehicles traveling 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 - 10 years after construction, selected materials should be used that comply with the requirements for lower layers in the paved road design standards.

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 9 – Structural Design: Paved Roads, Section 9.2.5*, 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: The general procedure for determining uniform sections is as follows:

- At least 5 DCP tests to 800 mm depth should be carried out per kilometer 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).
- Plot the data using the cumulative sum (CUSUM) technique. If the uniform sections delineated by the three parameters (DN_{150}, $DN_{150-300}$ and DSN_{800}) 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 9 – Structural Design Paved Roads, Section 9.2.5*, 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.

Pavement layer design

This makes use of the following procedure:

- Compare the relevant subgrade strength profiles with the necessary design given in Table 10-3, or the layer strength diagrams shown in Figure 10-7, 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 10-3, 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 only 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 10-3: Gravel road pavement design for different traffic categories (Soaked DN)

Layer, depth and DCP Structural Number	Traffic (Heavy vehicles/day)			
	≤ 2	2 – 6	7 – 20	21 - 60
	DN (mm/blow)			
150 mm Wearing course (Table 10-2)	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	14
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 HGV (Chapter 6)				

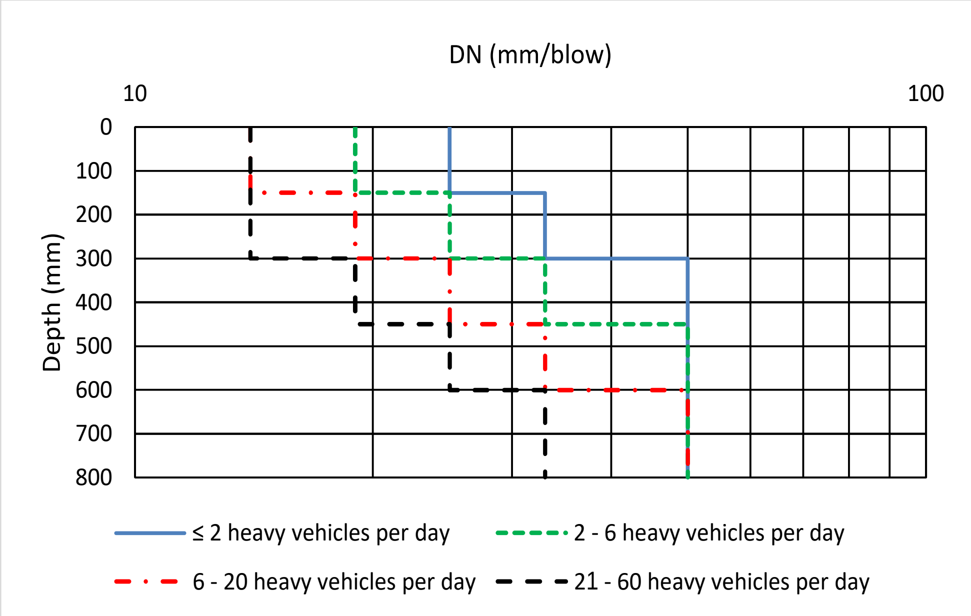


Figure 10-7: Layer strength diagrams for different traffic categories

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 10-2 and Figure 10-5, but if the potential for delayed maintenance (i.e. re-gravelling) exists, an additional 50 mm should be added as a buffer layer.

The relatively low strength of the support layers for the lower traffic roads have been shown to be sufficient to distribute the wheel loads. However, as the traffic increases, the support layer strengths approach the minimum strength for the wearing course material. 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 Zones E and F in Figure 10-5, 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 10-4.

Table 10-4: 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
Zone F	Good	15

The gravel losses shown in Table 10-4 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. Spot improvements should be considered on these sections.

Re-gravelling should take place before the underlying layer is exposed. The re-gravelling frequency, R, is typically in the range 5 years to 8 years.

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.

10.3.4 DCP-CBR Method

Materials

Material specifications for the DCP-CBR method are similar to, but slightly different from, those for the DCP-DN method.

The materials requirements for the gravel wearing course described in this section apply to both minor and major gravel roads but not to earth roads.

The main purpose of the gravel wearing course is to protect the structural layer. The wearing course should be designed in such a way as to enable an acceptably smooth ride (low roughness).

The material used for the wearing course should:

- Not be lost too soon (low gravel loss),
- Not be slippery,
- Not be greater than 40 mm maximum particle size.

The wearing course thickness depends on the annual gravel loss and the number of years between re-gravelling operations. Commonly, 150 mm is used at the construction stage and the layer is re-gravelled to 150 mm thickness during each operation. Table 10-5 and Figure 10-8 show the specifications and criteria for the selection of materials for use in the wearing course layer based on durability, rate of roughness progression, and ultimately whole-life costs (i.e. performance-based specifications).

Table 10-5: Recommended gravel wearing course specifications

Property	Specification
Maximum size (mm)	37.5
Oversize Index (% retained on 37.5 mm sieve)	< 5%
Plasticity Product	50 – 480 (min 280 preferred)
Grading Modulus	1.0 – 1.9
Soaked CBR at 95% BS Heavy	> 15%

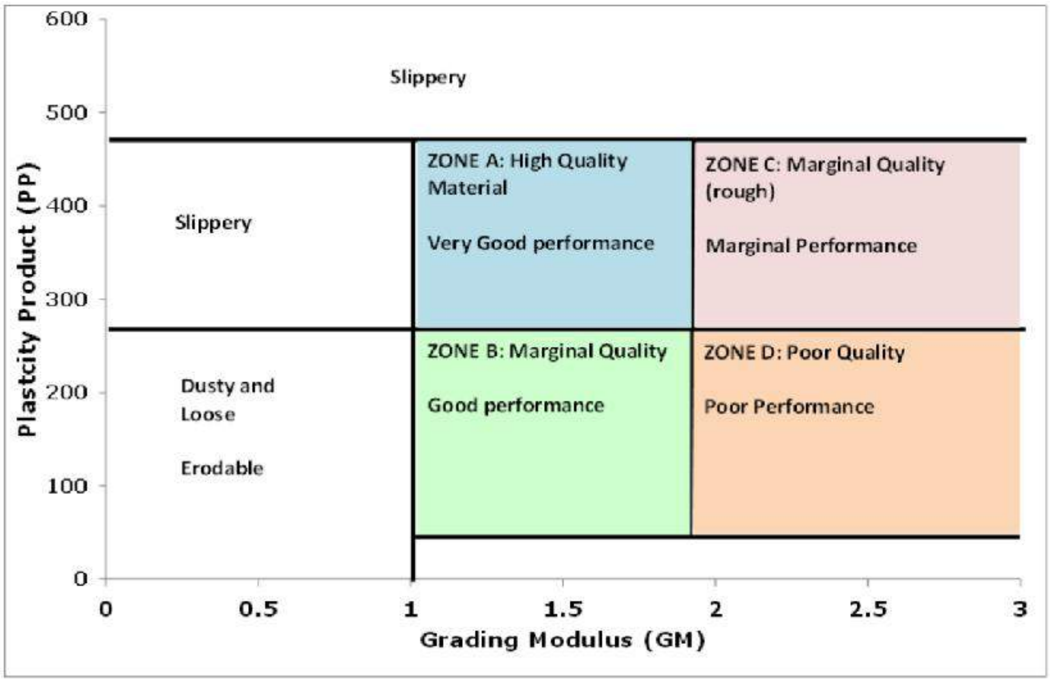


Figure 10-8: Selection chart for gravel wearing course material

where:

a) Ip 0.075 = the Plasticity Index of the material passing the 0.075 mm sieve

b) Plasticity Product (PP) = Ip 0.075 x P0.075 (Preferred range is 280-480)

c) Grading Modulus $GM = [300 - (P2 + P425 + P075)]/100$ (Preferred range is 1.0-1.9)

Where: $P_{2.36}$ = percentage passing the 2.36 mm sieve

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

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

The particle size distribution test for the material must be done using the wet sieving method.

Pavement Design

General: The basic principles and considerations for gravel road design remain the same as for the DCP-DN method. However, the DCP-CBR method distinguishes between minor (LVR1/2) and major gravel roads (LVR3/4/5), for which slightly different design criteria are applied.

Determination of subgrade strength: For both minor and major gravel roads, catalogue designs have been developed based on subgrade class (as for paved roads) and ADT (minor) or CESA (major).

A DCP survey as described for the DCP-DN method must, therefore, be carried out for the determination of uniform sections based on the Structural Number (SN) for each test point. The subgrade strength is then determined as described in *Chapter 9 – Structural Design Paved Road, Section 9.2.5*.

Minor Gravel Roads: Minor gravel roads (LVR 1 and LVR 2) are designed with two layers, a sub-base and a wearing course. If there is no choice, then these layers will be the same material.

- The design chart is based on the AADT (not CESA) of the road and assumes that the traffic includes not more than approximately 30% of commercial vehicles (small bus and larger)
- A nominal wearing course thickness of 150 mm of NG15 is assumed for both road classes and subgrade conditions with the sub-base thickness being influenced by the subgrade class;
- Drainage, but not necessarily geometry, is upgraded to acceptable minimum levels during construction. This can be achieved by building up the formation to an appropriate height to achieve the h_{min} requirements given in Table 10-1.
- The recommended sub-base thicknesses and wearing course material strengths for different subgrade and traffic conditions are shown in Table 10-6.

Table 10-6: Design chart for minor gravel roads

Subgrade Class (CBR %)	**LVR1/LVR2(1)**
S2 (3-4%)	Layer 1 – 150 mm WC Layer 2 – 200 mm NG15(2)
S3 and S4 (5-14%)	150 mm WC
S5 (15-29%)	Use in-situ material

Notes: (1) If more than 10 heavy vehicles/day, design as a major gravel road
(2) If NG30 material is available, the thickness can be reduced to 150 mm

Major Gravel Roads: For major gravel roads, the approach is as follows:

- The subgrade should be prepared in the same way as for a low volume sealed road.
- It is assumed that the wearing course will be replaced at intervals related to the expected annual gravel loss and before the structural layer is exposed to traffic and itself begins to wear away.
- The geometry and drainage are upgraded to acceptable minimum levels during construction. This may require the introduction of a fill layer between the compacted in situ subgrade and the wearing course.

Major gravel roads are likely to incur high maintenance costs in some circumstances, namely:

- When the quality of the gravel is poor.
- Where no sources of gravel are available within a reasonable haul distance.
- On road gradients higher than about 6%.
- In areas of high and intense rainfall.
- When the road shape has deteriorated as a result of previous inadequate or poor maintenance.

In these circumstances, spot improvements will almost certainly be justified, and, in some cases, it may prove to be more economical to build a fully paved road at the outset.

The design procedure consists of the following steps:

- Determine the traffic volume and traffic loading (*Chapter 6 - Traffic*).
- Determine the strength of the subgrade at the appropriate moisture condition.
- Establish the quality of the gravel that is to be used (Table 10-5). If only very poor gravel is available, blending with another gravel or soil to improve its properties may be an option.

- Determine the thickness of the gravel base that is necessary to avoid excessive compressive stresses in the subgrade from Table 10-7 or Table 10-8.
- Calculate the thickness of the wearing course based on the expected rate of gravel loss and a realistic choice of the frequency of re-gravelling.

If the in-situ subgrade is classified as S4 or stronger, it is only necessary to reshape and recompact it before applying an NG20 or NG15 base.

Table 10-7: Gravel base thickness for major gravel roads – strong gravel (NG20)

Subgrade Strength Class CBR (%)	Traffic Classes (mesas)				
	TLC1 (<0.01)	TLC2 (0.01-0.1)	TLC3 (0.1-0.3)	TLC4 (0.3-0.5)	TLC5 (0.5-1.0)
S2 (3-4)	170	200	240	280	375
S3 (5-7)	150	175	200	230	295
S4 (8-14)	125	150	180	205	260

Table 10-8: Gravel base thickness for major gravel roads – weak gravel (NG15)

Subgrade Strength Class CBR (%)	Traffic Classes (mesas)				
	TLC1 (<0.01)	TLC2 (0.01-0.1)	TLC3 (0.1-0.3)	TLC4 (0.3-0.5)	TLC5 (0.5-1.0)
S2 (3-4)	190	220	260	300	400
S3 (5-7)	170	190	220	250	310
S4 (8-14)	150	170	175	190	280

Note: *This is the additional depth of compacted subgrade material

The following points should be noted:

- The thicknesses required increase considerably if the gravel is weak, hence stronger gravels should generally be used if they are available at a reasonable cost.
- On relatively weak subgrades (S2 and S3), the use of strong gravels (G45) should be avoided because of the poor structural "balance" of such pavements. Instead, the use of an improved subgrade layer should be considered. However, for gravel roads, if G45 is the only nearby readily available material, it can be used.
- Where the available gravel is not homogeneous, it will be necessary to substitute a particular class of gravel with one or more different classes of gravel of appropriate thickness. The following conversion factors may be used for this purpose.

  G45 = 1.5 x G15

  G30 = 1.3 x G15

Thus, a 200 mm layer of G45 material could be substituted with a 300 mm layer of G15 material. For effective compaction of the gravel layer, it is necessary to restrict the loose thickness of gravel to a maximum lift of about 200 mm. Thus, any of the gravel layers that require a compacted thickness of more than 150 mm must be compacted in more than one 200 mm lift.

10.4 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 traditional CBR testing. 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. 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.

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Appendix: Design Examples

Design example DCP-DN method

The use of the DCP method for the design of a typical unpaved road being upgraded from an existing track is illustrated below. The expected traffic is between 7 and 20 heavy vehicles per day.

DCP tests were carried out every 500 m (every 200 m would have been preferable) and the results analysed using AfCAP LVR DCP programme as described in *Chapter 9 – Section 9.2.5*. The results were then tabulated in a spreadsheet as illustrated in Table A-1, and the CUSUMs calculated for all of the DSN_{800}, $DN_{150-300}$, $DN_{301-450}$ and $DN_{451-800}$ values. This data was then used to identify the uniform sections as illustrated in Figure A-1. Four distinct uniform sections are shown by the majority of the plots (0 to 2.5 km, 2.5 to 4.5 km, 4.5 to 7.5 km and 7.5 to 9.0 km).

Table A-1: Spreadsheet showing the CUSUM calculation

Cusum Analysis Unpaved Example																
Test			DSN800			0-150 mm			151-300 mm			301-450 mm			451-800	601-800
no	Chainage	Position	DSN	DSN-Avg	Cusum	DN	DN-Avg	Cusum	DN	DN-Avg	Cusum	DN	DN-Avg	Cusum	DN	DN
1	0.000	RHS	179	88.30	88.30	0.68	-18.72	-18.72	2.59	-18.41	-18.41	2.03	-16.78	-16.78	1.23	
2	0.500	RHS	357	266.30	354.60	0.68	-18.72	-37.43	0.58	-20.42	-38.83	0.92	-17.89	-34.68	0.45	
11	1.000	RHS	188	97.30	451.90	4.01	-15.39	-52.82	4.20	-16.80	-55.62	4.54	-14.27	-48.95	6.38	
3	1.500	RHS	160	69.30	521.20	0.56	-18.84	-71.65	4.06	-16.94	-72.56	2.21	-16.60	-65.55	1.06	
4	2.000	RHS	150	59.30	580.50	3.85	-15.55	-87.20	9.10	-11.90	-84.46	22.11	3.30	-62.25	4.80	
115	2.500	LHS	134	43.30	623.80	4.55	-14.85	-102.05	4.10	-16.90	-101.36	8.50	-10.31	-72.57	18.70	
5	3.000	LHS	43	-47.70	576.10	25.95	6.55	-95.49	21.52	0.52	-100.83	22.94	4.13	-68.44	16.07	
7	3.500	RHS	34	-56.70	519.40	54.00	34.60	-60.89	46.80	25.80	-75.03	20.90	2.09	-66.35	17.20	
8	4.000	LHS	52	-38.70	480.70	15.54	-3.86	-64.74	40.00	19.00	-56.03	18.90	0.09	-66.26	12.30	
9	4.500	LHS	33	-57.70	423.00	29.10	9.70	-55.04	21.20	0.20	-55.83	22.10	3.29	-62.98	33.50	
111	5.000	LHS	90	-0.70	422.30	6.40	-13.00	-68.04	8.80	-12.20	-68.02	7.70	-11.11	-74.09	15.60	
14	5.500	LHS	79	-11.70	410.60	5.20	-14.20	-82.23	11.30	-9.70	-77.72	14.10	-4.71	-78.80	21.90	
114	6.000	RHS	83	-7.70	402.90	7.60	-11.80	-94.03	8.50	-12.50	-90.22	16.80	-2.01	-80.81	11.95	
10	6.500	LHS	54	-36.70	366.20	12.30	-7.10	-101.12	11.80	-9.20	-99.42	16.40	-2.41	-83.23	21.20	
12	7.000	LHS	48	-42.70	323.50	31.60	12.20	-88.92	18.10	-2.90	-102.31	12.00	-6.81	-90.04	18.30	
13	7.500	RHS	37	-53.70	269.80	22.30	2.90	-86.02	15.70	-5.30	-107.61	17.80	-1.01	-91.05	29.30	
15	8.000	RHS	36	-54.70	215.10	53.40	34.00	-52.01	46.40	25.40	-82.21	22.20	3.39	-87.66	15.30	
117	8.500	RHS	21	-69.70	145.40	27.70	8.30	-43.71	44.20	23.20	-59.01	47.00	28.19	-59.48	44.40	
116	9.000	LHS	15	-75.70	69.70	45.00	25.60	-18.10	43.00	22.00	-37.00	45.00	26.19	-33.29	75.00	
112	9.500	RHS	21	-69.70	0.00	37.50	18.10	0.00	58.00	37.00	0.00	52.10	33.29	0.00	41.30	
			90.70			19.40			21.00			18.81			20.30	

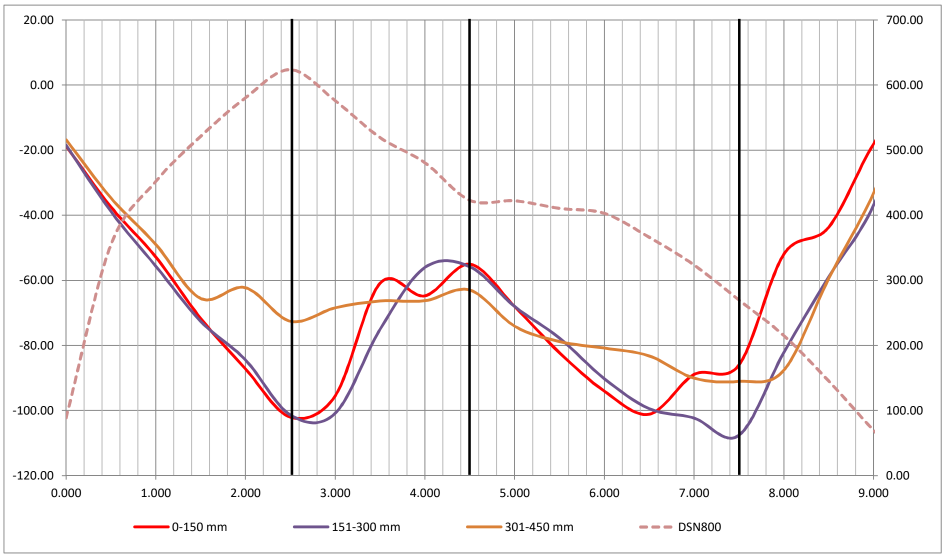

Figure A-1: Plot of CUSUMs for different parameters

As the DCP survey was carried out towards the end of the wet season, it was assumed that the pavement was in its weakest in-service condition, which was confirmed by moisture measurements and Laboratory DN tests. The DN values were thus compared directly with the required layer strength profiles in Table 10-3 as shown in Table 10-3A-2.

Table A-2: Comparison of Layer Strength (soaked) profile against design catalogue

Design class 7-20 Hvpd	Spec. DN/Layer mm	Section 1 Km 0+000 – 2+500	Section 2 Km 2+500 – 4+500	Section 3 Km 4+500 – 7+500	Section 4 Km 7+500 – 9+000
0-150 mm	14	4.0	39	22	48
151 – 300 mm	19	4.2	43	16	51
301 – 450 mm	25	8.5	22	17	49
451 – 600 mm	33	6.4	19	22	44
601 – 800 mm	50	6.9	20	25	45
DSN800	33	150	34	37	21

Adequate	Inadequate

Different upgrading requirements results were obtained for each of the uniform sections as shown in Table A-3:

Table A-3: Upgrading requirements

Design class 7-20 Hvpd	Spec. DN/Layer mm	Section 1 Km 0+500 – 2+500	Section 2 Km 2+500 – 4+500	Section 3 Km 4+500 – 7+500	Section 4 Km 7+500 – 9+000
0-150 mm	14	4.0	≤ 14	22	48
151 – 300 mm	19	4.2	≤ 19	16	51
301 – 450 mm	25	8.5	22	17	49
451 – 800 mm	33	6.4	19	22	44
601 – 800 mm	50	6.4	20	25	45
DSN800	33	150	≥ 33	≥ 33	21

Adequate	Marginal / can be improved	Inadequate	New base	New subbase

Design considerations

- **Section 1:** The existing conditions are structurally adequate and the road needs only to be shaped and re-compacted. It is still, however, necessary to confirm that the material to be used for the wearing course (upper 150 to 250 mm) complies with the requirements of Table 10-2 to ensure good functional performance.
- **Section 2:** The upper 300 mm of the existing structure is inadequate. As the material is not even strong enough for the support layers, it would need to be removed or improved with some form of mechanical or chemical treatment. Material complying with the requirements of Table 10-2 would need to be imported for the wearing course.
- **Section 3:** The upper 150 mm of the structure is inadequate and would need to be replaced or improved, for instance by mechanical stabilisation. The remainder of the structure is adequate.
- **Section 4:** This area is a major problem, possibly located on a moist stream bed or marshy area, and would need to be carefully designed with a totally imported structure of at least 800 mm thick. This would probably include a rockfill overlain with selected materials and even possibly some geotextile separation and drainage layers. An alternative route bypassing this area would in many cases probably be more cost-effective.

Design Example DCP-CBR Method

The following is assumed:

- The Traffic Load Class in the above example was determined at TLC2 over a 15-year period.
- The uniform sections determined through a CUSUM analysis of SNP (*see Chapter 9 – Design example DCP-CBR method)* are the same as for the DCP-DN method.
- The DCP data represents the pavement in a soaked condition. The subgrade classes for the uniform sections can then be calculated using the TRL relationship between DN and CBR:

$$\text{Log}_{10}\,\text{CBR} = 2.48 - 1.057\ \text{Log}_{10}\,\text{DN}$$

The subgrade classes will then be as shown in Table A-4:

Table A-4: Design subgrade classes for uniform sections

Design class TLC2	Section 1	Section 2	Section 3	Section 4
	Km 0+500 – 2+500	Km 2+500 – 4+500	Km 4+500 – 7+500	Km 7+500 – 9+000
CBR	71	20	13	6
Subgrade class	S6 (>30)	S5 (15-29)	S4 (8-14)	S3 (5-7)

The base thickness will then depend on the quality of the available gravel (Tables 10-7 to 10-9) as shown in Table 10-14.

Table A-5: Gravel base thickness (mm) for different gravel qualities

Design class TLC2	Section 1	Section 2	Section 3	Section 4
	Km 0+500 – 2+500	Km 2+500 – 4+500	Km 4+500 – 7+500	Km 7+500 – 9+000
G45	R&R*	125	150	200
G30	100	100	175	200
G15	150	150	225	250

*R&R = Rip, shape and re-compact existing gravel base

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11.1 Introduction

11.1.1 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.

11.1.2 Purpose and Scope

The main purpose of this chapter is to provide a broad overview of:

- The various types of 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.
- The outline design of both bituminous and non-bituminous 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.

11.2 Bituminous Surfacings

11.2.1 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.

11.2.2 Main Types

Terminology for different surfacing types varies for different countries within the region. For purposes of this Manual, Figure 11-1 illustrates the main types of bituminous surfacings that are potentially suitable for use in Malawi.

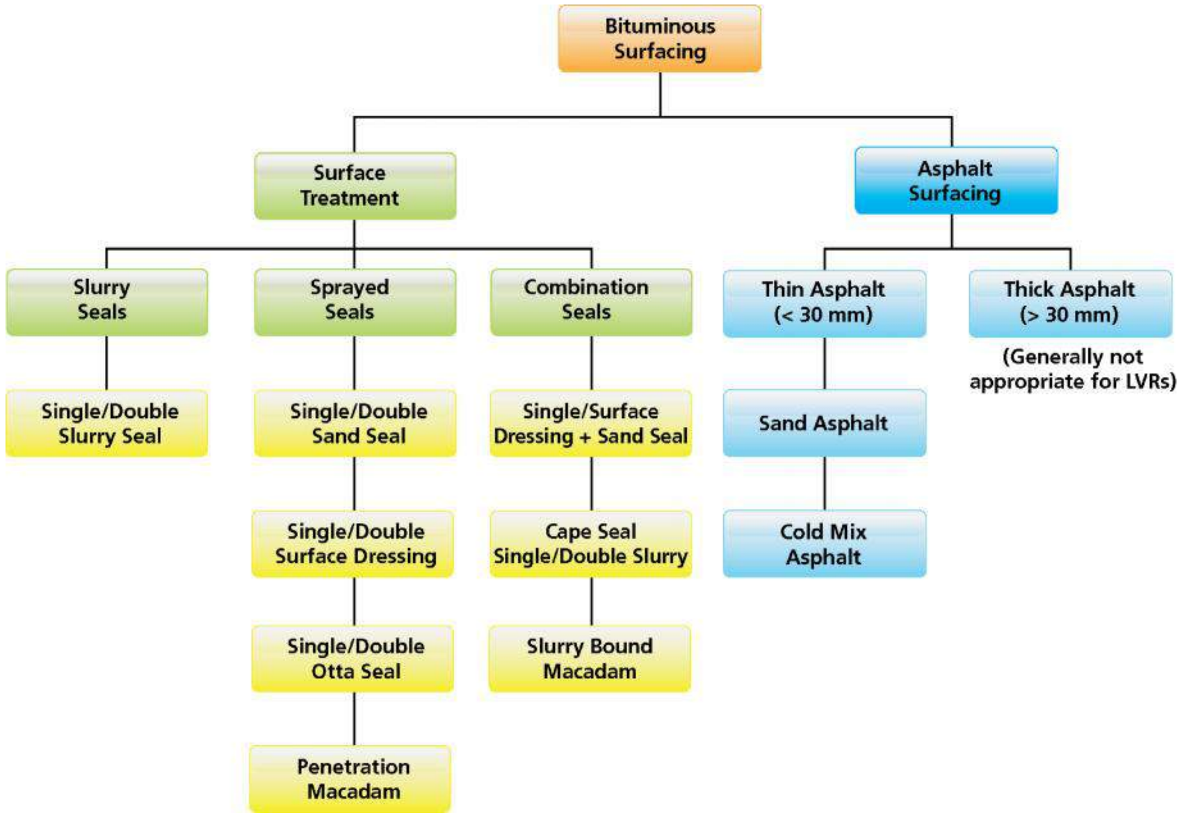


Figure 11-1: Terminology and categorisation of bituminous surfacings

Some of the typical types of bituminous surfacings used on LVRs are shown in Figure 11-2.

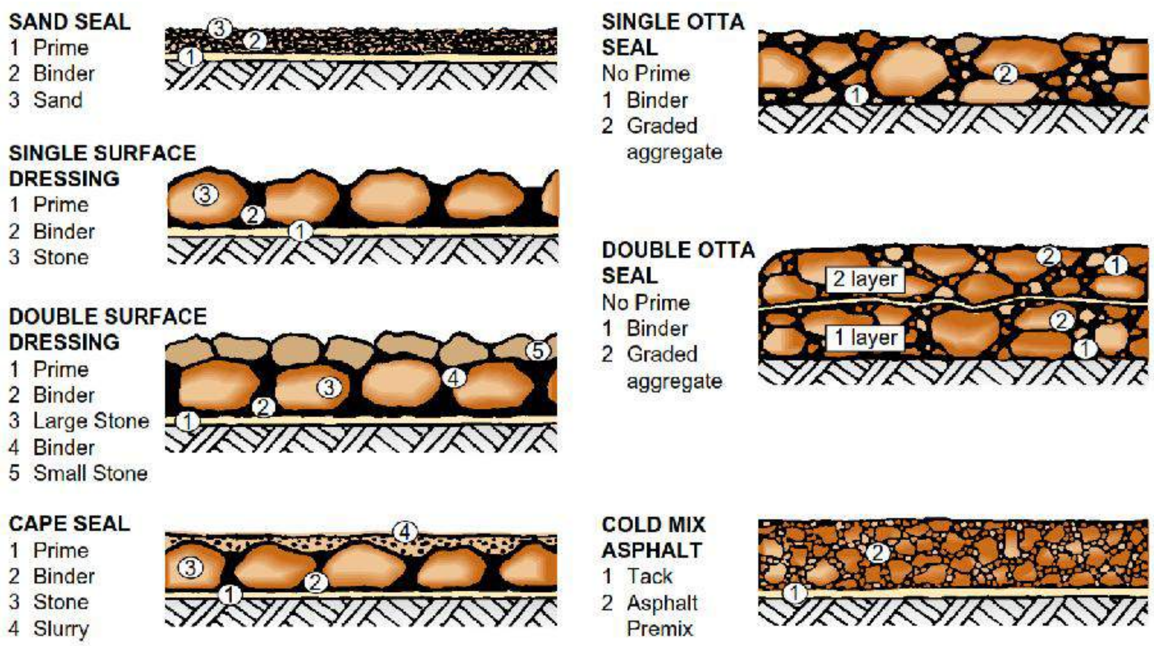


Figure 11-2: Common types of bituminous surfacings

11.2.3 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 11-3.

Figure 11-3: 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 11.1 indicates the relative difference in the required properties between the various surfacing types.

Table 11-1: 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 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.

11.2.4 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 seals, which are typically used as temporary or holding measures in a phased surfacing strategy, have much shorter service lives (generally < 10 years) than double/combination seals (generally > 10 years).

Table 11-2: Typical lives of bituminous surfacings

Type of surfacing	Typical service life (years)
a) Thin seal / phased strategy	
Single Sand Seal	2 – 3
Double Sand Seal	3 – 6
Single Slurry Seal	3 – 5
Single Surface Dressing	5 – 7
b) Double / combination seal strategy	
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

11.2.5 General Characteristics

The general characteristics of the different types of bituminous surfacings are summarized in Table 11-3.

Table 11-3: 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 years -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 11-3 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 in Malawi.
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 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 11-3 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 years – 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 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 in Malawi.
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 in Malawi.
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.

11.2.6 Design

General: The design of a surface treatment is usually project-specific and related to such factors as traffic volume, climatic conditions, available type and quality of materials. Various methods have been developed by various authorities for the design of surface treatments. Thus, the approach to their design, as described in this section, is generic, with the objective of presenting typical binder and aggregate application rates for planning or tendering purposes only. Where applicable, reference has been made to the source document on the design of the particular surface treatment for detail design purposes.

Prime coat: A prime coat is used to provide an effective bond between the surface treatment and the existing road surface or underlying pavement layer and is essential for good performance of a bituminous surfacing. This generally requires that the non-bituminous base must be primed with an appropriate grade of bitumen before the start of construction of the surface treatment. However, for an Otta seal and Penetration Macadam a prime coat is normally not required.

Typical primes are:

- **Bitumen primes**: Low viscosity, medium curing cutback bitumen such as MC-30, MC-70, or in rare circumstances, MC-250, can be used for prime coats.
- **Emulsion primes**: Bitumen emulsion primes are not suitable for priming stabilized bases as they tend to form a skin on the road surface and not penetrate into the top of the base.

The choice of prime depends principally on the texture and density of the surface being primed. Low viscosity primes are necessary for dense cement or lime stabilized surfaces, while higher viscosity primes are used for untreated, coarse-textured surfaces. Emulsion primes are not recommended for saline base courses.

The grade of prime and the nominal application rates to be used on the various types of road bases are presented in Table 11-4.

Table 11-4: Typical prime application rates in relation to road base type

Pavement surface	Prime	
	Grade	**Rate of application (l/m2)**
Tightly bonded (light primer)	MC-30	0.7 – 0.8
Medium porosity (medium primer)	MC-30 / MC-70	0.8 – 0.9
Porous (heavy primer)	MC-30 – MC 70	0.9 – 1.1

Binders: There are a wide variety of binders that may be used for bituminous surfacings depending primarily on the type and function of the surfacing – a choice that would be influenced by such factors as traffic characteristics, pavement structure (bearing capacity), road geometry (gradient, curvature, inter- sections, etc.) and the environment. A correct choice of binder is, therefore, crucial for achieving good performance of the surfacing.

The binder used in a surface treatment must fulfil a number of important requirements including the capability of:

- being easily sprayed;
- "wetting" the surface of the road in a continuous film;
- quick setting, i.e. not running off a cambered road or forming pools of binder in local depressions;
- easy "wetting" and adhering to the aggregate chippings at spray temperature;
- resisting traffic forces and holding the chippings at the highest prevailing ambient temperatures;
- remaining flexible at the lowest ambient temperatures, neither cracking nor becoming brittle enough to allow traffic to "whip off" the chippings;
- resisting premature weathering and hardening.

Figure 11-4 shows the permissible range of binder viscosity that applies to the construction of surface treatments. In Malawi, mean daytime temperatures in the summer range from about 20°C to 35°C and in the winter from about 12°C to 26°C. In general, penetration grades of bitumen are most appropriate for the higher road temperatures and cutback grades for the lower road temperatures.

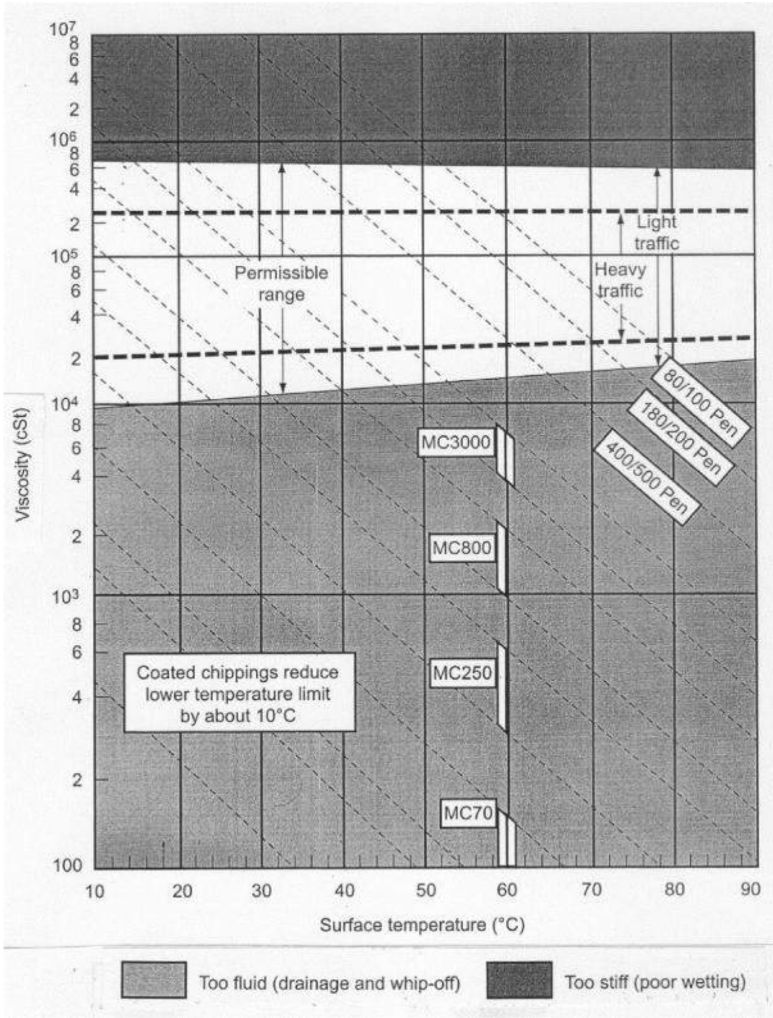


Figure 11-4: Surface temperature/choice of binder for surface treatments

The following are the types of binders that are generally appropriate for use in Malawi.

- **Penetration grade bitumen:** Refined and blended to meet specific requirements. They are relatively stiff and are graded by their penetration. Penetration grades used for Asphalt Concrete and Surface Treatment are shown in Table 11-5.

Table 11-5: Penetration grade bitumen for Asphalt Concrete and Surface Treatment

Asphalt concrete	Surface treatment
40/50	70/100
50/70	150/200

- **Cutback bitumen:** Produced by diluting a penetration grade bitumen with a flux oil such as kerosene or diesel oil to produce the desired viscosity. After construction, the diluents evaporate with time and the binder reverts to its original penetration grade. A cutback varies in behaviour according to the type of cutter or flux used as the diluent. Cutback bitumen used for sealing works and prime are:
  - MC 3000
  - MC 800
  - MC 70 and MC 30

Possible environmental impacts should be considered when using cut-back bitumen.

- **Bitumen emulsion:** Consists of dispersions of bitumen in water. Hot bitumen, water and emulsifier are processed in a high-speed colloid mill that disperses the bitumen in the water in the form of small droplets. They have a low viscosity compared to the bitumen from which they are produced and can be workable at ambient temperatures.

- **Modified bitumen: P**roduced by adding natural rubber or a polymer modifier to bitumen with resulting improvement in their service performance by enhancing such properties as their durability, resistance to ageing, elasticity and/or plasticity. The most common polymers used in modified bitumen are styrene butadiene styrene (SBS), polybutadiene (PBD) and ethylene-vinyl acetate (EVA).

Adhesion agent: The successful performance of a bituminous seal depends not only upon the strength of the two main constituents – the binder and the aggregate – but also upon the attainment of adhesion between these materials - a condition that is sometimes not achieved in practice.

The main function of an adhesion agent is to facilitate the attainment of a strong and continuing bond between the binder and the aggregate. However, if the aggregate is dusty, the adhesion agent will be ineffective, and in such a case, the aggregate should be pre-coated.

Pre-coating materials: Surfacing aggregates are often contaminated with dust on construction sites and, in that condition, the dust tends to prevent actual contact between the aggregate and the binder. This prevents or retards the setting action of the binder, which results in poor adhesion between the constituents. This problem can be overcome by sprinkling the aggregate with water or, alternatively, by using an appropriate pre-coating material, which increases the ability of the binder to wet the aggregate and improve adhesion between binder and aggregate.

Sand Seal

Design: There are no formal methods for the design of Sand Seals with the binder and aggregate application rates being based on local experience. Typical constituents for sand seals are:

Binder: The following grades of binder are typically used:

- MC-800 cut-back bitumen.
- MC-3000 cut-back bitumen.
- Spray-grade emulsion (65% or 70% of net bitumen).

Aggregate: The grading of the sand may vary, but the requirements of Table 11-6 must be met. However, in the case of a relatively high proportion of motorbikes, a coarser sand or grit may be considered (max. 7 mm) in order to improve the skid resistance when wet.

Table 11-6: Grading of sand for use in Sand Seal

Sieve size (mm)	Grading (% by mass passing)	
	Natural river sand	Crusher dust
10	100	100
5	85-100	85-100
1.18	20-60	20-80
0.425	0-30	-
0.300	0-15	-
0.150	0-5	0-30
Sand equivalent ≥ 35% (SANS 838)		

For planning or tender purposes, typical binder and aggregate application rates for Sand Seals are shown in Table 11-7.

Table 11-7: Binder and aggregate application rates for Sand Seals

Application	Net bitumen application rate (l/m²)	Aggregate application rate (m³/m²)
Double Sand Seal used as a permanent seal	1.2 per layer	0.010 – 0.012 per layer
Single Sand Seal used as a cover over an Otta Seal or Surface Dressing	0.8 – 1.0	0.010 – 0.012
Single Sand Seal used as a maintenance remedy on an existing road	0.6 – 1.0	0.010 – 0.012

Slurry Seal

Design: The detailed design of a Slurry Seal surfacing is presented in the *Sabita Manual 28 – Best Practice for the Design and Construction of Slurry Seals, June 2010*. The design is based on semi-empirical methods or experience with the exact proportions of the mix being determined by trial mixes.

Binder: The binder typically used is an anionic or cationic emulsion or quick setting cationic emulsion produced from 80/100 pen. grade base bitumen. Stable grade anionic and cationic emulsions are used when the slurry mixes are being laid by hand. If the crusher dust used in the slurry comes from acidic rocks a cationic emulsion is preferred.

Aggregate: The aggregate grading for conventional slurry mixes is presented in Table 11-8.

Table 11-8: Aggregate grading for conventional slurry mixes

Sieve size (mm)	Percentage passing sieve, by mass	
	Fine type	Coarse type
10		100
5	100	85-100
2	85-100	50-90
1.18	60-90	32-70
0.425	32-60	20-44
0.150	10-27	7-20
0.075	4-12	2-8
Sand equivalent ≥ 35 (SANS 838)		

For planning or tender purposes, the typical composition of the slurry may be based on the mass proportions indicated in Table 11-9.

Table 11-9: Nominal Slurry Seal mix components

Material	Proportion (Parts)
Fine aggregate (dry)	100
Cement (or lime)	1.0 – 1.5
60% stable grade emulsion	20
Water	+ / - 15

Surface Dressing

Design: Design methods for both single and double Surface Dressings are presented in *Overseas Road Note 3 (2nd edition, 2000): A guide to surface dressing in tropical and sub-tropical countries.*

The design is based on the concept of partially filling the voids in the covering aggregate. This is controlled by the natural orientation of the chippings as they lie on the road surface with their 'least dimension' in the vertical direction. Thus, the Average Least Dimension (ALD) of the chippings is the parameter that mainly determines how much bitumen is required. Corrections to the spray rate need to be subsequently carried out to take account of site conditions as described in the guide. These conditions include traffic level, the hardness of existing road surface (controlling embedment of the chippings), shape and condition of chippings, downhill or uphill road gradient, the grade of bitumen, and climate.

Typical constituents for Surface Dressings are:

Binder: The bituminous binder can consist of any of the following:

- 70/100 or 150/200 penetration grade bitumen.
- MC 3000 grade cutback bitumen.
- Spray grade anionic (60%) or cationic (65% or 70%).
- Modified binders (polymer modified and bitumen rubber).

Aggregate: The aggregate for a Surface Dressing shall be durable and free from organic matter or any other contamination. Typical aggregate properties and grading requirements for Surface Dressings are given in Table 11-10.

Table 11-10 : Aggregate properties for bituminous Surface Dressings

Sieve size (mm) Material property	Nominal aggregate size			
	20 mm	14 mm	10 mm	7.1 mm
	Grading (% by mass passing			
25	100			
20	85-100	100		
14	0-35	85-100		
10	0-5	0-30	85-100	100
7.1	-	0-5	0-30	80-100
5	-	-	0-5	0-40
2	0-2	-	-	0-5
0.425	< 0.5	< 1.0	< 1.0	< 1.5
0.075	< 0.3	< 0.5	< 0.5	< 1.0
Flakiness Index	Max 20	Max 25		Max 30
10% $FACT_{dry}$	AADT > 1000: min 160 kN			
	AADT < 1000: min 120 kN			
10% $FACT_{soaked\ 24\ hrs}$	Min 75% of the corresponding TFV_{dry}			

In view of the fact that design of surfacing seals is based on aggregate size, grading and ALD, no departure from the standard specifications is recommended for these parameters for aggregates used in seals on LVSRs.

Binder application: For planning purposes, typical binder and aggregate application rates for Single and Double Surface Dressings are given in Table 11-11.

Table 11-11: Nominal binder and aggregate application rates for Surface Dressings

Item	Double Surface Dressing		Single Surface Dressing	
	20 mm / 10 mm	14 mm / 7.1 mm	14 mm	10 mm
Aggregate application rates (m^3/m^2)				
1st layer	0.015	0.011	0.012	0.010
2nd layer	0.009	0.007		
Hot spray rates of 70/100 penetration grade bitumen (l/m^2)				
AADT < 200	1.7 + 1.3	1.5 + 0.8	1.6	1.3
AADT 200 - 1000	1.5 + 1.0	1.2 + 0.7	1.3	1.0

These specifications apply in situations where the surfacing stone meets conventional specifications. In situations where the materials are marginal, the following design procedure should be mandatory.

Design procedure:

Laboratory tests

1. Sample surfacing stone from the selected quarry or quarries and carry out laboratory tests. Compare the laboratory tests with the specifications given in Table 11-10.
2. Calculate application rates.
   a) Determine the Average Least Dimension (ALD) from the chart (Figure 11-5) using the value of the measured flakiness index and the nominal size value obtained from the sieve analysis. The intercept of ALD line and a straight line drawn from the aggregate size scale to the flakiness scale should be read as the ALD value of the surfacing stone. Alternatively, place a random sample of 100 stones on a flat table. The ALD is their vertical height measure from their most stable face. Measure this for each stone with callipers and take the average.
   b) Determine the weighting factor, the sum of the individual factors given in Table 11-12.
   c) Determine the binder application rate using the following formula:

$$R = 0.625 + (0.023xF) + [0.0375+ (0.0011 \times F)] \times ALD$$

Where: F = Overall weighting factor (see Table 11-12 below)
ALD = Average least dimension
R = Rate of binder application in kg/m2

This formula is correct for MC3000 grade bitumen. Adjustment factors for different cut-back bitumen, penetration grade bitumen and emulsions are required (see TRL's ORN 3 for a more detailed description).

   d) Calculate the application rate for the chippings or surfacing stone.

A rough estimate of the application rate for the chippings can be obtained using the following formula assuming the density of loose aggregate to be approximately 1.35 kg/litre:

$$\text{Chipping application rate} = 1.364 \times ALD$$

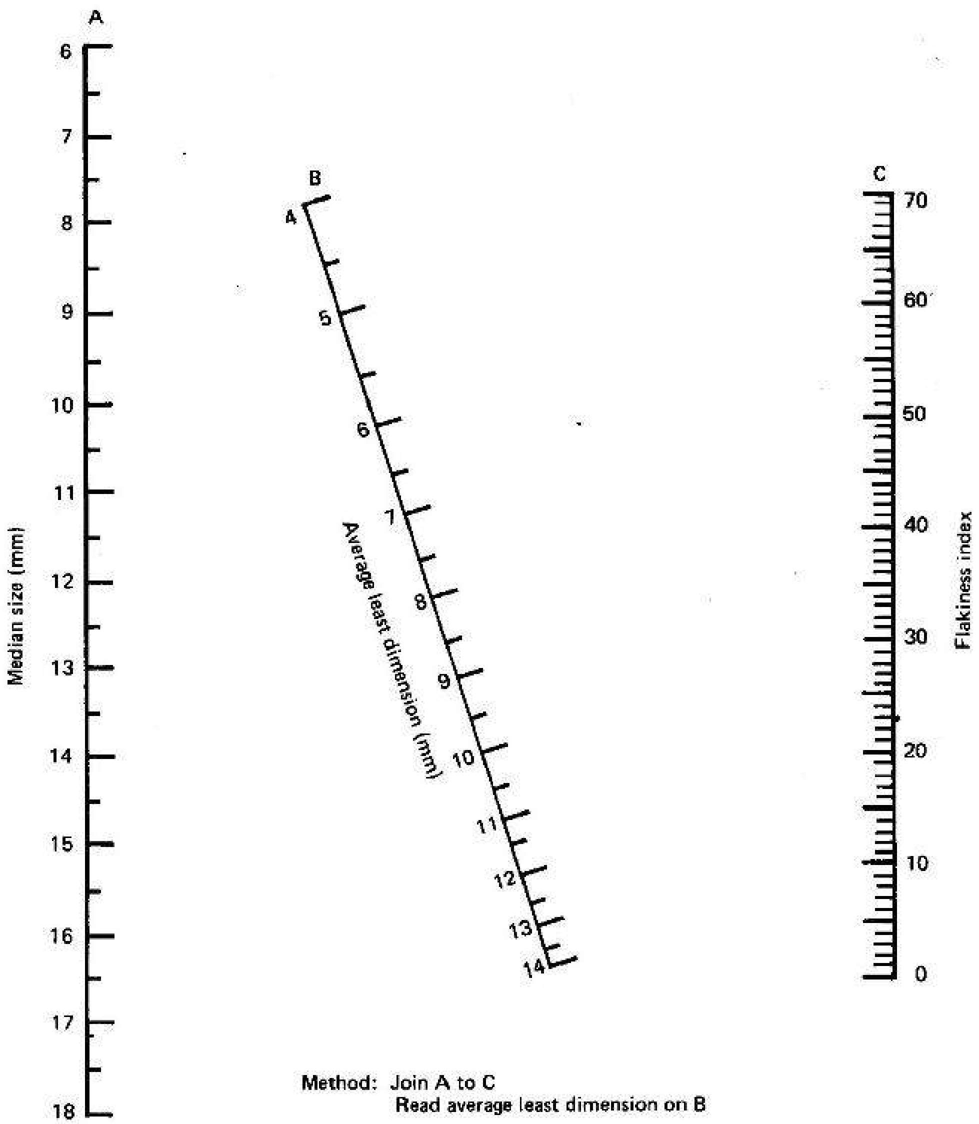


Figure 11-5: Determination of Average Least Dimension (ALD)

The weighting factor, F, is obtained from Table 11-12.

Table 11-12: Determination of weighting factor (F)

Description	Factor (F)	Description	Factor (F)
Traffic vehicles/lane/day		**Climatic conditions**	
Very light (0 – 20)	+8	Wet and cold	+2
Light (20 – 100)	+4	Tropical (wet and hot)	+1
Medium light (100 – 250)	+2	Temperate	0
Medium (250 – 500)	0	Semi-arid (hot and dry)	-1
		Arid (very hot and very dry)	-2
Existing surface		**Type of chippings**	
Untreated or primed	+6	Round/dusty	+2
Very lean bituminous	+4	Cubical	0
Lean bituminous	0	Flaky	-2
Average bituminous	-1	Pre-coated	-2
Very rich bituminous	-3		

Conversions from hot spray rates in volume (litres) to tonnes for payment purposes must be made for the bitumen density at a spraying temperature of 180°C. For planning purposes, a hot density of 0.90 kg/l should be used until reliable data for the particular bitumen is available.

Shoulders and steep grades: The design of bituminous surfacings for shoulders or steep grades (typically > 5%) follows, in most respects, the same general principles as that for the road carriageway. However, because of the much-reduced trafficking of the shoulders, and the tendency for the surfacing to dry out more quickly than on the carriageway, higher bitumen spray rates are required on shoulders, typically of the order of + 10% of that used on the travelled way. In contrast, because of the slower moving traffic on steep grades, lower bitumen application rates are required, typically of the order of – 10% of that used on flat grades.

Otta Seal

Design: The design of an Otta Seal relies on an empirical approach in terms of the selection of both an appropriate type of binder and an aggregate application rate. Full details of the design methods are given in the *Norwegian Public Roads Administration, Publication No.93 - A Guide to the use of Otta Seals (1999)*, which is based on extensive experience in Sub-Saharan Africa. The bitumen and aggregate application rates in this Manual incorporate the latest recommendations for the design and construction of Otta Seals.

Binder: The choice of binder in relation to traffic and aggregate grading is given in Table 11-13, where the shaded cells indicate the preferred grading in relation to traffic.

Table 11-13: Choice of Otta Seal binder in relation to traffic and grading

AADT at time of construction	Type of bitumen		
	Open Grading	Medium Grading	Dense Grading
>100	150/200 pen grade	150/200 pen grade in cold weather	MC3000 MC800 in cold weather
<100	150/200 pen grade	MC 3000	MC 800

Table 11-14 gives the recommended hot spray rates for primed base courses.

Table 11-14: Nominal binder application rates for Otta Seal for un-primed bases (l/m²)

Type of Seal		Open grading	Medium grading	Dense grading	
				AADT < 100	AADT > 100
Double	1st layer	1.7	1.8	1.8	1.7
	2nd layer	1.6	1.4	2.0	1.9
Single w/ Sand cover seal	Fine sand	0.8	0.7		0.9
	Crusher dust	0.9	0.8		0.8
	1st layer	1.7	1.8	2.0	1.9
Single		1.8	1.9	2.1	2.0
Reseal (single)		1.7	1.8	2.0	1.8

The following points should be noted with regard to the binder application rates:

- On primed bases, the spray rate for the first layer shall be reduced by 0.2 l/m^2.
- Hot spray rates lower than 1.6 l/m2 should not be allowed.
- Binder for the sand seal cover shall be MC 3000 for crusher dust or coarse river sand and MC 800 for fine sand.
- Where the aggregate has a water absorbency of more than 2%, the hot spray rate should be increased by 0.3 l/m2.

Aggregate: Crushed and uncrushed material or a mixture of both can be used. The grading of the aggregate should fall within, and should desirably be parallel to, the grading envelope. Although the envelope is relatively wide, the preferred maximum size is 16 mm (19 mm can be tolerated for the Double Otta Seal), and the maximum fines content (material passing the 0.075 mm sieve) should preferably not exceed 10%. The recommended grading in relation to the traffic level is indicated in Table 11-15.

Table 11-15: Alternative Otta Seal grading requirements

AASHTO Sieve (mm)	Preferred grading		
	Open AADT < 100	Medium AADT >100	Dense AADT > 100
	% passing	% passing	% passing
20	100	100	100
14	50 - 82	68 - 94	84 - 100
10	36 – 58	44 - 73	70 - 98
5	10 - 30	19 - 42	44 - 70
2	0 - 8	3 - 18	20 - 48
1.18	0 - 5	1 - 14	15 - 38
0.425	0 - 2	0 - 6	7 - 25
0.075	0 – 1	0 - 2	3 – 10

The aggregate shall meet the specification requirements shown in Table 11-16.

Table 11-16: Specifications for Otta Seal aggregate

Test	Requirement	
	< 100 vpd	> 100 vpd
10% FACT (kN)	Min.90	Min. 110
Wet/dry 10% FACT ratio	Min. 0.60	Min. 0.75
Water absorption (WA) (%)	Max. 2.0 In case of higher (WA), increase hot spray rate with 0,3 l/m^2	

The aggregate application rate for different gradings is presented in Table 11-17.

Table 11-17: Nominal Otta Seal aggregate application rates

Type of Seal	Aggregate Application Rates (m^3/m^2)		
	Open Grading	Medium Grading	Dense Grading
Otta Seals	0,015 – 0,017	0.015 – 0,017	0.018 – 0.022
Sand Cover Seals	0.012 – 0,014		

The following points should be noted with regard to the aggregate application rates:

- Sufficient amounts of aggregate should be applied to ensure that there is some surplus material during rolling (to prevent aggregate pick-up) and through the initial curing period of the seal.
- Aggregate embedment will normally take about 3 weeks – 6 weeks to be achieved where crushed rock is used, after which any excess aggregate can be swept off. Where natural gravel is used, the initial curing period will be considerably longer (typically 6 - 10 weeks).

Rolling: In the construction of Otta Seals the following factors should be given particular attention.

- As a rule of thumb, it should be assumed that a good result will be achieved when the bitumen can be seen being pressed up in-between the aggregate particles, sparsely distributed in the wheel tracks of the chip spreader or truck wheels.
- Sufficient rolling of the Otta Seal must be achieved. A minimum of two pneumatic-tired rollers with a minimum weight of 12 tonnes or more is essential. Such rollers are particularly well suited to kneading the binder upwards into the aggregate particles, and to apply pressure over the entire area. A minimum of 30 passes with a pneumatic-tired roller is required over the entire surface area, shoulders included, on the day of construction.
- After the initial rolling is completed (on the day of construction), it may be an advantage to apply one pass with a 10-12 tonne static tandem steel roller to improve the embedment of the larger aggregate. During this process, any weak aggregate will be broken down and will contribute to the production of a dense matrix texture. Table 11-18 summarizes the minimum rolling requirements.

Table 11-18: Minimum rolling requirements for an Otta Seal

Rolling after treatment	Minimum requirements
On the day of construction	30 passes with pneumatic roller (weight > 12 tonnes) + 1 pass with a static steel
For each of the next three days after construction	30 passes with pneumatic roller (weight > 12 tonnes)
2-3 weeks after construction	Sweep off any excess aggregate

- Commercial traffic should be allowed on the surfaced area immediately following completion of the initial rolling with the pneumatic roller(s). This will assist further in the kneading of the binder/aggregate admixture.
- To minimise the “whipping off” of surfacing aggregates and damage to windscreens, a maximum speed limit of 40 km/hour - 50 km/hour should be enforced immediately after construction and sustained for minimum 3 weeks - 4 weeks when any excess aggregate should be swept off.

Figure 11-6: Extensive rolling of an Otta Seal is essential to achieve a good result

Cape Seal

Design: The design of a Cape Seal is a combination of a Single Surface Dressing plus a Slurry Seal. The design is similar to that for a Surface Dressing and Slurry Seal, as described above.

Typical constituents for Cape Seals are:

- Binder: A variety of binder types may be used for constructing a Cape Seal.
- Aggregate: The same requirements are required as for Surface Dressings and Slurry Seals.

For planning purposes, typical binder and aggregate application rates for Single Surface Dressings are as shown in Table 11-19.

Table 11-19: Nominal binder and aggregate application rates for a Cape Seal

Nominal size of aggregate (mm)	Nominal rates of application for tendering purposes	
	Binder (litres of net cold bitumen per m^2)	Aggregate (m^3/m^2)
14 20	0.8 1.1	0.019 0.013

Cold Mix Asphalt

Design: The Cold Mix Asphalt (CMA) is, in many respects, similar to an Otta Seal in that a graded aggregate is used. However, the binder used is an emulsion, rather than a hot-applied penetration grade or cut-back bitumen.

Binder: Different types of the emulsion may be used for different circumstances. Table 11-20 describes the binder options.

Table 11-20: Binder options for CMA

Type of emulsion		Description
K3-60/65/70	Slow setting cationic emulsion	The preferred binder option. Works well with most aggregates and gives adequate working time before setting. Suitable for labour-based work.
K2-60/65/70	Medium setting cationic emulsion	Not commonly used but may work if construction is relatively quick. Fines content must be kept to a minimum to prevent "balling" of the fines and inadequate coating of the coarse fractions.
SS-60/70	Slow setting anionic emulsion	Not commonly used but may be used with natural gravels with high fines content. Very long setting time.

Note: MS = Medium setting, SS = Slow setting (all anionic emulsions)
K2 = Medium setting, K3 = Slow setting (all cationic emulsions)
The numbers (60, 65, 70) denotes the bitumen content in the emulsions

Cationic emulsions coat most aggregates more efficiently than anionic emulsions and have better adhesion properties. They are, therefore, normally the preferred binder for CMA.

"Breaking" of the emulsion is the loss of water from the emulsion. Determining whether an emulsion has broken is very easy as the colour turns from brown to black when it breaks. The "breaking" process is influenced by the environmental conditions: the incident wind velocity, humidity and temperature. In cold and damp conditions, the breaking time will be longer than in hot and sunny weather.

For more information, refer to the SHELL Bitumen Handbook of 2015.

Aggregate: Normally, crushed aggregates are preferred for CMA, but natural gravels may also be used under circumstances where crushed aggregates are not readily available.

The grading and strength requirements for crushed aggregate are shown in Table 11-21 and Table 11-22: Minimum aggregate strength requirements for Cold Mix Asphalt.

Table 11-21: Recommended grading envelope for CMA

Recommended aggregate grading envelope	
Sieve size (mm)	**Percentage by weight passing**
14	100
10	80-95
7.1	58-85
5	40-68
2	19-40
1.18	12-28
0.425	6-16
0.300	4-13
0.150	2-9
0.075	1-7
Sand equivalent min. 40	
Mineral filler: Cement, lime or other non-plastic material if required to improve grading and/or workability	
Flakiness Index Max 30%	

Table 11-22: Minimum aggregate strength requirements for Cold Mix Asphalt

Aggregate strength requirements	AADT at time of construction	
	<100	**>100**
Min Dry 10% FACT	90 kN	110 kN
Min Wet/Dry strength ratio	0.60	0.75

For natural gravels, the grading, shape (flakiness) and strength requirements are not critical. The recommended minimum 10% FACT is 90 kN with a wet/dry ratio of 0.60.

Mix proportions: The residual bitumen content in CMA shall be within the limits specified in Table 11-23. The exact percentage to use shall be determined after the Engineer's approval of the mix design and the result of field trials.

Table 11-23: Residual bitumen content in CMA

Aggregate grading	0/14
Residual bitumen content	5.5 – 7.0%

The reactivity of the aggregate with the cationic emulsion varies between different parent rock types. Thus, trial mixes must be carried out to ensure that no "balling" of the emulsion with the fines in the aggregates occurs as this would prevent proper coating of the coarse aggregates. If "balling" occurs, the fines content must be reduced towards the lower boundary of the recommended grading envelope

For tendering purposes, the following mix proportions shall be used:

- Maximum aggregate batch volume: 40 litres.
- K3 65 Cationic Emulsion: 6.5 litres.
- Water: 1 litre (when using dry aggregate).

For every new source of aggregate, trial mixes should be done before surfacing operations start, and if necessary, the mix proportions adjusted. It is important that all aggregates are evenly moist before the emulsion is added.

Construction: Mixing can be done in concrete mixers or by hand tools in purpose-made mixing pans, which are easier to transport and clean. Mixing pans also makes for easy scaling up or down the production by adding or subtracting pans and labourers as required.

The mixing must be done swiftly and thoroughly while ensuring that all the aggregates are coated. When the mixing is completed, the mix must quickly be placed on the road in between the guide rails and levelled to the top of the guide rails before the emulsion starts to break (turn from brown to black), after which point the mix gets sticky and difficult to spread.

Figure 11-7: Levelling and compaction of CMA

Compaction should be undertaken with a double drum steel roller, as shown in Figure 11-7. Rolling can commence once the guide rails have been removed, and the initial breaking of the asphalt has commenced for the full depth of the layer. This period will be affected by the ruling weather conditions but can normally be done within ½ hour.

The first compaction is done with the roller in static mode. After 2 hours to 3 hours, the final compaction is done with the roller in vibrating mode.

The recommended minimum compacted thickness of CMA is 20 mm, which is achieved by the use of 25 mm guide rails, as shown in Figure 11-7.

Care must be taken to plan the CMA works to prevent washout by rain before the emulsion has set properly.

11.2.7 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 11-24.

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 11-24: 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

11.3 Non-bituminous Surfacings

11.3.1 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 later in this chapter.

11.3.2 Main Types

The main types of non-bituminous surfacings are summarized in Figure 11-8.

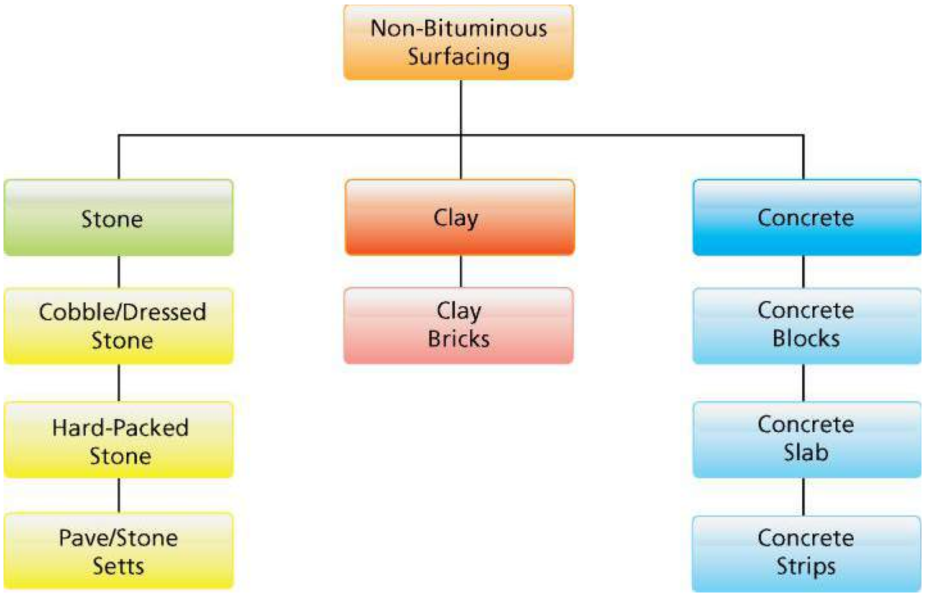
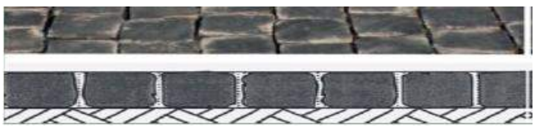


Figure 11-8: Terminology and categorisation of non-bituminous surfacing types

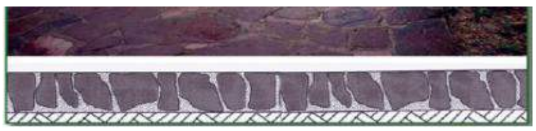
Some of the typical types of non-bituminous surfacings are shown in Figure 11-9.

Cobble Stone/Dressed Stone

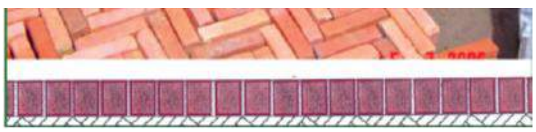
Hand Packed Stone



Pave/Stone Setts



Mortared Stone



Fired Clay Brick

Unreinforced Concrete

Figure 11-9: Common types of non-bituminous surfacings

11.3.3 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 to 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. Reference is made to *Chapter 9 – Structural Design: Paved Roads,* Section 9-4, Design of Roads with Non-discrete Surfacings, until further locally relevant evidence is available.

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.

11.3.4 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.

11.3.5 General Characteristics

The general characteristics of a range of non-bituminous surfacings that may be considered for use in Malawi are summarized in Table 11-25.

Table 11-25: 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.

Surfacing	Characteristics (Cont'd)
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 (good practice is to lay the bricks with narrow face up to improve strength). • 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.
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.

11.3.6 Design

The design approach for non-bituminous surfacings is similar to that of the more traditional bituminous surfacings, in that design inputs are principally traffic volume, subgrade soil condition and other environmental factors.

The detailed design of non-bituminous surfacings is dealt with in *Chapter 9 – Structural Design: Paved Roads*.

Figure 11-10: Concrete strip road with dangerous edge drop

The non–bituminous surfacings and their typical thicknesses and strength requirements are given in Table 11-26.

Table 11-26: Common thicknesses and strength requirements for non–bituminous surfacings

Type of surfacing	Typical thickness (mm)	Crushing strength (MPa)
Cobble Stone/Dressed Stone	150 - 200	20
Hand Packed Stone	150 - 300	20
Pave Stone Setts	100	20
Fired Clay Brick *	70 - 100	20
Concrete Blocks	70	25
Un-Reinforced Concrete (URC)	150 -200	20
Lightly Reinforced Concrete	150 - 200	25
Concrete Strips	150 - 200	25
Mortared Stone	70	20

Note: *Water absorption < 16% of their weight of water after a 1-hour soaking.

11.3.7 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.
- In waiting lanes for weigh bridges or toll booths.
- When very low maintenance capability is likely.
- When very long service life is required.
- Where natural stone is in plentiful supply.

11.3.8 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.

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Chapter 12 – Life-cycle Costing

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12.1 Introduction

12.1.1 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 12-1, for a given analysis period, one alternative might entail the use of a relatively thin, inexpensive pavement which requires multiple strengthening interventions (Alternative B), whilst another alternative might entail the use of a thicker, more expensive pavement with fewer interventions (Alternative A).

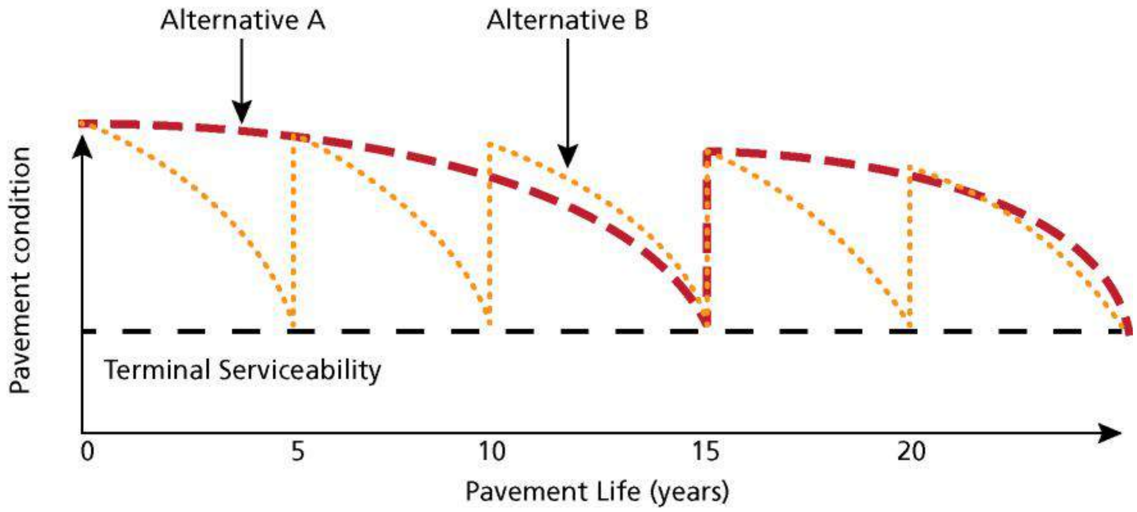

Figure 12-1: Alternative pavement options

To make the most effective use of the available resources, the designer is required to find which alternative will serve the needs of road users for a given level of service at the lowest cost over time. This task can be achieved through the use of a life-cycle economic evaluation, often referred to as "life-cycle" or "whole-of-life" costing.

12.1.2 Purpose and Scope

The main 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 methods of carrying out an LCC analysis and considers the necessary inputs to the analysis.

The scope of the chapter is on LCC analysis of alternative road pavements and surfacings and options for upgrading. However, the principles of this analysis can also be applied to comparing road projects involving alternative alignments, or alternative maintenance strategies, etc., which are outside the scope of this Manual.

12.2 Life-cycle Cost Analysis

12.2.1 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.

12.2.2 The NPV Method

The NPV method is used for undertaking an LCC analysis of alternative projects. 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:

$$NPV = \sum_{i=0}^{n-1} \frac{b_i - c_i}{(1 + \frac{r}{100})^i}$$

Where n = project analysis period in years
i = current year, with i = 0 in the base year
b_i = sum of all benefits in year i
c_i = sum of all costs in year i
r = planning discount rate expressed as a percentage

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 indicated that the project is economically justified at the given discount rate and, the higher the NPV, the greater will be the benefits from the project.

12.2.3 Components of a LCC Analysis

The principal components of LCC analysis are illustrated in Figure 12-2. These components are 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.

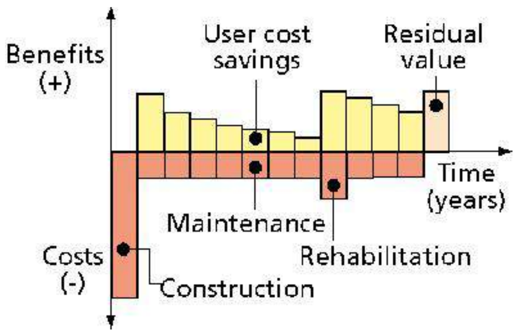

Figure 12-2: Distribution of costs and benefits during the life cycle of a road

When considering different upgrading options of similar structural capacity and surfacing type, the difference in vehicle operating costs (VOC) benefits may be considered negligible. In such cases, only the initial construction costs and the future maintenance costs of each option need to be considered in the LCC analysis.

The components of a LCC analysis associated with a particular design alternative are listed below and illustrated in Figure 12-3.

- Analysis period.
- Structural design period.
- construction/rehabilitation costs.
- Maintenance costs.
- Road user costs.
- Salvage value.
- Discount rate

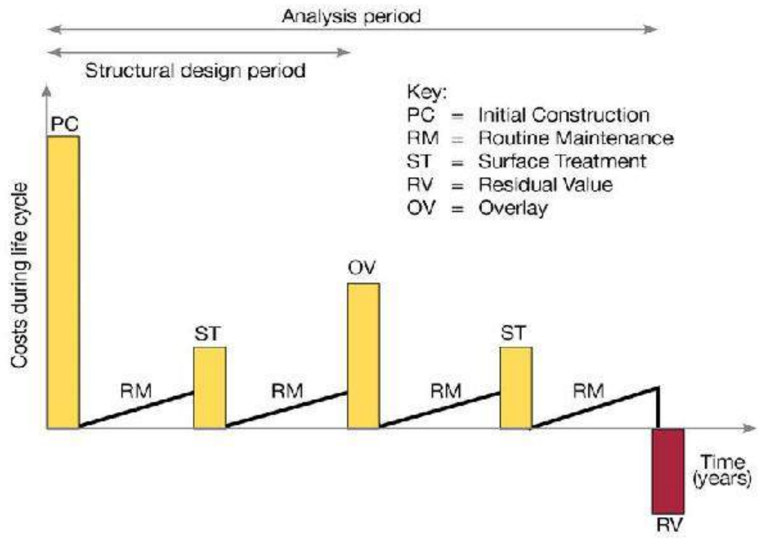

Figure 12-3: Components of a typical life cycle cost analysis

12.2.4 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 12.3)
6) Compute NPV of both options
7) Analyse results, including sensitivity analysis, if warranted
8) Decide on preferred option, i.e. the option with the highest NPV.

In view of the uncertainty of future costs, e.g. hauling distances for gravel, aggregates, 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 the end of which 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, 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, tyres, 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 12-2).

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 future construction cost to subbase level, discounted to the base year.

Discount rate

This rate must be selected to express future expenditure in terms of present values and costs. It is usually based on a combination of policy and economic considerations.

12.3 Selection Design Standard

12.3.1 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 minimise total life-cycle costs, as illustrated in Figure 12-4. Such an analysis can be undertaken using an appropriate techno-economic model, such as the World Bank's Highway Design and Maintenance Standards (HDM) model or, preferably, the Low Volume Road Economic Decision (RED) model which is customised to the characteristics of LVRs.

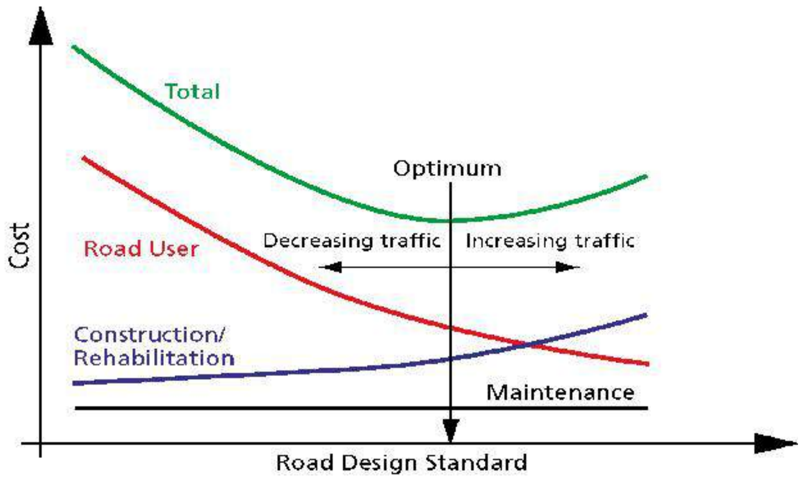

Figure 12-4: Economic analysis of optimum road design standard

As indicated in Figure 12-4, the optimum road design standard varies in relation to traffic level and the associated relative mix of construction, maintenance and user costs. Thus, as illustrated in Figure 12-5, 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 higher traffic pavement would incur higher initial construction costs but lower maintenance and VOC.

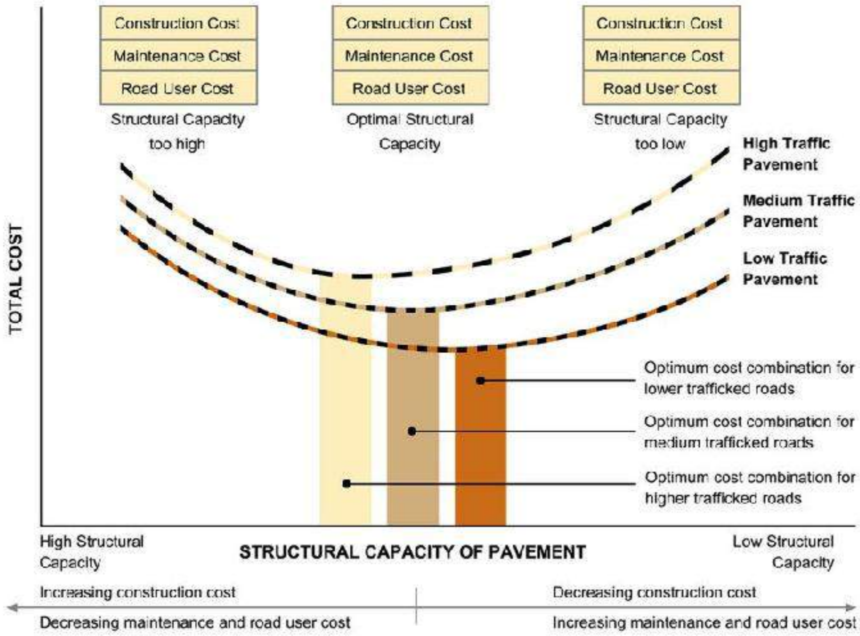

Figure 12-5: Combined cost for various pavement structure capacities

12.3.2 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 12-6 and Figure 12-7, both 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 12-6: Gravel road option

Lower construction costs, higher maintenance and road user costs.

Figure 12-7: Paved road option

Higher construction costs, lower maintenance and road user costs.

The typical components of the LCC analysis are illustrated in Figure 12-8. Such an analysis can be undertaken using an appropriate appraisal model, such as the World Bank's Roads Economic Decision (RED) model. This model, which is customized to the characteristics and needs of low-volume roads, performs an economic evaluation of road investment options using the consumer surplus approach. Using RED, the VOC relationships may need to be calibrated for local conditions. The option with the higher NPV would be the preferred one.

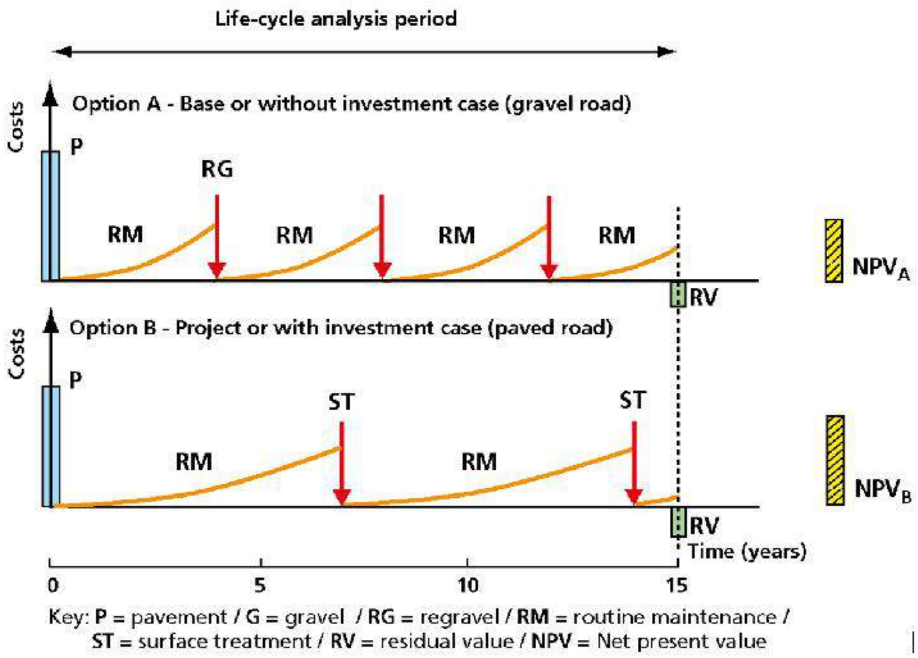

Figure 12-8: Typical components of a LCC: Gravel versus Paved road

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.

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, approximate figure of > 300 vpd to typically in the range 100 vpd – 200 vpd depending on road environmental conditions. Some of the factors influencing this reduced traffic threshold for upgrading are given in Table 12-1.

Table 12-1: 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 from non-motorised 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 12-9, which reflects the outcome of recent research carried out in the southern African region 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 for Sub-Saharan Africa, which indicated a first-generation bitumen surface at traffic of over 200 vpd.

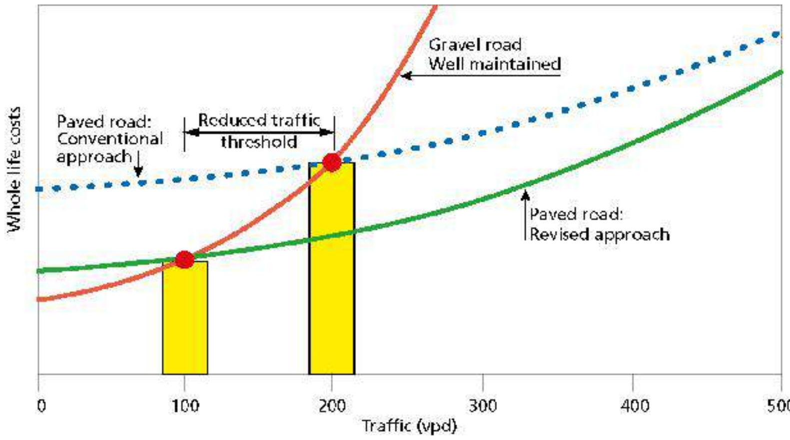

Figure 12-9: Break-even traffic levels for paving a gravel road: Traditional versus revised approaches.

12.3.3 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 12-10, Table 12-2 and Table 12-3 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 an 8% real discount rate.

As indicated in Table 12-2 and Table 12-3, the Single Otta Seal + Sand Seal Option has the lowest 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.

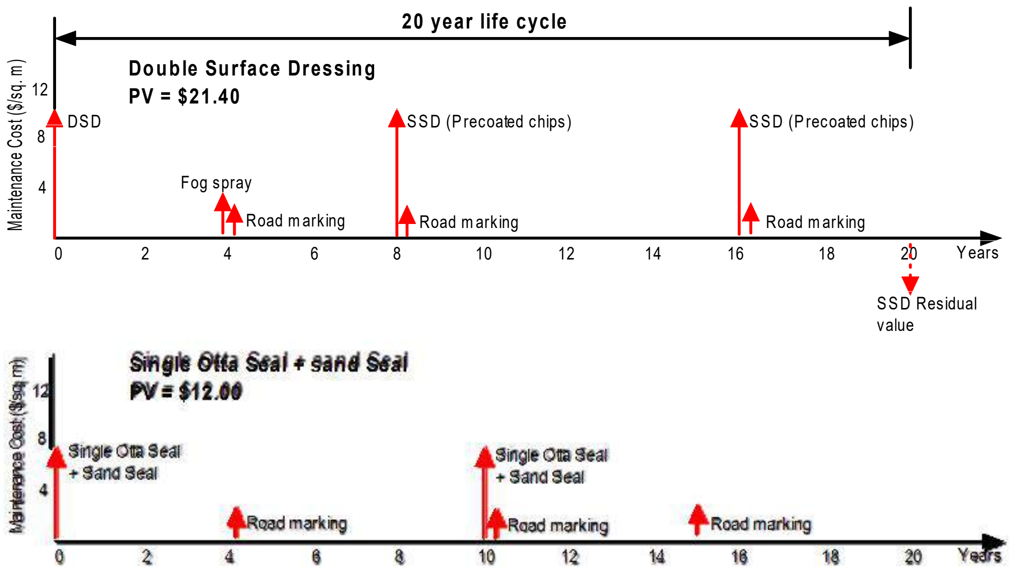

Figure 12-10: LCC comparison between a single Otta Seal + Sand Seal and a DSD

Table 12-2: Life-cycle cost analysis for Double Surfacing

Activity	Years after construction	Base Cost/m² ($)	8% Discount Factor	PV of Costs/m² ($)
1. Construct Double Chip Seal	-	10.00	1.0000	10.00
2. Fog spray	4	02.00	0.7350	1.47
3. Road marking	4	00.96	0.7350	0.71
4. Single Chip Seal (pre-coated)	8	10.00	0.5403	5.40
5. Road marking	8	00.96	0.5403	0.52
6. Fog spray	12	2.00	0.3971	0.79
7. Road marking	12	00.96	0.3971	0.38
8. Single Chip Seal (pre-coated)	16	10.00	0.2919	2.92
9. Road marking	16	00.96	0.2919	0.28
10. Residual value of surfacing	20	(5.00)	0.2145	(1.07)
				Total 21.40/m²

Table 12-3: Life-cycle cost analysis for Single Otta Seal + Sand Seal

Activity	Years after construction	Base Cost ($)	8% Discount Factor	PV of Costs ($)
1. Construct single Otta Seal + sand Seal	-	7.25	1.00	7.25
2. Road marking	5	0.96	0.6806	0.65
3. Single Otta reseal	10	7.25	0.4632	3.36
4. Road marking	10	0.96	0.4632	0.44
5. Road marking	15	0.96	0.3152	0.30
Assume life span of 20 years. Thus, no residual value.				0.00
				Total 12.00/m²

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Section 4 Miscellaneous

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13.1 Introduction

13.1.1 Background

The concepts and technology for cost-effective provision of LVRs have undergone significant advances over the last 20 to 30 years towards what now constitutes appropriate technical solutions for pavement and geometric design, as presented in Volume 1 and 2 of this Manual. Nonetheless, there are several aspects of LVR provision, which are intricately linked to successful LVR project implementation, that are not covered in-depth in this manual. However, for most of these aspects of project implementation, through planning and procurement to construction and maintenance, relevant reference documents, guidelines and manuals are available and should be used – both by clients, designers and contractors – as and when required.

13.1.2 Purpose and Scope

The purpose of this chapter is thus to highlight some of the most important aspects that should be borne in mind during planning, project execution and maintenance of LVRs.

The scope of the chapter is limited to a brief discussion of the key aspects. The bibliography provides references to the most relevant source documents, where available, on the various topics.

13.2 Engineering Adaptations to Climate Change

13.2.1 General

The projected climate change, as highlighted in *Chapter 3 – Physical Environment*, is likely to have several significant effects on the road infrastructure of most countries, but particularly on the low volume rural road network in Sub-Saharan Africa, where the roads are more vulnerable. 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 incorporating the necessary climate adaptation measures, but it will 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 prioritise them for adaptation measures. The priority would be based on 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 assist with the prioritisation, it is necessary to carry out visual assessments (in addition to the conventional routine assessments for pavement management purposes) of existing roads with particular attention being paid to the expected 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 and only those that are deemed to be particularly vulnerable should be identified and, wherever possible, improved. For existing LVR infrastructure, it is thus. in most cases, a question of "retrofitting" the most vulnerable facilities to a climate-resilient condition, but this is a costly option.

The most important and cost-effective countermeasure to meet the challenges of a changing climate is to provide the required funding for, and to carry out, regular routine and periodic maintenance in line with international best practice. This will ensure that large parts of the road infrastructure network will be functional and 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, various steps need to be followed:

- Identify the expected changes in climate along the road from the best available maps, i.e. those with the most detailed predictions. These changes will mostly 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.

- Assess the expected effects of the above changes on the infrastructure.
- Assess the road, road environment (earthworks and pavement drainage structures), and any larger drainage structures. This should follow a standard assessment protocol as outlined in the AfCAP or other relevant manuals, and concentrate on those issues not normally assessed during standard visual assessments for Road or Bridge Management Systems.

One of the biggest problems at the moment 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.

13.2.2 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 does not need to 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 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 recommended for steep gradients (> 8-10%).
- **Additional, enlarged culverts or improved culverts:** 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**: Required where the alignments are low and are being 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.
- **Re-alignment:** Re-alignments where an identified climate impact hazard and consequent engineering risk may be most cost-effectively overcome by avoidance.
- **River/stream crossing**: Existing fords or low-level bridges might be replaced by climate-resilient structures such as vented fords, or submergible multiple culverts.

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

13.3 Environmental and Social Impact Issues

13.3.1 General

The construction, rehabilitation, upgrading, maintenance and usage of road infrastructure can result in considerable negative or unwanted impacts on the natural and human environment. These impacts must be prevented or minimised in the implementation and usage of such road infrastructure projects. To do so, it is necessary to analyse the construction, maintenance and routine operations associated with a road project to identify the potential environmental and social impacts that might arise and to provide information to help design mitigation and/or preventative measures to ensure the protection of the natural and social environment and cultural heritage of Malawi.

In its broadest sense, the term environment includes both the natural or "bio-physical" environment (flora, fauna and physical features) as well as the human environment (socio-economic and cultural factors) and the interaction between them. As illustrated in Table 13-1, there are four key cornerstones of the environment:

- Ecological
- Economic
- Social, and
- Physical.

Each of the cornerstones of the environment includes a range of factors that should be considered at all stages of the planning cycle.

Table 13-1: Cornerstones of the road environment

Road Environment	**Ecological**	- impact on flora and fauna - deforestation - disturbance of natural eco-systems - decrease in biodiversity - threats to exotic and non-indigenous species - depletion of scarce material resources - regressive or progressive soil erosion
	Economic	- capital costs (design and construction) - maintenance costs - flood damage costs - loss/degradation of agricultural/arable land - sterilisation of land for future use - land value reduced (designated borrows, severed farms)
	Social	- severance/dislocation of local communities - adverse impacts on women and children - disturbance of cultural, historical and burial sites - conflicts arising from changing land use/ownership of land - traffic accidents - health and safety (e.g. danger to humans, especially children, and wildlife from drowning in borrow pits) - potential transmission of communicable diseases - construction impacts
	Physical	- aesthetic - e.g. loss of natural beauty and scars on landscape - destabilisation of soils and slopes - noise, air, water pollution - dust impact - interruption and modification of natural drainage systems

Although most LVR projects by design minimise their environmental footprint by mostly following the alignment of already existing earth and/or gravel roads, certain activities, such as the opening of borrow pits and extraction of construction materials, inevitably impact the natural and social environment in one way or another.

13.3.2 Typical Causes of Environmental Problems

The following are typical causes of environmental problems related to the provision of LVSRs.

Design defects

These create problems when they are not anticipated, and the project fails to include remedial measures. For example, measures taken to keep water off the road can cause problems elsewhere. Upstream preventative measures can interfere with natural river flows. Drains concentrate and speed up flow, sometimes causing flooding and soil erosion downstream.

Poor project documentation

Unless remedial measures are clearly reflected in project documents (e.g. Terms of Reference or contract documents), they may not be fully implemented. This often results from the use of standard contract documents that do not allow for the special circumstances of the project. For example, standard contract documents for roads often include specifications requiring the contractor to "cut and dispose of soil within the transverse profile", or to "carry surplus material to spoil". When such specifications are inadvertently applied in steep, mountainous terrain – particularly if hillsides are intensely cultivated – the dumping of soil over the edge of the road formation can have devastating results.

Presence of construction activities

Temporary site works are typically characterised by borrow pits, ruts in the road created by vehicle wheels and drainage ditches, which provide ideal breeding grounds for insects (particularly mosquitoes). Construction workers may kill local fauna for the pot, while the canteen refuse normally associated with construction camps may be a source of air pollution as well as breeding places for insects.

Weak environmental institutions

Successful remedial measures depend on the effectiveness of local environmental institutions, including those dealing with the regulation of land use. For example, when new roads are being constructed in undeveloped areas, it should be mandatory for the project to be cleared by the local planning agency responsible for dealing with the planned and spontaneous development that may occur in response to the project. However, such provisions will only be effective if the local environmental agency has the skills, manpower and authority to ensure that the contractor complies with the requirements. If the poor performance of local environmental institutions is likely to affect project implementation, this should be evaluated and attended to before the project is approved.

13.3.3 Regulatory Framework

The principal environmental law in Malawi is the Environmental Management Act (EMA) of 2018. This Act, together with the Malawi Environmental Impact Assessment (EIA) Guidelines (1997) provides the regulatory framework through which environmental issues are assessed.

The Environmental Affairs Department (EAD) has the mandate to approve environmental and social impact assessments of projects and provide the approval for the project to go to the construction stage. This takes place within the framework provisions of other regulatory, legislative and policy provisions including the Constitution of the Republic of Malawi (1995), the Malawi National Land Policy (2002), the National Environmental Policy (2004), the National Construction Policy (2014), the National HIV/AIDS Workplace Policy (2010), the Physical Planning Act (2016), the Water Resources Act (2013), the Land Act (2016) and the Public Health Act (1996) amongst others.

13.3.4 The ESIA Process

Environmental and Social Impact Assessment (EISA) is not an outcome. Rather, it is a process for improving the quality of the outcome and can be applied to any proposal. The process is flexible and adaptable and can be tailored to suit the circumstances of any road project. In essence, an ESIA evaluates foreseeable impacts, both beneficial and adverse, and helps to reveal mitigating measures and alternatives as well as to optimise positive impacts while reducing or limiting negative impacts.

The main components of the ESIA include:

- Establishing the need for the project;
- Identifying and quantifying the full range of potential impacts on the natural and social environments;
- Formulating remedial procedures for avoiding, mitigating and compensating for these impacts;
- Reflecting remedial measures in the project documents; and
- Ensuring that remedial measures are complied with during the project implementation.

The ESIA process is usually integrated into the project cycle as a means of improving the quality of the outcome. Community involvement in this process is important and necessary as it seeks to solicit information, views and concerns that only the affected community can provide. It can take many forms and fit into the process at any or all stages, depending on the type of project. It can involve a broad range of interest groups and individuals or it may only require limited involvement. The process is outlined in Table 13-2.

Table 13-2: A framework for ESIA

Phase of project cycle	Activity	Objective
Project identification	Initial Screening	- Register “danger” signals. - Avoid unnecessary investigations where impacts are likely to be minimal.
Feasibility	Environmental Appraisal	- Predict main impacts. - Assess importance of effects. - Indicate key mitigating measures. - Present implications to decision makers.
Design	Environmental and Social Impact Assessment	- Predict in detail likely impacts, including cost implications. - Identify specific measures required to avoid, mitigate or compensate for damage. - Present predictions and options to decision makers.
Commitment & Negotiation	Environmental Enforcement	- Ensure environmental mitigation measures are included in the contract documents.
Implementation	Environmental Monitoring	- Ensure environmental mitigation measures are being complied with during construction.
Operations & Maintenance	Environmental Audit	- Assess the extent of implementation in a project against the requirements derived from the ESIA. - Ensure lessons learned are incorporated in future projects.

The initial stages of any road project must include a screening exercise to determine what level of an environmental and social impact assessment must be done. For projects which are large in physical extent and/or pose particularly sensitive bio-physical or socio-economic concerns, a full Environmental and Social Impact Assessment (ESIA) would be required, out of which a full ESIA report would be produced. For projects which are smaller in physical extent, or which have less extensive potential impacts, a less extensive assessment is required, and an abbreviated ESIA report will be sufficient.

The project proponent, for example, the Roads Authority or any other road agency such as councils under the Ministry of Local Government and Rural Development for the various classes of road, would be the responsible entity to ensure that an environmental and social impact assessment is carried out in accordance with the national legislation and a report submitted to Environmental Affairs Department (EAD). The road agencies should include this component in design consultancy assignments. It is important for a design team leader to provide adequate information to the team carrying out the environmental and social impact assessment, to ensure that appropriate mitigation measures can be included in the Environmental Management Plan to be included in the submission to EAD.

Relevant information from a design team that may be necessary for the ESIA process would include:

- Road alignment and reserve width;
- Classification of materials along the alignment, with an indication of their erodibility
- Gradients along the alignment;
- Locations of potential borrow pits and quarries;
- Likely routes for borrow materials overhaul;
- Locations of potential water sources;
- Anticipated construction period;
- Technology to be used for road construction;
- Types of materials to be incorporated in road construction;
- Quantities of materials to be incorporated in the road works; and
- Types and quantities of waste to be generated.

The ESIA regulations state that all major roads outside urban areas, the construction of new roads and major improvements over 10 km in length or over 1 km in length if the road passes through a national park or game reserve area, need a full ESIA. It is important to note that each individual project must be assessed on its own merits and application made to the Environmental Affairs Department to approve the level of assessment (full or abbreviated ESIA).

13.4 Borrow Pit Management

13.4.1 General

While the effects of opening and operation of borrow pits for extraction of construction materials are an integral part of the ESIA process described above, borrow pits present a range of potential problems that warrant special considerations, as discussed below.

The identification and development of good sources of pavement construction material at regular intervals along the length of a LVR is essential for achieving cost-effective construction and ongoing maintenance operations.

Up to 70% of the construction cost of a typical LVR may relate to pavement materials production and supply. Also, aggregate replacement costs are often as high as 60% of the maintenance costs of

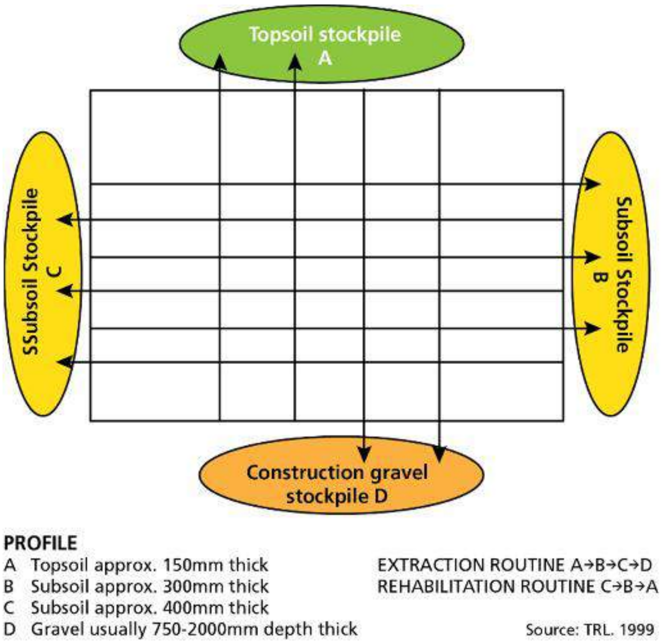

Figure 13-1: Recommended procedure for removal of overburden and stockpiling

an unpaved road. There are, therefore, significant cost-benefits that can be achieved by implementing improved borrow pit management procedures and material supply strategies.

Proper management of material sources is essential to ensure that the best quality available materials are used in the top layers of the pavement structure. The efforts made to locate these, often scarce materials for base, are of little use if this material is wastefully used in earthworks layers. Too often (and in particular for borrow pits located for low volume roads), borrow pit excavation is carried out with only the plant operator present and no correct supervision. In many cases, this results in good quality gravel getting contaminated and having to be spoiled. Good management of materials (including skilful supervision during all operations in the borrow pit) is, therefore, a critical operation in LVR construction.

An awareness of the potentially damaging effects (negative impacts) that borrow pits and quarries may have on the local environment is also required so that mitigating measures may be incorporated in the tender documents for enforcement during the construction operations.

13.4.2 Main Factors to Consider

There are many, and varied, factors to consider when locating and exploiting borrow pits, such as:

- Minimising environmental impact by reducing the dust and noise pollution for residents in the area.
- Preparation of a borrow pit plan to ensure that topsoil and overburden is stockpiled for later use in restoration of the borrow pit and separation of different material types/qualities in different stockpiles to ensure that materials are not contaminated.

Figure 13-2: Children exposed to risk of drowning and water-borne disease

- Drainage of the borrow pit to:
  - ensure accessibility and that the materials are not getting soaked;
  - prevent accidental drowning of children.
- Ensure adequate security for people and livestock by fencing, if required, and avoidance of steep cut slopes.
- Prevention of landslides where borrow pits are located on hill slopes.
- Ensuring the safety of workers and plant operators. Workmen and plant operators should receive suitable training that covers safe working practices in borrow pits and quarries. Appropriate safety clothing should be provided and may include hard hats, protective boots and road safety vests. The use of these should be mandatory.
- Minimising water course pollution.

13.5 Pavement Cross-section

13.5.1 General

Due to the low traffic levels and few incidences (on average) of vehicles meeting each other on LVRs, shoulders in the normal sense is generally not recommended (ref. *Geometric Design Manual – Part A: Rural Roads, Section 2.3.4*). Vehicles on LVRs tend to drive towards the centre of the paved surface. Thus, the structural function of shoulders, i.e. to provide lateral support for the pavement and to prevent moisture ingress into the pavement under the outer wheel track, is normally not as essential on LVRs as it would be on higher-order 2-lane roads.

However, where shoulders are provided on sections with a high incidence of NMT, the shoulders fulfil several important functions that are enhanced when they are sealed – a feature that is strongly recommended, where possible. The advantages of sealing shoulders include:

- Provision of better support and moisture protection for the pavement layers and also reduces erosion of the shoulders (especially on steep gradients).
- Improved pavement performance by ensuring that the zone of seasonal moisture variation does not penetrate under the outer wheel track.
- Reduced maintenance costs by avoiding the need for re-gravelling at regular intervals.
- Reduced risk of road accidents, especially where the edge-drop between the shoulder and the pavement is significant or the shoulders are relatively soft.

13.5.2 Width of Shoulder Sealing

This should be such as to ensure that any moisture variation is contained within the shoulders and does not penetrate the outer wheel track of the pavement. The recommended minimum paved shoulder width will depend on the extent of lateral infiltration into the shoulder at the wettest time of the year, normally +/- 1.0 m. However, narrower widths would be acceptable on single-lane carriageways where vehicles tend to use the central portion of the road.

13.5.3 Type of Shoulder Surfacing

Ideally, the same type of surfacing used on the carriageway should be extended over the shoulder as well, particularly where it is likely to be heavily trafficked as in towns and built-up areas. Where a two-layer surface treatment is used on the carriageway, at least the first layer should be extended to the shoulder with an appropriate second cover seal which may be of a different type to that used on the carriageway. The objective is to achieve a durable, close-textured surfacing on the shoulder. This will generally require an increased binder application rate to cater for the lesser trafficking of shoulder compared with the carriageway.

13.6 Labour vs. Equipment

13.6.1 General

In view of the emergence of labour-based approaches as a viable alternative to the more traditional plant-based approaches, the planning of such projects merits special consideration as a means of providing much-needed employment. Without appropriate technical and financial planning from the inception of a project, serious problems may ensue, which may ruin the initiative and bring into disrepute the practicability and objectives of labour-based projects.

Many items need to be investigated in terms of their suitability for labour-based methods of construction or maintenance. Contractual aspects need to be established and appropriate designs are undertaken. Such planning must extend beyond engineering technology and the practicality of construction and consider such factors as the financing and management of labour-based projects.

In the final instance, the use of labour-based methods in the provision of LVRs is a policy issue and the government may formulate policies favouring its use on LVR projects.

13.6.2 Project Design

Many LVR projects lend themselves readily to the applications of labour-based methods (LBM) in that the alignments normally follow the existing ground level quite closely, thereby avoiding large quantities of earthworks for which it would be uneconomic to enforce the use of LBM.

In the design of small structures, different options are available, some of which would permit the use of labour and others would not. Thus, from the onset, the designer is faced with the choice of producing designs that are, or are not, conducive to the the use of labour. In many cases, a combination of labour- and equipment-based technology would be the optimal solution.

13.6.3 Construction Strategy

General

LVRs can be constructed using a variety of work methods, plant and equipment which range from relatively inexpensive, simple hand tools to relatively expensive and sophisticated plant. This makes it possible for such roads to be constructed using either labour- or equipment-based methods of construction. Both methods offer advantages and disadvantages, depending on a wide range of factors. Further, the choice of method may be open to the contractor or may be dictated by the policy objectives of the client. Whatever the case, the contractor must devise an appropriate construction strategy to comply with the requirements of the project in the most efficient and cost-effective manner.

Labour-based technology

Labour-based technology can be defined as the construction technology which, while maintaining cost competitiveness and engineering quality standards, maximises opportunities for the employment of labour (skilled and unskilled) together with the support of light equipment and with the utilisation of locally available materials and other resources. This implies that labour-based methods do not exclude the use of machines. It is acknowledged that some work activities require inputs of conventional equipment. When applied in road construction, certain activities such as gravel transport and compaction still require trucks and compaction equipment. Nonetheless, there is a significant amount of work that can be carried out using labour.

The objective of this approach is to maximise the number of job opportunities per unit of expenditure. However, despite the substantial potential benefits offered by labour-based construction, a number of myths and problems still prevail in the minds of many people concerning this technology. These need to be fully appreciated if the labour-based approach is to be successfully deployed on LVR projects.

Common myths:

a) *Standards should be lowered to allow for labour-based methods.*

It commonly believed that standards must be lowered to accommodate the use of LBM. This is certainly not the case if projects have been designed for the use of LBM and the required training of contractors and labourers are incorporated in the project design. Examples abound with successfully executed LBM projects for construction of low volume sealed roads, as illustrated in Figure 13-3 and Figure 13-4 below.

Figure 13-3: LVSR base and CMA surfacing constructed by LBM

Figure 13-4: Shutter system for construction of ETB base by LBM

b) *Labour-based construction is out-of-date and incompatible with the modern world.*

Where employment opportunities are scarce, labourers enjoy working on well-designed and managed projects when they are afforded the required training and the remuneration is fair, particularly when task-work is used.

c) *The required quality standards cannot be attained by labour-based methods.*

Apart from large mass earthworks and large high-level structures, virtually all activities can be constructed by LBM to the required quality standards, e.g. most bituminous surfacings, pavement layers, drainage works, clearing, small structures, etc. Quality is achieved through proper site management, training, appropriate design and using the right tools for the job.

d) *Labour-based construction is only for welfare relief schemes.*

Poorly managed relief programmes have contributed to the bad reputation of LBM for infrastructure provision and should not be promoted.

e) *Poorly trained contractors will never understand tender procedures.*

A policy for the use of LBM must encompass the required training for the target group of contractors. Simplified tender- and contract documents should be used.

f) *Voluntary labour can be used to keep costs down.*

Voluntary labour on LVR construction projects would make it impossible to enforce the required workmanship standards and should not be promoted.

As shown above, these myths are ill-founded and should not prejudice consideration of labour-based construction where appropriate.

Suitability of construction activities for labour-based works

Provided the site is well managed, and the site staff and workers are afforded the required training, most construction activities can be carried out to the required quality standards by LBM exclusively or in combination with light plant and equipment. However, other activities, such as haulage of materials over long distances (typically > 1 km), are not. Moreover, construction activities such as manipulating heavy precast sections, are not possible without the use of the appropriate kind of machinery.

Figure 13-5: Tree removal and de-stumping

Figure 13-6: Screening of aggregates

To facilitate the increased use of labour-based methods, emphasis should be placed on developing designs to increase labour tasks that also develop marketable skills. Examples include:

- Stone pitched rather than concrete drains.
- Cast in-situ rather than precast culverts.
- Stone rather than concrete retaining walls.
- Cast in-situ rather than precast kerbs.

It should be appreciated, however, that some construction operations cannot be undertaken by labour if standards are not to be compromised. For example:

- Heavy compaction plant is required for road bases and sub-bases, but lighter plant that is more compatible with LBM can be used successfully if layer thicknesses are reduced.
- Sheep's-foot and pad-foot rollers are essential for compacting clay.
- A grid roller is usually required for in situ break-down of oversize material.
- Pre-collapse of collapsible soils normally requires heavy impact or heavy vibratory rollers.

Labour-based projects usually employ a relatively large number of labourers. In such a situation, the site management staff require to be particularly good "people-managers" with a strong managerial as well as technical background. They need to be familiar with local traditions and social structures in order to avoid disputes on site that could threaten the progress of construction and the sustainability of the project.

Equipment-based technology
Some projects, especially large ones, may require heavy plant and equipment for various reasons:

- Large volumes of earthworks may need to be moved.
- Haul distances are long and large quantities of fill and pavement materials may be required.
- Large volumes of materials are required from borrow pits or quarries which have to be excavated and adequately rehabilitated after completion.
- Heavy watering and compaction may be required to achieve specified in-situ densities.
- Crushing of pavement and surfacing materials, where specified, may be required.
- Large quantities of concrete or asphalt may be required.

Generally, the overall size of the project and the volume of materials to be moved within a fixed construction period, are the governing factors when determining whether labour-based or plant-intensive methods are to be used. However, even the largest plant-intensive projects can accommodate many labour-based tasks within the works, and the designer should always try to incorporate these into the contract documents, where required, to assist with the government's aim of job creation and poverty reduction.

13.7 Compaction

Effective compaction of the existing running surface of the earth and/or gravel road, which is to be upgraded, is one of the most cost-effective means of improving the structural capacity of the LVR pavement. A well-compacted running surface possesses enhanced strength, stiffness and bearing capacity, is more resistant to moisture penetration and less susceptible to differential settlement. The higher the density, the stronger the layer support, the lesser the required thickness of the overlying pavement layers and the more economical the pavement structure. Thus, there is every benefit to achieving as high a density and related strength as economically possible in the subgrade and pavement layers.

Maximizing the strength potential of a subgrade soil can be achieved, not necessarily by compacting to a pre-determined relative compaction level, as is traditionally done but, rather, by compacting with the heaviest plant available to attain the highest uniform level of density possible ("compaction to near refusal") without significant strength degradation of the particles. In so doing, there is a significant reduction in permeability as well as a beneficial gain in density, strength and stiffness, with the latter correlating directly with longer pavement life, as illustrated in Figure 13-7. For these compelling reasons, where the higher densities can be realistically attained in the field from field measurements on similar materials or other established information, they should be specified in the tender documents.

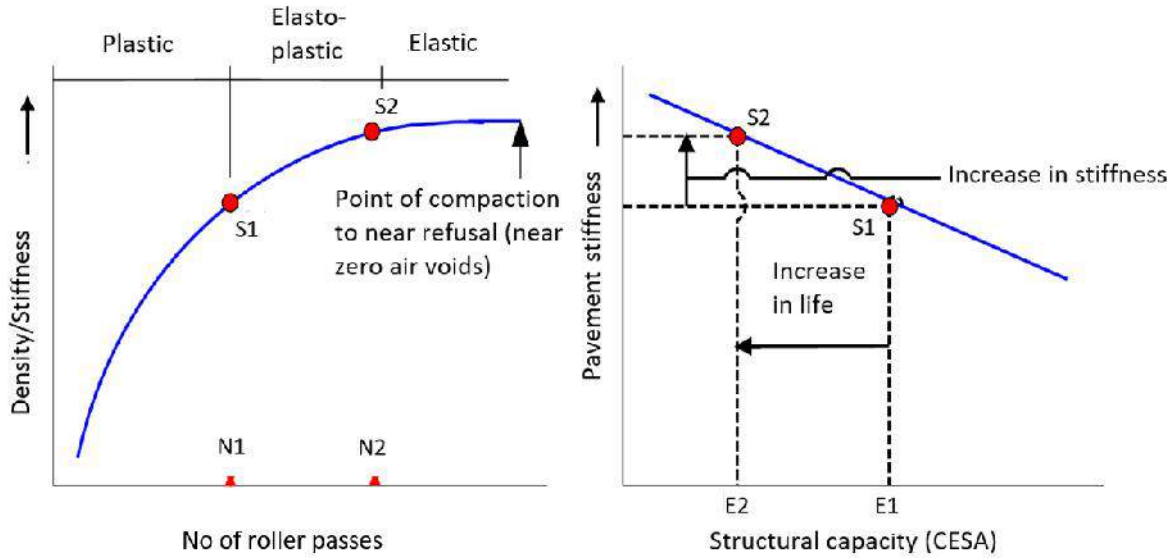

Figure 13-7: Benefits of compaction to refusal

13.8 Quality Assurance and Control

13.8.1 General

Good quality control/quality assurance (QC/QA) practices are essential to obtain satisfactory results on any road project. This is particularly the case for LVRs where naturally occurring, inherently variable materials are being used and it is essential that the underlying design assumptions are achieved on site. This includes critical factors such as the use of materials of acceptable quality and attainment of the minimum compaction requirements and pavement layer thicknesses specified. Unless these, and other specified requirements are met, such as an adequate quality surfacing to waterproof the pavement structure, and effective internal and external drainage, then the premature failure of the LVR is likely.

In developing a QC/QA system, it should be borne in mind that LVR projects in Malawi are often relatively small in size and widely scattered in remote areas with limited facilities. Moreover, the speed of construction is relatively slow and the available resources, as well as skills with small contractors, are at a relatively low level. It is, therefore, necessary that while developing a suitable QC/QA system for construction, such constraints are borne in mind. Thus, the types of quality control and their frequency must be judiciously selected so as to be achievable under the prevailing conditions.

13.8.2 Approach

There are various means of ensuring that an acceptable quality of the final LVR is achieved. Each is separate, and each has an important role to play. Together, they consist of a suite of procedures that work together to ensure good quality. A clear distinction should be drawn between the following:

- Quality Plan (QP).
- Quality Assurance (QA).
- Quality Control (QC).
- Production Control (PC).
- Acceptance Control (AC).

The differences and functions of each component of the Total Quality Management System (TQMS) are explained below.

13.8.3 Components of a TQMS

Quality Plan (QP)

This refers to a written plan submitted by the contractor, which is reviewed and approved by the client/ supervising engineer. This document clearly demonstrates how the contractor will control the processes used during construction in order to meet the requirements set out in the technical specifications. The QP will typically include the sequence of tests (QC tests) to be performed on the materials intended for use at a prescribed frequency, with the objective of demonstrating that the intent of the specification is being satisfied.

The tender documents should include the requirement that the contractor must present his project Quality Plan that he/she intends to follow during the working process.

Quality Assurance (QA)

QA is the documentation required to show that the contractor is following the Quality Plan. It incorporates standard procedures and methodologies and applies to all site activities aimed at significantly reducing or eliminating, non-conformance before it occurs. The QA Plan should be followed by everyone on site and checked by both the supervising engineer and the contractor. QA activities are determined before construction work begins and are performed throughout construction. Components of a QA system typically include process checklists and construction methodology (contractor's work plan).

Quality Control (QC)

This refers to measured quality-related attributes associated with the construction of various aspects of the project. QC is generally concerned with measuring properties and checking that specifications have been met consistently throughout the project. It does not in itself create higher quality. Examples of quality control activities include site inspections, field and laboratory testing. Such activities are performed after the work has been completed. A good practice for QC is the development and use of a checklist for monitoring and inspecting the construction of the pavement system.

Production Control

This is carried out by the contractor for the purpose of satisfying himself that chosen methods and materials meet the specified standards. Production control serves as an early warning for the contractor and helps reduce his risk of failure and associated additional cost to himself of remedial work. The contractor may be obliged to submit results from the production control to the supervising engineer and may, in some cases, these may be taken as part of the acceptance control.

Acceptance Control

This is carried out by the supervising engineer to check compliance with the specified standards and to enable payments to be made. Acceptance control makes use of confirmed QA and QC testing. Results from acceptance control will normally form part of the as-built data, which provides the basis of the road inventory kept by the responsible road agency.

Quality control supervision, therefore, comprises two principal elements, namely site inspection and laboratory and in-situ testing. A large component of the latter is compaction control and testing.

Site Inspection

The works are inspected visually to detect any deviation from the specified requirements. Visual assessment is an essential element of pavement layer approval, particularly in the identification of oversize material in lower pavement layers or in a gravel wearing course. Physical measurements of thickness, widths and crossfall are an essential element of this assessment. This activity is supplemented by simple in-situ checking of specified procedures; for example, the temperature of bitumen and spray rates, concrete slump, etc.

Laboratory and in-situ testing

Materials, as well as the finished product, are subject to laboratory testing for such characteristics as density and strength. On larger projects, it may be possible for the contractor to set up and maintain a basic field laboratory for routine tests for quality control testing required on a day to day basis. The field laboratory will normally have test equipment that does not require an electric power supply and is relevant to the project specifications. There are also portable field test kits, such as the Gravel Test Kit supplied by CSIR of South Africa, which includes simple equipment for basic control tests.

Field compaction control

One of the most critical quality control activities is the field density compaction tests, the outcome of which could have a significant bearing on the performance of the road. The Sand Replacement Test, sometimes used in conjunction with Nuclear Density testing, is commonly used for compaction control on LVR projects. Alternatively, these tests may be replaced for quality control purposes by the easier-to-perform DCP test, initially in conjunction with in situ density and moisture content testing for correlation purposes to establish a “target” DN value, as illustrated in Figure 13-8 below, which signifies that satisfactory density has been achieved. Using the DCP does, however, rely on careful control of the compaction moisture content during construction and the control tests must be carried out immediately after compaction before the layer dries back.

It should be noted that the Target DN for compaction quality control will always be higher than the Laboratory DN which is used for design, due to the lesser confinement of the material in the field compared to the confinement in the CBR mould as well as the pore pressure which may be present.

Compaction control is typically based on absolute requirements and spot tests. However, the number of such tests is often too low for a high level of statistical reliability and therefore does not necessarily ensure a well-defined quality of the product. It is for this reason that a statistical approach to quality control should be adopted, particularly for the larger LVR projects. Works and materials are accepted or rejected based on the requirements given in the SATCC Standard Specification Road Works or other such specifications as provided/approved by the RA.

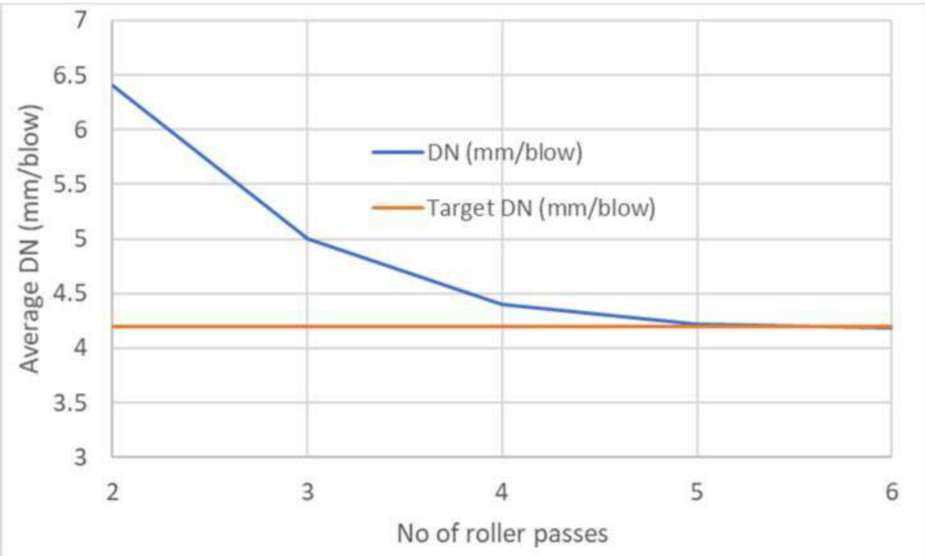

Figure 13-8: Establishment of Target DN for compaction quality control

Quality Assurance/Quality Control in practice

The construction specification establishes the framework for QC/QA. With a method specification, the individual responsible for QC would document the equipment utilised and continuously monitor its activities during operation. The QA may be by certification of QC tests and reports with an intermittent inspection.

With an end-result specification, the individual responsible for QC would perform frequent testing at the start of the process, testing for changed conditions, and some testing for verification. The QA would typically be a prescribed number of tests for a specific quantity of materials at random locations.

Statistical processing of the test data may be used to determine the acceptance of the results and are addressed in the SATCC Standard Specifications for Road Works. Statistical methods of QC offer the following advantages:

- Both the quality level and the variability of the product are taken into account in the assessment, thus providing greater production flexibility and potentially more economic gain;
- The need for engineering judgment is reduced since no such judgment is required to assess the material against the acceptable control limits determined; and
- The introduction of a conditional acceptance range below the previous coincident acceptance/rejection limit will result in greater procedural flexibility and reduced numbers of disputes in the case of products of marginal quality.

Quality Control with limited resources

The resources available for quality control in the construction of LVRs are sometimes limited, depending upon the size of the contract. In such a situation, it is important to utilise as efficiently as possible whatever resources are available by supplementing conventional control methods with other practical procedures as described below.

- Stockpiling as a means of selecting qualities and ensuring the known quality of the materials being used (stockpiling carried out preferably in the borrow pit rather than on the road).
- Good management procedures to ensure that materials are used to their full potential and to prevent rejection of materials after transportation to the road.
- Control by observation of construction procedures by an experienced practitioner.
- Proof rolling (e.g. with loaded trucks) to test the stability of layers before proceeding with further construction.
- Use of methods for direct strength measurements by correlation with known parameters (e.g. probing methods such as DCP and others).
- Laboratory testing for ‘calibration’ of method specifications.
- Laboratory testing of typical material sources for ‘calibration’ of visual observations.

In addition to the above, the following quality control activities are not expensive to undertake and can make a significant difference to the quality of the constructed LVR:

Bituminous surfacing:

- Visual inspection of all surfacing equipment.
- Check that spray nozzles are not blocked.
- Check that bitumen spray temperature is correct.
- Ensure that the spray rate is correct.
- Ensure that sheets for start and end points are in place.
- Ensure that longitudinal and transverse joints are correctly done.
- Ensure that rolling is timely and correctly carried out.

Base finishing:

- Ensure that the base is not too rough and/or too open.
- Ensure that ‘biscuit’ layers are not present. The use of a geological hammer/pick can be used to identify such flaws.
- When priming is carried out, ensure light watering takes place prior to spraying prime.

Pavement layers and subgrade:

- Test stockpile prior to transporting material to the site.
- Ensure that the tested stockpile(s) is the only one(s) used.
- Prepare a method specification for compaction control, e.g. by using a DCP.
- Prepare a watering plan for adding water to the material.

Compaction Quality Control

Compaction is a crucially important aspect of road construction as it substantially influences the long-term durability and performance of the pavement structure and, hence, the whole-life cost of the road. Thus, this aspect of the construction process must be carefully controlled on site to ensure that the specified densities are met in a consistent manner. The main aspects that need to be considered, other than routine material control testing, are the field density attained, the layer thickness and surface finish of the pavement layer being compacted.

The pavement layers (or gravel wearing course) should be compacted at least to the minimum specified density assumed in the pavement design, but preferably to refusal in view of the benefits that can be obtained in higher material strengths from such higher compaction densities. This requires that the materials be processed at or about Optimum Moisture Content and rollers of adequate mass are used. Trial sections should be constructed using the materials and plant that will be used on site, and density increase monitored for each pass of the roller. Compaction control can make use of any density determination method (nuclear, sand replacement, etc.) or, alternatively, use of the DCP as discussed above. Project specifications and payment items in construction contracts will have to reflect this approach.

13.9 Maintenance

13.9.1 General

Road maintenance is an integral component of the LVR provision process, the type and cost of which is influenced significantly by decisions made during the preceding planning, design and construction phases. Proper maintenance contributes to the preservation of the road asset and to prolonging the road's life to its intended service duration. Without adequate maintenance, roads deteriorate rapidly, become dangerous and costly to use and, ultimately, the costs to the economy are substantial.

Whereas the design and construction of LVSRs are dominated by engineering issues, maintenance is essentially a multi-dimensional issue in which the policy, management and technical aspects dominate and are influenced by political, social and institutional issues. For example, the government may formulate a policy to use maintenance works as a poverty alleviation tool through appropriate community involvement.

As highlighted in Section 13.2 above, LVRs are particularly vulnerable to lack of or deferred maintenance due to the extensive use of local, often moisture-sensitive materials, for the pavement construction. This vulnerability is further exacerbated by the projected climatic changes in the coming decades. To ensure that LVRs are regularly and adequately maintained should, therefore, be of the highest priority for road agencies.

13.9.2 Importance of maintenance

The case for maintenance is a compelling one. Having spent time, effort and money in planning, designing and constructing a LVR, it is vital to ensure that the asset is preserved by timely and effective maintenance. Although the pre-construction and construction phases are extremely important, in economic terms, they are only the "cost" phases from which no benefits are derived. Indeed, the major benefits are derived from the maintenance and operation phases of the project cycle, which occupy by far the largest proportion of the total project cycle, during which the benefits for which the road was originally constructed will be realised.

Thus, it makes sense to prolong the life of the road as much as possible by carrying out effective and efficient maintenance in a timely manner in order to:

- prolong the useful life of the road and the period of time during which benefits are incurred;
- postpone as long as possible the need for capital replacement;
- reduce the cost of operating vehicles on roads; and
- help to keep the roads open and enable greater regularity, punctuality and safety of road transport services.

The first two factors correspond most directly to the interest of the roads agencies, the third to that of operators of vehicles, and the fourth, more generally, to that of the inhabitants of the area traversed by the road.

13.9.3 Typical Maintenance Activities

Maintenance activities are either cyclic, reactive, or indeed both, and can be of a routine or periodic nature. Cyclic activities are those that are carried out at regular intervals. Reactive activities are those that are carried out in response to an occurrence e.g. erosion, carriageway marking or a condition defect exceeding values dictated by maintenance standards, e.g. rutting greater than a given value.

Table 13-3: Typical maintenance activities

Works category	Maintenance activity	Type	
		Cyclic	Reactive
Routine maintenance	**General**		
	Grass cutting	X	X
	Removal of obstacles		X
	Culvert clearing / repair	X	X
	Bridge clearing / repair	X	X
	Drain clearing	X	X
	Erosion control / repair		X
	Carriageway markings		X
	Repairing road signs		X
	Pavement		
	Pothole repairs		X
	Surface patching (local sealing)		X
	Crack sealing		X
	Edge repairs		X
Periodic maintenance	Rejuvenation seal		X
	Resealing		X
	Shoulder re-gravelling / reshaping		X

Many of the activities in Table 13-3 can be carried out cost-effectively using labour-based methods. Some periodic maintenance work may still require specialised equipment, e.g. bitumen sealing operations, but labour-based methods can be used for many activities.

Various forms of maintenance contracts can be applied, e.g. "length-man" or performance-based contracts, with the view to minimise organisational/institutional arrangements and costs.

13.10 Overload Control

Incidences of vehicle overloading can have a significant negative impact on the performance of a LVSR. The designs for LVSRs are based on fatigue failure i.e. repetitive loading for long periods of time. The overloading of individual trucks may cause irreversible plastic deformation which will lead to excessive rutting or general deformation of the pavement. In the same manner, the surfacing may fail prematurely through the crushing of aggregate, stripping, shear failures, bleeding and other types of distress.

Figure 13-9 illustrates the adverse impact of overloading on a road pavement and the consequent need for unnecessarily high maintenance and rehabilitation costs. Naturally, every effort should be made to limit the amount of overloading (illegal loading). Ideally, overload control on LVRs should be enforced through the implementation of vehicle load regulations by the use of portable weighbridges. Physical measures of a permanent nature in the form of steel gantries or barriers could also be considered to prevent access to heavy vehicles on LVRs, especially on typical access roads. However, such measures may require political acceptance and amendment of existing regulations.

While the design process should account for the amount of heavy vehicle axle loads in determining the design traffic loading (refer to *Chapter 6 – Traffic*), the specific effects of the very heavy abnormal axle loads on the pavement must be considered in finalising the design. The challenge is that the traffic loading characteristics for LVSRs are not well known and is often difficult to predict for the design life of the individual road.

In situations where overloading is likely to occur, special attention must be given to the quality and strength of all the pavement layers during construction. Amongst other measures, there may be justification in adopting a higher traffic design loading, which will result in the need for increased quality and thickness of the base and subbase/subgrade layers. The specific measures that the Engineer may deem necessary should, ideally, be based on either proven local practice or at least specialised advice/analysis in order to maintain a well-balanced structure.

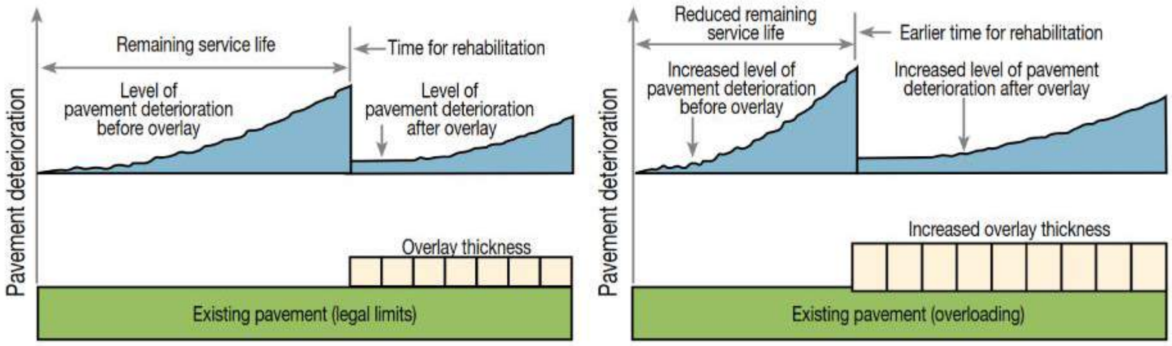

Figure 13-9: Impact of overloading on pavement performance

Examples of situations where overloading and associated pavement damage is likely to occur include:

- Mining output and transportation of very heavy equipment, both of which cause high stress and strain on the pavement and the surfacing.
- Transportation of agricultural output. Medium trucks are the most commonly used, but there are cases where very heavy trucks, including articulated trucks (18 tons or more), have been observed.
- Logging activities. Logging trucks are usually heavily overloaded, and their control is difficult. Logging trucks, the drivers of which are fully aware that their vehicles are overloaded, often have a tendency to travel at night to avoid detection.
- Construction activities and haulage of materials from quarries and borrow pits. Haulage trucks or tipper trucks can affect the performance of low volume roads and surfacings significantly.

In view of the above, the current and future functionality of the LVSR should be considered to evaluate these risks. This should be assessed at the design stage and also on an ongoing basis when the road is in-service because mitigating measures can be carried out as part of maintenance. This may include situations where there is a sudden increase in heavy vehicles. Regular assessments and re-evaluations of in-service conditions are required. This should be followed by the reprogramming of maintenance activities and designing interventions to suit the in-service conditions. Also, where reseal cycles are scheduled periodically at predetermined intervals the period between interventions may need to be shortened accordingly. If the road is upgraded for a higher traffic volume in terms of traffic loading, then strengthening of the pavement and/or surfacing may be necessary, e.g. a stronger surfacing may be considered for reseal to cater for increased traffic. In the same manner, the cycles can be lengthened accordingly if the traffic decreases significantly.

Figure 13-10: Overloaded timber truck

13.11 Bibliography

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Glossary of Terms

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.

Design Speed
The maximum safe speed that can be maintained over a specified section of road when conditions are so favourable that the design features of the road govern the speed.

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.

Distributor
A vehicle comprising an insulated tank with heating and circulating facilities and a spray bar capable of applying a thin, uniform and predetermined layer of binder.

Drainage
Interception and removal of ground water and surface water by artificial or natural means.

Dressed Stone
See Cobble Stone

Drift
A stream or river crossing with a hard platform at bed level for vehicles to traverse and over which the stream or river water can flow. The structure allows for complete overtopping.

Earth Road
See ENS.

Embankment
Constructed earthworks below the pavement raising the road above the surrounding natural ground level.

ENS (Engineered Natural Surface)
An earth road built from the soil in place at the road location, and provided with a camber and drainage system

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.

Fog Spray
A light application of diluted bitumen emulsion to the final layer of stone of a reseal or chip seal or to an existing bituminous surfacing as a maintenance treatment.

Formation
The shaped surface of the earthworks, or subgrade, before constructing the pavement layers.

Geocells
Typical cellular confinement systems are made with ultrasonically-welded high-density polyethylene (HDPE) or Novel Polymeric Alloy strips that are expanded on-site to form a honeycomb-like structure which may be filled with sand, soil, rock or concrete.

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.

Peri-urban
Immediately adjoining an urban area or village area.

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.

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.

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.

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.

Tack Coat
A coat of bituminous binder applied to a primed layer or to an existing bituminous surface as a preliminary treatment to promote adhesion between the existing surface and a subsequently applied bituminous layer.

Traditional Methods of Pavement Design
Refers to well-established methods of pavement design that were developed primarily for high volume roads and do not cater for the specific requirements of low volume roads.

Ultra-Thin Reinforced Concrete Pavement (UTRCP)
A layer of concrete, 50 mm thick, continuously reinforced with welded wire mesh.

Vented Drift
A low-level structure constructed across streams or rivers with openings to permit water to flow below road level. The vented rift may become submerged in flood conditions.

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.